Ministry of Mines and Energy
Secretariat of Energy Planning and Development

Manual for Hydropower Inventory Studies of River Basins

2007 edition

CEPEL Grupo Eletrobrás

PROJETO ESTAL Ministério de Minas e Energia

Portuguese Edition (December 2007)

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PREFACE

The publication of this Manual for Hydropower Inventory Studies of River Basins is part of a broader resumption of planning by the Ministry of Mines and Energy, as part of its responsibility in the government for devising policies for the energy sector, in line with the guidelines laid down by the National Council for Energy Policies (CNPE).

This Manual is the outcome of the work undertaken in a broad-based review and update of the previous manual published in 1997 by ELETROBRÁS. It reflects current Brazilian experience in Inventory Studies of river basins and the changes that have taken place over the last ten years in the country's electricity sector, especially with regard to legislation, the environment, water resources and institutional changes.

The review of the Manual was initiated in 2004 by the Ministry of Mines and Energy, which subsequently contracted Electric Energy Research Center (CEPEL) to coordinate and consolidate the work under its supervision. The project also received support from the World Bank in the form of an Energy Sector Technical Assistance Loan (ESTAL). A working group was set up of professionals with technical expertise from different companies with experience in Inventory Studies, as well as representatives from sector associations.

One important new ingredient in the latest edition of the Manual is the inclusion of Integrated Environmental Assessments as part of a broader approach addressing issues of sustainable development. In addition, the methodology takes account of the multiple uses of the waters in the river basins under study, in line with the National Plan for Water Resources.

It is, then, with great satisfaction that the Ministry of Mines and Energy presents electricity sector agents with this new Manual for Hydropower Inventory Studies of River Basins, an up-to-date tool that incorporates the latest concepts, methodologies and technical advances produced in Brazil and internationally for assessing hydropower potential. It will surely prove fundamental for the planning and development of new hydropower projects in Brazil, and become an essential reference in the fulfillment of the country's energy requirements for the medium and long term, as set forth in the 2030 National Energy Plan.

Finally, the Ministry of Mines and Energy takes this opportunity to express its gratitude for the collaboration of the institutions involved in this process, which put every effort into ensuring its success by lending the expertise of their most highly qualified technical personnel, representatives of Brazil's acknowledged competence in studies and designs for hydropower projects.

Brasília, December 2007

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Appendices
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PREFACE TO ENGLISH LANGUAGE VERSION

The Portuguese language 2007 edition of this Manual for Hydropower Inventory Studies of River Basins (*Manual de Inventário Hidrelétrico de Bacias Hidrográficas)* was launched in December 2007 after two years of work by a multidisciplinary team coordinated by the Electric Energy Research Center (CEPEL) and made up of professionals from the Brazilian electricity sector and other sectors that interface with hydropower development in Brazil. At that time, it was understood that this Manual was important for inventory studies in Brazil because it would present the latest concepts, methodologies and technical advances from Brazil and abroad for use in assessments of hydropower potential.

Since 2008, several inventory studies have been carried out or reviewed based on the 2007 edition of the Manual and using the SINV computer system for the energy and socioenvironmental studies. The SINV computer system, developed by CEPEL, was updated during the review of the Manual. The utilization of the Manual and the SINV system assures quality and homogeneity in the results of studies of this nature, while also simplifying the analysis required at the approval stage.

The Ministry of Mines and Energy's decision to invest in the translation of the Manual into English came in response to the several presentations given of the results of inventory studies at national and international forums and the use of this Manual in inventory studies of bi-national river basins. It also aims to help divulge the country's efforts towards developing its hydropower potential in a sustainable way and show the technical, institutional and legal advances of the Brazilian electricity sector. Given the technical nature of the project, the Ministry again contracted CEPEL, this time to coordinate the translation.

Around the world, the sustainable development of hydropower is also being used to foster social and economic development, especially for local communities, and as an important tool in the fight against global warming. Given that Brazil's environmental legislation is one of the most stringent in the world, the approach the country is adopting to face the challenge of continuing to expand its energy and electricity matrices based on renewable resources is to take socioenvironmental issues into account from the very earliest planning stage. The basic approach in hydropower inventory studies of river basins is, then, to find a balance between cost-effective energy production and socioenvironmental considerations, while also taking account of multiple water uses.

By bringing out an English translation of the 2007 edition of the Manual for Hydropower Inventory Studies of River Basins, the Ministry of Mines and Energy intends to offer the international community a basic reference to be considered in the portfolio of possible energy solutions in developing countries, which should be of particular interest to multilateral financing and development organizations.

The use of this Manual will highlight opportunities for the sustainable development of hydropower. This involves taking account of environmental issues, and enhancing synergies between hydropower development, efficient water use, local and regional social and economic development, improved access to clean energy, job creation and reduced emissions.

Brasília, June 2010

Márcio Pereira Zimmermann
Minister of State of Mines and Energy
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WORKING GROUP

This Manual is the outcome of a review process that started in 2005 when CEPEL (Electric Energy Research Center) was contracted by the Ministry of Mines and Energy by means of the World Bank's Energy Sector Technical Assistance Loan (ESTAL). The review is based on the 1997 manual published by Eletrobrás (*Manual de Inventário Hidrelétrico de Bacias Hidrográficas*).

In order to coordinate the review process, a Working Group was set up, made up of a Core Group, an Executive Group and an Advisory Group. The members of the Core Group were representatives from the Ministry of Mines and Energy (coordinator), CEPEL (Executive Secretary), Eletrobrás (*Centrais Elétricas Brasileira*) and EPE (Energy Research Company). The aim of this group was to assist the ministry in its decision-taking processes.

The brief of the Executive Group was to undertake an analysis of the 1997 manual and propose improvements. As such, it was subdivided into five theme-based groups: institutional, engineering studies, energy studies, water resource studies and socioenvironmental studies. Professionals from different companies from the electricity sector with experience in Inventory Studies were invited to sit on these subgroups, as well as representatives from ANEEL (National Electricity Agency), the Ministry of the Environment, ANA (National Water Agency) and other environmental and water resource entities.

The Advisory Group was formed in order to obtain contributions from sector associations, universities and consultants specialized in Inventory Studies.

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ABBREVIATIONS USED

Abbreviation	Name or explanation in English	Name in Portuguese
ABNT	Brazilian technical standards association	Associação Brasileira de Normas Técnicas
ANA	National Water Agency	Agência Nacional de Águas
ANEEL	National Electricity Agency	Agência Nacional de Energia Elétrica
ART	Document that defines the limits of the professional services to be rendered by members of CREA and by what entity they will be rendered	Anotação de Responsabilidade Técnica
BNDES	National Development Bank	Banco Nacional de Desenvolvimento Econômico e Social
CEPEL	Electric Energy Research Center	Centro de Pesquisas de Energia Elétrica
CMSE	Brazilian Electricity Sector Monitoring Committee	Comitê de Monitoramento do Setor Elétrico
CNI	National Confederation of Industries	Confederação Nacional da Indústria
CNRH	National Council for Water Resources	Conselho Nacional de Recursos Hídricos
COCAR	Cartography Committee, Ministry of Planning, Budgets and Management	Comissão Nacional de Cartografia Ministério do Planejamento, Orçamento e Gestão
CONAMA	National Council for the Environment	Conselho Nacional do Meio Ambiente
CONCAR	National Cartography Council	Conselho Nacional de Cartografia
COPEL	Paraná Energy Company	Companhia Paranaense de Energia
CPRM	Department of Mines and Energy mineral research entity (Geological Survey of Brazil)	Companhia de Pesquisa de Recursos Minerais
CREA	Regional Council for Engineering, Architecture and Agronomy	Conselho Regional de Engenharia, Arquitetura e Agronomia
CUR	Reference Unit Cost	
DEA	Distributed Environmental Assessment	
DERs	Highway departments (states)	Departamento de Estradas de Rodagem
DHN	Brazilian Navy Department of Hydrography and Shipping	Diretoria de Hidrografia e Navegação
DNIT	National Department for Infrastructure and Transportation	Departamento Nacional de Infra-Estrutura e Transporte
DNPM	National Department of Mining	Departamento Nacional da Produção Mineral
DSG	Brazilian Navy Geographical Survey	Diretoria de Serviço Geográfico
EAP	economically active population	
EAR	residual stored energy	Energia Armazenada Residual
EIA	Environmental Impact Assessment	Estudo de Impacto Ambiental
EMBRAPA	agricultural research company	Empresa Brasileira de Pesquisa Agropecuária
EPE	energy research company	Empresa de Pesquisa Energética
FCP	Ministry of Culture foundation for African-Brazilian affairs	Fundação Cultural Palmares
FGV	social science research and education entity	Fundação Getúlio Vargas
FUNAI	entity that divulges information about Brazilian indigenous peoples	Fundação Nacional do Índio
GIS	geographic information system	
GTON	North Region Technical Group	Grupo Técnico Operacional da Região Norte
HDI	Human Development Index	
IA	Negative socioenvironmental index of a cascade option on the environmental system	
IAC	Negative socioenvironmental index of a cascade option on each synthesis component in the study area	
IAE	Positive socioenvironmental index of a cascade option relating to each element	
IAp	Positive socioenvironmental index of a cascade option on the environmental system	
IBAMA	Brazilian Environmental Institute	Instituto Brasileiro de Meio Ambiente e dos Recursos Naturais Renováveis
IBGE	Brazilian Institute of Geography and Statistics	Instituto Brasileiro de Geografia e Estatística
IEA	Integrated Environmental Assessment	
INCRA	institute for land reform	Instituto Nacional de Colonização e Reforma Agrária
INMET	meteorology institute	Instituto Nacional de Meteorologia
INPE	institute for space research	Instituto Nacional de Pesquisa Espacial

Abbreviation	Name or explanation in English	Name in Portuguese
INPRA	socioenvironmental research institute	Instituto Internacional de Pesquisa e Responsabilidade Socioambiental Chico Mendes
IPARJ	Rio de Janeiro based institute for anthropological research	Instituto de Pesquisas Antropológicas do Rio de Janeiro
IPEA	entity that carries out economic research	Instituto de Pesquisa Econômica Aplicada
IPHAN/MinC	Ministry of Culture heritage protection agency	Instituto do Patrimônio Histórico e Arqueológico Nacional
ISA	socioenvironmental NGO	Instituto Socioambiental
ISS	Tax on Services	Imposto sobre Serviços
LI	Installation License	Licença de Instalação
LP	Preliminary License	Licença Prévia
MAPA	Ministry of Agriculture, Fisheries and Supplies	Ministério de Agricultura, Pecuária e Abastecimento
MCidades	Ministry of Cities	Ministério das Cidades
MDA	Ministry of Agrarian Development	Ministério do Desenvolvimento Agrário
MDIC	Ministry of Development, Industry and Foreign Trade	Ministério do Desenvolvimento, Indústria e Comércio Exterior
MDS	Ministry of Social Development and Fight against Hunger	Ministério do Desenvolvimento Social e Combate à Fome
MI	Ministry of National Integration	Ministério de Integração Nacional
MMA	Ministry of the Environment	Ministério de Meio Ambiente
MME	Ministry of Mines and Energy	Ministério das Minas e Energia
MT	Ministry of Transport	Ministério dos Transportes
MTur	Ministry of Tourism	Ministério do Turismo
NA	water level	Nível d’água
OEMAs	state environmental entities	Órgãos Estaduais de Meio Ambiente
ONS	entity responsible for coordinating and controlling the operation of electricity generation and transmission facilities included in the SIN	Operador Nacional do Sistema Elétrico
OPE	Standard Eletrobras Cost Estimate	Orçamento Padrão da Eletrobrás
OTEP	technical education and research organizations	Organizações Técnicas de Ensino e Pesquisa
PBA	Basic Environmental Plan (part of installation licensing process)	Plano Básico Ambiental (parte do processo de obtenção da Licença de Instalação)
PCA	Environmental Control Plan (part of operational licensing process)	Plano de Controle Ambiental (parte do processo de obtenção da Licença de Operação)
PIE	Independent Electricity Generator	Produtor Independente de Energia Elétrica
PNRH	National Plan for Water Resources	Plano Nacional de Recursos Hídricos
PPA	Multi-Year Plan	Plano Plurianual
PRH	Water Resources Plan	Plano de Recursos Hídricos
RAIS	annual list of information on society	Relação Anual de Informações Sociais
RIMA	Environmental Impact Report	Relatório de Impacto Ambiental
SBE	Brazilian Society of Speleology	Sociedade Brasileira de Espeleologia
SEAP	Department of Aquaculture and Fishing	Secretaria Especial de Aqüicultura e Pesca
SEMA	state departments of the environment	Secretarias estaduais de meio Ambiente
SEPPIR	Department for Racial Equality	Secretaria Especial de Políticas de Promoção da Igualdade Racial
SERH	state departments of water resources	Secretarias Estaduais de Recursos Hídricos
SIN	National Interconnected System	Sistema Interligado Nacional
SINDUSCON	civil construction industry union	Sindicato da Indústria de Construção Civil
SIPAM	Amazon protection system	Sistema de Proteção da Amazônia
SisEvapo	system for evaluating net evaporation from reservoirs	Sistema de Avaliação da Evaporação Líquida dos Reservatórios
SISNAMA	National system for coordinating environmental actions	Sistema Nacional do Meio Ambiente
SIUC	information system on conservation units run by IBAMA	Sistema de Informações de Unidades de Conservação/IBAMA
SPHAN	precursor of IPHAN	Serviço de Patrimônio Histórico e Arqueológico Nacional
SRHU/MMA	Department of Water Resources	Secretaria de Recursos Hídricos
TAR	reference power rate	Tarifa Atualizada de Referência
UNDP	United Nations Development Programme	
WHO	World Health Organization	

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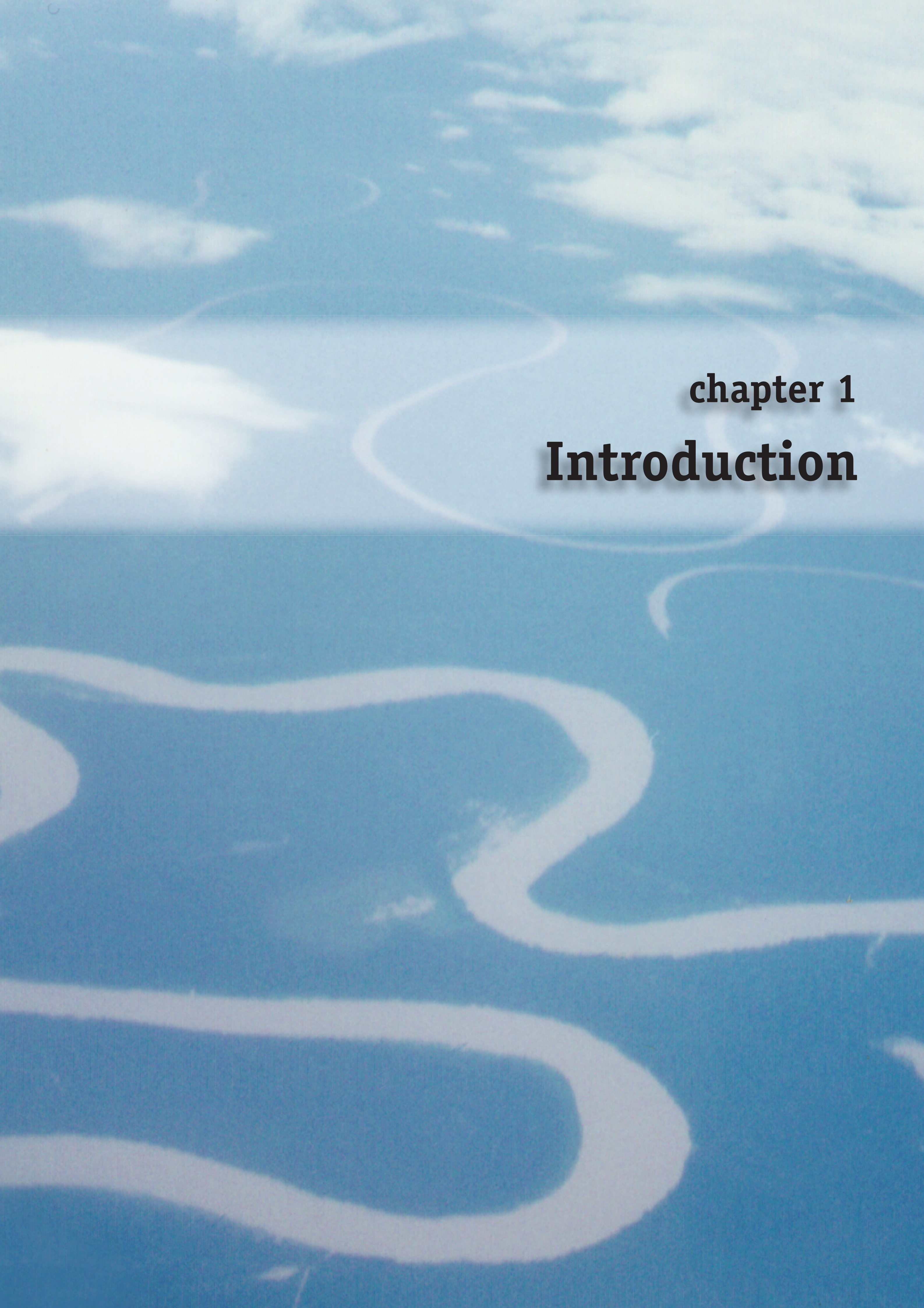
CHAPTER 4

chapter 1

Introduction



CHAPTER 1

1.1 HYDROELECTRIC POWER

Over the years Brazil has adopted a policy of harnessing its hydroelectric potential, which has made it self-sufficient in electricity by drawing on a low-cost renewable source of energy and developing national technology.

As the electricity industry is a major user of water, it has the responsibility and duty to plan the use of this resource as an input for the cost-effective, optimized production of electricity, alongside the other users of water.

This is a topic that is currently legislated and regulated by several government entities and allows for the involvement of different agents. Appendix 1 presents an overview of the issues concerning hydroelectric power and its potential from a Brazilian and international perspective, as well as the institutional context in which hydropower inventory studies are undertaken in Brazil. It also explains certain key issues involved in the relationship between institutions and the legal processes and procedures needed for hydropower generation at this stage of planning. Appendix 1 is organized into the following topics:

- Hydroelectric Power around the World

This topic presents an assessment of key issues affecting hydroelectric power in the world today. It addresses topics raised by institutions such as the International Hydropower Association, the World Commission on Dams, the World Research Institute and the International Rivers Network, providing an overview of the arguments for and against impoundment. The issues concerning dams around the world are presented in brief, illustrated by a quantitative and qualitative analysis of existing dams designed to show that the issue is not restricted to energy generation alone. In order to inform those people who may be in charge of undertaking Hydropower Inventory Studies about any potential areas of conflict, the main arguments against hydroelectric power on a world level are also presented. There is also a discussion of the impact of large-scale dams, concluding that the issues at stake are not just a function of the scale of a project.

Some of the international data on hydropower are presented, which show its contribution to the international energy grid. Data are also provided on the world's largest electricity and hydropower producers. The contrasts between the electricity systems in different countries are highlighted, giving special attention to the exceptional make-up of the Brazilian grid. A general picture is also given of the countries with the potential to develop hydroelectric power. A description is provided of some of the features that set hydroelectric power apart from other sources of energy, which can, if developed appropriately, make hydropower plants even more competitive.

- Hydroelectric Power in Brazil

Brazil's hydroelectric potential and the feasibility of its being harnessed are illustrated in an overview of the different hydroelectric projects classified according to their stage of development, location, scale, river basin and impounded area, drawing on data from SIPOT[1]. A historical view of hydropower projects in the country is given, covering the introduction of the system, the creation of the companies and the overall growth of the electricity industry.

The current state of the hydropower system in the country is also presented, as well as its prospects for growth and the role of the interconnected transmission system, which links up different river basins in Brazil and is designed to optimize the country's water resources by integrating them through a wide variety of dispatch configurations. This is particularly important for ensuring the feasibility of hydropower projects in the north of Brazil.

1 SIPOT – Information System on the Hydroelectric Potential of Brazil, Eletrobrás. *sipot@eletrobras.com*.

A brief analysis is also made of the impact social and environmental aspects have on hydropower generation in Brazil, based on the Environmental Master Plan (II PDMA) and several changes to the relevant legislation.

- Hydroelectric Power Projects and the Institutional Organization of the Electricity Sector

  This section shows how legislation pertaining to the electricity sector has changed since the first reforms were made in 1995, when acts 9.074 and 8.987 were passed to legislate the granting of electricity contracts.

- Institutional Organization

  This shows the different entities and agents and their respective powers.

- Legislation Pertaining to Harnessing Hydroelectric Potential

  In 1995, Act 8.987 brought in major changes to the way concessions were granted and authorization was given for the rendering of public services. It regulates article 175 of the Federal Constitution of 1988, which states that concessions for public services and works and authorization for public services must be subject to public tender. This item discusses the implications of this change of focus, covering the following topics:

- Concessions for projects in the electricity industry;
- Resolutions for Hydropower Inventory Studies;
- Financial compensation;
- National Policy for Water Resources;
- National Policy for the Environment.

1.2 OBJECTIVE

The aim of this Manual for Hydropower Inventory Studies is to present a set of criteria, procedures and instructions for undertaking Hydropower Inventory Studies of river basins.

The hydroelectric potential of a river basin, as defined in this Manual, corresponds to the potential that can be technically, economically, socially and environmentally harnessed, taking into account the multiple water uses in the river basin under study.

The techniques and methods used for this purpose have been developed and refined in Brazil since the early 1960s. The methods described in this Manual are the result of the experience acquired in hydropower developments in Brazil.

This new edition of the Manual for Hydropower Inventory Studies is based on the 1997[2] edition and includes some additional items:

- methods and criteria for social and environmental studies in compliance with Integrated Environmental Assessments (IEA);
- a chapter dedicated to specific analyses concerning the IEA of the cascade selected in the Final Studies;
- methods and criteria for considering, in the Final Studies, the potential positive socioenvironmental impacts arising from the introduction of the hydropower plants;
- updated methods and criteria for building up the scenario for multiple water use in the light of the National Plan for Water Resources;
- updated procedures for appraising, quantifying and calculating the costs involved;
- updated unit prices;
- altered multi-objective methodology for selecting the cascade in the Final Studies so that positive socioenvironmental impacts can be included;
- updated version of the SINV[3] program, which is used for the energy and socioenvironmental studies in compliance with the procedures in this manual.

2 Manual de Inventário Hidroelétrico de Bacias Hidrográficas, Eletrobrás, 1997.

3 SINV – System used for the socio-environmental and energy studies and for the studies used to select the cascades in hydropower inventory studies, CEPEL, 2007.

1.3 EXPANSION PLANS FOR THE BRAZILIAN POWER SYSTEM

Planning the Brazilian energy sector involves taking decisions about the expansion of the production/generation system, the transportation system, and the storage of energy (at hydroelectric power plants and in fuel stocks). It is by this process that supply is adjusted to the demand forecast in different analyses taking into consideration the country's different sources of energy and fuel imports and exports. Overall, it requires several kinds of studies with different aims and time-frames depending on the specific aim.

In view of the nature of Brazil's electricity system and the rest of the energy sector, its expansion is planned in three distinct stages involving different kinds of studies as set out below.

- **Long-Term Studies** covering up to 30 years, involving analyses of different strategies for developing the country's different energy systems and the future energy supply mix. The priorities are also established for the country's technological and industrial development, as well as a program of engineering studies designed to assess the technical, economic and socioenvironmental feasibility of different energy developments. The findings include recommendations for river basins to be prioritized in Hydropower Inventory Studies, guidelines for short-term studies, and estimates of marginal costs for long-term expansion plans.

  The main variables in these studies are: the prospects for the domestic economy and the respective demands of the energy market; the availability of primary energy sources and the options for importing energy and energy sources; trends in technology development, especially concerning renewable energy sources; the environmental impacts of projects; and conservation and efficiency projects for energy use and production. These studies are carried out every four to five years on average, and their findings serve as an input for energy strategies and policies, which are consolidated in the studies for the National Energy Matrix and the National Energy Plan.

- **Medium-Term Studies** for the electricity sector covering up to 15 years, which include analyses of the different options for expanding power generation and transmission adjusted to the market's energy requirements. These studies are developed for specific planning cases, such as the studies for and design of interconnections between the systems in different regions of the country, the introduction of hydropower from the Amazon to the National Interconnected System, and the calculation of marginal costs for designing hydropower plants. The studies meet the specific expansion needs of the national electricity system.

  The main variables in these studies are the same as those for the short-term studies below.

- **Short-Term Studies** covering at least the next ten years, which present decisions concerning the physical expansion of energy supply, pinpointing specific projects and their time-frame for introduction, with analyses of supply conditions for the market. The physical targets and the expansion programs are established with a view to future auctions for the purchase of energy from new generation developments and future auctions for new transmission facilities. The technical, economic and socioenvironmental feasibility studies required for new generation projects are identified.

  Other sources of energy are also analyzed, taking into consideration the current priority of developing the plan with a focus on energy integration, especially for oil and oil products, liquid fuels, natural gas, renewable energy sources and coal.

  The main variables in these studies are: the demands of the energy market; the criteria for assuring supply and minimizing investment costs; the time-frame for new developments, taking into consideration the engineering and environmental studies involved. These studies are carried out every year and are consolidated in the Ten-Year Expansion Plan.

By this means, the expansion of Brazil's energy system is planned by consolidating these studies on two distinct levels: the Ten-Year Expansion Plan and the National Energy Plan, the latter of which is strategic in nature, providing energy guidelines and policies and setting the groundwork for shaping the Ten-Year Plan.

The quality of the expansion plans depends on the coherence and homogeneity of the respective Hydropower Inventory Studies and the compatibility between them, even when they are carried out by different technical teams.

1.4 STUDIES AND DESIGNS REQUIRED FOR NEW HYDROELECTRIC PROJECTS

The development of new hydroelectric power plants involves five distinct stages (see Figure 1.4.01).

The first of these is the **Estimate of Hydroelectric Potential**, when a preliminary analysis is made of the river basin, looking into its topographical, hydrological, geological and environmental features to identify whether it could be harnessed for power generation. This analysis, which is based purely upon pre-existing data, is office-based and provides a first indication of the potential and estimated cost of developing the river basin, as well as prioritizing topics for the next stage.

The second stage is the **Hydropower Inventory Study**, when different cascades comprising a number of individual projects are devised and analyzed for the river basin. These are then compared to select the one that gives the best relative development cost, energy benefits and socioenvironmental impacts. The analysis draws on secondary data, which are supplemented by field data and involves basic cartographic, hydrometeorological, energy, geological, geotechnical, socioenvironmental and multiple water use studies. Based on this analysis, a number of possible cascades are identified, including their key characteristics, cost/benefit ratios and socioenvironmental indexes. As part of the Hydropower Inventory Study, the projects from the final selected cascade are submitted to an Integrated Environmental Assessment (IEA), which is needed for environmental licensing purposes. These projects are then included in the national list of approved projects, and are available for inclusion in energy expansion plans.

Next is the **Feasibility** stage, which includes more detailed studies to assess aspects of technical, energy, economic and socioenvironmental feasibility, the results of which serve to identify the optimal projects, which are then made available for auction. These studies include field work and calculations of the basic dimensions of the project and reservoir, its area of influence and the local and regional infrastructure needed for it to be built. These also include multiple water use analyses and assessments of socioenvironmental interference. The results of these studies are compiled in an Environmental Impact Assessment (EIA) and an Environmental Impact Report (RIMA) for each project, which are submitted to the respective environmental agencies as part of the application for a Preliminary License.

The next step in developing a project, once a bid process has taken place, is the **Basic Design**. The project analyzed in the feasibility studies is characterized in detail to identify more precisely its technical requirements, the technical specifications for the civil construction and the hydro and electromechanical equipment required, as well as the associated social and environmental programs. A Basic Environmental Plan (Projeto Básico Ambiental) must be prepared on the basis of the recommendations in the EIA before an Installation License can be issued and the construction work contracted out.

The final stage is the **Executive Design**, in which the civil construction and the hydro and electromechanical equipment are defined in detail so that the construction work can be carried out and the equipment assembled. This stage also includes all the steps concerning the impoundment, including the socioenvironmental programs to prevent, minimize or offset any social or environmental damage, after which an Operating License can be applied for.

Once the plant has been built, the reservoir is filled and the plant is commissioned. Once generation begins, the plant is monitored closely to adjust any issues that may be identified. The plant can only enter service once it has received an Operating License.

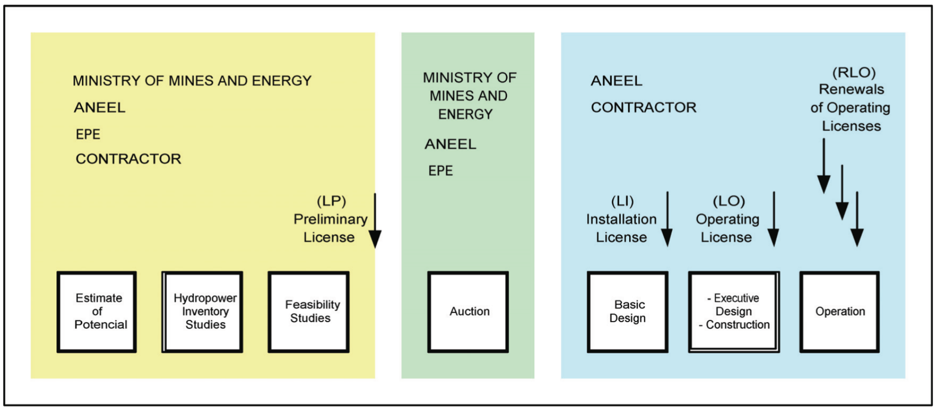


Figure 1.4.01 – Stages of introducing a new hydropower project

1.5 PHASES OF HYDROPOWER INVENTORY STUDIES

Hydropower Inventory Studies are carried out in four phases:

- Planning

Initially, it is necessary to plan and organize the study, listing the different studies that need to be undertaken and estimating how long they will take and their cost. At the end, a management report is produced that contains the work plan. The flowchart in Figure 1.5.01 contains the activities involved at this stage.

- Preliminary Studies

Different cascades are proposed and analyzed for harnessing the hydroelectric potential, making a preliminary estimate of their generation potential and costs and any negative socioenvironmental impacts involved, all using secondary data. The aim of the Preliminary Studies is to select the most promising cascades from a socioenvironmental, energy and economic perspective, so that these can then be subject to more in-depth analysis at the next phase. The flow chart in Figure 1.5.02 shows the activities in this phase.

- Final Studies

The aim of the Final Studies is to establish what construction work and equipment would be required to fully harness the hydroelectric, socioenvironmental and economic potential of the basin.

The analysis in this phase is in-depth, and takes into account the positive socioenvironmental impacts. There are also supplementary field studies undertaken for the projects included in the cascade options selected in the previous phase. The flow chart in Figure 1.5.03 shows the activities in this phase.

- Integrated Environmental Assessment of the final selected cascade

The aim of this phase is to supplement and consolidate the socioenvironmental studies for the cascade selected in the Final Studies, highlighting the cumulative and synergistic effects resulting from the negative and positive impacts brought about by the set of projects involved. It is in this phase that the socioenvironmental guidelines are drawn up so that the studies required for designing the projects and future socioenvironmental studies for the river basin can be undertaken, and for the purposes of environmental licensing for the developments. The flowchart in Figure 1.5.04 shows the activities in this phase.

Figure 1.5.01 – Flow Chart of the Planning Phase

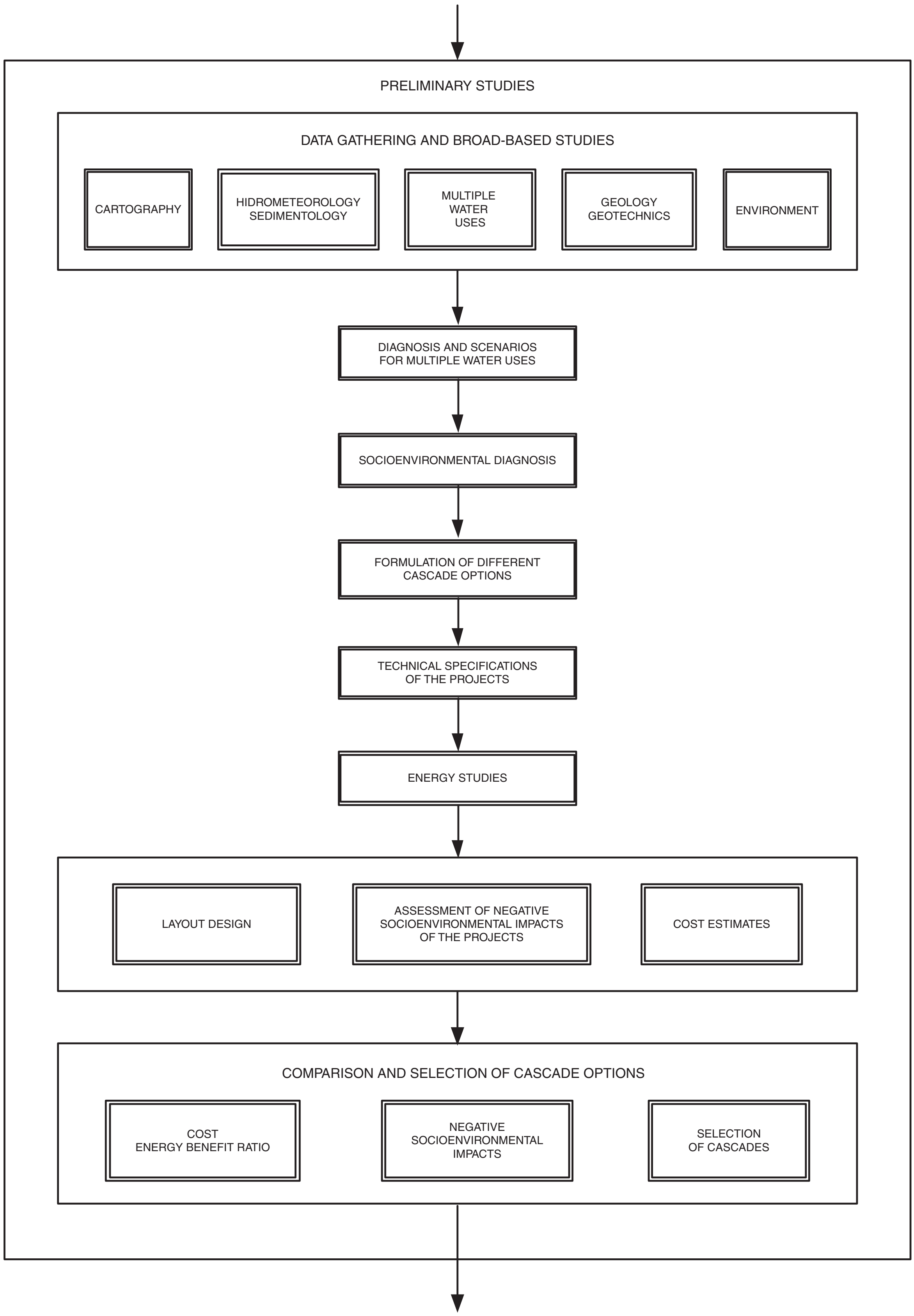


Figure 1.5.02 – Flow Chart of Preliminary Studies

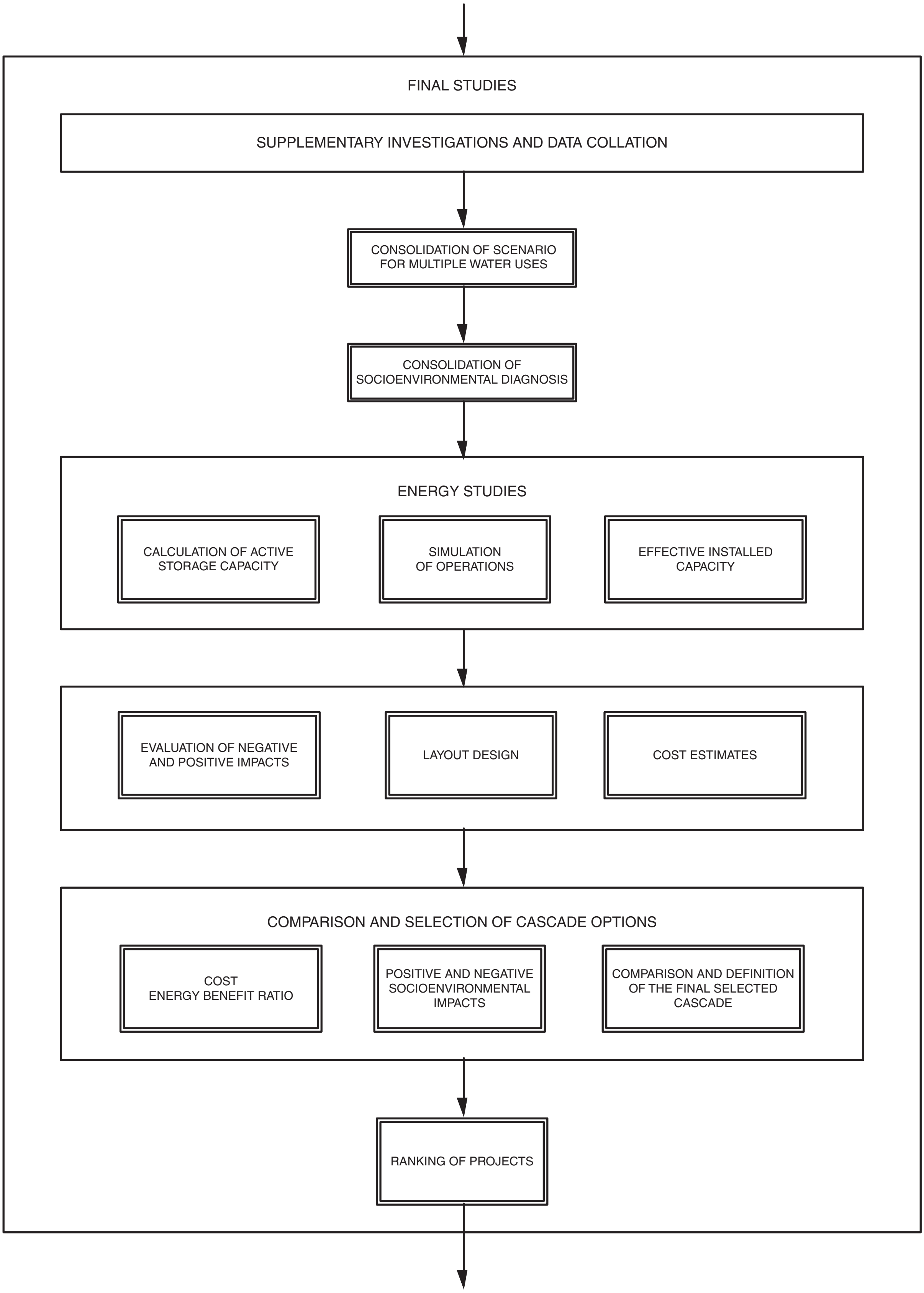


Figure 1.5.03 – Flow Chart of Final Studies

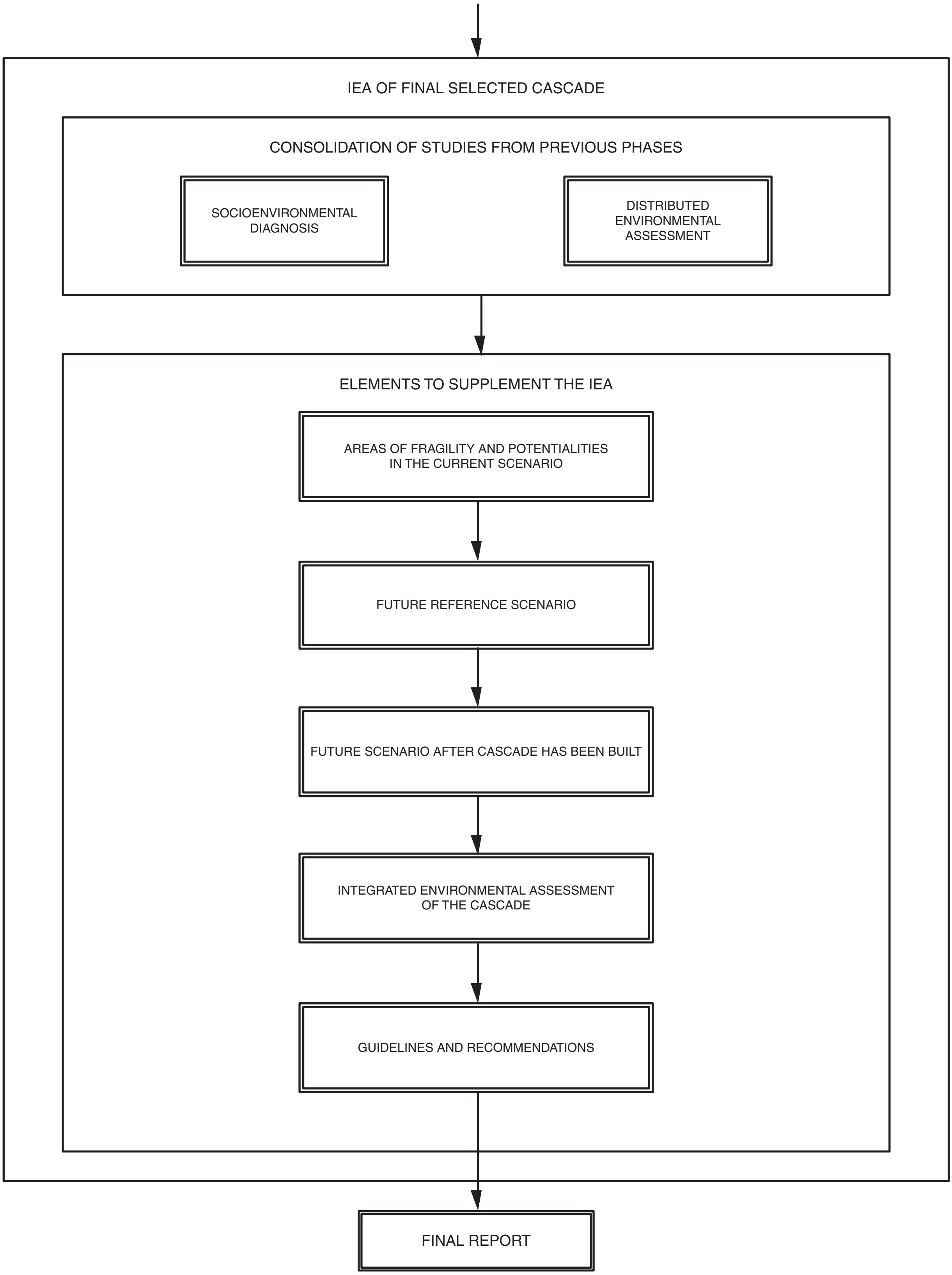


Figure 1.5.04 – Flow Chart for IEA of Final Selected Cascade

1.6 PROCEDURES FOR THE PREPARATION AND APPROVAL OF HYDROPOWER INVENTORY STUDIES

Table 1.6.01 shows the institutional and legal procedures required for preparing a Hydropower Inventory Study.

Table 1.6.01 – Institutional and Legal Procedures for preparing a Hydropower Inventory Study

	Activity	Entity	Regulation	Mandatory
1	Apply to have the Hydropower Inventory Study registered by ANEEL	Main Stakeholder	ANEEL Resolution 393/98 articles 6 and 9	Yes
2	Register the Hydropower Inventory Study	ANEEL	ANEEL Resolution 393/98 art. 6	Yes
3	Notify the Ministry of Mines and Energy (MME) about the beginning of the studies	Main Stakeholder		Yes
4	Notify the other ministries about the beginning of the Hydropower Inventory Study in the river basin	Ministry of Mines and Energy (MME)		Yes
4	Register *Anotação de Responsabilidade Técnica* (ART) with CREA	Professional Responsible	CONFEA Resolution 425/98	Yes
5	Notify ANEEL if the Inventory Studies are abandoned	Main Stakeholder	ANEEL Resolution 393/98 Art 11	Yes
6	Authorize field studies	ANEEL / FUNAI / INPRA / Brazilian Navy / state departments and others		Yes
9	Notify river basin committees, ANA or state water resource and environmental administrators (when applicable) of the beginning of studies	Main Stakeholder	Act 9433/97 and ANEEL Resolution 393/98 Art 13	Yes
10	Carry out the Hydropower Inventory Study in accordance with the terms of this Manual and use the SINV system for energy and socioenvironmental studies and for selecting the cascade options	Main Stakeholder		Yes
10.1	Planning			
	Establish procedures for monitoring the studies	MME / EPE	Act 10.847/2004 Art. 4	Recommended
10.2	Preliminary Studies			
	Submit progress report to MME / EPE and ANEEL	Main Stakeholder	Act 10.847/2004 Art. 4 and ANEEL Resolution 393/98 Art. 10	Yes
	Technical meeting to be called by MME for presentation of the results of this stage	MME		Yes
10.3	Final Studies			
	Send the water resource administration body (ANA or state entity) the information about the consumption of water for other purposes and streamflow information for each project site.	Main Stakeholder		Recommended
	Send ANEEL the streamflow information for the project sites for the final cascade selected using the format described in Annex G.	Main Stakeholder		Yes
	Public meeting to be called by MME to present the findings concerning the final cascade selected and the IEA, and its guidelines and recommendations	MME		Yes
11	Submit studies to ANEEL	Main Stakeholder		Yes
12	Notify receipt of studies	ANEEL	ANEEL Res. 393/98 Art. 14	Yes
14	Give approval of studies	ANEEL	ANEEL Res. 393/98 Art. 14	Yes

1.7 SCOPE OF THE MANUAL

This manual provides guidelines for the studies needed for Hydropower Inventory Studies of river basins anywhere in Brazil. In each real situation, the features of the basin in question should be analyzed by adapting the basic methodology to each case as efficiently and pragmatically as possible, taking into account specific features in each case and consulting the government entities responsible for approving the studies.

It is recommended that an integrated study be made of each river basin, identifying the potential for regulating its streamflow over several years so as to ensure that its economic-energy efficiency is optimized.

Overall, this Manual can only be used for projects that are greater in size than small hydros (over 30 MW). It should be noted that in river basins which could contain some plants over 30 MW as well as others of a smaller scale, these should all be included in the Hydropower Inventory Study.

When Hydropower Inventory Studies are reviewed and/or updated, all available information should be drawn on, giving special attention to the latest socioenvironmental studies and the expected cost of the projects, using the methods set forth in this Manual.

1.8 CONTENTS OF THE MANUAL

This Manual contains seven chapters, one appendix and seven annexes. In each chapter, the references are included as footnotes. All the references are also listed at the end of each chapter and there is also a selected bibliography.

Chapter 1 consists of this introduction and is supplemented by Appendix 1. Chapter 2 sets out the basic criteria, while Chapter 3 gives information on planning Inventory Studies. Chapter 4 contains the procedures for Preliminary Studies, and Chapter 5 contains the procedures for Final Studies. Chapter 6 provides the supplementary procedures for IEA's, and Chapter 7 provides a model for drafting the final report.

Annexes A, B, C, E and G contain lists of the spreadsheets and graphs that are provided in digital format. Annex D sets out the features of the SINV system, version 6.0. Annex F gives a summarized method for IEA's used in three previous Inventory Studies.

1.9 BIBLIOGRAPHY

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chapter 2

Basic Criteria

CHAPTER 2

The basic criterion for all Inventory Studies is to maximize economic-energy efficiency, while minimizing negative socioenvironmental impacts and also taking into account any positive socioenvironmental impacts to be gleaned by building hydroelectric projects in the river basin. When these studies are carried out, energy, economic, multiple water use, socioenvironmental and engineering technique criteria must therefore be set.

The energy and economic criteria are directly related to optimizing the hydroelectric potential of the river basin in question, while respecting the portions of head and flow required for other water uses. These criteria will have a direct impact on the goal of maximizing economic-energy efficiency, which, in the comparison of different options, is translated into their respective cost / energy benefit ratios (R$/MWh).

The criteria for multiple water uses involve taking into consideration the other uses of the water resources in the basin, minimizing any conflict and ensuring the efficient use of the water by estimating the amount of head and flow available for electricity generation. As such, a diagnostic study should be prepared during the Preliminary Studies to serve as a source of data for designing a scenario of future multiple water uses for the river basin, which will be used in the Final Studies.

The engineering criteria refer to the use of established solutions in devising the layout of the projects. Users of this manual will have access to a set of information and procedures which will ultimately provide the capacity for the structures to be dimensioned and costs and quantities estimated both simply and quickly. The costs quoted for civil construction and equipment are the mean of the unit prices being charged recently in Brazil. The equipment includes the latest electromechanical technologies.

The criteria of a socioenvironmental nature concern the analysis of the negative and positive socioenvironmental impacts which each cascade option would have on the area under study. The analysis of negative socioenvironmental impacts is necessary to ensure that such impacts are minimized and is instrumental in the selection of the best cascade. The analysis of positive socioenvironmental impacts is a factor that influences the final selection of the best cascade.

The socioenvironmental criteria should be taken into consideration when the different cascades and projects are devised, so that the different cascades can be compared and selected in view of two indexes which express the intensity of the negative and positive impacts individually on the area under study.

As this is a study that is used for sector planning, the main benefit is the generation of electricity, which is achieved by maximizing the economic-energy efficiency of the river basin. However, the analysis of potential positive impacts is also included with a view to elucidating how social and economic development both locally and regionally could be boosted by the introduction of the hydropower projects. This analysis feeds into the subsequent stages of the planning cycle for these projects (Feasibility Studies, Basic Design, etc.) and identifies what coordination needs to be established between different institutions for this potential to be realized.

The positive socioenvironmental impacts are only included in the analysis for the purposes of making the final selection of one cascade in the Final Studies. In the Preliminary Studies, when a number of cascades are shortlisted to be analyzed in greater detail in the Final Studies, the focus should still be on maximizing economic-energy efficiency and minimizing negative socioenvironmental impacts, and any that fail to meet either or both of these criteria should be eliminated.

It should be noted that in order to take full account of these four groups of criteria, the aspects that are included in the cost/benefit indexes as construction costs (control and compensation costs) should not be included in the negative socioenvironmental impact index. In other words, this index should only cover externalities brought about by the negative impacts identified in the study area, such as changes to the way of life of the local people or lost natural habitats.

For the assessments of socioenvironmental criteria and multiple water uses, the local social and economic development trends should be taken into account, as set out below:

- for evaluating multiple water uses in the river basin, forecasts should be prepared on a time frame that is compatible with the National Plan for Water Resources. The future scenario should be comprised of the criteria set out in item 2.2.
- the socioenvironmental assessments undertaken in the Preliminary and Final Studies must take into account the current socioenvironmental situation in the region under study and its development trends as identified in the socioenvironmental diagnostic study, including any regional development policies, plans or programs, be they by federal, state or municipal entities, as well as all water resource plans and any other sector plans or programs.
- in the Integrated Environmental Assessment (IEA) of the cascade selected in the Final Studies, scenarios must be prepared for the future development of the region under study following the guidelines set out in chapter 6 and in compliance with the recommendations in this chapter.

2.1 ENERGY CRITERIA

The evaluation of the energy potential of the cascades under study should take into account the following basic criteria:

2.1.1 Reference System

A reference system is defined as the set of electricity generation plants which the energy benefits of the cascade options under examination should be quantified against. The reference system should be the power system that the projects under study would be connected to and must be defined by the concession-granting authority.

2.1.2 Critical Period

The critical period of a reference system is the period of time from the streamflow records when the reservoirs of the system were fully used, without any deficit, at the beginning of which time they were full and at the end they were totally depleted, without their replenishing completely during the period in question. The critical period to be adopted in the Inventory Studies will be defined by the concession-granting authority. In 2007, when this manual was published, the critical period adopted for the National Interconnected System ran from June 1949 to November 1956 (90 months).

2.1.3 Energy Benefits

When one or a group of hydroelectric power plants are integrated into a system, this brings about three kinds of energy benefits: firm energy, secondary energy and peaking capability.

A system's **firm energy** is the greatest load the system can provide without any deficits occurring under the worst hydrological conditions recorded for the series of natural streamflows.

Secondary energy is the energy that is available only during favorable hydrological periods, and can be used to meet the needs of consumers with intermittent load requirements or to substitute thermal power generation, bringing about fuel savings.

Peaking capability is the maximum amount of energy a system can produce during peak demand periods. It is directly correlated to the plants' installed capacity, so when calculating it, one should also take into account all losses caused by reduced head when reservoirs are depleted and/or the level of the tailrace is high, as well as scheduled and unscheduled shutdowns.

In Inventory Studies, firm energy is the most important variable when analyzing the benefits of any given cascade. In this manual, secondary energy is not taken into consideration at this stage, basically because of the great uncertainty as to its effective use. However, it may be taken into account if the concession-granting authority deems it desirable, in which case the authority will be responsible for determining how it should be calculated. By the same token, in isolated hydrothermal systems, secondary energy may be a decisive factor in correctly dimensioning a hydroelectric project.

Peaking capacity benefits are included indirectly. All projects are pre-dimensioned to have a common reference capability factor (item 2.1.4), which allows the respective energy benefits to be compared consistently from the perspective of the corresponding peaking capacity benefits.

Firm Energy

In Inventory Studies, the firm energy from one hydropower plant or group of plants is the mean energy generated by the plant or group of plants obtained in simulations of the operation of the system during the critical period of the reference system, as described in item 5.3.1. In the Preliminary Studies, rough estimates can be made based on the formulas given in items 4.6.1 and 4.6.2.

Firm Energy Contribution

The firm energy contribution from one hydropower plant or group of plants is the extra firm energy generated when this project or group of projects is added to the others in the river basin. Generally, this increase is the same as or greater than the firm energy of the projects analyzed, since the addition of new projects can sometimes cause the contribution by other plants in the same river basin to rise because of the effects on their regulation reservoirs. Should the water level in the tailrace canal of the project in question be lower than the water level in the reservoir immediately downstream, there may be energy losses for those immediately upstream.

As with firm energy, the firm energy contribution can be calculated by simulating the operation of the system during the critical period of the reference system, as described in item 5.3.1. In the Preliminary Studies, rough estimates can be made based on the formulas given in item 4.6.3.

Minimum Firm Energy Contribution

Before construction work begins, a minimum firm energy contribution must be set for any new project, below which level it will not be included in any cascade option. This minimum will vary according to the region and the reference system, and should be set by the concession-granting authority.

2.1.4 Energy Dimensioning

In the energy dimensioning of projects, the maximum reservoir drawdown must be determined, as must the installed capacity and the reference head. The installed capacity is the combined capacity of all the generators. The reference head is the net head at which the turbines, with their distributors fully open, will supply the installed capacity. Generally speaking, the optimal values for a given project will depend on the cascade it is part of. However, in Inventory Studies there is no need to consider the outcome of the different energy dimensioning activities for a given project in every cascade it is part of; it is enough to carry out the energy dimensioning of the project for the cascade where it will have the highest installed capacity. This approximation can be used when there is no great difference between the different energy dimensions calculated. However, when the energy dimensioning of a given project for two different cascade options gives rise to quite different results, it is recommended that two projects be created and analyzed separately for the same dam site.

Maximum Drawdown

When the hydroelectric potential of a river basin is being harnessed, every effort should be made to ensure that the projects are designed to have some storage capacity so that energy can be generated during drier periods by harnessing the water stored during rainy periods. However, when reservoirs are drawn down too low, there is a loss of head and a resulting drop in generating capacity. A value for maximum drawdown or live storage should therefore be set for each project by a process of optimization, based on the firm energy contribution by the project when it is integrated into the cascade taken as the basis for the energy dimensioning. Even so, special attention should be paid to the reservoir replenishment time, which should not exceed 36 months after the end of the critical period. The procedure for calculating this in the Preliminary and Final Studies is described in items 4.6.6 and 5.3.4, respectively.

Reference Head for Inventory Studies

For the purposes of Inventory Studies, the reference head is taken as the average net head of the project, which is defined as the difference between the mean level in the reservoir and the water level in the tailrace at a flow rate that is 10% greater than the mean flow during the critical period for the reference system, or the maximum normal water level of the reservoir downstream, when this is greater than the previous level, minus hydraulic losses (item 4.6.1). When using energy simulation models, the water level in the tailrace for calculating the reference head is the mean water level in the tailrace of the project throughout the critical period of the reference system.

Reference Capacity Factor (F_k)

When analyzing peaking capacity benefits, the energy calculations for the projects make use of the same reference capacity factor so that the comparisons between the energy benefits of different projects are consistent. This factor represents the ratio between the sum of the firm energy from the projects in the reference system and their respective installed capacity.

The capacity factor can be calculated by the following expression:

$$F_k = f_c*(1-P_p)*(1-r)/(1-P_e) \qquad (2.1.4.01)$$

Where:

f_c	load factor of the market to be supplied by the reference system;
P_p	peak power loss factor in the transmission system;
P_e	power loss factor in the transmission system;
r	generation reserve factor of 15%.

In the absence of more reliable information, it is recommended to use F_k = 55%.

Installed Capacity (P_i)

This is obtained by applying the reference capacity factor to the firm energy to be obtained from the project when it is integrated into the cascade option taken as the basis for the energy dimensioning, as shown below:

$$P_i = \frac{Ef_i}{F_k} \qquad (2.1.4.02)$$

Mean Net Head

When undertaking energy dimensioning for hydropower projects, generation of the installed capacity should be assured with the turbine distributor fully open and with the mean net head, corresponding to the level of reservoir drawdown representative of the drawdown level during the operation of the project during the critical period of the reference system. The mean net head is the mean of the net heads calculated for the project under study in simulated operations of the system throughout the critical period of the reference system, as per item 5.3.1, assuming the project is part of the cascade taken as the basis for the energy dimensioning. Estimates can be used in the Preliminary Studies as shown in the formulas in item 4.6.1.

2.2 CRITERIA FOR MULTIPLE WATER USES

By the terms of Act 9.433 of January 8th 1997, the National Policy for Water Resources is based, among other criteria, on the principle that the management of all water resources should always provide for multiple uses of the waters. Although Hydropower Inventory Studies do not go into the same the breadth or detail as river basin plans, in the assessment and comparison of the different cascades in the Final Studies they must still take into account multiple water uses and the interaction between these and the proposed hydropower projects so as to minimize potential conflicts and ensure the most efficient use of the resources available. This being the case, the current uses of the waters should be identified and a long-term scenario should be designed for their multiple uses in the river basin. This is the scenario used for the assessments of the energy benefits and the positive and negative socioenvironmental impacts of the different cascades. The assumptions used to build up the scenario for multiple water uses are also applied to the scenarios for the integrated environmental assessment of the cascade that is selected (chapter 6).

2.2.1 Diagnosis and Scenario for Multiple Water Uses in the River Basin

In order to design a scenario for long-term multiple water uses that is in line with the National Policy for Water Resources, a diagnostic study must first be made of the current uses of the waters in the river basin under study, drawing on pre-existing data and information.

Having done this, a scenario is designed that is compatible with the time frame of the National Policy for Water Resources. One of the cornerstones of this future scenario should be whether the projected uses of the waters in the river basin are actually reasonable. Likewise, the analysis of existing sector plans must be underpinned by the principle of feasibility. Evidently, as the design of a future scenario has a strong subjective element, there must be clear motivations and justifications used throughout, especially when existing plans are not followed in the full.

As such, the primary source of information should be the data gathered in the Planning stage (item 3.1.3), especially: a) estimates of water consumption prepared by the National Water Agency (ANA) and by the state entities with responsibility for water resources; b) the National Plan for Water Resources, state plans for water resources and river basin plans; c) integrated development master plans, irrigation programs, navigation studies, flood control studies, and water supply studies for human, animal and industrial use; and d) non-consumptive uses, such as tourism, leisure, preservation of the landscape and environmental conservation areas; all with a view to obtaining a realistic, balanced portrayal of the potential for development in the water basin.

2.3 SOCIOENVIRONMENTAL CRITERIA

The criteria adopted for the socioenvironmental studies adapt the methodological instruments and widely-used procedures for environmental impact assessments to the requirements and specificities of Hydropower Inventory Studies. These are supplemented by the procedures used in Integrated Environmental Assessments. Together, these criteria (presented below) serve to systematize the information gathered about the main socioenvironmental issues, identifying areas of environmental fragility and socioeconomic potentialities, as well as the main synergistic and cumulative effects resulting from the introduction of the cascade selected. They should also influence the design of the projects and the formulation of the cascade options, while also supplying information for the cost estimate of the construction work and providing the means for the cascades to be compared and selected using a multi-objective approach.

The results of the socioenvironmental studies should indicate guidelines for future studies, environmental licensing, the building of the hydroelectric projects and, in particular, the issues to be focused on during the Feasibility stage.

2.3.1 Study Area

The study area should be defined in such a way as to allow for an analysis of the socioenvironmental processes inherent to the river basin being studied, which should be included in its entirety. It should also make it possible to identify and assess the impact processes resulting from the harnessing of its hydroelectric potential.

The borders of the study area should be defined at the Planning stage, and may be adjusted as the work proceeds. In this process, the specific features of the socioenvironmental processes being addressed should be respected, which often spill out beyond the physiographic limits.

2.3.2 Environmental System

An environmental system is the set of elements in the study area, including their features or qualities, the functions they play in the different processes and how they interact.

When analyzing an environmental system, the physical, biotic, social, cultural, economic and political processes should be taken into account, as well as the way these are interrelated and how they impact on the physical space, all of which requires a multi- and inter-disciplinary approach.

2.3.3 Synthesis Components

An analytical structure is used to represent the environmental system, comprising six elements, called **synthesis components**: aquatic ecosystems (and water resources), territorial ecosystems, ways of life, territorial organization, regional economy and indigenous peoples / traditional communities.

These **synthesis components** are structured according to the interactions amongst different elements from the environmental system. The assumptions behind their conceptual definition and the structuring of their analytical content are:

- to develop an understanding of the integral nature of the processes by which the socioenvironmental elements interact;
- to elucidate the most important issues arising from the interactions between the projects/cascades and the study area; and
- to enhance the capacity to select or differentiate between the different cascades under study.

The term "synthesis component" is intended to express a sense of interaction between the different environmental elements, or **characterization elements**, that comprise the synthesis component in question, giving an idea of the combined processes involved in the analysis. It should be noted that some characterization elements exist in more than one synthesis component, taking on different functions according to the processes inherent to each one. These elements are not grouped under the categories traditionally used in socioenvironmental studies (physical, biotic and human environments), but rather each synthesis component will represent a synthesis of characterization elements from these three categories.

Physical Processes and Features

Physical processes and features are an integral part of all the synthesis components. They have stable interrelations and features, and can be of a permanent or temporary nature. As such, these are the elements which ensure the continuation and interaction of the biological and human relations. As physical processes and features provide support and interconnectivity for the socioenvironmental processes, they are not regarded as a separate synthesis component, but are a basic element in the analyses of the six synthesis components adopted.

2.3.4 Sub-Areas

The studies undertaken at the diagnostic study stage should involve the spatial segmentation of the reference situation in each synthesis component in the study area into sub-units, here called **sub-areas**, by analyzing their similarities and differences. The sub-areas are continuous units of land which contain particular relationships and processes that mark them out from the others and which determine their relationship with the dynamics of the synthesis component in the study area as a whole. By using this methodology it is possible to identify the impacts of each project and how they interact with the synthesis components in each sub-area, while also providing a broad view of the set of impacts caused by the projects in each sub-area and those which extrapolate the borders of these areas.

2.3.5 Sensitivity, Fragility and Potentialities [1]

- The **sensitivity** of an area can be defined as "*the extent to which environmental systems and ecosystems react when they are affected by human action, in such a way that their qualities are altered.*" [2] In this manual, the term is used to identify and map out the most sensitive areas in the sub-areas of the river basin, expressing the integrity of the natural resources, the qualitative aspects of the landscape and the different socioeconomic situations in terms of their different degrees of sensitivity.

1 The definitions presented in the IEA's for Doce river basin, EPE/ Sondotécnica, 2007

2 Iara Verocai, 1990, Vocabulário Básico de Meio Ambiente, RJ, quoted in EPE/ Sondotécnica - AAI do Rio Doce, 2007

- The **fragility** of an area can be defined as "*the degree of susceptibility to damage in the face of given actions.*"[3] In this manual, the term is used to identify and map out impacts resulting from the introduction of hydropower projects in areas characterized as sensitive. This will be done as part of the IEA for the alternative selected.
- The **potentialities** of an area are associated to the existence of aspects that could benefit as a result of the introduction of hydropower projects, i.e. which represent opportunities for developing the socioeconomic conditions in the study area.

2.3.6 Assessment of Socioenvironmental Impacts

The aim of the assessment of socioenvironmental impacts is to help compare and select the cascade options and pinpoint the main socioenvironmental issues related to the individual projects and groups of projects.

In the analysis of the socioenvironmental impacts of any cascade, all the projects are considered jointly.

The criteria adopted for assessing the impacts and obtaining the socioenvironmental impact indexes for the purposes of the proposed studies are set out below:

- **Negative Socioenvironmental Impacts:** potentially unfavorable alterations caused by a project or group of projects on a synthesis component or on the socioenvironmental system, taking the current state of the study area and its development trends as a reference.

  The assessment of negative socioenvironmental impacts should include identifying any unfavorable alterations, any actions which could prevent the impacts from occurring either fully or partially (control), any actions that could mitigate the consequences of the impacts (mitigation) and any actions that may offset the impacts when reparation is not an option (compensation). These actions will be translated into costs, then incorporated into the overall cost of the project as socioenvironmental costs (item 2.7.3). The negative socioenvironmental impacts that cannot be controlled or the residual impacts when some control, compensation or mitigation is possible (degradation costs) will be evaluated and translated into a negative socioenvironmental impact index, which will be associated to the objective of minimizing negative socioenvironmental impacts.

- **Positive Socioenvironmental Impacts:** potentially favorable alterations caused by a hydroelectric project or group of projects, taking the current state of the study area and its development trends as a reference. This only relates to the socioeconomic impacts by which the hydropower projects may make a positive contribution to local or regional socioeconomic development, as described in item 5.4.2.

  Assessments of positive socioenvironmental impacts should include identifying favorable alterations, which are then translated into a positive socioenvironmental impact index. This is one factor that is considered in the final selection of one cascade, which takes place during the Final Studies.

  Any effect the environment may have on the planned projects should, whenever possible, be identified and incorporated into the engineering plans. However, these are not included in the socioenvironmental indexes.

- **Impact Process:** this is a set of alterations that could be triggered by a project or group of projects on the pre-existing natural and social processes in the study area; any given impact process is associated with environmental impacts that are interrelated on the level of the synthesis component.

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3 Angel Ramos, 1987, apud Iara Verocai, Vocabulário Básico de Meio Ambiente, RJ, 1990 – quoted in EPE/ Sondotécnica – AAI do Rio Doce, 2007

- **Impact Indicator:** this is an instrument that guides the assessment of the socioenvironmental impacts of a project or group of projects on a synthesis component, thereby establishing the focus of the analysis. Impact indicators are a combination of one or more variables, or **assessment elements**, used to characterize one or more effects to be expected at a site or sites in a river basin.

  Impact indicators are formulated by identifying the main impact processes and organizing the data into **assessment elements**. These correspond to the previously defined **characterization elements**.

  Descriptions are provided in items 4.3 and 4.8 of the characterization elements and assessment elements that have been devised to cover the vast majority of situations encountered across Brazil. However, the information to be used in each specific study should be selected and supplemented as necessary.

  The assessment elements selected to formulate the impact indicators in each case should be such that the cascades being compared can be effectively differentiated, without losing sight of the overall set of environmental processes involved. It is also important to strike a balance between quantitative and qualitative assessment elements, so that the analysis does not merely include the most readily quantifiable aspects. The assessment elements must also be capable of highlighting the processes that are cumulative and synergistic.

- **Assessment Criteria:** for each of the indicators, assessment criteria must be defined, which serve to determine the degree of impact involved.

- **Socioenvironmental Index:** this is the numerical figure that will represent the intensity of the socioenvironmental impact. It runs on a scale from **zero** (minimum impact) to **one** (maximum impact).

  The negative socioenvironmental index of a project or group of projects is the result of assessing the negative impacts on a synthesis component. Meanwhile, the negative socioenvironmental index of a cascade for the same synthesis component is obtained by combining the negative socioenvironmental indexes of the projects that make up the cascade, using the procedures described in items 4.8, 4.11.2, 5.4 and 5.8.2.

  The negative socioenvironmental index of a cascade represents its total negative impact on the study area, translating the extent to which it meets the objective of minimizing the socioenvironmental impacts. In order to obtain this index, all the negative socioenvironmental indexes for all the synthesis components should be combined, using the procedures and mechanisms set described in items 4.11.2 and 5.8.2.

  The positive socioenvironmental index for the socioenvironmental system under analysis is calculated per cascade option in the Final Studies, using the procedures and mechanisms described in item 5.4.2, which already take account of the cumulative and synergistic effects in the assessment. The positive socioenvironmental impact of a cascade is an aggregate of the indexes for each of the aspects, giving the total positive socioenvironmental impact of the cascade on the study area, as described in item 5.8.3. It should be used in the final selection of one cascade.

Cumulative and Synergistic Impacts

Cumulative and synergistic impacts are caused by the combination of one or more human actions with other past, present or future actions, triggering changes to the environment. Cumulative impacts result from the aggregate interaction of these alterations in a given space with time. Impacts are considered synergistic when the result of these interactions brings about an alteration to a given space that is different than the sum of the alterations.

Cumulative and synergistic impacts should focus primarily on permanent alterations, since temporary impacts will fade with time and their cumulative effect will be reduced.

2.3.7 Integrated Environmental Assessment

The cascade selected in the Final Studies should be the object of an Integrated Environmental Assessment (IEA) with the purpose of highlighting the cumulative and synergistic effects resulting from the negative and positive socioenvironmental impacts brought about by the group of projects it includes, as identified in the Preliminary Studies and taken into account in the selection of the cascade in the Final Studies. This assessment seeks to identify any areas of fragility or potentiality in the river basin under study, and should involve preparing future scenarios for the development of the river basin as described in item 6.5. As a result, guidelines should be prepared to be incorporated in future socioenvironmental studies for hydropower projects, which will serve to enhance future environmental licensing processes and provide recommendations for future projects.

2.4 CRITERIA FOR SELECTING DAM SITES

In the study to find sites suitable for dams, all the sections with rapids and waterfalls should be given special attention, as should any site where there is a marked narrowing of the river valley. Likewise, any restrictions imposed by physical, social and economic factors must also be addressed.

For each dam axis, the maximum water level the reservoir could reach should be calculated.

Maps and river profiles should be prepared for the sites, providing the basic information needed to formulate the different cascade options. The criteria to be adopted in each case will depend on the technical assessment of the topographical, geological, geotechnical, hydrological and socioenvironmental parameters.

2.5 CRITERIA FOR PROJECT LAYOUTS

Each site that is chosen for a hydropower plant has its own unique topographical, geological and hydrological features. As no two sites are the same, designing the right layout is a creative process that is normally the result of an iterative process, where several different cascades are devised, dimensioned and have their costs estimated in order to find the best one. By definition, the best layout for any hydropower project will be the one that manages to organize all the different elements in such a way that an adequate level of safety is assured, as well as ease of operation and maintenance, and all at the lowest overall cost. However, for the sake of standardization and whenever feasible, a set of basic criteria can be used to account for the majority of the solutions used in Brazil. The main recommendation for Inventory Studies is to adopt conservative, robust project layouts.

Project Elements

First, it is important to list all the elements that may be included in the layout of a hydropower project:

- **dam** – this is a structure made of earth or concrete that is built across the river valley from the abutment of one bank to the other with the purpose of raising the water level in the river to the normal maximum level of the reservoir;
- **dike** – usually made of earth, this structure closes off any saddles, so as to prevent any water flowing out of the reservoir;
- **river diversion system** – this is normally next to the dam and has the purpose of diverting the water from the river through canals, galleries, sluiceways, tunnels or even narrowing the river bed so as to allow the structures to be built on the river bed in the absence of water;
- **hydraulic conveyance facilities** – this includes headrace canals, intakes, conduits or tunnels, and possibly also surge tanks or forebays, pressure penstocks, a surface or underground powerhouse, and tailrace canals or tunnels. The purpose of the hydraulic conveyance facilities is to channel the water in such a way that mechanical energy can be transformed into electrical energy;
- **spillway structure** – this comprises an approach channel, a gated or ungated spillway, an energy dissipator and a downstream channel. As with the hydraulic conveyance facilities, the spillway structure can be near or far from the dam, depending on the specific features of the site under study;
- **bottom outlet** – structure containing gates or valves to release the waters downstream from the dam;
- **navigation system** – these are structures that allow cargos or passengers to navigate upstream or downstream, overcoming the difference in water level brought about by the impoundment;
- **fish passage system** – this is a structure that provides the means for aquatic fauna to migrate upstream or downstream, overcoming the difference in level brought about by the dam.

Dam

The location of the dam axis and the hydraulic conveyance facilities are crucial in finding the most economical layout for projects on rivers with marked differences in water levels, including waterfalls and rapids. In these cases, the dam axis should normally be upstream from the concentrated head as this will reduce the height of the dam and therefore its cost.

- **Types of Dam** – In Inventory Studies, the kind of dam to build depends greatly on the topography of the site and the geotechnical conditions of the foundations along the axis, not to mention the availability of building materials nearby. As this stage tends to involve auger boring, inspection pits and occasionally geophysical surveys, the geotechnical information available about the real state of the foundations is very limited. For this reason, projects involving traditional dams should be planned, rather than other kinds, such as concrete arch dams, multiple arch dams or buttress dams. Rockfill dams with concrete facing may be considered, providing there is no doubt as to the quality of the sound bedrock for the foundations of the plinth.

- **Geomechanical Parameters of the Foundations** – As the dam foundations are generally the single most important geological factor, they must have geomechanical parameters that are as good as or better than the parameters for the body of the dam. Based on this criterion, conventional or roller compacted concrete dams should not have soil or weathered bedrock foundations, but only foundations of good quality sound rock. Similarly, the foundations for rockfill dams can be made of weathered rock provided it can provide sufficient support.
- **Impermeability of the Foundations** – When homogeneous soil is used for earthfills, a cut-off trench must be built that reaches the impervious layer of the foundations. For rockfill dams with a central or inclined clay core, the cut-off must be a continuation of the core, going downwards until it reaches the top of the sound bedrock.
- **Materials Management** – Another important geotechnical factor is to strike a balance between the excavations required for the structures, the volumes of rockfill and earthfill and the materials to be used to make up the concrete. As this balance depends on the real construction schedule, there may be a need to keep intermediate stocks or to use extra borrow areas.

  These factors can raise costs and distort the original estimates. For this reason, it is best to aim for a flexible project layout and factor in losses of between 10% and 20% as a function of the use of material from excavations, depending on the size of the construction work. The need to move this material from one bank to the other should also be taken into account.
- **Construction Aspects** – Another important geotechnical factor has to do with the constraints on using different kinds of dam. For instance, earthfill or rockfill dams with a clay core should not be built in regions where it may rain all year round.

River Diversion

The river diversion scheme should be planned to have the capacity to discharge the design flood. The choice of diversion will depend on aspects of the project layout, such as the kind of dam, its height and length, and the kind of spillway, both of which depend on the particular topography of the site, as well as the estimated design flow and the geological features of the region.

Generally speaking, the choice of the diversion system boils down to the choice of the kind of structure to discharge the design flood (item 4.1.2) during the second phase of the diversion. The first phase of diversion, when necessary, comprises cofferdams used for the construction of the structure for the second phase.

The cheapest and most widely used method for diverting a river is to use **sluiceways**. Built through concrete dams or under the ogee crest of the spillway, sluiceways are normally used in **broad river valleys**. Normally, the river is first narrowed by a first-phase lengthwise cofferdam, which excludes water from one section of the river so that the dam or spillway, which will contain the sluiceways, can be built. Next, once the part of the structure needed to divert the river has been built, the river is diverted through the sluiceways, and the narrowed part of the river is cut off with second-phase cofferdams so that the construction of the dam and/or spillway can be completed in this newly dewatered section. Notwithstanding the complexity of the logistics, the use of a definitive structure for diverting the river and the shortening of the construction schedule more than offset the time and cost of building and removing the cofferdams.

Project layouts with abutment spillways normally involve diverting the river through **tunnels**, and are used in **narrow river valleys**. The tunnels, along with the approach and downstream channels, can normally be built without using cofferdams. Once they are built, the river bed is closed off and construction of the dam begins. When dams are planned that will involve diverting the river through tunnels, it is worth checking the economic feasibility of using them as bottom outlets, with a view to reducing the construction costs for the spillway system.

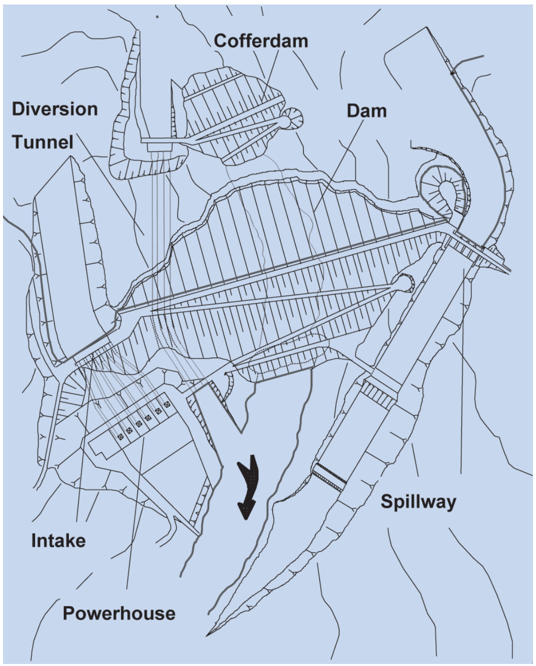

Figure 2.5.01: Typical Layout in a Narrow River Valley (Foz do Areia Hydropower Plant - Governador Bento Munhoz da Rocha, Iguaçu river, South of Brazil)

Generally speaking, galleries are recommended for areas with small design flows and when there is space on the lower part of the abutment for them to be built in the dry or when it is not geologically feasible to use tunnels.

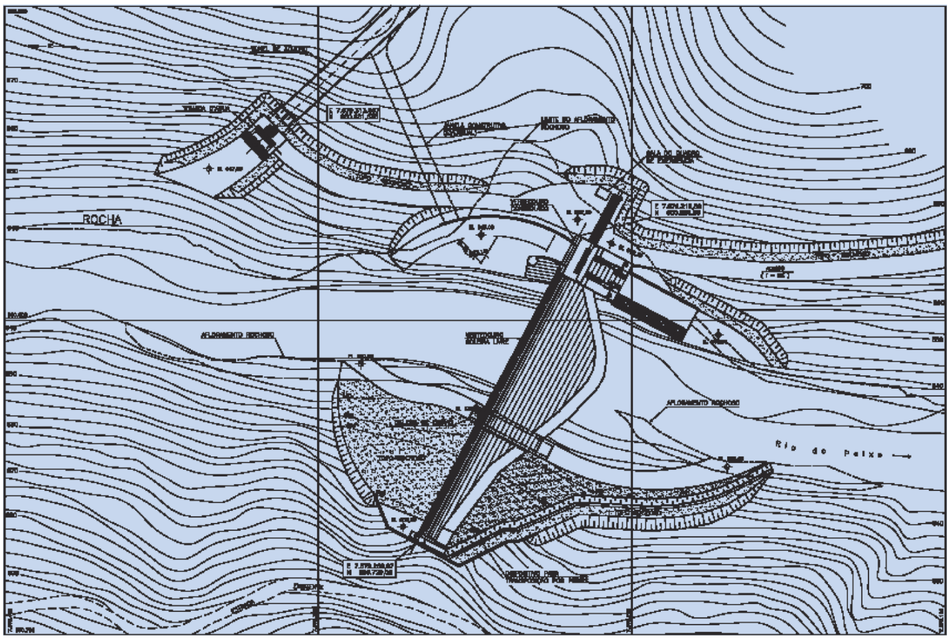

Figure 2.5.02: Design of Picada Hydropower Plant (Peixe River, Southeast of Brazil)

One particular kind of layout in **broad river valleys** with a low dam, when the ogee crest of the spillway is not high enough to have sluiceways built, is to do the diversion in two phases but with the second phase over the leveled or unleveled crest of the spillway.

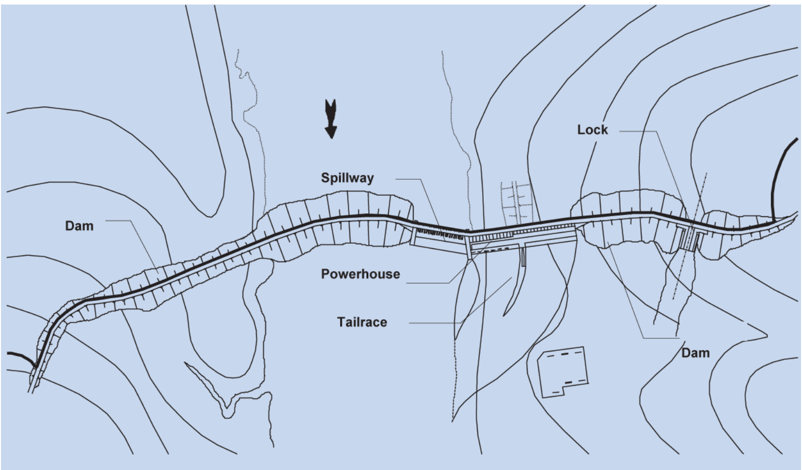

Figure 2.5.03: Typical Layout in a Wide River Valley (Tucuruí Hydropower Plant, Tocantins River, North of Brazil)

Spillway Structure

The spillway structure should be designed to pass the design flood (item 4.1.2) without exceeding the maximum normal water level in the reservoir and reducing the flood storage capacity. This limitation is attenuated at the Feasibility stage, when more information is gathered about the reservoirs, floods and the possibility of discharging flood flows without causing damage downstream. Whenever possible, spillways should be ungated or have tainter gates; normally, there is no need to have emergency spillways or fuse plug spillways to reduce the flood discharge capacity required by the authorities. These limitations are reviewed at the Feasibility stage, when more and better quality information is made available about the topography of the reservoir and the location of the structures, as well as on the floods and the geological state of the foundations. Space permitting, a spillway with a high ogee crest is used; otherwise, use an abutment spillway.

For layouts which have sections with a low flow rate between the dam and the tailrace canal, bottom outlets or valves should be planned to assure that the flows are ecologically and sanitationally acceptable. Using bottom outlets should only be considered if factors downstream require outlet conditions that cannot be met by the ungated surface spillway.

Hydraulic Conveyance Facilities

The hydraulic conveyance facilities should be developed in such a way that the powerhouse or the tailrace canal is downstream from the concentrated head so as to harness the head to the utmost. The total length of the conveyance facilities must also be kept to a minimum to achieve the most practical and economical solution. It is very important to reduce the length of the sections subject to the highest pressures, since the cost per meter of these sections is generally very high, be they pressure penstocks or tunnels. In Inventory Studies, when there is limited information available on the real geotechnical conditions of the bedrock, large-scale underground construction work should be avoided, such as long headrace tunnels, underground surge chambers, pressure tunnels, underground powerhouses and tailrace tunnels.

Navigation System

When the studies indicate that the river is navigable, locks or other navigation facilities should be an integral part of the project from the outset, taking into account the criteria required for river navigation in their layout.

Fish Passage System

Whenever the studies indicate that structures are required to allow aquatic fauna to migrate upstream or downstream, these should be included in the layouts.

2.6 ECONOMIC PARAMETERS

The economic parameters used in Inventory Studies are presented below:

Reference Date for Cost Estimates

This is the date of reference for all the monetary values quoted in the cost estimates.

Useful Life of the Facilities

This is the period of time that a hydroelectric power plant is economically useful, which is normally taken as 50 years.

Discount Rate

This is the rate used to calculate the present value of future disbursements for investments, and to determine the annual costs of these investments. This rate is set by the concession-granting authority. It should be used uniformly throughout the Inventory Study.

Reference Energy Unit Cost (CRE)

This is the generation cost, expressed in R$/MWh, above which any firm energy contributed by a project or group of projects ceases to be economically competitive with other sources of energy. It is the pure energy production cost in the reference system over the long term. This cost is provided by the concession-granting authority.

Reference Peak Energy Cost (CRP)

Expressed in R$/kW/year, this is the value above which the benefit of adding extra peak capacity to conventional power plants ceases to be economically competitive. It is the pure peak capacity cost in the reference system over the long term. This cost is provided by the concession-granting authority.

Reference Unit Cost (CUR)

Expressed in R$/MWh, this is the generation cost above which the energy generated by a plant or group of plant ceases to be economically competitive. It is the long-term cost in the reference system considered for energy production at a capacity factor Fk, by combining the source of energy and the source of peak energy in the reference system. This cost is calculated by the following expression:

$$CUR = CRE + \frac{CRP}{8.76.F_K} \tag{2.6.01}$$

Annual Operating and Maintenance Cost (COM)

Expressed in R$/kW/year, this is the amount of money required to operate and maintain hydroelectric power plants. In order to estimate it, a mathematical function should be employed that relates installed capacity, P (MW), to the annual operating and maintenance cost, COM (R$/kW/year). The curve presented below is based on information collected by ANEEL for the 2007 review of the Energy Optimization Rate[4].

$$COM = a \times P^{-b} \tag{2.6.02}$$

4 ANEEL, *Estudo de Custos Unitários de O&M das UHE - Composição da TEO - Tarifa de Energia de Otimização*. Andrade&Canellas, São Paulo, SP, 2006

Where:

P	installed capacity of a hydroelectric power plant, in MW
a	87.343
b	0.3716

The reference date for the COM's used to adjust the curve presented in equation 2.6.02 was December 2006. The concession-granting authority should be consulted to find out if any change has been made to the figures. If not, the COM should simply be updated using the General Price Index (Índice Geral de Preços – Disponibilidade Interna, or IGP-DI).

2.7 CRITERIA FOR DIMENSIONING AND COSTING PROJECTS

The methods used for calculating the size of structures and equipment for the projects in the cascades under study and the costs involved are different in the Preliminary Studies and Final Studies. Details are provided in items 4.10 and 5.7, respectively.

When a group of cascades are selected at the end of the Preliminary Studies, comparisons are made between them and those that are uncompetitive or dominated are rejected. For this reason, even though basic preliminary data can be used at this stage, the more accurate the cost estimate of each project by reducing uncertainties, the more reliable the selection of the cascades to go through to the Final Studies will be.

For this reason, whenever possible, it is recommended that in the Preliminary Studies, the spreadsheets provided be used for dimensioning the structures for the Final Studies. It is enough to have a minimum of data from the field for the purposes of filling any gaps for which assumptions cannot be made.

2.7.1 Calculating the Size of Structures and Equipment

In the Preliminary Studies, the size of most of the structures need not be calculated. The external dimensions of the main structures are determined only for the purposes of designing the general layout of the projects.

For the Final Studies, more in-depth information is gathered on the topographical and geological conditions, which makes it possible to design layouts in greater detail. Not only are the external dimensions of the main structures decided on, but the hydraulic pre-design is done using graphs and equations. The same applies for the equipment.

2.7.2 Estimating Engineering Costs

In the Preliminary Studies, the basic method for estimating costs is to use graphs from which the overall or unit costs are obtained for the construction work, services or equipment as a function of one or more parameters. The aim is to estimate the costs of the construction work and equipment in a simplified manner so that a cost estimate for the project can be formulated quickly and simply using standard costs, without the need to identify the specific features of each site.

In the Final Studies the quantity of services, supplies and equipment are determined for each structure using graphs, formulas and tables. Cost estimates are calculated for each structure by applying unit prices to the quantities of services, supplies and equipment required. Some of the costs of sets of services and equipment are given parametrically using global values.

2.7.3 Socioenvironmental Costs

The socioenvironmental costs that will effectively be included in the overall cost of the projects and incorporated into the cost/benefit indexes must be estimated. These are:

- control costs (incurred to prevent the partial or total occurrence of the negative socioenvironmental impacts of a project);
- mitigation costs (incurred in reducing the consequences of negative socioenvironmental impacts);
- compensation costs (incurred in compensating for socioenvironmental impacts caused by a project when reparation is impossible);
- monitoring costs (incurred in following up and assessing socioenvironmental impacts and programs); and
- institutional costs (incurred in preparing the socioenvironmental studies for the different stages of the project and the studies required for the environmental agencies (EIA/RIMA and PBA) for environmental licensing purposes and public meetings.

In both the Preliminary Studies and the Final Studies, the criterion for estimating socioenvironmental costs involves using global or unit costs. Some unit prices can be obtained from secondary sources and supplemented by field studies, especially the cost of land and rural and urban land development.

2.8 CRITERIA FOR SELECTING CASCADES

The basic criterion for the selection of cascades is to maximize economic-energy efficiency while minimizing negative socioenvironmental impacts. As maximizing economic-energy efficiency generally comes into conflict with minimizing negative socioenvironmental impacts, when cascades are compared and selected these aspects must be evaluated as part of a multi-objective approach.

2.8.1 Preliminary Studies

In the Preliminary Studies, the aim of the comparison and selection of the cascades is to eliminate those that are not competitive either because their economic-energy efficiency, or because of their negative socioenvironmental impacts. The two indexes used to make this comparison are the cost/energy benefit index and the negative socioenvironmental impact index, which are calculated for each cascade using the methods in items 4.11.1 and 4.11.2.

As the aim here is to select a group of cascades to be studied in greater depth in the Final Studies, it is recommended that they should not be compared by taking the aggregate sum of the indexes mentioned above, but by identifying how each cascade stands up in relation to the core objectives of *maximizing economic-energy efficiency and minimizing negative socioenvironmental impacts*. By so doing, the need to establish the relative importance of the objectives is avoided at this stage. The selection should be made (see item 4.11.3) on the basis of eliminating the cascades that fail to meet either objective in isolation, and identifying the undominated cascades from among those that remain (cases in which there is no other cascade with a lower cost/energy benefit index and negative socioenvironmental index).

2.8.2 Final Studies

In the Final Studies, the cascades are compared and selected with a view to identify one cascade to be submitted to the subsequent studies required for planning the expansion of the electricity industry. This choice should be made by ranking the cascades, taking the basic criterion of maximizing economic-energy efficiency while minimizing negative socioenvironmental impacts, while also taking into account any positive socioenvironmental impacts that could be achieved by building the hydropower projects in the river basin.

The cascades should be ranked according to a preference index ***I***, obtained by the weighted sum of the cost/energy benefit index and the negative socioenvironmental impact index. The weights used should be established in such a way as to take into account the relative importance attributed to each of the objectives, reflecting the context in which the analysis takes place and the period when the studies are undertaken (item 5.8.4). In order to define these weights, apart from the opinion of the experts directly involved in the studies, the results of the technical meeting where the partial results are presented at the end of the Preliminary Studies should be considered, as described in item 2.9.

In order to make the final choice of one cascade, it is proposed that an additional analysis be made, by which the positive socioenvironmental impacts in the study area (expressed by the positive socioenvironmental impact indexes) are incorporated to the cascades that have already been ranked, resulting in the modified preference index ***I´***, as described in item 5.8.4.

2.9 COMMUNICATION AND PUBLIC MEETING

The following procedures should be followed with a view to informing and consulting different sectors of society throughout the Inventory Studies:

a) At the planning stage, notify all environmental agencies, water authorities, and any committees, associations or institutions concerned with the management of water resources that the study is taking place; provide a list of objectives, activities involved, analyses and fieldwork to be undertaken in the water basin.

b) At the end of the Preliminary Studies, a technical meeting is held, to be called by the Ministry of Mines and Energy, where the results of the Preliminary Studies are presented.

c) At the end of the studies, a public meeting is held, called by the Ministry of Mines and Energy, to present the cascade selected and the findings of the IEA's and the associated guidelines and recommendations.

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chapter 3

Planning

CHAPTER 3

The aim of the Planning phase is to organize the subsequent stages of the Hydropower Inventory Study, which are the Preliminary Studies and Final Studies, as well as the Integrated Environmental Assessment. At the end of this phase, a management report is produced containing an action plan for the Hydropower Inventory Studies, including its schedule and costs.

Local and regional cartographical, geological, geotechnical, hydrometeorological, sedimentometric and socioenvironmental data are gathered at this stage, as well as costs.

By analyzing these data, it is possible to:

- gauge what field studies and investigations will need to be made at later stages;
- identify likely sites for dams;
- propose different cascade options; and
- estimate the energy potential of the area under study.

In this Planning stage, previously identified parameters, restrictions and factors are taken into account, which impose limits on the development of the work.

In relatively well known areas, this phase will be based mostly on bibliographical and documental information gathered from private and governmental entities. Field reconnaissance trips in this phase are of an expeditious nature and are designed only for gathering and confirming relevant information that is quickly and easily obtainable.

In less well known areas, the scarcity of ready information may mean that even at the Planning stage more in-depth field work has to be undertaken in order to obtain results of an acceptable level of accuracy.

During this phase, a communiqué about the studies, describing the activities, analyses and surveys to be undertaken in the river basins, should be issued to the environmental and water resource entities and also to any committees, associations or other institutions involved in managing the water resources, explaining the objectives of the studies, establishing communication channels and making it easier to gather the data which will be needed in due course.

3.1 DATA GATHERING AND ANALYSIS

The best starting point for the data gathering stage is to contact the following institutions, which safeguard information on their particular areas of expertise:

- ANA – Agência Nacional de Águas (National Water Agency): hydrometeorological and sedimentologic data;
- ANEEL – Agência Nacional de Energia Elétrica (Brazilian Electricity Regulatory Agency): data on prior studies and guidelines on the subject;
- CNRH – Conselho Nacional de Recursos Hídricos (National Council for Water Resources): legislation pertaining to water resources and information on river basin committees;
- CONAMA – Conselho Nacional do Meio Ambiente (National Council for the Environment): environmental legislation;
- CONCAR – Conselho Nacional de Cartografia (National Cartography Council): cartography legislation and standards;
- CPRM – Companhia de Pesquisa de Recursos Minerais (Brazilian Geological Survey): hydrometeorological, geological, hydrogeological and mineral data;
- DHN – Diretoria de Hidrografia e Navegação da Marinha do Brasil/Ministério da Defesa (Brazilian Navy's department of hydrography and navigation/Ministry of Defense): cartographic information concerning navigable waterways and licenses required;
- DNIT – Departamento Nacional de Infra-Estrutura e Transporte (National Department of Infrastructure and Transportation): information on transportation infrastructure;
- DNPM – Departamento Nacional da Produção Mineral (National Department of Mining): geological and mineral mapping, and applications for mining activities;
- DSG – Diretoria de Serviço Geográfico do Exército Brasileiro/Ministério da Defesa (Brazilian Army's geography service/Ministry of Defense): planimetric and altimetric information, conventional maps and image maps of the region of interest and others;
- EMBRAPA – Empresa Brasileira de Pesquisa Agropecuária (Brazilian farming and ranching research company): information about the agricultural suitability and use of the land;
- EPE – Empresa de Pesquisa Energética (Energy Research Company): hydropower inventory studies, hydropower planning studies;
- FCP – Fundação Cultural Palmares/Ministry of Culture (Palmares Cultural Foundation): information on *quilombola* communities (former runaway slave communities);
- FGV – Fundação Getúlio Vargas (Getúlio Vargas Foundation): economic and social indicators;
- FUNAI – Fundação Nacional do Índio (National Foundation for Indigenous Peoples): location of and information about indigenous lands and peoples;
- IBAMA – Instituto Brasileiro de Meio Ambiente e dos Recursos Naturais Renováveis (Brazilian Institute for the Environment and Renewable Natural Resources): environmental information;
- IBGE – Fundação Instituto Brasileiro de Geografia e Estatística (Brazilian Institute of Geography and Statistics): socioenvironmental, geodetic, topographic, cartographic and remote sensing information, as well as data from the RADAMBRASIL project, among others;
- INCRA – Instituto Nacional de Colonização e Reforma Agrária (National Institute for Colonization and Land Reform): information on plans for human settlements and demarcation of *quilombola* lands;

- INMET – Instituto Nacional de Meteorologia (National Institute of Meteorology): climatic and meteorological data;
- INPE – Instituto Nacional de Pesquisa Espacial (National Institute for Space Research): remote sensing;
- INPRA – Instituto Internacional de Pesquisa e Responsabilidade Socioambiental Chico Mendes (Chico Mendes Institute for Research and Socioenvironmental Responsibility);
- IPHAN – Instituto do Patrimônio Histórico e Arqueológico Nacional (National Institute for Historical and Archaeological Heritage): information on archaeological sites;
- MMA – Ministry of the Environment: information on conservation areas and priority areas for biodiversity conservation;
- state environment agencies;
- state water resource entities: licenses;
- SBE – Sociedade Brasileira de Espeleologia (Brazilian Society of Speleology): recording of caves;
- SEAP – Secretaria Especial de Aqüicultura e Pesca (Department of Fishing and Fisheries): plans;
- SEMA – Secretarias Estaduais de Meio Ambiente (state departments of the environment;
- SEPPIR – Secretaria Especial de Políticas de Promoção da Igualdade Racial (Department for the Promotion of Racial Equality);
- SERH – Secretarias Estaduais de Recursos Hídricos (state departments for water resources);
- SIPAM – Sistema de Proteção da Amazônia (Amazon Protection System): information on cartographic and socioenvironmental information on the Amazon;
- SPHAN – Secretaria de Patrimônio Histórico e Arqueológico Nacional (National Department of Historical and Archaeological Heritage); and
- SRHU/MMA – Secretaria de Recursos Hídricos (Department of Water Resources / Ministry of the Environment): planning and management of water resources.

There are other municipal and state entities which also have information of relevance, the most important of which are those that manage water resources.

Information should also be gathered on:

- socioenvironmental impact studies for projects in the river basin;
- socioenvironmental studies (environmental control reports, basic environmental plans, monitoring reports, etc.);
- integrated regional plans, municipal master plans, existing sector plans (e.g. plans for new roads, railroads, waterways, etc.);
- master plans for river basins;
- strategic or integrated environmental assessments of the river basin under study; and
- miscellaneous studies (reports, technical papers, theses, etc.) which address the river basin either directly or indirectly, highlighting the following areas: mineral resources, seismicity, wildlife, vegetation, geology, geotechnics, geomorphology, pedology, hydro-climatology, hydrometeorology, limnology, ichthyology, sociology, economics, anthropology, indigenous peoples, traditional communities, archaeology and paleontology.

3.1.1 Cartography

The quality of the planning depends on the quantity, range, representativeness and consistency of the data available. Generally, the following information should be sought.

- Topographic maps (planimetric and altimetric) and thematic maps – these can be obtained from the public cartography agencies or mapping companies. Lists of charts, maps and plans of the area of interest can be gathered, then they can be selected for each end use according to their scale. An analysis should be made of the reliability of these documents by assessing the method used to produce them and their cartographic accuracy. This is necessary for deciding whether they can be used at later phases;
- Planimetric and altimetric support points – survey of the existence of geodetic services from the responsible entities. The accuracy, availability and integrity of the basic support available should be checked. Geodetic or topographic studies undertaken by government entities or private companies can be used, provided they are compatible with the degree of accuracy required in the technical standards pertaining to the topics that need to be addressed;
- Geographical information systems – information can be obtained from public and/or private companies, which can be selected according to the cartographic data used to prepare it and the quality of the associated database.

Other documents of use, especially for later phases, are:

- Remote sensing images – request information from INPE, IBGE, DSG and other institutions and companies with information of this nature, checking what coverage exists of the area of interest, especially: cloud cover, the dates when the images were taken, what kinds of sensors were used to produce them, the spatial, geometric, spectral and temporal resolutions, the mapping scales available, and the existence of stereo pairs;
- Aerial photographs – request information from public entities or mapping companies. The information should be analyzed for its usefulness at later stages. The existence and availability of aerial photographs of the area of interest should be checked, as well as any mosaics or orthophoto maps, and non-metric aerial photographs; and
- Geoid map – request information from IBGE and universities.

The analysis of the information gathered should be based on clear criteria, not only to check the quality of the information (according to the methods used) but also to verify the projection system used and the feasibility of cross-referencing (the altimetric and planimetric data) between them.

Should it be necessary to make a specific or supplementary survey of the river basin to fill any gaps for the Final Studies, the information should be produced in compliance with the Technical Standards for National Cartography, decree 5.334 of January 6th 2005.

To sum up, by the end of this stage, an assessment should have been made of the documents gathered and a list prepared of any services still needed.

3.1.2 Hydrometeorology

The hydrometeorological and sedimentologic data to be gathered correspond to recorded daily streamflow, sediment discharge and meteorological information – rainfall, winds, sunshine, temperatures, etc. – from the gauging stations that exist in the river basin that can provide reliable data from a sufficiently long period of time. The main information available should be gathered, such as forms to describe the gauging stations, summaries of the streamflow and sediment discharge measurements, records of water level observations or limnigrams, and rainfall records or pluviograms, as well as any analyses of consistency and data gap filling undertaken.

All the gaps in the data for given periods of time or parts of the area under study should also be identified, taking into account any data from the region with similar behavior. The network of river gauging stations should be characterized in terms of the kind of data they gather and their characteristics.

In the absence of data in sufficient quantity or of sufficient reliability, it will be necessary in the Preliminary Studies to produce the monthly average flows and extreme flows based on regional studies or deterministic models, while also drawing on any river gauging or rainfall data available not only from the river basin under study but from contiguous areas with similar hydrological and hydro-geological features.

At the end of this phase, a map of the basin should be prepared showing the existing river gauging stations and possible dam axes for study. The availability of data should be indicated using a bar chart, as well as a list of the stations, the respective periods of data available and the kind of data collection undertaken.

The data needed for the studies can be obtained from ANA, INMET, ANEEL, EPE, state water resource management entities, and companies and entities which use the water resources, such as electricity and water utilities.

In order to better understand the sedimentology of the basin, sedimentometric data and data on the erosion conditions in the basin should be collected using erodibility maps and maps of land use and deforestation, as well as information obtained from the field reconnaissance. The sections of river either upstream or downstream where erosion and silting processes could be critically altered by the introduction of reservoirs should also be identified. It is important to consult studies contained in technical reports of any nature for dam projects.

Should it be necessary to install new river gauging stations, rainfall gauges or sediment measurement devices to supplement the existing ones, their location should be decided upon only after consulting ANEEL resolution 396/98. The entities that operate the hydrometric network in the region and the generation companies that operate in the basin should also be contacted to find out if they have any interest in operating and maintaining these gauging devices.

3.1.3 Multiple Water Uses

The collection of data and information about the different water uses is designed to identify the potentialities of the river basin under study and to assess its compatibility with the National Plan for Water Resources and any river basin plans that may exist, as well as any sector or integrated plans available. The aim is to provide inputs for drawing up the scenario for the use of the water in the river basin, which is necessary for assessing the energy benefits of the cascade options and the socioenvironmental impacts. Table 3.1.3.01 below presents the kind of information that should be collected and the main entities that can supply information on different water uses that may interfere with the hydropower generation project. Future uses of the reservoir should also be considered, including tourism, aquaculture and the legal status of the body of water, which could imply in restrictions on its use.

Table 3.1.3.01 – Water uses and sources of information

USE	SOURCES and INFORMATION	ENTITIES
Irrigation	– National Plan for Water Resources – state plans for water resources – plans for the water resources in the river basin – municipal master plans – record of licenses granted – inventory of irrigable areas – crop irrigation coefficients – consumption statistics – plans for the introduction of irrigation projects – record irrigation projects and their beneficiaries – soil types and crop calendars – farming census (per municipality – IBGE)	SRHU/MMA ANA/MMA IBGE MAPA MI river basin committees regional agencies state entities municipal entities
Livestock Breeding	– National Plan for Water Resources – state plans for water resources – plans for the water resources in the river basin – municipal master plans – record of licenses granted – consumption statistics – requirements for livestock breeding (heads of cattle, horses, asses, sheep, goats and pigs) – farming and ranching census (*Censo Agropecuário*) per municipality – IBGE	SRHU/MMA ANA/MMA IBGE MAPA MI river basin committees regional agencies state entities municipal entities
Flood Control	– National Plan for Water Resources – state plans for water resources – plans for the water resources in the river basin – survey of soil use on the floodplains – survey of works existing for flood control or prevention – large–scale drainage plans – flood control studies for the river basin	SRHU/MMA ANA/MMA IBGE ANEEL ONS MI river basin committees regional agencies state entities municipal entities OTEP
Navigation	– National Plan for Water Resources – state plans for water resources – plans for the water resources in the river basin – municipal master plans – survey of navigable sections and river ports – statistics on the movement of people and goods in the region – river navigation plans	SRHU/MMA ANA/MMA IBGE MI Ministry of Transportation / DNIT state departments of transportation river basin committees regional agencies state entities municipal entities
Human Water Supply	– National Plan for Water Resources – state plans for water resources – plans for the water resources in the river basin – municipal master plans – population growth statistics – urban and rural demographic forecasts – record of licenses granted – agriculture incentive plans in rural areas	SRHU/MMA ANA/MMA IBGE Ministry of Cities river basin committees regional agencies state entities municipal entities water and sewage treatment companies

Sanitation	– National Plan for Water Resources – state plans for water resources – plans for the water resources in the river basin – municipal master plans – record of licenses granted – urban and rural demographic forecasts – National Information and Sanitation System – national basic sanitation survey	SRHU/MMA ANA/MMA IBGE Ministry of Cities river basin committees regional agencies state entities municipal entities water and sewage treatment companies
Industry	– National Plan for Water Resources – state plans for water resources – plans for the water resources in the river basin – municipal master plans – record of licenses granted – plans for thermoelectric power plants – industry census – annual industry survey	SRHU/MMA ANA/MMA IBGE MDIC MME DNPM/MME CNI (Confederação Nacional da Indústria) federations river basin committees regional agencies state entities municipal entities
Tourism and Leisure	– National Plan for Water Resources – state plans for water resources – plans for the water resources in the river basin – municipal master plans – record of licenses granted – tourism plans	SRHU/MMA ANA/MMA IBGE MDIC Ministry of Tourism Ministry of Sports river basin committees regional agencies state entities municipal entities
Aquaculture	– National Plan for Water Resources – state plans for water resources – plans for the water resources in the river basin – municipal master plans – record of licenses granted – sector plan	SEAP/PR MAPA SPU/MPOG MMA ANA Port Authorities
Ecosystem Maintenance	– state plans for water resources – plans for the water resources in the river basin – studies of minimum ecological flow	entities that manage water resources and the environment

3.1.4 Geology and Geotechnics

The geological and geotechnical data to be collected should allow for a characterization of the foundation and excavation conditions for building the structures and the natural building materials to be used for the projects and any information that could supplement the environmental studies. Geological and geomorphological information about the river basin should be collected on sources of erosion, mineral resources, the stability of hillsides, natural and induced seismicity, watertightness and configuration of the reservoirs.

The basic data for the Planning phase are essentially the documents obtained from existing sources in the form of geological and geomorphological maps, geological and geotechnical studies and maps, remote sensing images (radar, multispectral images, etc.) and aerial photographs. The following basic information must be gathered:

- geotechnical data on hydropower plants planned and installed in the region and/or under similar geological conditions to those in the study area, as well as any roads and large-scale works in the region;
- geological maps, geomorphological maps, maps of mining potential and seismotectonic maps, with their scale, coverage area, the entity that produced them and the year in which they were produced; and
- data on mineral resources from DNPM in the form of maps which identify where there are areas of research and mining activities, and lists of these processes.

This information should be submitted to a detailed analysis to check its quality and relevance to the studies. After this is done, the information may, if necessary, be checked on site by air reconnaissance and field trips.

If there are no suitable maps or geological information on the region under study, preliminary photo-geological maps should be prepared of the areas of interest in the rivers to be studied, which will help identify sites or sections that could be impounded, and also to identify the different sections of the floodplain and any geological features of interest.

Table 3.1.4.01 below sets out the geological, geomorphological and geotechnical data required for the Preliminary Studies and Final Studies that should be collected and analyzed during the planning phase.

In the Preliminary Studies, it is only necessary to produce estimates of the geological and geotechnical parameters. During the Final Studies, however, these characteristics must be determined with a high enough degree of accuracy as to assure the integrity of the foundations of the structures and the availability of natural building materials.

By the end of this phase, an assessment should have been made of the documents gathered and a list prepared of any services still needed.

Table 3.1.4.01 – Geological, Geomorphological and Geotechnical Activities Required in the Hydropower Inventory Studies

Planning stage	Preliminary studies	Final studies
– Register and obtain data on: · hydropower plants planned and installed in the region and/or under similar geological conditions to those in the study area, as well as any roads and large-scale works in the region; · geological maps, geomorphological maps, maps of potential mineral deposits and seismotectonic maps – supply scale, coverage area, entity and year; · data on mineral resources from DNPM in the form of maps which identify where there are areas of research and mining and lists of these processes. – Analyze the material available. – Draw up a list of services necessary.	– Make geological interpretations of the remote sensing images and aerial photos with the aim of: · supplementing the existing geological and geomorphological maps; · providing inputs for defining the best sites for dams; · indicating potential sources of natural building materials (sand and gravel deposits in the river bed quarries, borrow areas); · indicating areas of the river basin that are potentially susceptible to erosion; · indicating areas with natural hillside instability around the reservoir; and · carrying out expeditious geological and geotechnical mapping of the dam sites. – Drill at least inspection wells and auger bores at the dam sites – Present geological and geotechnical sections of the prospective dam axes, indicating the likely top of the bedrock and the kinds of materials encountered at the site. – Assess the potential for natural building materials (borrow areas, sand, gravel and quarries). – Undertake geological and geomorphological mapping of the region along the rivers and roads. – Map out mineral resources. – Prepare a seismotectonic map of the basin.	– Supplement the information obtained in the Preliminary Studies on areas of regional interest and potential sites for dams that were approved for the Final Study phase. – Carry out supplementary investigations as required, such as boreholes, trenches, auger bores and geophysical soundings. – Provide comparative parameters between the different kinds of rock existing in the dam foundations. – Assess the mineral resources that could be influenced by the reservoirs, using data from DNPM.

3.1.5 The Environment

The socioenvironmental data should help identify the most significant issues, especially those that could represent constraints or opportunities, influencing the choice of dam sites and identifying the preliminary set of cascade options, while also serving as inputs for the work plan and cost estimates for the next stages.

The data should be collected primarily from public entities, state-owned companies, specialized federal, state and municipal government agencies, universities and research institutes. The information will include academic publications, documents, maps, charts and statistics.

Some of the socioenvironmental aspects can be studied using the same databases as those used for the cartography, hydrometeorology, multiple water use, geology and geotechnics studies, as defined in items 3.1.1, 3.1.2, 3.1.3 and 3.1.4.

Once gathered, the data should be catalogued and assessed as to its consistency and accuracy. It should be checked whether all the basic data is available that could seriously interfere in the schedule of the Hydropower Inventory Studies. This assessment should take into account the data used to formulate the core components defined in preparing the socioenvironmental diagnosis during the Preliminary Studies, as in item 4.3.

Given that socioenvironmental issues are taken into account in the selection of dam sites and the preliminary identification of cascade options, the socioenvironmental issues set out in Table 3.1.5.01 below warrant special attention.

Table 3.1.5.01 – Significant Socioenvironmental Issues

Socioenvironmental issues	Minimum content	Sources of information
groups of indigenous peoples	Locate indigenous lands, indicate their legal status, population and ethnicity	FUNAI, ISA, theses and academic papers, *Anuário Estatístico do Brasil* (Statistical Yearbook of Brazil) published by IBGE, specific legislation
Former *quilombo* communities, groups of ethnic minorities or traditional communities	Locate areas and population sizes	Movimento Negro Unificado (Unified Black Movement), SEPPIR, Palmares Foundation, INCRA, specific literature, specific legislation
Conservation Areas	Locate, classify, characterize, identify legal status and any existing conflicts	INPRA, state environment entities, specific literature, specific legislation
Heritage Properties	Locate historical, cultural, archaeological, landscape, speleological and ecological heritage sites/properties	IPHAN, state and municipal heritage protection entities
Seats of municipal and district authorities	Locate and identify the functional hierarchy and population	IBGE, state and local authorities
Densely populated rural areas	Identify land usage, population density, land ownership and production data	IBGE, local authorities and producers' associations
General infrastructure and basic sanitation	Locate roads, railroads, river ports, landing strips and power lines, nearby bridges and roads	Road maps, IBGE, DNIT, DERs, local authorities and state entities
Existence of minerals of economic and strategic value	Identify existence and classification of mining licenses	DNPM, CPRM, RADAM-Brasil project, state and municipal entities
Mineral water sources	Locate and characterize	DNPM, state and municipal entities
Industrial and agri-industrial facilities	Locate and characterize	EMBRAPA, EMATER, confederation of industries, state and municipal entities, IBGE, MDA.
Commercial fishing	Characterize in general terms (organization of the activity, quantity and species fished, markets for the catches)	IBAMA, state environmental protection entities, IBGE, SEAP.
Rare, endemic or threatened species	Locate and identify on a local and regional level	IBAMA, INPRA, state environmental protection entities, theses and scientific publications
Priority areas for the conservation of biodiversity	Locate and identify on a local and regional level	MMA, INPRA, state environmental protection entities, National Congress Environment Commission

Socioenvironmental issues	Minimum content	Sources of information
Fragile areas or areas of ecological interest	Identify locations, characteristics, uses and area occupation	IBAMA, INPRA, state environmental protection entities, local authorities, municipal and state departments
Ichthyofauna	Migratory routes, breeding grounds, fish farms	Research institutes, universities and environmental entities
Economic activities	Economic activities that could be affected, such as fishing, agriculture, etc., including estimated sums involved	MAPA, SEAP, IBGE, state and municipal departments
Organized Civil Society	Conflicts and means of organization	Main NGOs, social movements and associations that are active in the region

The items identified here are only the main impacts that may take place when a hydropower plant is built. Depending on the specific nature of the river basin, some of the items may be disregarded, while others could arise that require attention.

The following steps are recommended:

- to gather, collate, organize and georeference the data. In this case, the use of geographic information systems provides greater speed and flexibility for the analyses required, while also helping build up the digital database, which is of great importance in keeping information updated and assuring ease of information recovery;
- whenever possible, to gather secondary data to prepare thematic maps using the same coverage area and scale which are compatible with the other maps needed for the Hydropower Inventory Study, all of which should be connected to a single database;
- to prepare a summarized map, which will set out all the information on one single map, providing the first environmental characterization of the river basin, highlighting the aspects that could pose any major restriction and highlighting any existing or potential problems or conflicts. This map should be used when prospective sites are picked for dams and when the study area is demarcated for the subsequent stages, as well as in preparing the work plan for the field reconnaissance.

3.2 IDENTIFICATION OF DAM SITES

A **map of potential dam sites** should be prepared, drawing on the analysis of the data gathered in item 3.1 and the restrictions identified. The potential dam sites should primarily be identified from planimetric and altimetric maps. Additionally, aerial photographs and remote sensing images should be used. At this first research stage, special attention should be paid to all sections of the river with rapids and waterfalls, and all the areas where the river valley narrows sharply. Likewise, attention should be given to the restrictions imposed by the physical and socioenvironmental factors identified.

For each dam axis, it should be determined what the highest water level could be for the reservoir.

Plans and profiles of the river should be made at these sites, which will serve as a basis for preparing the different cascade options. The criteria to be adopted in each case will depend on the technical assessment of the topographic, geological, geotechnical, hydrological and socioenvironmental parameters.

3.3 FIELD RECONNAISSANCE

Once the geological and hydrological data, the physical and socioenvironmental factors of relevance, the planimetric and altimetric maps of the different study areas and the map of potential dam sites have been gathered and prepared, field reconnaissance activities should be planned, which may be aerial, land-based and river-based.

Field reconnaissance must be carried out by a multidisciplinary team with the main aim of confirming, adding and/or ruling out potential dam sites and the effects of the restrictions identified. It should also confirm, add or disregard any of the logistical support points for the future studies.

During the field reconnaissance activities, a detailed examination should be made of the general morphology of the region and compared with the available information. Any sections where there is a sharp narrowing of abutments, gorges or other geographical features of this nature should be studied. The sites previously pinpointed as potentially suited to having dams built on them should be inspected in greater detail and estimates should be made of the maximum head permitted at the sites.

It should be checked whether there are any river gauging stations in the region and what facilities they contain, as well as any sandbanks or stretches of river susceptible to erosion.

The main uses of the water resources should be identified at this stage. These include existing water consumption, fishing areas, any beaches, whether commonly frequented or not, tourist areas, wastewater discharge, transport upstream, downstream or from bank to bank, rainwater drainage networks, etc.

The field reconnaissance for the socioenvironmental aspects should be expeditious, as in the other areas of study. It is important for planning the socioenvironmental studies, as it permits an overview of the river basin as a whole. Particular attention should be given to any aspect that affects the aquatic and terrestrial ecosystems and any local inhabitants that might be affected, as well as the state of any riverside woodlands, and any constructions within the river channel, since these elements could have a direct influence on the complexity of the studies required at the later stages and the length of time needed to do them.

All the observations made by the multidisciplinary team should be consolidated, especially to include new data and to confirm or contradict information from prior studies. The thematic maps and other data should be reviewed so that the new information can be included.

3.4 CASCADE OPTIONS

Taking the information obtained in 3.1, 3.2 and 3.3, the greatest possible number of cascade options should be considered, and the energy potential of each cascade and each individual project should be estimated.

The cascade options should normally include the creation of regulating reservoirs in the upstream stretches. The maximum heights of the dams should be compatible with the physical characteristics and the nature of the foundations at each site.

The cascade options should harness the entirety of the available head, considering the limitations imposed by the physical and socioenvironmental factors identified.

The possibility of diverting part of the flow to other basins or vice versa should be considered if there is anything that justifies such a solution.

3.5 PLANNING REPORT

A technical and management report should be prepared, containing predictions of the results to be achieved and the resources needed to fulfill the objectives of the Hydropower Inventory Study. In order to provide guidelines for the subsequent phases of the Hydropower Inventory Studies (Preliminary Studies, Final Studies and Integrated Environmental Assessment), this report should contain:

- an evaluation of the energy potential;
- an evaluation of the restrictions and constraints imposed on the potential plants; and
- the work plan, including activities, schedule and costs.

3.5.1 Socioenvironmental Aspects and Water Resources

The data gathered on socioenvironmental aspects and on water resources, and the analyses made of these should be consolidated and divulged in a specific item of the report, including:

- a preliminary socioenvironmental characterization of the river basin;
- a preliminary characterization of the multiple water uses and land use, highlighting any existing or potential problems or conflicts;
- any restrictions or constraints imposed on likely hydropower projects;
- the thematic maps; and
- the map that summarizes all the main data.

3.5.2 Work Plan and Cost Estimate

The data gathered and the cascade options formulated are used to decide upon the tentative perimeters of the reservoirs and dam sites. Based on this material, a proposal should be prepared listing and describing the services to be undertaken to fulfill the procedures required for the different phases of the Hydropower Inventory Studies (Preliminary Studies, Final Studies and Integrated Environmental Assessment), containing:

- definition of the study area, as per item 2.3.1;
- definition of the specifications for and estimate of the extent of the mapping work required;
- definition of the number and location of new river gauging stations and rainfall gauges;
- estimate of the number of topological surveys and water depth soundings;
- estimate of the number and kind of geological and geotechnical surveys required;
- estimate of the number of measurements of streamflow and sediment discharges;
- identification of the complementary studies required to fulfill the requirements for formulating the synthesis components, as set forth in item 4.3;
- indication of the surveys to be carried out to estimate the socioenvironmental costs, as set out in items 4.10.1 and 5.7.1;
- cost estimate of the technical meeting and public seminar where the results will be communicated.

If the river basin does not have an adequate network of gauging stations and/or if an intense sediment transportation process is identified, the installation of supplementary gauging facilities should be planned as should campaigns to gather hydrological and sedimentological data for the studies. It is recommended that the campaigns be carried out during the Preliminary Studies so that the results are ready by the beginning of the Final Studies.

Special attention should be paid to scheduling the studies that require longer periods of observation (e.g. factors subject to seasonal variations) or special conditions for them to be undertaken (e.g. logistic support, laboratory tests) to assure that the results are available when they are needed.

Finally, based on the estimate of services to be carried out, schedules, costs, human resources, equipment, structure and logistics required can be prepared and obtained. A technical team should be picked for each task or activity, according to the skills required in each case.

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chapter 4

Preliminary Studies

CHAPTER 4

During the Preliminary Studies, the different cascades for harnessing hydroelectric power are subject to a preliminary, expeditious evaluation, including estimates of the costs and socioenvironmental impacts associated with their use, based on secondary data. The basic data gathering activities and different studies planned in the previous stage are also undertaken, which are not only needed for the preliminary analysis of the cascades, but also provide information necessary for the Final Studies. Besides identifying the possible cascade options, the overall technical specifications of each project are agreed on, including their preliminary design, the dimensions of the main equipment needed, and the estimated installation costs. Using the results of the energy studies, the studies of multiple water uses, the socioenvironmental studies and cost estimates, a shortlist of the most promising cascades is drawn up for more in-depth studies at the next phase. At the end of the Preliminary Studies, a technical meeting is held where the results are presented (item 2.9).

The socioenvironmental studies are carried out with the main aim of incorporating socioenvironmental aspects throughout this stage in order to:

- help draw up the shortlist of cascade options and devise the best design for the projects, taking into account the main socioenvironmental issues identified;
- supply the information needed to estimate the socioenvironmental costs (quantitative costs and corresponding unit prices), by identifying the likely negative socioenvironmental impacts incurred by each project and cascade under study;
- calculate a negative socioenvironmental impact index for each cascade, so they can be compared using a multi-objective approach and the ones to go through to the Final Studies can be selected;
- enable the identification of areas of sensitivity, with a view to indicating critical areas for the introduction of future projects and also areas that offer social and economic potentialities to be addressed in the IEA.

The methodology for assessing socioenvironmental impacts at this stage is presented in this chapter in such a way that it can easily be combined with the energy and engineering studies. There are three main phases of studies: socioenvironmental diagnosis, assessment of impacts per project, and calculation of the negative socioenvironmental impact index for each cascade option. This methodology is incorporated into the SINV system. The data generated at each stage can be stored in the system and then used later on to formulate the socioenvironmental impact indexes used for comparing and selecting the different cascade options at the end of the Preliminary Studies.

The socioenvironmental diagnosis generates basic information on the study area relating to the synthesis components and their characterization elements. The procedures for the diagnoses are set out in item 4.1 (**Data Gathering and Studies**) and item 4.3 (**Socioenvironmental Diagnosis**). The results of these analyses are compiled in maps per synthesis component, which should mark out the areas of sensitivity and any areas with socioeconomic potentialities that could be leveraged by the schemes. This information provides a basis for formulating the different cascade options and for analyzing the projects within these cascades.

According to this methodology, all the projects under consideration for the study are first studied individually. This means that each project is assessed without taking into account the effects of any others in the same river basin. The impacts are identified, predicted and assessed, and impact indexes are attributed to each project and each synthesis component. The procedures for this are described in item 4.8 (**Assessment of Negative Socioenvironmental Impacts per Project**). The impact indicators and assessment elements to be used when evaluating the impacts on each synthesis components are also presented. The information on the impacts is used for estimating the **socioenvironmental costs** (item 4.10.1).

The procedures for obtaining the negative socioenvironmental impact index for each cascade under study are described in item 4.11.2 (**Negative Socioenvironmental Index**). Initially, the indexes for the projects should be combined per synthesis component to obtain the impact index for each cascade on each synthesis component. These indexes are then combined to give the impact index of the cascade in question on the socioenvironmental system, which is a consolidation of the socioenvironmental assessment of the cascade, representing its performance in meeting the objective of *minimizing negative socioenvironmental impacts*.

4.1 DATA GATHERING AND STUDIES

4.1.1 Cartography

In order to prepare the topographic maps and carry out all the surveys required for Inventory Studies, the planimetric and altimetric data must be linked to geodetic marks and benchmarks from the Brazilian Geodetic System (Sistema Geodésico Brasileiro), as defined by the IBGE. The forms to be adopted should be compatible with the levels of precision needed to transpose the coordinates to the marks to be implanted at the sites of interest.

The following documents or their equivalents should be referred to:

- "Especificações e Normas Gerais para Levantamentos Geodésicos" [General Specifications and Standards for Geodetic Surveys] approved by IBGE Resolution PRP-22 of 07/21/83 and ratified by COCAR Resolution 02/83 of 07/14/83, published in the official government publication (*Diário Oficial da União*) on 07/27/83.
- "Especificações e Normas Gerais para Levantamentos GPS: Versão Preliminar" [General Specifications and Standards for GPS surveys: Preliminary Version] approved by IBGE Resolution 5 of 03/31/93, which was included in chapter 2 of PRP-22 (General Specifications and Standards for Geodetic Surveys) of 07/21/83.
- "Padronização de Geodetic marks: Instrução Técnica" [Standardization of Geodetic Marks: Technical Instructions] approved by Service Standard NS-DGC 001/2005 issued in January 2006 by the Geoscience Department of IBGE.
- ABNT NBR 13.133 standard on "Execução de levantamento topográfico" [Conducting Topographic Surveys] issued on 06/30/ 94.
- ABNT NBR 14.166 standard on "Rede de Referência Cadastral Municipal – Procedimento" [Municipal Registration Reference Network – Procedures] of 08/01/98.
- Regulatory instructions for the technical standards for national cartography, established by Decree 89.817 of June 20th 1984, published in *Diário Oficial da União* on June 22nd 1984 and subsequent amendments in Decree 5.334 (01/06/05) which amends the text of article 21 of these instructions.
- IBGE Resolution RPR 1/2005 of February 25th 2005, which alters the characterization of the Brazilian Geodetics System, defining SIRGAS2000 as the source of reference data.
- Ministerial Directive 121/MB of April 23rd 2003 – "Instruções Para Controle dos Levantamentos Hidrográficos Pela Marinha do Brasil" [Instructions for the Control of Hydrographic Surveys by the Brazilian Navy].

The methods recommended for obtaining the basic cartographic data are:

- aerial photogrammetry;
- interferometric radar;
- laser profiling;
- satellite imaging;
- topographic surveys.

The methods listed above should be used either individually or in combination to prepare cartographic outputs that are compatible with the uses required for the inventory study in question. They should take into account the specific features of the region in question, especially the following:

- vegetation: characteristic biome, crops, pasture, etc.;
- topography: whether the region is mostly flat or hilly;
- mean height estimates for the dams;
- interference in towns, cities, villages, roads, railroads, indigenous areas, conservation areas, etc.

In terms of the degree of precision required, the following requirements should be observed.

A topographic map should be prepared at a scale of 1:10,000 with contour lines at five-meter intervals, classified as Class A for its standard of cartographic excellence (Padrão de Exatidão Cartográfica) according to the Brazilian Cartography Standard. In cases where the area to be surveyed is very large or where it can be justified on technical grounds, scales of up to 1:25,000 can be used with ten-meter intervals between the contour lines. When the head of a project is low (around 20 m), the scale should be 1:5,000 with contour lines at five-meter intervals or less.

In view of the intended purpose of these outputs, it should give special attention to hydrography items, providing the correct identification of the main water course and its tributaries, showing the places where there are changes in their slope, narrowings, gorges, rapids and waterfalls, as well as any civil constructions associated to them.

The altimetric survey should represent the relief in such a way that the details are identified that could affect the results of the profiles of the water courses, the calculation of the capacity of the reservoirs and the assessment of the physical interferences the reservoirs would have on the landscape. To this end, the following recommendations should be followed:

- Point features should be added to the tops of all elevations, the bottom of all depressions, in saddles and in areas that are especially flat (which cover over 2 cm on the scale of the map);
- Point features should be added to the water level along the rivers, especially where there is a change in slope, such as at the beginning and end of rapids, at the top and bottom of waterfalls, any lakes or artificial reservoirs, and, whenever visible, at the points of confluence with the main tributaries;
- Point features should be added to engineering structures, such as bridge decks, the crest and foot of dams, berths in harbors, roads and railroads that cross or run parallel to the water courses, etc.;
- Supplementary contour lines whenever the slope of the land creates contours that are more than 2 cm apart on the scale chosen for the map, or wherever the slope changes abruptly, such as edges of plateaus, cliffs, etc.

Any field work that needs more precise data should be done in separate surveys. Below is a list of the main kinds of information to be gathered in this way:

- Longitudinal profile: includes determining the planimetric and altimetric coordinates of the water levels of the river and its main tributaries, and other natural or artificial elements of importance to the studies. The profile design should be referenced to a particular date in order to ensure uniform flows, and should contain the accrued distances from the mouth and point features for each important element. In order to minimize errors during the data gathering process, the survey should ideally be carried out during dry periods, and the dates and times should be recorded for each water level reading taken. Below is a list of the water levels of the main elements to be taken:

- top and bottom of each waterfall and rapids;
- mouth of the tributaries;
- boundaries of conservation areas and indigenous lands;

- bridges, with the coordinates of the abutments and elevation of the deck;
- ferry crossings;
- local communities, traditional mining activities, etc.;
- staff gauges;
- transmission lines, measuring the coordinates of the nearest towers and if possible including the voltage of the line(s);
- other elements deemed of importance.

- Topographic and bathymetric surveys of the river bed and abutments along the prospective dam axes, with cross-sections where necessary to characterize the morphology of the abutments and adjacent areas, providing data to be used in the studies for the project layouts.
- Cross-referencing of the geological/geotechnical investigations with the hydrological surveys.

These studies should be in compliance with the specifications for leveling (Class IIIN) and planimetry (IVP) as set out in ABNT standard 13.133. A list should be supplied of all the feature points, with their planimetric and altimetric coordinates and the date the information was gathered.

Final Outputs

For each site considered and using the topographic data available, the following should be prepared:

- the respective water level elevation / surface area curve and water level elevation / reservoir volume curve for each site for each of the different cascades, to be used in the energy studies, studies into multiple water uses and socioenvironmental studies;
- calculation of the planimetric and altimetric location of the information gathered from the geological and geotechnical investigations, the river gauging stations and the features of interest identified in the socioenvironmental studies; and
- calculation of the drainage areas for all the river gauging stations for each sub-basin of interest and at each potential site for a project.

4.1.2 Hydrometeorology

The hydrometeorology studies are started at this phase so that there is consistent, homogeneous information available on the whole river basin by the time the Final Studies begin, and also to provide information of great enough precision to formulate and shortlist the cascade options during the Preliminary Studies.

The hydrometeorological studies are therefore designed to characterize the elements needed to estimate the energy potential and calculate the dimensions of the structures.

The elements required for estimating the energy potential are the records of the monthly average natural flows and the normal water levels at the potential dam sites.

Physiographic and Climatological Characterization of the Basin

Before the basin can be characterized, aside from its location, several other of its physiographic features must be defined, such as its surface area, perimeter, hypsometric curve, the shape of the basin, drainage density, channel slope, vegetation, pedological features, current land use, relief, and others.

Some of these elements are important for interpreting the results of the hydrology studies, as they can directly influence the hydrometeorology of the basin and consequently the flow characteristics, hydrology and sedimentology of the main water course.

The most representative meteorological data should be gathered: rainfall, temperature, humidity, radiation, winds, evaporation, pressure and any others of significance for characterizing the climate of the region under study.

Net Evaporation

When reservoirs are built, the water balance of water courses is altered. In the area flooded for a reservoir, evapotranspiration is replaced by evaporation.

The monthly net evaporation should be determined for each month of the year, providing twelve values per year expressed in millimeters.

In the Inventory Studies of hydroelectric potential, net evaporation rates should be calculated for each reservoir. This is the difference between the real evaporation of the reservoir and the real evapotranspiration in the reservoir area before it was built. For projects linked up to the National Interconnected System (Sistema Interligado Nacional, SIN), the net evaporation is estimated by using the method set out in the System for Evaluating Net Evaporation from Reservoirs (Sistema de Avaliação da Evaporação Líquida dos Reservatórios – SisEvapo).[1] For projects in semi-arid areas of the northeast of Brazil, the real evaporation of the reservoir is obtained by collecting data from evaporation tanks, while real evapotranspiration in the reservoir area is estimated by the water balance method (ONS, 2004).

The SisEvapo system is being extended to the other river basins in the country. Its use is recommended for reservoirs in basins for which the SisEvapo system is prepared to calculate net evaporation. The information needed to use the system should be included in the final report of the survey. For those basins for which the system is not yet adapted, the methodology should be described in the final report of the survey along with all the data utilized.

Net evaporation should be calculated for all the projects in the basin, including existing ones. It is used to calculate the natural mean monthly flows (in the case of existing projects) and in the energy studies.

Flow Records from River Gauging Stations

The selection of river gauging stations for the hydrology studies should take into account their location in the river basin, the observation period, the existence of data gaps, the consistency of the data and of the rating curves, and the density of stations in the river basin.

The limnimetric and stream discharge readings should be taken into account in determining the rating curves and the series of mean daily and monthly discharge rates.

When there is insufficient data or no data from river gauging stations, a hydrometeorological model of the river basin can be used to fill any gaps and/or extend the natural flows recorded at the stations. As a last resort, when the basic data from the stations are inadequate, data from stations in river basins with similar hydrological features can be used.

The relevant entities should be informed of any alterations found in the rating curves and drainage areas at the official river gauging stations, which should be accompanied by detailed technical justifications.

Records of Mean Monthly Natural Flow

The natural flow is the flow along a section of river in the absence of the action of man in the basin upstream from this section. The kind of action that could affect it is the regulating of the water course by the action of reservoirs, diversions to other river basins and withdrawals for different uses.

1 CEHPAR/LACTEC. *Programa SisEvapo v2.0. Relatório Técnico*. Curitiba, 63 pages. The SisEvapo system can be obtained from the Ministry of Mines and Energy.

For each dam site under study, a series of mean monthly natural flows must be calculated, which should be derived from the flows recorded at one or more river gauging stations along the same water course or in the same river basin.

The mean monthly natural flows at the planned sites for projects should start from 1931 and go under at least two years prior to the conclusion of the Inventory Study, even in river basins whose projects are not yet connected up to the National Interconnected System (SIN).

In determining the mean monthly natural flows for each project site, the consumptive uses should be estimated (item 4.2.1) and, when applicable, the influence arising from the operation of the reservoirs and the effect of net evaporation on the reservoirs. The monthly values for consumptive uses should be added to the mean monthly natural flow values obtained thus far.

The hydrological data for the projects under study should be compared with the respective data from the other projects in the same river basin, both upstream and downstream, which will allow any negative increments or incompatible intermediate contributions to be identified and consequently corrected.

The methodology used for determining the series of data must be described, with an explanation of how the flows were obtained for each period. When extra data are needed to fill gaps in the readings from river gauging stations, it should be stated in the methodology what studies were carried out, and the months for which the data were added should be flagged.

Studies of Minimum Flows

It is important to assess minimum flows as these values will be used in the studies for the filling and operation of the reservoirs and for defining the discharges downstream to meet the requirements for the multiple water uses and the environmental needs.

The minimum flows should be assessed by analyzing the daily mean flows statistically. Flow duration curves can be presented for identifying the characteristic values and calculating the occurrence probability of the flows and their duration.

When there is no data available, flows can be estimated by drawing correlations with basins of similar hydrological features for which there are data available, or by making an approximate analysis of the rainfall/runoff ratio.

Flood Studies

The only figures needed for designing the structures at this stage are the flood flows for the spillways and diversion works. A statistical analysis should be done of the daily extreme flows, whenever there are reliable records of these data. When such data are not available, the parameters required can be estimated by drawing a correlation with river basins of similar hydrological features for which data are available, or by making an approximate analysis of the rainfall/runoff ratio.

The probable maximum flood, which is required to design the spillway, is calculated as the maximum flow that would occur once in every 10,000 years. For the diversion works, it is normal to adopt the values corresponding to the peak flows with recurrence intervals of 25, 50 and 100 years. As a suggestion, two-parameter exponential distribution and Gumbel distribution can be used for determining flood flows.

Rating Curve in the Tailrace Canal

In the Preliminary Studies, a schedule must be drawn up of the field studies necessary to measure the discharge, inspect the river gauging stations, install staff gauges and carry out echosounding surveys of the topography, which will provide data for the rating curve in the tailrace canal to be used in the Final Studies.

When reliable data are not available for the Preliminary Studies, the water level in the tailrace canal of each project should be estimated, assuming this level as corresponding to a flow that is 10% greater than the mean during the critical period, or the maximum normal water level of the reservoir downstream, if this level is higher.

Studies into Sediment Transport and Silting in Reservoirs

Existing data, information and studies can be used to assess the silting characteristics of the reservoir and predict the useful life of the project, as well as to study what sediment control measures will be needed.

Once the data have been obtained, the next steps in assessing silting include determining the mean annual solid discharge at the project site, obtaining the reservoir's sediment retention efficiency and evaluating its apparent specific weight.

The **mean annual solid discharge** (D_{st}) at the dam site is the sum of the suspended load and bed load. Generally, the bed load does not leave the reservoir through the spillway or through normal discharge processes. Therefore, it is crucial to measure the bed load discharge, especially in the case of small and medium-sized reservoirs.

In Brazil, it is normal only to measure the suspended load discharge. Therefore, total solid discharge should be determined using a coefficient that takes into account the bed load discharge. This coefficient should be adjusted according to experience acquired in other studies.

The solid discharge measurements can be presented in daily or sporadic values. When the data are sporadic, a sediment rating curve is plotted, correlating solid discharge to stream discharge.

When there are no sedimentological data, a value can be adopted by using regionalized data. The *Guia de Avaliação de Assoreamento de Reservatórios*[2] [Guide for Assessing Silting in Reservoirs] contains procedures for regionalizing sediment data. Overall, it is recommended that the mean annual suspended load discharge data from river gauging stations within the river basin itself and/or from neighboring basins should be used in the analysis by correlating them with the respective drainage areas. Sediment load regionalization should be used with caution and must be confirmed by measurements taken *in situ*.

It is recommended that a calculation be made of the amount of solid material transported annually throughout the project's useful life (50 years) so as to foresee any alterations in the production of sediment in the basin arising from the action of man.

The **sediment retention efficiency** (E_r) of a reservoir is the ratio between the solid discharge that is retained in the reservoir and the total outflow discharge.

The **apparent specific weight** (γ_{ap}) of deposits in a reservoir varies with time due to compacting by the weight of the water and the weight of the sediment itself. In order to determine γ_{ap} the grain size of the material transported must be known.

Once the annual mean solid discharge - D_{st}(t/year), the sediment retention efficiency - E_r and the apparent specific weight - γ_{ap}(t/m^3) have been obtained, the annual sediment volume, S_a(m^3/year) can be calculated by:

$$Sa = \frac{D_{st} \times E_r}{g_{ab}} \quad (m^3/s) \tag{4.1.2.01}$$

2 Guia de Avaliação de Assoreamento de Reservatórios, ANEEL, 2000.

The silting rate, T_a(year) of a reference sediment volume V(m^3) in the reservoir can be calculated by:

$$T_a = \frac{V}{S} = \frac{V \times \gamma_{ap}}{D_{st} \times E_r} \quad (\text{year}) \tag{4.1.2.02}$$

For the purposes of this Manual, the reference sediment volume is the volume that corresponds to the highest of the following three elevations: the sill of the water intake or the sill of the headrace canal or the highest point in the structure designed for sediment retention, when there is one.

- Useful Life of a Project

The useful life of a project is the period of time before which sediment deposits start to interrupt the generation of electricity. This happens when the sediment deposits exceed the reference sediment volume.

In Inventory Studies, this is calculated by taking the volume of sediment for twice the project's useful life. As this is normally 50 years, the sediment volume is calculated for 100 years. So:

$$S_{100} = 100 \times S_a \ (m^3/\text{year}) \tag{4.1.2.03}$$

The volume of sediments in 100 years is compared to the reference sediment volume. The volume of sediments calculated should be less than the reference sediment volume. If it is not, steps should be taken to minimize silting in the reservoir.

Effects on Sediment Transport Downstream from Dams

The erosion effects downstream from a dam are primarily a function of changes to flows, reduced solid load transported and the reduced grain size of the bed load downstream from the dam. If this material has a larger grain size, the sediments with a smaller grain size flowing out from the dam will be transported further downstream, and a layer of coarser material will be left, called the "armor layer", which is practically stable. If the material on the river bed is fine, there will be more sediment transported downstream, until a new stable slope is reached for the bed.

Some formulas and models are available in the *Guia de Avaliação de Assoreamento de Reservatórios* that can be used for assessing the degradation the river bed will suffer based on the diameter of the armor layer or the length of the stable slope.

Sediment Control

If structures have to be designed for sediment control purposes, they should be analyzed and designed interactively when the project designs are decided upon (items 4.7 and 5.5).

4.1.3 Geology and Geotechnics

The geological and geotechnical studies help identify the location of likely dam sites and their associated structures, and are also used in the socioenvironmental studies.

Services

In the area of influence of future reservoirs, geological photo-interpretation should be carried out of radar and satellite images and aerial photographs to supplement existing geological and geomorphological maps or be used to prepare new ones. If necessary, the areas of future reservoirs should be inspected by land, water or air with a view to consolidating the data from thematic maps, identifying:

- general geological and geomorphological conditions;
- areas at risk of erosion, identifying present sources of erosion;
- areas with potential instability of the hillsides;
- areas with mineral resources, including traditional mining;
- areas where watertightness could be compromised; and
- areas at risk of natural or induced seismic shocks in the study area.

At this phase, a general geological survey of the area of influence should be undertaken with a view to providing inputs for future study phases.

The main **geological and geotechnical parameters** to be covered at the potential dam sites for design purposes and for preliminary estimates of civil engineering costs are:

- mean soil coverage until the top of the bedrock;
- predominant type of rock;
- general characteristics of the foundations;
- availability of natural construction materials, such as quarries, clay deposits, soil deposits, natural sources of sand and gravel deposits;
- susceptibility to landslides;
- preliminary assessment of silting potential of the reservoir's area of influence; and
- assessment of the degree of mining activity both upstream and downstream from the reservoir's area of influence.

Of particular importance for the dam construction cost estimates are the assessments of the state of the foundations. The foundations should be investigated first with a view to identifying and analyzing their surface geology. Depending on the findings of this analysis and the provisional layout for the structures necessary for the project, other expeditious surveys can be undertaken to confirm the hypotheses formulated. Two methods for these are auger borings and inspection pits. The results are presented in the form of geological and geotechnical sections for each potential dam site.

The estimates of natural building materials available – deposits of sand and gravel, rock and soil – are based on indications of their location, quantity and volume.

Supplementary investigations should be planned for the selected sites to be undertaken in the subsequent study phases.

4.1.4 Environment

The gathering of socioenvironmental data and information should supplement the information gathered at the study planning stage (Chapter 3), addressing the different synthesis components selected to represent the socioenvironmental system and helping in the analysis of the positive and negative impacts and the needs for Integrated Environmental Assessments.

The basic data required are set out per synthesis component in item 4.3. These data are both quantitative and qualitative. Most of the data required can be found in secondary sources (official databases, universities, research centers, etc.). However, the secondary information for some elements and/or components and in some regions should be checked in the field or against existing satellite images or aerial photos or by other means whenever there is not enough to build up a picture of the region for the analyses, and when it is indispensable for the studies to be undertaken.

When field trips have to be undertaken to obtain qualitative and quantitative data on a given aspect, they should only be done after the available sources of secondary data have been exhausted, so that a sufficiently accurate picture of the situation can be built up for the analyses and field trips to be clearly focused. The field work should target issues that have already been identified as necessary for producing the knowledge required.

In order to build up a picture of the region for each synthesis component, involving quantitative and qualitative information, there must be an analytical and interpretative aspect to the work, which means that experienced professionals must be included in the teams.

It is important that as of this stage in the survey, the team should work in integration using an interdisciplinary approach, as this will allow a rounded picture of the synthesis components to be built up and the inter-relationships between the elements in the socioenvironmental system to be identified.

The variables and parameters needed for the socioenvironmental characterization will alter with time and space in a given river basin. To identify the socioenvironmental aspects and their integration, an appropriate scale must be established to give a representation that takes into account most of the indicators.

The scale of the study must be such that the set of projects under analysis can be addressed in conjunction. Different scales from this can also be used to analyze topics and items of particular importance, using existing official maps. The data and information should be compatible with the scale of the study, and thematic maps should be prepared with a suitable scale for the socioenvironmental items of relevance and for the local assessment (e.g. subdivisions of the basin).

The scale to be adopted for the data analysis should be in keeping with a total view of the water basin. Other scales can be used to represent the findings, depending on the theme in question.

The spatial information must be stored in a geographical database that is compatible with the database kept by the concession-granting authority (MME).

4.2 MULTIPLE WATER USES

4.2.1 Diagnosis of Multiple Water Uses

The aim of the diagnosis of multiple water uses is to determine the historical record of withdrawals for consumptive uses for each project site under study, and also to estimate future restrictions on these projects imposed by the use of the waters by other sectors.

The idea is also to identify the potentialities for multiple water uses in the water basin under study and verify this potentiality against the provisions of the National Plan for Water Resources, state plans, water basin plans and any sector and integrated plans available.

Quantitative and qualitative information should be gathered for the diagnosis, most of which are available from secondary sources (table 3.1.3.01). When the secondary data are not up-to-date or are insufficient for the depth of analysis required, field studies should be carried out.

With a view to assessing whether there will be any need to consider the multiple water use scenario at the Preliminary Study phase, it should be identified whether the multiple water uses in the river basin under study could cause any major alteration to the choice or assessment of the different cascade options. This could be if the water use scenario for a given river basin indicates that the water availability is far below the natural flow, or there are major water level restrictions.

Based on the data collected, all the different water uses in the basin should be identified, including those that could have a major interference in electricity generation.

Consumptive Uses

The monthly flows for consumptive use at prospective project sites are the result of summing the flows of all water withdrawals, minus returned waters upstream from the site in question. The effective consumptive flows should be estimated for each of the following uses:

- urban water supply;
- rural water supply;
- livestock;
- irrigation; and
- industrial uses.

In order to determine the monthly consumptive uses, the diagnoses contained in any existing water resource plans should be taken into account. Alternatively, the methods to estimate flows for consumptive uses adopted by the National Interconnected System[3] can be used, or technical studies can be developed in-house. In this case, the methodology should be described and justified in the Inventory Studies report.

As an outcome of this work, the monthly consumptive flows for each prospective dam site under study are prepared, covering the same period as the records for the natural monthly mean flows, i.e. from 1931 until at least two years prior to the conclusion of the Inventory Studies. This means that it will often be necessary to estimate the water withdrawal volumes from years past and extrapolate them to the data obtained.

3 ONS, *Estimativa das Vazões para Atividades de Uso Consuntivo da Água nas Principais Bacias do Sistema Interligado Nacional.* FAHMA-DREER, 2003

If there are any sector plans or water basin plans, the information contained in them should be analyzed from the perspective of multiple water uses with a view to building up a long-term scenario.

For each use – irrigation, urban and rural water supply, livestock and industrial uses – information should be collected on consumption forecasts, identifying the areas where the water uses are concentrated, locating population areas and the places where water withdrawals are needed that will be benefited or hampered by each prospective reservoir.

Non-Consumptive Uses

When it comes to water uses that do not involve any withdrawals but which compete with hydropower generation, efforts should be made to identify where the areas of use are and the consequent restrictions imposed on the electricity sector for ensuring the efficient use of the waters. In general these uses are:

- navigation;
- flood control;
- tourism and leisure;
- aquaculture and fishing; and
- ecosystem maintenance.

If there are any sector plans or water basin plans, the information from these should be analyzed from the perspective of multiple water uses with a view to building up a long-term scenario.

This information includes:

- navigation: number of kilometers and location of sections covered in the plan and influenced by each prospective reservoir, historical freight transportation data, data on the number of people in the region, and programs devised to adapt the waterway infrastructure to meet these requirements;
- flood control: geographic area and location of urban and rural communities protected and benefited by each prospective reservoir and historical data on critical events in the region;
- tourism and leisure: location of tourist areas influenced by the introduction of each prospective reservoir;
- aquaculture and fishing: location and capacity of existing and planned projects that are influenced by each prospective reservoir;
- ecosystem maintenance: identification of sections and respective values of flows required to maintain the ecosystems.

4.2.2 Scenario of multiple water uses in the river basin

The scenario of multiple water uses should start to be built up during the Preliminary Studies, once the diagnosis has been done, and should be finished during the Final Studies. However, in the case of water basins with multiple water uses that could significantly alter the selection and/or assessment of the different cascade options, these studies should be considered at the Preliminary Studies stage. Examples of these are river basins where the scenario of multiple water uses makes it necessary to redesign projects or even rule out dam axes that would otherwise be included in the cascades.

In the Final Studies, the different cascades are compared against the long-term scenario of multiple water uses. Since for each projected scenario it is necessary to undertake several studies, it is acceptable to build up one single scenario, which should be the average or expected scenario and which will not take into account any extremes of abundance or scarcity.

This scenario is built up from a set of physical, social, economic and political data relating to a specific period for the purposes of a future study.

In order to prepare this scenario, the information obtained at the planning phase must be drawn on (items 3.1.1, 3.1.2, 3.1.3, 3.1.5 and 3.3), as must the information from the Preliminary Studies (items 4.1.1, 4.1.2, 4.1.4, 4.2.1 and 4.3).

For the electricity sector, this scenario should be prepared in order to address its long-term planning needs. However, as the National Plan for Water Resources (PNRH) is the basis for the use of all water resources, it is acceptable to prepare a scenario with a time frame that is compatible with the PNRH itself.

The scenario should, for each section of river in the water basin under study, specify the portions of flow and head affected by the multiple uses of the waters in such a way that energy generation is hampered, including:

- net flow losses due to water withdrawals for consumptive uses and diversions to other river basins, when applicable;
- flows uses in operating locks, when applicable;
- flows used for navigation;
- volumes for reservoirs and restriction flows for flood control;
- minimum flows needed downstream to meet multiple water use and environmental needs;
- potential restrictions on reservoir operation as a function of their use for tourism and leisure.

If the basin under study is already covered by a water basin plan that is in compliance with existing legislation, the creation of this scenario of water uses should take into consideration all the uses identified in the plan and projected uses in the future.

It should be noted that all the information, data and assumptions used in drawing up the scenario must be set forth clearly and transparently in a specific item in the final report, which will be used to assess the positive and negative socioenvironmental impacts of the final selected cascade.

Consumptive Uses

The series of consumptive uses obtained in item 4.2.1 should be projected forwards to the last year of the time frame for the long-term scenario. In the energy studies, the twelve values for this year are subtracted from the historic monthly natural flow records.

When building up the scenario of consumptive uses of the waters, a number of references should be used, such as:

- projections made by official entities, such as IBGE and IPEA;
- water resource plans, when available;
- official master plans for development (PPA [multi-year plan], state plans, sector plans, etc.);
- studies already undertaken to meet environmental requirements;
- studies or methodologies for estimating scenarios of consumptive water uses in the river basin, such as the methodology used by the ONS;
- maximum flows that water management bodies designate for other water uses.

A water balance should be prepared that shows the extent to which the waters are affected at each prospective dam site in the study area.

Non-Consumptive Uses

- Navigation

When incorporating navigation activities into the scenario of non-consumptive uses, all water resource plans, waterway navigation plans and different sector plans should be taken into consideration insofar as they influence in some way transportation on the river.

Based on these plans, projections should be estimated of the movements of freight and people on the waterways in the region, and any programs that have been devised as part of broader waterway infrastructure development plans to meet these needs should be consulted.

The sections of river should be identified that will be considered in the plans as waterways, along with the size of the vessels and traffic expected. All the projects under study along these sections should be designed in such a way that vessels can be raised or lowered to sections at different levels, when required.

When sections of river are identified as navigable either before or after the introduction of the prospective reservoirs, sector studies should be used to estimate the type and quantity of vessels to be transported between different water levels.

When flows are interrupted by locks to the point of influencing power generation capacity, calculations of these values should be made based on the type and quantity of vessels and the difference in water level, and presented in the form of a vector of 12 monthly values for each project for use in the energy studies.

- Flood Control

In order to determine the flood storage capacity of the reservoirs for flood control in the long-term scenario for the river basin, the first thing to do is to identify the main existing or planned areas of occupation that are subject to flooding by overspills from the main channel, whether they are upstream or downstream from the projects, during major floods.

Generally speaking, macro drainage plans provide for a set of local works to protect the most vulnerable areas by using reservoirs to reduce flood peaks. An analysis of existing plans and studies will help build up the scenario, which should take account of the total flood storage capacity to be allocated at each site under study.

In the Final Studies, for each cascade option, the total flood storage capacity for each flood control point should be distributed amongst the upstream reservoirs in proportion to the mean annual peak flow and the reservoirs' live storage, using the formula below:

$$Vesp_i = \alpha_i \, . \, Vesp$$

$$\alpha_i = \frac{\overline{Qp}_i \, . \, Vu_i}{\sum_j \left(\overline{Qp}_j \, . \, Vu_j \right)} \qquad (4.2.2.01)$$

where:

Vesp	total flood storage capacity;
α_i	coefficient of the proportion of flood storage capacity in reservoir i;
Qp_i	mean flood peak at the site of reservoir i;
Vu_i	live storage of the reservoir.

If there is more than one flood control point in the basin, then only the largest of the flood storage capacities needed to assure the protection of each flood control point should be taken account for each reservoir. Where the streamflow regime of the basin is seasonal, changes to the flood storage capacities should be considered for each reservoir, starting the rainy season with zero flood storage capacity, reaching the required value only in the month with the most rainfall, and returning to zero at the beginning of the dry season. A curve for the flood storage capacity of each reservoir throughout the year is then plotted.

In the Preliminary Studies, only the flood storage capacity of the month when the critical period of the reference system begins should be taken into account (item 4.6.1).

- Tourism and Leisure

  The scenario for tourism uses should be based on official information and take into account the fact that a new reservoir does not necessary imply in an influx of leisure users, as this depends on other agents which are often only private.

  This means that when reservoirs are planned for flow regulation or flood control, the tourism potentialities of the region where the reservoir would be built should be identified, as should any potential conflicts that might arise from the shared use of the reservoir banks and impounded waters. The potential restrictions on the operation of the reservoirs should also be identified as a result of their use for tourism.

- Aquaculture and Fishing

  When the scenario for aquaculture uses of the reservoirs is prepared, the sector plan and water basin plan should be consulted in order to gather the information needed to characterize the aquaculture facilities and to project their growth in the long term.

  Once the characteristics required for the installation of the aquaculture facilities have been established, their location at each planned reservoir can be estimated. Any potential conflicts over the use of the waters should be identified, in terms of their quality, access to the facilities and operation of the reservoirs.

  When the flows that are affected by the fish passage systems are significant to the point of influencing the energy generation capacity of the project, these values should be calculated from the sizes of the structures and presented in the form of a vector of 12 monthly values for each project, to be used in the energy studies.

- Ecosystem Maintenance

  In order to prepare the scenario for ecosystem maintenance, the ecosystem conservation and preservation requirements should be considered, as well as the needs of any traditional communities living downstream from the intervention in the water course.

  When building up this scenario, it is necessary to consider the minimum dilution flows necessary to ensure the body of water stays within the class in which it has been classified, and to minimize the environmental interference that could be caused by altering the streamflow regime.

  In order to calculate the flows necessary for maintaining the ecosystems, the regional water management entity should be consulted.

4.3 SOCIOENVIRONMENTAL DIAGNOSIS

The studies to be carried out as part of the socioenvironmental diagnosis aim to:

- supply the information required for formulating the different cascade options and designing the projects;
- build up a reference situation for assessing the positive and negative socioenvironmental impacts of the projects and cascade options;
- provide information necessary to characterize the main socioenvironmental features of the basin (the most sensitive areas, potentialities, main uses of the waters and land, socioeconomic aspects) which give a broad view of the most significant socioenvironmental impacts to be caused by the projects and cascades, highlighting the projects' cumulative and synergistic effects in order to be able to compare the different cascades and fulfill the requirements of the Integrated Environmental Assessment.

The studies should be conducted in such a way as to produce knowledge about the study area so that a reference situation can be created for the purposes of comparing the socioenvironmental impacts inherent to the different cascades. As such, the studies must aim to gain an overall understanding of the current reality in the study area, highlighting its past and future trends and providing a spatial representation of the most significant aspects required to make the comparative analysis. An integrated focus should be adopted in the studies, seeking to lay bare the processes of greatest importance in structuring the socioenvironmental dynamics of the study area over space and time.

It is not necessary to develop an in-depth characterization of the area for these studies. Indeed, the idea is to interpret and process secondary data and a sufficiently large set of primary data to build up a comprehensive reference situation in order to analyze the socioenvironmental impacts of the projects and cascades. This should enable the identification of the most significant socioenvironmental processes affected by the interaction of the project in the region and those aspects that should be looked into in greater depth. In particular, when knowledge of the study area is being built up, it is important to apprehend the views of the different social groups on the issues under analysis.

The diagnosis should be structured and organized according to the synthesis components, which are described in detail in items 4.3.1 to 4.3.7.

The studies are developed by consolidating, analyzing and spatially representing the data and information pertaining to each synthesis component, giving special attention to the issues already identified during the planning phase (Chapter 3). Also, as the synthesis components involve interactions between different elements from the socioenvironmental system, an interdisciplinary approach should be used to integrate the analysis of each synthesis component with that of the others in an overall attempt to comprehend the environmental processes in the study area. This means that the characterization of the synthesis components may take into account the fundamental inter-relationships between their elements.

As an input for the Integrated Environmental Assessment, the socioenvironmental diagnosis should highlight the past and future development trends of the region so that a scenario of its future development can be built up. Also, those socioenvironmental processes that are systemic or are deemed more significant from a regional perspective should be highlighted, while it is also necessary to have a reference situation for the analysis of each project. The following items should be addressed:

- the potentialities of the river basin: its natural resources, main socioeconomic activities, production trends, uses of the waters and land, landscape and tourism aspects, existing plans and programs for the region, and the socioeconomic potentialities that can be leveraged by the introduction of hydroelectric power projects in the region;

- areas under socioenvironmental management: better preserved areas with original vegetation, degraded areas, areas for biodiversity conservation, and areas with restrictions or conditions on their use, such as conservation areas or indigenous lands;
- areas of environmental sensitivity: areas that are most sensitive to the presence of hydroelectric projects should also be identified and located. Whenever possible, a classification should be created for different levels of sensitivity;
- existing and potential conflicts: related to the use of the waters and the soil, the biodiversity conservation strategies, and the policies, plans and programs that exist for the region's development.

  Potential conflicts are understood as problems that in one way or another would be worsened or would arise if a hydroelectric project were built, such as:
- conflicts brought about by the resettlement of urban and rural communities;
- substitution of land uses, breaking down of social relations and economic output;
- property speculation;
- interference in archaeological, historical and cultural heritage;
- areas of conflict over land use;
- interference in the natural resources available for development;
- loss of tourism potential;
- loss of natural resources (minerals, biodiversity);
- conflicts over the multiple uses of the waters (item 4.2);
- interference in indigenous lands and federal, state or municipal conservation areas.

Spatial Representation

As Inventory Studies involve making comparisons between different groups of projects organized in different ways inside a given water basin, the dimension of physical space is of great importance. Meanwhile, in order to fulfill the requirements of the socioenvironmental diagnosis, a reference situation must be built up that can be used for the analyses on two levels: of the projects and of the cascade options that are selected. In this sense, the results of these studies for each synthesis component should be represented spatially using the following procedures:

- georeferencing of the information from each synthesis component in the study area in order to make it feasible to integrate the characterization elements in a single map, making up a reference situation that can provide an understanding of the issues inherent to each synthesis component. The most sensitive areas and areas of conflict should be marked out, and in the case of the synthesis components related to socioeconomic aspects, the areas where the existence of any potentiality that could be harnessed by the introduction of the projects should also be marked out. The synthesis component maps and the thematic maps used to draw them up must be kept in the Geographic Information System (Sistema de Informações Geográficas);
- segmentation of the reference situation for each synthesis component in the study area into sub-areas by making an analysis of their similarities and differences. The sub-areas are continuous units of land which contain particular relationships and processes that mark them out from the others, and which determine their relationship with the dynamics of the synthesis component in the study area as a whole. The indicators and criteria used to segment the area into sub-areas must be given;
- weighting of each sub-area according to the importance of the processes by which it is characterized to the dynamics of the synthesis component in the study area as a whole. The weights are given according to the repercussions of the processes occurring in each sub-area on the study area, highlighting those

aspects that exceed the boundaries of a single sub-area. The weights are attributed on a scale of zero to one, and their sum should equal one. Given the specific features of each synthesis component and each river basin under study, different criteria can be adopted for the weighting of the sub-areas, though these must always be described and justified in the study.

This mechanism makes it possible to formulate a basis for identifying the impacts of each project and how it interacts with the synthesis components in each sub-area, while also giving an overview of the combined impacts of the projects in each sub-area and those that extrapolate the boundaries of these areas.

With this procedure, it is possible to formulate a reasonable basis for analyzing the impact processes inherent to each possible cascade option without, however, failing to address the most significant processes inherent to each project.

Results of the Diagnosis

The map of each synthesis component should be accompanied by a description that highlights the attributes that were instrumental in defining the boundaries of each sub-area, putting each one within the wider context of the study area and its interrelationships with the other sub-areas. This description should also highlight any aspects of note or particularly sensitive areas from a social or environmental perspective, as well as any potentialities that could be leveraged by the introduction of the projects and any existing or potential conflicts, all of which will be used in formulating the different cascades and the design of the projects.

At the end of the diagnosis, the analyses of all the synthesis components must be consolidated. Using an interdisciplinary approach, the interactions between the processes inherent to the synthesis components should be identified and investigated, building up the general scenario of the socioenvironmental conditions within the study area. When deemed necessary, these interconnections can also be represented on a single map (**synthesis map**).

The data, information and findings of the diagnosis are also fundamental inputs for the first stage of the Integrated Environmental Assessment.

The information produced at this stage should be inputted to the SINV system for ease of comparison and selection of cascade options at the end of the Preliminary Studies.

Synthesis Components

The synthesis components adopted to represent the socioenvironmental system are:

- aquatic ecosystems;
- terrestrial ecosystems;
- ways of life;
- territorial organization;
- regional economy; and
- indigenous peoples / traditional communities.[4]

As the physical processes and features provide the support and interaction between the environmental processes, they are not called synthesis components, but viewed as basic elements for the analyses of all the six synthesis components.

4 Decree 6,040 of February 7th 2007 – National Policy for the Sustainable Development of Traditional Peoples and Communities.

Likewise, all the historical, cultural, archaeological, speleological, landscape and ecological heritage is taken as a characterization element and consequently related to the synthesis components.

Items 4.3.1 to 4.3.7 below set out the theoretical framework and content of the physical processes and features for each of the synthesis components, as well as the characterization elements used to structure them. The tables provide a summary of these elements and indicate other sources of information.

4.3.1 Physical Processes and Features

It can be seen from the structure for the socioenvironmental system presented in items 2.3 and 4.3 that while the physical processes and features are not a synthesis component, they are nonetheless an essential element that works in conjunction with these components, in that they are what ensure the continuity and interaction of the biological and human relations.

While the main elements of river basins are their slopes and stream channels, they must be seen from a broader perspective as a complex system that contains processes of diverse natures which interact amongst themselves and which vary with time and space, forming what one might call a landscape unit.

This means that river basins can sometimes behave like a substrate for the occurrence and distribution of plant and animal species and sometimes like a resource and precondition for the development of human activities.

Given that the physical features are inevitably connected to the biological and socioeconomic features, priority must be given in the socioenvironmental diagnosis of the physical processes and features which make these interactions most evident. In this sense, the surveys and the depth of the analyses must be compatible with the contents of the synthesis components.

- **Geological Features** – The geological approach involves gathering and analyzing information that can help identify at least the following aspects: geological units and structures, associated lithologies and mineral potential. A correlation should be drawn between the geological evidence and the socioenvironmental segmentation of the study area. The criteria to be used for segmenting the area geologically are the strength of the materials, elements of the relief that impose some kind of restriction, and the existence of minerals. The information gathered for item 4.1.3 should be given priority in this process.
- **Geomorphological Features** – The main geomophological features and morphodynamic processes should be identified (dissection and deposition forms and processes). The geomorphological segmentation should prioritize the analysis of the different reliefs and processes at play, the degree of stability, erosion and deposition processes. The information gathered for item 4.1.3 should be given priority in this. In order to assist with the analysis of the river habitats, the processes that correlate to the main morphological features should be identified and detailed, such as altitude, channel slope, valley profile (flat-bottomed, V-shaped, U-shaped, wide floor), channel pattern (straight, meandering, braided), presence of rapids and white water, presence of islands, and sedimentation / erosion zones.
- **Pedologic and Edaphic Features** – The main soil types in the area should be identified, detailing their physical, chemical and structural features so as to identify their usage potential and limitations. The soils that are suited to agriculture and forestry should be identified, as should their susceptibility to erosion. The information gathered for item 4.1.3 should be given priority in this analysis.
- **Hydrology and Climatology** – This topic involves describing and characterizing the streamflow regime and climatic features of the study area, the surface waters and the ground waters (or available surface and ground waters). This characterization should be based on the studies from item 4.1.2.

- **Water Quality** – This element is important for the analysis of the following synthesis components: aquatic ecosystems, regional economy and ways of life. As such, the analysis should address the following elements:
- preservation of biological diversity – water quality indicators should be used to classify the bodies of water according to the ecological features required for ensuring the preservation of their biological diversity, as set out in item 4.3.2;
- water uses – the water quality indicators used should be capable of characterizing the water potablity and purity levels needed for domestic water supply and economic activities in general;
- water-borne diseases – the indicators for this item should identify sources of contamination by domestic wastewaters and the regional epidemiological profile.

The study of the water quality should start by identifying the main activities in the basin and any potentially polluting elements discharged into the water bodies. Many of these data can be obtained by consulting the secondary sources kept by environmental entities in some Brazilian states and from water quality entities, when available. However, the spatial location of the activities must be based on the land usage patterns that can be identified in satellite images. Land use should be mapped out using the same criteria as the studies for the Regional Economy synthesis component and the other synthesis components.

When there are no pre-existing water quality data or the data that do exist are insufficient to meet the objectives of this study, preliminary field studies must be carried out, including at least two campaigns at times chosen to capture significant variations that could affect the physical and chemical quality of the waters. These periods could be different depending on the biological and geographical features of the river basin or the human occupation of the region, but could be picked to represent the dry season versus the rainy season, the sowing season versus the harvesting season, etc.

The sampling points must be picked according to: the land use in the river basin; the physical characteristics of the river channels; the hydrological and hydrogeological features (including aquifers); the position of the prospective dams; the optimization of the sampling points in terms of their ease of access and evenness of spatial distribution.

Results of the Diagnosis

An analysis of the abovementioned elements is made, based on which a matrix of physical and natural interrelations is built up.

The association between the geological segmentation and the endogenous processes that formed the relief, superimposed on the external processes defined by the streamflow and climatic patterns, erosion areas, and sediment transport and deposition gives a picture not just of the interactions between these processes but also of the physical segmentation of the landscape that makes up the river basin.

When considering the Aquatic Ecosystem and Terrestrial Ecosystem synthesis components, the physical processes and features in the study area provide the basis for the biological interactions that take place within the landscape, and so are a central element in analyzing the biological and geographical segmentation of the study area, and are translated into details of the different habitats which jointly form the river basin.

When analyzing the Aquatic Ecosystems, the information about the morphology of the river, the altitude, channel slope, channel pattern, valley profile, presence of rapids and white water, presence of islands, sediment accumulation/deposition zones and geological substrate are cross-referenced. The habitats along the river channel should be segmented so as to highlight the physical characteristics of importance for evaluating the biological diversity. The water quality is subject to special analysis in the Aquatic Ecosystems synthesis component.

Physical aspects are of importance to the socioeconomic and cultural aspects in that they are one of the features that define how occupation processes develop and how resources are appropriated, and thus how the territory is organized.

The characterization and analysis of the physical features and processes for evaluating the **Ways of Life**, **Territorial Organization**, **Regional Economy** and **Indigenous Peoples/Traditional Communities** synthesis components is directly linked to the following factors: agricultural suitability under different kinds of territorial management, mining and landscape potentialities, and state of degradation of the resources. The analysis of these features is based on four fundamental elements: **morphodynamic processes** (dynamics of erosion and deposition), **flood dynamics**, **relief segmentation** and **physical and chemical properties of the soils**.

The **morphodynamic processes** represent the whole dynamics of erosion and deposition within the river basin, adding erosion processes to the stream processes. This makes it possible to assess both the effects brought about by the introduction of the projects, and any restrictions imposed on land use or economic activities.

The **flood dynamics** are important not only for understanding the morphodynamic processes, but also with regard to the social groupings that are directly connected to the river. They involve a whole range of social and economic relations that are fundamental to the social reproduction of these groups, which depend on the seasonal variations of the floodplains.

The aim of the **relief segmentation** is to highlight the particularities of the units, providing inputs that will be used in the analysis of the restrictions and potentialities impinging on territorial organization and occupation.

The **physical and chemical properties of the soils** are important for making erodibility analyses and identifying whether the soil is arable.

At the end of this analysis, enough data must have been gathered to characterize the following aspects:

- physical segmentation;
- arable soil and erodibility;
- mineral resources;
- geomorphological heritage; and
- water quality.

This information will be represented on thematic maps with a scale that is compatible with the maps for the engineering studies and maps of the synthesis components, as described below:

- **physical segmentation** – mapping the main features and processes in the natural physical environment, highlighting the particular features of the geomorphological environments with the respective relief segmentation; processes such as erosion and deposition; geological and pedological characteristics of the materials; correlated structures;
- **arable soil and erodibility** – mapping of areas of arable soil and degrees of erodibility;
- **mineral resources** – mapping of the mineral resources (in terms of their mineral potential);
- **geomorphological heritage** – including formations of outstanding beauty, caves, waterfalls, etc.;
- **water quality** – primarily the mapping of stretches of river with markedly different water qualities.

4.3.2 Synthesis Component: Aquatic Ecosystems

The Aquatic Ecosystems synthesis component encompasses a multiplicity of processes and relationships that take place within the biophysical environment. In view of the complexity inherent to any study of ecosystems, conceptual frameworks and methodologies must be developed that ensure the diagnostic analysis is compatible with the scale of the work at the stage of the Inventory Studies that has been reached, without, however, compromising the systemic content of this synthesis component. It was decided that the focus here would be on **environmental factors essential for maintaining biological diversity**, prioritizing those elements that allow for a spatial assessment, and taking as a reference any studies being developed within the area of biogeography.

This synthesis component draws on information relating to the physical and biotic structures and the biological data on species, so as to permit the **identification of the different levels of ecological significance that exist amongst the different habitats that make up the study area**. Ecological significance is understood here as the potential of the system under analysis to present a greater level of biological diversity or endemism than the other subsystems.

In this case, the choice of sub-units of analysis should be made in advance, selecting those areas that represent combinations of natural features, meaning that they contain the ecological processes associated with the functioning and structure of biotic communities. Within this guiding principle, it was decided to adopt the sub-basins as the sub-units of analysis in the drainage network (sub-areas), while understanding that they could be grouped together or kept separate depending on the characteristics of each area studied. The main channel should, however, be considered as a single sub-unit of analysis (sub-area). It can, however, be subdivided in situations where there is any physical interference in its course caused by a geographical barrier that gives rise to systems with independent ecological features.

The characterization elements selected for structuring this synthesis component are described below and shown in summary form in table 4.3.2.02, at the end of item 4.3.2.

Riparian Forest (riverside forests, riparian forests, floodplain forests, igapó forest[5])

Riparian forests are very important for regulating the ecological interactions between the terrestrial and aquatic ecosystems. Their state of conservation and the spatial distribution of the vegetation in the sub-basins within the study area and along the main channel should therefore be investigated.

This can be undertaken using remote sensing equipment at a compatible scale. Field reconnaissance work should be carried out to identify the levels of conservation.

Water Quality

The physical, chemical and biological parameters to be measured should be chosen bearing in mind two guidelines:

- they must allow the rivers' water quality to be measured for the characteristics that could compromise the biodiversity, such as dissolved oxygen, pH, nitrogen and phosphorous content, suspended matter, toxic compounds, heavy metals, phytoplankton, zooplankton and macrobenthos. These are not necessarily the same factors to be investigated when studying water quality for the purposes of public water supplies or for other uses that require higher levels of purity and potability;

5 Igapó forest: part of the Amazon forest that grows near rivers on low-lying ground that is permanently flooded. The trees are tall but have low branches, which means there are rarely any vines or shrubs. (Ref: NETO, E. F. Dicionário Prático de Ecologia, Ed. Aquariana, São Paulo, 2001).

- they must allow simplified models to be used to assess the water quality according to the regional specificities of the river basin under study, taking into account the existence of sources of pollution and their concentration.

The water quality should be classified in order to represent the relative levels of quality encountered, providing a range of water quality levels that illustrates the different biological conditions of socioenvironmental quality.

It is recommended that the sub-areas be classified into three different levels:

Class A – systems whose ecological features are not compromised and in which there is no pollution;

Class B – systems whose ecological features are compromised to some extent by interference from sources of pollution;

Class C – systems whose ecological features are greatly compromised by the intensity of the levels of pollution.

When possible, this classification should be compared with the CONAMA water classification.

If necessary, the water quality analysis may also look into the levels of dominance between indicator species and the build-up of metals in species from different levels in the food chain.

Stream Physiography

The physical variables deemed useful for characterizing the aquatic ecosystems are chosen for their capacity to indicate the variability of the habitats and their capacity to support these systems. These are: stream order, drainage density, physical diversity of the river habitats and presence of river pools.

- **Stream Order**: The relationship between the stream order and the abundance of ichthyofauna implies that in high-order streams there are aquatic communities with greater biodiversity than those in low-order streams. In order to obtain the features that determine the biodiversity, the order of each sub-basin and of the main basin must be known.

Stream systems can be classified differently, but studies that use this as a factor for understanding the ecology of the aquatic fauna have mostly adopted the Strahler method (1952)[6], which is also recommended in this Manual.

According to Strahler, the smaller stream channels with no tributaries are first-order streams; second-order streams arise when two first-order streams join and only receive first-order tributaries. Third-order streams arise from the meeting of two second-order streams, and can receive first- and second-order tributaries; fourth-order streams arise when two third-order systems join, and so on. This system of classification is shown in Figure 4.3.2.01.

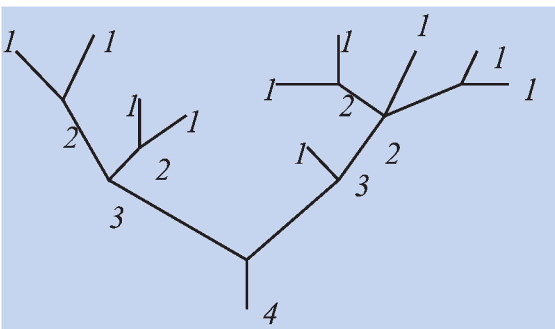


Figure 4.3.2.01 – Representation of Stream Orders in River Basins

6 STRAHLER (1952), "Dynamic Basis of Geomorphology", Geological Society American Bulletin, USA.

This means that each sub-basin should be classified according to its stream order and its status within the set of sub-basins under analysis.

- **Drainage Density**: This is the ratio of the number of confluences and the drainage area of each sub-basin.
- **Physical diversity of the main channel**: The correlation that exists between diversity of habitats and wealth of species leads to the expectation that main stream channels with a high diversity of habitats along its course will have fish communities composed of a greater number of species than could be expected from homogeneous rivers. The recognition of habitats along the river channel in each sub-area begins with studies of the physical features and processes, using variables such as altitude, channel slope, valley profile, thalweg profile, the presence of rapids and white water, the presence of islands, etc. These variables are directly related to the hydrodynamics of the system and the different kinds of processes that take place in the stream channels, and consequently to the aquatic fauna, determining the dispersion and spatial occupation of the species and influencing the structure of assemblages.

  The use of the Shannon index (S)[7] is recommended for generating a value to express the relationship between the different habitats in the sub-areas, as shown below:

$$S=\sum_{i=1}^{n} P_i.\log P_i \tag{4.3.2.01}$$

where:

P_i	percentage of the surface area of the sub-area that is occupied by each habitat identified;
n	number of habitats in the sub-area.

- **Heterogeneity of river habitats**: Once the different habitats that make up the main channel have been identified, it is important to observe the heterogeneity that exists in the drainage area associated to each one, as this is an important factor in assessing their capacity to sustain a high wealth of species. The analysis of this factor can be done by interpreting maps at a scale of 1:50 000 or 1:100 000, considering all the physical features mentioned previously for each habitat analyzed (stream order, drainage density, physical diversity of the river channel, etc.) so as to identify different scenarios of heterogeneity.
- **Ecologically Strategic Habitats**: In many situations, the biological diversity of a given region is enhanced by the presence of unique environmental conditions that operate by buffering impacts and maintaining different taxons and young fauna, while also allowing for the presence of endemic groups and the reproductive success of numerous species.

  These habitats of strategic ecological importance include river pools, river beaches, rivers with unique physiography, etc. The analysis begins by confirming the effective use of these habitats in the region under study. Once this has been done, the area or extension of land covered by these systems in each sub-area must be obtained, and its representativeness for the management of the local biodiversity must be assessed.

Biological Data

The focus of this item is on analyzing vertebrates, especially fish, by identifying the main species. Aside from the unquestionable ecological importance of ichthyofauna to river basins, the systemic features necessary for this group of fauna to survive make them excellent indicators of biodiversity.

Given the difficulties inherent to obtaining a biologically representative sample of wealth of fish species in river basins, it is suggested that the analysis of the biodiversity of fish species be done using

7 Shannon (1949) – "The Mathematical Theory of Communication", Urbana, University of Illinois Press, 117 pp.

secondary data that can be supplemented by field work designed to obtain the information listed in table 4.3.2.01.

Table 4.3.2.01

Classification	Geographic Distribution	Meso-Spatial Distribution	Environmental Distribution	Size	Migratory Habits
Species/Genus	Endemic	Headwaters	Still Waters	Large	Non-Existent
	Not Endemic	Lowlands	Running Waters	Medium	Optional
		Intermediate Sections		Small	Necessary

Although there may be many freshwater fish species that need to migrate upstream to complete their reproductive cycles, there is no need at this stage to do a detailed study of their migratory routes to the point of capturing species to tag them, or to study their stages of development and breeding grounds. Information on migratory routes and breeding grounds for the main characiform and siluriform fish can however be obtained in structured interviews with fishermen and riverside communities. This information, which is gathered in the field from people who traditionally use these resources, must be cross-checked against data obtained from the biogeographic analysis undertaken.

As mentioned before, in the areas where river pools are identified, eggs and larvae can be collected for the purposes of species identification. These data are easy to obtain and give a very precise idea of the breeding areas of rheophile species. By combining interviews with a biogeographic analysis and sampling from river pools, a characterization can be made of a satisfactory enough level for this stage of studies.

To supplement the abovementioned data, information should be gathered on fishing activities in the basins and the most productive areas identified. Fishing activities of significant local and regional importance are normally undertaken in fish-bearing rivers. Although this is an important economic activity and provides a source of subsistence for many Brazilian communities, there is a major dearth of reliable information on the real yield of fishing activities. However, it is possible to obtain data, albeit underestimated, for some basins on capture numbers, unloading sites and markets, workforce employed, etc.

In some basins it may be necessary to identify other groups of vertebrates (mammals, reptiles, birds) that could be impacted by the introduction of hydroelectric projects. The flooding of areas used for resting, feeding and breeding by birds, especially migrating species protected by international legislation, and the flooding of turtle breeding grounds, are examples of impacts that could take place. As these are exceptional circumstances, no specific prior study is proposed here. The technical teams involved in the Inventory Studies should decide under what circumstances studies of this nature need to be undertaken.

By cross-referencing the information on the physical diversity of the stream channels, the diversity of the river habits, and the habitats of strategic ecological importance with the biotic features, a representative picture can be built up of the likely fauna species in each river habitat. This makes it possible to identify those habitats where there is a greater concentration of biodiversity and/or where there are migratory, endemic or exclusive species.

Results of the Diagnosis

By collating the characterization elements, it should be possible to understand what factors are essential for maintaining the biodiversity of each sub-area. An integrated analysis of the physical and biotic elements to characterize them and determine their spatial distribution can be used to assess the different degrees of ecological significance existing in the different sub-areas.

In order to represent this component spatially, a map should be drawn up with the sub-areas marked out. Within each sub-area the most significant information from the perspective of ensuring its biodiversity should be represented. Annexed to this map, a characterization should be prepared of each sub-area that highlights its most significant features, points out any areas of sensitivity, and relates it to the other sub-areas and the river basin as a whole. The map will be used in the formulation of the different cascades and in identifying and assessing the impacts, and later in the IEA of the cascade selected.

The relative weights of the sub-areas, representing the importance of the processes they contain to the dynamics of the synthesis component in the study area should also be decided on at this point.

Table 4.3.2.02 – Characterization Elements of the Aquatic Ecosystems Synthesis Component

Synthesis component	Characterization elements	Source
Aquatic Ecosystems	• Riparian forests (riverside forests, riparian forests, floodplain forests, igapó forests, etc.): state of conservation and physical distribution. • Occurrence of macrophytes. • Water quality: – systems whose ecological features are not compromised and in which there is no pollution; – systems whose ecological features are compromised to some extent in response to the interference of sources of pollution; – systems whose ecological features are greatly compromised by the intensity of the levels of pollution. • Stream Physiography: – stream order (Strahler's Index); – drainage density; – physical diversity of the main river channel; – heterogeneity of river habitats; – ecologically strategic habitats. • Biological Data – biology and ecology of the most representative fish species (bibliographic data); – identification and distribution of main migratory routes, breeding grounds and feeding grounds; – identification and distribution of most productive fishing areas (volume captured – historic records, workforce employed); – identification of main species, especially migratory species and those associated to habitats with high hydrodynamics; – occurrence of other vertebrates (e.g. river-dwelling mammals, reptiles, birds).	• satellite images • existing maps and aerial • photogrammetric surveys • academic theses • scientific publications • data on water quality (environmental agencies, ANA, water management departments) • data on river fishing (IBGE, IBAMA, research institutes) • RADAM BRASIL project • general treaties on freshwater fish • supplementary field studies • SEAP • SRHU/MMA

4.3.3 Synthesis Component: Terrestrial Ecosystems

As with the Aquatic Ecosystems synthesis component, the complexity inherent to any study of terrestrial ecosystems makes it necessary to establish conceptual and methodological frameworks to bring the diagnostic analysis into line with the scale of the work needed for the Inventory Studies. This component will also focus on any environmental factors that are instrumental in ensuring the preservation of the biodiversity, prioritizing those elements that give a spatial assessment and taking studies being developed in the area of biogeography as a reference.

The characterization elements for structuring this synthesis component were selected in such a way as to help identify the different degrees of ecological significance that exist in the different habitats in the study area. Ecological significance is understood here as the potential of the system under analysis to present greater biological diversity than the other subsystems. The characterization elements used for this analysis are set out below and summarized in Table 4.3.3.02.

Vegetation and Land Use in the River Basin

The natural plant species in the basin should be identified and mapped out using remote sensing equipment and any pre-existing maps of vegetation or other maps or aerial photogrammetric surveys available. The interpretation and analysis should be done on a scale that will allow the interferences caused by the projects to be assessed. Depending on the size of the river basin, it may be necessary to prepare two maps: one with more systemic features, clumping together similar types of vegetation to try and get an overview of the level of degradation and conservation in the study area, and another looking into the physiognomic features of the flora in greater detail, including the diversity of the flora in each unit of analysis.

It is advisable for a field study to be undertaken to confirm the patterns mapped out and identify the extent and state of conservation of the vegetation. In particular, the extent and state of conservation of the riparian forests should be identified in view of their ecological importance in maintaining gene flow and as a habitat for many species in the terrestrial ecosystems.

The gathering and mapping of information on land use in the study area is carried out within the scope of the Regional Economy synthesis component.

Factors of pressure on ecosystems

In order to identify the factors that exert pressure on the ecosystems, data should be gathered on changes to the way natural resources are exploited and the expansion of farming lands. The studies should take as their point of reference the studies undertaken for the Regional Economy synthesis component, and are used in assessing the sustainability of these activities and the level of pressure exerted by man on the natural ecosystems.

Ecosystems of ecological interest

The ecosystems of particular ecological interest should be identified and mapped out. These are the most important ecosystems according to the function they exert in preserving biological diversity. They include: ecosystems that are important for maintaining population flows, such as riverside forests; ecosystems that support threatened species; ecotones, which serve as transition zones between two phytoecologically different regions, with species interpenetrated in their habitats; and conservation areas protected by law. In order to define these areas, information is used from remote sensing images, maps from the RADAM Project and from SIUC[8], and information on priority areas for biodiversity conservation.

Landscape Ecology

Information should be gathered from which an assessment can be made of the study area's capacity to maintain fauna species and the general level of insularity of the native vegetation. The extent to which the natural vegetation is insular is an indicator of any loss of biodiversity, as there is a strong correlation between biodiversity and area size. Thus, it is suggested that information be obtained by jointly analyzing the aspects relating to landscape ecology and the parameters relating to biodiversity.

There are two phytophysiognomic conditions of importance for preserving terrestrial fauna: (a) unaltered physiognomies that have maintained a high level of integrity and have therefore permitted the survival of the primary fauna species from the area; and (b) physiognomies distributed in patches which, as they provide for different forms of contact between different ecosystems, provide for the coexistence of more or less ombrophilic species.

8 Sistema de Informação de Unidades de Conservação (database on conservation areas).

The overall assessment of the landscape in each sub-area takes into consideration the following: the average form of remaining forest patches, the isolation of the patches, and the physiognomic classification of the patches.

- **Average form of remaining forest patches**: This parameter is used as an indicator of the capacity to sustain fauna, taking as a basis the principle of form and function developed by Thompson (1961). It is therefore expected that systems with a lower perimeter/area ratio have a greater capacity to retain their internal features (in this case, organisms) than systems when this ratio is higher. The average form of forest patches (FM) is given by the following relationship between perimeter (P) and area (Ap):

$$FM = \frac{\sum_{i=1}^{n} \frac{P}{2\sqrt{A\pi}}}{n} \tag{4.3.3.01}$$

where:

n	number of forest patches.

- **Isolation of forest patches**: By measuring the isolation of patches, represented by the distance between each remaining forest patch, it is possible to assess the level of insularity of the wildlife in a given sub-area. The isolation (**IM**) of each sub-area can be estimated by:

$$IM = \frac{1}{n}\sum d_{ij} \tag{4.3.3.02}$$

where:

n	number of forest patches in the sub-area, excluding riparian forest;
d_{ij}	distance between one patch i and its neighbor j in the sub-area.

- **Phytophisiognomic classification of patches**: this indicates the diversity of flora in the area under study. The IBGE classification is recommended for this. Any physiognomy that is found to be exclusive to a particular area should be highlighted.

Occurrence and distribution of animal species

Information about the likely occurrence of mammal, bird and reptile species in the study area can be gathered by combining data from expeditious field campaigns with any information available from secondary sources on neotropical fauna. This information is available in the specialized literature.

The assessment of the occurrence of animal species requires the information to be organized according to the categories described in table 4.3.3.01:

Table 4.3.3.01

Classification	Geographic Distribution	Spatial Distribution	Habitat	Status
Species/Genus	Endemic	Peripheral	Soil	Threatened
	Not Endemic	Central	Trees	Vulnerable
			Shrubland	Not Threatened
			Swampland	

This database structure covers a minimum number of aspects to be considered and encompasses taxonomic and ecological variables. It is recommended that new information be added so that ecological features found to be of relevance for better characterizing the wildlife in the area under study can be included.

When this data are related to the data from the physical characterization of the area, it will be possible to identify the likely occurrence of fauna in the different habitats in the study area. Attempts should be made to gather information on the taxonomic diversity of the vertebrates, the threatened species on official lists and the species which are more vulnerable to alterations brought about by man because they are restricted to particular forest patches.

Results of the Diagnosis

By collating the characterization elements, it should be possible to analyze the current status of the Terrestrial Ecosystems and understand what factors are essential for maintaining their biodiversity. The analyses will provide a spatial assessment of the elements required to ensure the continued biodiversity, which can be used to segment the study area into sub-areas according to criteria that give the most faithful representation of the biological processes and elements. These sub-areas may be correlated to a particular sub-basin, a landscape, a phytophysiognomic unit, or to a number of other aspects. Each spatial unit must be defined in such a way that it provides the greatest correlation between the elements and processes it is designed to represent.

In order to represent this component spatially, a map should be drawn up with the sub-areas marked out. Within each sub-area the most significant information from the perspective of ensuring its biodiversity should be represented. Annexed to this map, a characterization should be prepared of each sub-area that highlights its most significant features, points out any areas of sensitivity, and relates it to the other sub-areas and the river basin as a whole. The map will be used in the formulation of the different cascades and in identifying and assessing the impacts, and later in the IEA of the cascade selected.

The relative weights of the sub-areas, representing the importance of the processes they contain to the dynamics of the synthesis component in the study area should also be decided on at this point.

Table 4.3.3.02 – Characterization elements for the “Terrestrial Ecosystems” synthesis component

Synthesis component	Characterization elements	Source
Terrestrial ecosystems	• Phytophysiognomic description of the types of vegetation and land use in the river basin; •Factors that exert pressure on the ecosystems (logging, farming, ranching, deforestation); •Conservation Areas and other areas under legal protection, ecosystems of particular ecological interest, priority areas for biodiversity conservation, ecotones, areas containing rare or threatened species, ecosystems of importance for maintaining population flows (ecological corridors, biosphere reserves); •Landscape ecology (analysis of the form and connectivity of forest patches and their ecological representativeness for conservation of the species they contain); •Characterization of animal species per habitat and identification of endemic, threatened and rare species.	– satellite images – pre-existing maps of vegetation and land use – RADAM BRASIL project – existing maps and aerial photogrammetric surveys – farming census (IBGE) – INPRA, IBAMA, MMA, MAPA – academic theses – scientific publications – data on the development of deforested areas (IBGE, INPE, NGOs) – general literature on neotropical wildlife – supplementary field studies – state environment entities – EMBRAPA, EMATER

4.3.4 Synthesis Component: Ways of Life

This synthesis component encompasses the different ways human beings organize themselves to ensure their physical, social, political, cultural and emotional survival. It relates to the ways people occupy land, exploit the natural resources at their disposal, relate to each other in this process, and produce representations about the land they occupy. Particular world views are central to these forms of organization, as are the ways people view themselves (forms of representation). These are the formats that give meaning to the set of relationships that are re-confirmed on a daily basis – political, economic, cultural, affective, social relations, etc. It is not enough just to characterize a way of life to understand it; one must comprehend the forms mentioned here in their different manifestations, apprehend the most significant elements in the organization of a particular social group, and capture what gives it its particular identity, what gives it its particular place in space and time.

In order to apprehend the identity underlying a particular way of life[9], it is crucially important that a correlation be drawn between the social groups' survival strategies, which have to do with their material structure, and the historically constructed forms of sociality that form their socio-cultural structure. In this sense, the characterization elements selected to formulate this synthesis component must be analyzed in conjunction so that the different ways of life in the study area and the ways they are expressed spatially can be characterized.

These elements should be addressed qualitatively and quantitatively and interactions between them should be identified so as to (re)construct the reality that is under study. It is believed that by assessing this synthesis component it will be possible to address particular questions that are often lost in the midst of more easily quantifiable information; issues which qualify the social reality and, in most cases, tend to be overlooked.

The characterization elements are presented in table 4.3.4.01 and are clustered into the following aspects:

- demographic dynamics;
- living conditions;
- production system;
- social organization;
- institutions.

Demographic Dynamics

The aim of the analysis of demographic dynamics is to identify the behavior of a population by looking at its general features (resident population, sex, age, household status), how it is distributed spatially and its mobility, providing inputs for understanding the other aspects under analysis.

A good way of assessing population mobility is to look at the net migratory balance (difference between the growth rate and the vegetative growth rate), which is a trend indicator that reveals whether the area attracts or repels individuals.

Living Conditions

The analysis of living conditions involves assessing the public and private resources at the group's disposal to meet their basic needs, and the relationship between access to these resources and the available quality of life: healthcare, income, employment, education, sanitation, communication, energy, transport and leisure.

The main sources of information for these characterization elements are statistics provided by government agencies and international organizations such as the WHO or UNDP. However, this information should be supplemented by qualitative data that address the specific strategies employed

9 For instance, there are situations where "proximity to the river" is an important element for organizing time and space for some social groupings. It can help determine their collective identity and understand the combined relationships of which they are comprised, which would characterize a "river-dependent" way of life. However, proximity to the river is not always a determining factor in the collective identity of a social group; it may simply be one amongst many, and not even the main one.

by the social groups to improve their quality of life. A mixture of field studies and reference to academic papers, research and theses from universities and/or research institutes can be used to broaden the scope of information of a qualitative nature.

The Human Development Index (HDI), developed by the UNDP, can also help to formulate a situational diagnosis of the set of information concerning education, income and health (infant mortality and life expectancy).

In particular, when assessing a population's state of health, a situational diagnosis can be obtained alongside a description of the health/disease processes in the region by observing the disease profile (prevalence/mortality), the medical and hospital infrastructure and the endemic disease profile, while correlating these aspects with the socioeconomic indicators and main health indicators (infant mortality, life expectancy, consultations per inhabitant). By doing so, the factors and areas of risk for the people's health can be identified.

Production System

The following elements have to do with production, meaning the ways people and/or social groups produce the goods required to meet their material needs. This includes the ways they exploit natural resources and the way they undertake their production activities, which will indicate how the societies are organized and have developed historically. The elements should also provide the means to identify the patterns of capitalization and decapitalization in the social groups.

The different production systems in the study area are identified by observing how rural and urban output is organized and how they interact, taking into account the natural resources available and the environmental factors in the area.

The information on each of the dimensions within the production system can be researched in secondary sources and supplemented by consulting studies and theses on the topic, as well as gathering data from the field.

- **Natural resources and key environmental factors** – in view of the fact that the basis for the production system is the natural resources available for production purposes, it is necessary to observe the following: floodplains, arable land, flood dynamics, land uses, water uses, mineral uses, forest uses and fishing activities. This information can be obtained from the data gathered and analyses made about the physical processes and features in the study area, and from the following synthesis components: Aquatic Ecosystems, Terrestrial Ecosystems and Regional Economy.
- **Organization of production** – the ways urban and rural production is organized are indicative of the potential links between them, especially those that involve living in urban areas and working in the countryside, or vice versa. The ways production is organized are also indicative of the degree of capitalization involved, which can be observed from the ownership and value of equity, as well as the different forms of income generation.

The information pertaining to the organization of urban production can be gathered primarily from the IBGE census, especially the data on the economically active population and non-economically active population, sector of economic activity, condition and class of monthly average income. Censuses and data produced by states and municipalities should be used whenever they are available.

It should also be noted that in some rural areas, bartering is still common practice and there is little currency in circulation.

Social Organization

The ways people and/or social groupings are organized and define their parameters for coexistence should be observed. This has to do with the way people relate to each other. As this is a predominantly qualitative aspect, the sources of basic information are academic studies produced at universities and research institutes and field studies. It is also worth consulting lists of trade unions, cooperatives and NGOs published by IBGE.

In view of the specific nature of the topics covered in this element, its main function is to characterize how people's relationships are mediated, pointing out situations of conflict, informal associations, and any special relationships that may explain the way certain groups are organized, such as family ties, neighborhood relations, work relations, political relations, etc.

By observing this set of aspects and information about the historical occupation of the land, the main elements can be identified that form the group's socio-cultural identity, its forms of sociality (relationships within itself) and the way it is represented. It is also possible to identify how these forms of sociality are expressed in space (land) and time, giving precedence to certain forms of mediation and/or certain concrete references (geomorphological heritage, buildings, monuments, etc.), by which process they become representative heritage sites for that social group.

When it comes to historical, cultural, landscape, architectural, speleological and ecological heritage, what matters here is the value each social group attributes to it rather than any official recognition or legal status it may have. The importance and significance of each heritage site should be characterized as part of the process of cultural production and reproduction of the social groups. By doing so, the meaning of the heritage to the social group to which it belongs is highlighted, and this can be offset against its historical, cultural, landscape, architectural, speleological or ecological value to the whole set of relationships that give it its identity.

Institutions

The main public entities should be identified, as well as any forms of organized civil society and interest groups operating in the study area. All public policies for social welfare and poverty reduction in the region should also be analyzed.

Results of the Diagnosis

By collating the characterization elements relating to the historically constructed survival strategies and forms of sociality, it should be possible to identify the "ways of life" existing in the study area and apprehend what underlies their identity. It is also important to observe any vulnerability to change in the forms of social reproduction brought about primarily by the existence of situations of contradiction or conflict, conditions of capitalization and decapitalization, and the degree of social organization of the groups.

In order to represent this component spatially, a map should be drawn up collating the information from each characterization element, and this should be analyzed together with the maps containing the information about the physical processes and features in the region and the other synthesis components, in order to mark out the areas in which each way of life exists.

The resulting map should have the sub-areas marked out according to the way the space is occupied by the different ways of life identified. Annexed to this map, a characterization should be prepared of the way of life in each sub-area that highlights its most significant features and relates it to ways of life in the other sub-areas and the study area as a whole. The most sensitive areas to the introduction of new projects should be highlighted, as well as any with potentialities that could be harnessed by the projects. The map will be used in the formulation of the different cascade options, in identifying and assessing the impacts, and later in the IEA of the cascade selected.

The relative weights of the sub-areas, representing the importance of the processes they contain to the dynamics of the synthesis component in the study area should also be decided on at this point.

Table 4.3.4.01 – Characterization elements for the Ways of Life Synthesis Component

Synthesis component	Characterization elements	Source
Ways of life	• Demographic Dynamics: – occupation process (emphasis on demographics); – quantitative population data; – spatial distribution of the population (rural/urban households); – growth rates; – natural growth rates; – migration flow; – factors that attract or repel population flows. • Living Conditions: – quality of life (HDI and other basic indicators); – services available (education, health, energy, communication, sanitation, transport and leisure); – employment conditions, household income distribution, personal income distribution; – analysis of the living conditions of the groups and smallholders; – health indicators, especially water-borne diseases. • Production System: – organization of rural production; – organization of urban production; – mineral, pedological, water, forest and fishing resources available; – environmental factors in the sub-area (flood dynamics, floodplains, areas of erosion, suitability for agriculture, relief segmentation). • Social Organization: – historical occupation processes; – socio-cultural identity (habits, values, beliefs, historical/cultural heritage); – representations; – areas of conflict and social tension; – situations of conflict; – organization of space / time; – forms of sociality. • Institutions: – public entities operating in the area; – forms of civil representation; – interest groups; – vulnerability of local communities to changes brought about by modernity; – public policies for social welfare and poverty reduction in the region.	– Demographic Census (IBGE) – List of health establishments (IBGE) – Health statistics (IBGE) – State Statistics – Farming Census (IBGE) – FUNAI – EMBRAPA – EMATER – INCRA – SEPPIR – MDA – MDS – List of trade unions, cooperatives and NGOs (IBGE) – Social movements and associations that are active in the region – Direct Research – Academic papers, theses and studies

4.3.5 Synthesis Component: Territorial Organization

This synthesis component covers the processes that determine territorial organization and dynamics and consequently the landscape and occupation patterns. It encompasses the man-made forms and objects that are erected in the land and their interrelations, which can be used to understand how the land is used and occupied and how the different parts are interrelated by communication networks and the circulation of goods and people.

The selected characterization elements are designed to gather information on the way the space and the landscape are organized, highlighting the role played by the water bodies in this organization on the circulation fluxes, communication, and the political and administrative organization of the land. This information is presented in summary form in table 4.3.5.01 and organized into the following aspects:

- demographic dynamics;
- land occupation;
- circulation and communication;
- political and administrative organization; and
- territorial management.

Demographic Dynamics

When it comes to demographic dynamics, the aim is to analyze the following two aspects:

- Development of urban and rural populations per municipality.

The behavior of the population should be observed in order to identify the dynamics of population growth per se, emphasizing the contribution of migration flow. The indicators that tend to be used for this are: urban, rural and total growth rates, average geometric rate of annual growth and net migration balance.

- Structure and spatial distribution of urban and rural populations per municipality.

These elements allow an analysis to be made of the spatial distribution of the population, their mobility and the resulting patterns of urban occupation. The statistical indicators that tend to be used for this are: demographic density and level of urbanization.

Land Occupation

The dynamics and factors at play in land occupation should be investigated, and any factors that are related to the water resources should be highlighted. This should be done by observing the following factors:

- Historical process of land occupation in the river basin.

The information to be analyzed should build up a picture of the main processes involved in land occupation processes in the area, the agents responsible for this and the forms of land appropriation used. Local and regional secondary sources as well as interviews are the main sources of information.

- Environmental factors which induce or restrict land occupation.

The area under study should be analyzed to identify any factors that affect human occupation (a) of a restrictive nature, such as areas of erosion, steep slopes, areas that are flooded or prone to flooding, and areas occupied by special facilities (e.g. military facilities, existing power plants, etc.); and (b) of an inductive nature, such as areas set aside for urban occupation that have already been plotted out, areas where roads are to be built, areas supplied by transportation services, areas that are suitable for farming, and areas for new farming settlements.

This information can be taken from the surveys undertaken for the diagnoses of the Physical Processes and Features, Aquatic Ecosystems and Terrestrial Ecosystems, and by consulting government agencies.

- Characteristics and spatial distribution of different kinds of land use and their intensity of use.

The different land uses must be identified and located. Broad categories of land use (e.g. urban, rural) can be taken as a starting point for distinguishing subcategories, whose basic features and intensity of use must be defined. It may be useful to cross-reference the uses actually established with the uses proposed by plans and existing legislation. The expansion trends of urban centers, sanitation infrastructure, communication infrastructure, housing infrastructure, highways, waterways and railways should be analyzed.

Statistical and cartographic information should be gathered from local and regional government agencies on land use, harvests, areas used temporarily and permanently for growing crops, and the extraction of timber and non-timber forest products.

The cartographic information will mostly come from remote sensing images, which will need to be interpreted and analyzed and used to build up a historical record of land occupation. Municipal master plans and territorial organization plans are another rich source of information, and all of this should be supplemented by data gathered in the field.

- Function of water resources in territorial organization.

In order to examine the role of the waters in the local and regional context of the river basin, the physical distribution of the waters should be considered, as should their role in the circulation of people and goods and in the ways the land in the river basin is structured, and the effective uses of the waters.

This analysis is essentially qualitative and interpretative, which means it must be referenced against the historical and regional context and government policies for the region.

- Main water uses and estimate of user numbers per type of use.

The main water uses should be identified and included in a map of an appropriate scale, and the number of users for each type of use should be estimated, highlighting any existing or potential conflicts. After identifying the areas where the greatest concentration of users and agents are located, the causal relationships should be highlighted.

This information, of a primarily qualitative and interpretative nature, must also be gathered in interviews with local and regional agencies responsible for urban, social, environmental and water management.

The information must then be correlated with the information used to build up the scenario of multiple water uses prepared in item 4.2.

- Urban and rural relationships and resulting settlement patterns.

The settlement patterns and relationships between town and countryside specific to the region under study should be identified, using IBGE studies of regional divisions and functional urban regions.

- Existing and planned development programs.

All public and private investments that are either planned or already underway that have a significant impact on local or regional development must be identified and located. The main information of this nature can be gathered from local and regional government agencies and should be supplemented by interviews.

Circulation and Communication

The main flows of people, goods and services within the river basin should be characterized, along with their respective infrastructure and large-scale facilities and the role they play in territorial organization.

The main flows and the directions of these flows should also be represented. The following aspects should be considered:

- Location and characterization of urban areas: diversity and functional hierarchy.

  The main urban centers should be located and the capacity and range of their production, consumption and service facilities should be analyzed. This includes local and extra-local large-scale facilities, such as: storage facilities for farm produce and merchandise prior to their transportation; health, education and interurban passenger and freight transport service providers; bank, credit and financing establishments; leisure facilities of importance beyond just the local area; areas that supply vegetable and fruit produce and other merchandise. It should also cover cooperatives, religious institutions and government institutions that provide services.

- Location, characteristics and relative importance of the highway, waterway and railway systems.

  The functions carried out by roads, railroads and sections of the water courses used as waterways must be identified, mapped out and qualified. Most of the information required for this can be gathered from maps produced by federal, state and municipal government entities.

- Origins and destinations, and integration between forms of transportation.

  The main origins and destinations of the most important routes of people and goods and their respective modes of transportation should be identified. It is necessary to map out the points where the different forms of passenger and cargo transport interconnect (within the larger highway, railway, waterway and airway systems), their capacity and size.

  Most of the information can be obtained from the public entities in charge of administrating the transportation systems, covering volume of traffic, cargo flows, number of passengers per period, volume of cargo per period.

Political and administrative organization

Aspects of direct public administration should be examined (especially on a municipal level) and simultaneously related to the land and population in question. The following should be included:

- location of municipal and district seats of government;
- municipal land covered by the river basin and its ratio to the total surface area;
- location and coverage area of main municipal, state and federal public institutions.

The main offices of the local and regional public entities and the services provided by municipal entities in each administrative district should be listed and located.

- Constituency and representation of municipal, state and federal governments.

  For each municipality, the number of voters and their ratio to the total population should be identified, as should the total number of local councilors and their proportional representation in terms of numbers of state and federal congressional representatives.

  This information can be gathered from local authorities and the regional electoral commission (Tribunal Eleitoral Regional).

Territorial Management

All public policies and legislation pertaining to local and regional development should be identified with a view to characterizing how the political and institutional aspects are interrelated. The municipal, state and federal plans, programs and projects for social and economic development must also be examined.

A study should be made of the main public, private and third sector social agents of significance in the study area.

The most important sources of information are documents produced by the Ministry of Planning, the Ministry of National Integration, the Ministry of the Environment, the Ministry of Agriculture, the Ministry of Social Development and state departments of planning.

Results of the Diagnosis

By collating the characterization elements, it should be possible to understand and characterize the processes instrumental in determining the territorial organization and its occupation patterns. By identifying the levels of urbanization, the presence of urban centers with the capacity to polarize activities, road infrastructure, facilities for the circulation of goods and people, the maintenance of relationships of exchange and/or dependency with other regions, it is important to perceive which are the structural processes within this organization.

The following information should be mapped out in order to provide a spatial representation of the Territorial Organization synthesis component:

- political and administrative boundaries, municipal and district seats of government;
- population density in the municipalities;
- relative distribution and relative growth of the urban population;
- urban centers, functional hierarchy and level of urbanization;
- existing and planned highways, waterways and railways;
- origins and destinations of main flows of goods and people; integration between forms of transport;
- dominant patterns of land use and land occupation;
- intensity of occupation of farming land;
- occurrence of large-scale facilities and capacity to supply areas beyond the local area (silos, warehouses, health facilities, storage, etc.); and
- existing and planned large-scale projects for farming, industry and the extraction of non-timber forest products.

The resulting map should have the sub-areas marked out with a view to classifying the study area according to its level of integration. This integration can be observed by analyzing all the information gathered in conjunction. The following categories can be used:

- areas of incipient integration (low level of urbanization, poor levels of accessibility);
- areas in transition (proximity to roads, occurrence of activities that indicate potential for integration, growing level of urbanization, some integration between modes of transportation);
- integrated areas or areas of consolidated integration (high level of urbanization, urban areas with the capacity to centralize activities, good levels of accessibility, facilities on a scale to meet supra-local requirements).

Annexed to this map, a characterization should be prepared of each sub-area that highlights its most significant features and relates it to the other sub-areas and the study area as a whole. The most sensitive areas to the introduction of new hydropower plants should be highlighted, as well as any with potentialities that could be harnessed by the projects. The map will be used in the formulation of the cascades, in identifying and assessing the impacts, and later in the IEA of the cascade selected.

Table 4.3.5.01 – Characterization Elements of the Territorial Organization synthesis component

Synthesis component	Characterization elements	Source
Territorial organization	• Demographic Dynamics: – development of urban and rural populations per municipality; – structure and spatial distribution of urban and rural populations per municipality; – significant importance relative to total population; – level of urbanization. • Land Occupation: – historical process of land occupation; – environmental factors which encourage or restrict land occupation; – characteristics and spatial distribution of different kinds of land use and their intensity of use; – function of water resources in the territorial organization; – main water uses and estimate of user numbers per type of use; – urban and rural relationships, and resulting settlement patterns; – assessment and location of historical and cultural heritage and the main archaeological, paleontological and speleological sites; – existence of conflicts over land or water use; – existing and planned development programs. • Circulation and Communication: – location and characterization of urban areas: diversity and functional hierarchy; – location, capacity and reach of production, consumption and service facilities; – location, characteristics and relative importance of the highway, waterway and railway systems; – origins and destinations, and integration between forms of transport. • Political and Administrative Organization: – location of municipal and district seats of government; – constituency and representation of municipal, state and federal governments; – area covered by municipal land and its ratio to the total surface area; – location and reach of main municipal, state and federal institutions. • Territorial Management – political and institutional coordination: information on public policies and legislation for local and regional development – municipal, state and federal plans, programs and projects for social and economic development; – main social agents operating in the river basin (NGOs, churches, associations, etc.).	– Census of Demographics, Trade and Services (IBGE) – National Domestic Census (IBGE) (Pesquisa Nacional por Amostra de Domicílio) – Anuário Estatístico do Brasil – IBGE – Farming Census (Censo Agropecuário) IBGE – Municipal Agricultural Produce (Produção Agrícola Municipal) – IBGE – Municipal Ranching Survey (Pesquisa da Pecuária Municipal) – IBGE – Timber and Non-Timber Forest Products (Produtos de Extração Vegetal e Silvicultura) – IBGE – Brazil's Cities and Towns (Cidades e Vilas do Brasil) – IBGE – Division of Brazilian Territory (Divisão Territorial do Brasil) – IBGE – Division of brazil into Homogeneous Microregions (Divisão do Brasil em Microregiões Homogêneas) – IBGE – Division of Brazil into Functional Urban Regions (Divisão do Brasil em Regiões Funcionais Urbanas) – IBGE – List of Special Areas (Cadastro de Áreas Especiais) – IBGE – Municipal Master Plans – University Research and Theses – State Statistics – FUNAI – INCRA, SEPPIR – MMA, INPRA,OEMAS – federal and state conservation areas – Highway maps – Landsat and Spot Images – Environmental and Land Occupation Maps (Mapeamento Ambiental da Ocupação de Terras) – EMBRAPA – List of Health Establishments (Cadastro de Estabelecimentos de Saúde) – IBGE – Ministry of Finance, Internal Revenue Service – State and Municipal Departments of Finance

4.3.6 Synthesis Component: Regional Economy

This synthesis component covers the economic activities of particular importance to the economy and quality of life in the study area and the environmental resources that represent potentialities for supporting future economic activities.

The characterization elements selected for this component are designed to organize and interpret the information in such a way that an economic profile can be built up on a local and regional scale of both the market and subsistence activities, giving a general idea of what activities support the economy in the region in which the study area is situated.

The main assets and economic activities should therefore be identified, and the following aspects should be addressed, which are also summarized in table 4.3.6.01:

- economic activities;
- potentialities of the river basin; and
- municipal finances.

Economic Activities

The economic activities (market and subsistence) that best represent the economy in the region and quality of life of its inhabitants should be listed. However, rather than carrying out a traditional economic analysis, the objective is to:

- build up an integrated view of the economic activities in the study area;
- identify and qualify the relationships established with the natural resources;
- select quantitative indicators for the most significant activities;
- identify and locate within the area under study those activities of importance to the economy that impact on the local residents' quality of life;
- identify establishments and their areas of concentration involved in maintaining a given standard of living (e.g. food industry, essential sectors responsible for employment and income). In other words, the relationship, type and spatial location of these support goods and establishments should be identified;
- identify the economic activities that are directly connected to the river, either because of some functional link or because they are within the drainage basin of the future reservoirs;
- identify the factors that impinge on the location of the main economic activities, and their physical and spatial relationships with the suppliers of inputs and consumers (chronological development).

The characterization should be done on two levels: general and per sector. The general characterization should give a comprehensive overview of the economy in the study area. It should highlight the activities involved in the primary sector, especially those that have any connection with the waters.

The sector characterization should be done after the general characterization has been completed, providing a more in-depth, detailed perspective on the activities considered of especial importance to the study.

The characterization elements for the economic activities should quantify and correlate the following information:

- Production Structure:
  - primary sector: land structure, type of output, number of establishments, number of establishments, workforce, value of output, land area occupied, mining (number of mines and seams being exploited);
  - secondary sector: number of establishments, workforce, gross value, gross value and value of industrial output, development, relations between main industries and sectors;
  - tertiary sector: number of establishments, workforce, total revenues, VAT and service tax revenues (ICMS and ISS).
- Characteristics, capacity to create jobs and income and spatial location of the main industries and establishments.
- Economic activities involved in assuring the standard of living of the local residents (e.g. food industry and industries that require a large workforce).
- Markets served.
- Economic activities related to the water resources; number of people affected per economic use.
- Forms of resource appropriation (intensive/extensive, level of mechanization).
- Social and economic importance of the activities.

This information can be taken from secondary sources. For data on revenues from the circulation of goods and rendering of services, the main sources are the Ministry of Finance, the Internal Revenue, and state and municipal departments of finance.

Primary data must also be produced, which trade unions and cooperatives can help provide.

Resources and potentialities of the river basin

Any environmental resources with potential economic worth that could provide support for future economic activities (potentialities) must be identified, qualified and mapped out. Effective and potential economic uses of the waters must be highlighted.

- Characteristics and respective spatial location:
  - mineral resources;
  - areas of agricultural potential;
  - potential for energy, timber, fishing, non-timber forest products, biological products, genetic products and tourism;
  - species of economic, medicinal and nutritional value;
  - potential and effective uses of the water resources; number of people affected per use;
  - existing and planned development programs; and
  - planned and existing infrastructure and roads.

It is worth noting that the heritage-related aspects (historical, cultural, archaeological, speleological and ecological) are addressed here for their tourism and leisure potential, which is a reflection of their economic potential.

- Environmental factors that induce or restrict economic activities and factors that exert pressure on natural resources.

The analysis of these elements is overwhelmingly qualitative and is based on the information gathered on the physical processes and features, the aquatic ecosystems, terrestrial ecosystems, territorial organization and ways of life.

Municipal Finances

The economic and financial dimension of municipal administration can be assessed by consulting information about public revenues (municipal finances). It is worth identifying the municipal revenues that are directly related to the economic activities, the local population and the land in the municipality in question. The following should be studied:

- municipal tax revenues: taxes and improvement contribution (*contribuição de melhoria*);
- union and state revenues: the revenues to be considered should be those which are directly related to the area in question and the people living there, especially the funds received from the Municipal Fund (*Fundo de Participação dos Municípios*).

It is important to detect any gaps in the information, which will have to be filled for a clearer picture to be built up of any particular issue.

Results of the Diagnosis

By collating the characterization elements, it should be possible to gain an integrated overview of the existing activities and potentialities inherent to the economy of the region where the study area is, and to identify its structural elements.

In order to represent the Regional Economy component spatially, the occurrence of the main elements involved in the different economic activities (areas of production, areas where establishments are concentrated, occurrence of large-scale projects), support resources and potentialities should be compiled. One or more maps can be prepared for this purpose, containing the following:

- areas of farming and ranching;
- large-scale existing or planned farming or forest product extraction projects;
- existing or planned industrial areas and industrial districts;
- areas of agricultural potential;
- areas where the tertiary sector is concentrated;
- areas of original vegetation;
- occurrence of mineral, energy, timber, non-timber forest products, biological or genetic resources; species of economic, medicinal or nutritional value;
- markets served and their relative importance to the local and regional markets;
- areas of tourism and leisure interest; and
- areas under legal protection (conservation areas, indigenous lands).

An overview of the water uses should also be mapped out, indicating the areas of concentrated water use, location of agents responsible for consumptive uses, any points of conflict, and flows of commercial and non-commercial navigation. This information should be brought into line with the information gathered for the Territorial Organization synthesis component (item 4.3.5) and used in building up the scenario of multiple water uses (item 4.2).

The resulting map should have the sub-areas marked out as defined in the introduction to item 4.3, in function, for example, of the occurrence of similar production structures, homogeneous consumption patterns, similar workforce distribution, concentration of economic activities and/or abundance of a given natural resource with economic potential. The most sensitive areas to the introduction of new projects should be marked out, as well as those that offer potentialities that could be harnessed with the introduction of a new project.

Annexed to this map, a characterization should be prepared of each sub-area that highlights its most significant features and relates it to the other sub-areas and the study area as a whole. The map will be used in the formulation of the different cascades, in identifying and assessing their impacts, and later in the IEA of the cascade selected.

The relative weights of the sub-areas, representing the importance of the processes they contain to the dynamics of the synthesis component in the study area should also be decided on at this point

Table 4.3.6.01 – Characterization elements for the Regional Economy Synthesis Component

Component	Characterization Elements	Source
Regional economy	• Economic activities (general and sector characterizations): – description, job and income creation capacity and location of main industries and establishments; – production structure; – primary sector: land structure, economic output, number of establishments, economically active population (EAP), people in employment, value of output, land area occupied; – secondary sector: number of establishments, EAP, people in employment, gross value, value of industrial output, chronological development of main industries and sectors; – tertiary sector: number of establishments, EAP, workforce, total revenues, VAT and service tax revenues (ICMS and ISS); – economic activities involved in assuring the quality of life of the local residents (e.g. food industry and sectors that require a large workforce).; – economic activities related to water resources; – forms of appropriation of resources (intensive / extensive, level of mechanization); – markets served and social and economic importance of the economic activities . • Resources and Potentialities of the River Basin: – Characteristics and respective spatial location; – mineral resources; – areas of agricultural potential; – potential for energy, timber, fishing, non-timber forest products, biological products, genetic products and tourism; 1) species of economic, medicinal and nutritional value; 2) effective and potential uses of the water resources; 3) planned and existing infrastructure; 4) existing and planned development investments and programs; – environmental factors that induce or restrict economic activities and factors that exert pressure on natural resources. • Finances – collection of municipal taxes; – share of federal and state tax revenues.	– Industry, Trade, Services and Farming Censuses – IBGE – Demographic Census – IBGE – Inventory Study (Pesquisa de Estoques) – IBGE – National Study of Basic Sanitation (Pesquisa Nacional de Saneamento Básico) – IBGE – Municipal Livestock Produce – IBGE – Municipal Agricultural Produce – IBGE – Timber and Non-Timber Forest Products (Produtos da Extração Vegetal e Silvicultura) – IBGE – Annual List of Information on Society (RAIS) – IBGE – Register of Plant Species and Products of Economic Importance (Cadastro de Espécies Vegetais e Produtos de Importância Econômica) – IBGE – survey of natural resources (RADAMBRASIL) – IBGE – economic indicators – FGV – register of special areas – IBGE – road maps – satellite images – university papers and theses – state and municipal departments – National Plan for Water Resources – ANA – EMBRAPA – EMATER – MAPA – SEAP – IBAMA – MDA – INPRA

4.3.7 Synthesis Component: Indigenous Peoples / Traditional Communities[10]

The aim of this synthesis component is to highlight the presence of groups that merit special treatment as they are protected by federal legislation for their specific cultural features. It is designed to help gain an understanding of how these groups are organized and how their social and cultural reproduction takes place. In this sense, the approach is similar to that used in the Ways of Life synthesis component.

The selected characterization elements are intended to make it possible to generate knowledge about what lends a given indigenous group its logic and meaning so that the mechanisms of their social reproduction can be understood. In other words, an effort is made to comprehend the relationship between each group's survival **strategies** and **forms of sociality** in order to identify the situations that tend to provide the logic and meaning that underpins their social reality (contradictions/conflicts).

The main sources of information are documents produced by government entities and research institutes, as well as academic papers and theses, all of which is to be supplemented by field research.

The selected characterization elements are described below.

Ethno-Historical Aspects

The aspects grouped here are designed to observe the differences and specificities of the ethnic groups and to generate an understanding of the historically formed relationship between the indigenous group and their environment. The following characterization elements should be addressed:

- archaeological knowledge of the region;
- history of the group.

Demographic Aspects

The demographic behavior of the indigenous population should be characterized in such a way that any changes over time and with regard to the territory can be identified in order to understand how they (re)adapt to new situations, looking in detail at the following characterization elements:

- demographic size and density;
- appraisal of demographic indexes.

Ethno-Ecological Aspects

These aspects are closely related to the cultural traditions from the perspective of the values that underpin the relationship between indigenous groups and the natural environment. They highlight the relationship between the indigenous population and the territory, for which the values that guide this relationship should be observed, as well as the way they appropriate this resource and the others at their disposal. The cultural patterns should be observed, as should the mythologically-founded explanations and sanctions and the world views that make up a group's cultural identity. This identity is formed by the basic parameters of historical heritage, community ties and antinomic relations with national society. The following characterization elements should be addressed:

- values and beliefs;
- sacred sites;

10 The procedures set out here were developed for indigenous peoples, but could be adapted to any group whose cultural features mean that they require special treatment, as is the case of former quilombo communities (originally formed by runaway slaves).

- values underlying the relationship between the indigenous peoples and nature (ethno-ecological values);
- size, nature and historical construction of the territory;
- appraisal of rates of territory loss;
- geomorphological heritage;
- ways by which natural resources (minerals, land, water, forest) are appropriated;
- assessment of the potential of the land for sustaining the group's social reproduction.

Material Conditions for Survival

In view of the fact that indigenous groups and other groups with culturally specific features can be economically autonomous, semi-autonomous or integrated, their forms of production, distribution and consumption must be characterized so they can be classified as to how integrated to the market they are. In order to do so, their forms of economic production must be observed, as must their land use, their knowledge of the fauna and flora, and the efficiency of their use of these vis-à-vis the use of commercial products. The following characterization elements should be addressed:

- socioeconomic dynamics of the interethnic region;
- forms of integration with the market;
- legal status of the territory (indigenous lands that are demarcated, in the process of being demarcated and under application for demarcation; *quilombo* lands that are demarcated, in the process of being demarcated and under application for demarcation, etc.);
- environmental factors impinging on the river basin (floodplains – flood dynamics, areas of erosion, suitability for agriculture, relief segmentation).

Social, Cultural and Political Organization

The ways groups are organized should be highlighted (solidarity/reciprocity vs. rivalries), and it should be identified whether there is any intra-group political unity and/or between different ethnic groups, and to what extent this is represented institutionally. The characterization elements below should provide a good characterization of the relationships between the indigenous people and national society (interethnic relationship), evaluating any changes that may have taken place and their effects on the territory:

- forms of religion and their relationship with the surrounding society;
- ethnic unit;
- interactions with other groups;
- linguistic affiliation;
- reciprocal solidarity / rivalries;
- types and nature of contact with society (interethnic relationship).

Results of the Diagnosis

By collating the characterization elements relating to the survival strategies and forms of sociality, the situations that express the logic that underpins a given group's social realities can be identified (situations of conflict, invasions of territory, status of legal protection, group organization, limits of ethno-ecological conditions).

In order to represent this component spatially, a map should be drawn up representing the information from each characterization element, and this should be analyzed together with the maps containing the information about the physical processes and features in the study area so that the land in which each ethnic group is present can be marked out.

The lands occupied by each group identified should be mapped out in the study area. A description characterizing each ethnic group should be annexed to this map. The most sensitive areas to the introduction of new hydropower projects should be highlighted. Unlike the other synthesis components, sub-areas are not used as a spatial unit of analysis, since the processes involved in this component are not compatible with such a breakdown of the area. This means that for this synthesis component, there is only one spatial unit of analysis, the study area as a whole, in which the indigenous lands and/or lands occupied by traditional communities should be mapped out.

Table 4.3.7.01 – Characterization Elements of the Indigenous Peoples / Traditional Communities synthesis component

Synthesis component	Characterization elements	Source
Indigenous peoples/ Traditional communities	• Ethno-Historical Aspects: – archaeological knowledge of the region; – history of the group. • Demographic Aspects: – demographic size and density; – evaluation of demographic loss rate. • Ethno-Ecological Aspects: – values and beliefs; – sacred sites; – values underlying the relationship between indigenous peoples and nature; – size, nature and historical construction of the territory; – appraisal of rates of land loss; – geomorphological heritage; – ways by which natural resources are appropriated (minerals, land, water, forest); – potential of the land for sustaining the group's social reproduction. • Material Conditions for Survival: – socioeconomic dynamics of the interethnic region; – forms of integration with the market; – legal status of the lands; – environmental factors impinging on the lands (floodplains – flood dynamics, areas of erosion, suitability for agriculture, relief segmentation). • Social, Cultural and Political Organization: – forms of religion and their relationship with the surrounding society; – ethnic unit; – interactions with other groups; – linguistic affiliation; – reciprocal solidarity / rivalries; – types and nature of contact with society at large (interethnic relationship).	– Anuário Estatístico do Brasil – IBGE – Information on indigenous lands available at the land register (Diretoria Fundiária) of FUNAI – Instituto Socioambiental – ISA – Instituto de Pesquisas Antropológicas do Rio de Janeiro – IPARJ – Academic papers and theses – Field Research – SEPPIR – INCRA – MDS – MDA – Fundação Cultural Palmares/ Ministry of Culture – United Black Movement (Movimento Negro Unificado) – IBAMA, INPRA, MMA

4.4 FORMULATING THE CASCADE OPTIONS

Based on the data collected and studied in items 4.1 and 4.2 and the socioenvironmental diagnosis in item 4.3, the sites for dams and cascade options formulated in item 3.4 should be reassessed. At this phase, sites for dams and/or cascades may be added or ruled out.

The height of the dams should be compatible with the topographical, geological, socioenvironmental features at each site, which will also influence the design of the dam sites upstream.

Although the cascade options should ideally harness all the head available, it is important at this stage to use the studies undertaken and the socioenvironmental diagnosis to identify any restrictions that may make any project or cascade unworkable or too costly. These include:

- towns, cities or other communities;
- significant cultural heritage sites;
- industrial areas and/or areas with other important economic activities;
- mineral deposits or mines of high worth and/or strategic importance;
- indigenous lands and lands occupied by former *quilombos*;
- conservation areas;
- areas containing monuments of historical and/or cultural importance; and
- areas of environmental importance, such as primary forests, patches of specific kinds of vegetation or breeding areas of rare species.

Should any of the cascades involve diverting the waters to a different sub-basin, a very careful assessment should be made, since this will have socioenvironmental impacts in both the supplying sub-basin and the receiving sub-basin. These impacts must be carefully assessed at this phase to confirm the advantages or drawbacks.

Generally speaking, the cascade options should include regulating reservoirs along the sections further upstream so that the projects further downstream can benefit from them and the energy potential of the cascade can be enhanced. The formation of regulating reservoirs must be assessed with great care for each river basin. The benefit of each reservoir will be quantified in the energy studies.

4.5 TECHNICAL FORM FOR PROJECTS

The technical form should be filled out for each dam site containing all the pertinent data and information available according to the phase reached in the studies, using the form in Annex E.

4.6 ENERGY STUDIES

The energy studies undertaken during the Preliminary Studies are designed to assess the potential for energy generation of each project under study and the energy benefits for the reference system in question. The aim is to provide a preliminary design of the main features of the reservoirs and turbine-generator sets, and an assessment of each project's economic competitiveness and that of each cascade option as a whole.

When the different projects for an inventory study are decided upon, the characteristics of the turbine-generator sets will not yet have been selected. In order to do this, there are specific procedures for making approximate energy evaluations, which are set out below (items 4.6.1 to 4.6.6). These are only based on productivity coefficients and types of turbines, and assume that all the natural hydropower from the river during the critical period of the reference system will be harnessed, plus the reservoirs' live storage, minus evaporation loss. However, under real operating conditions, the restrictions imposed by the turbine-generator sets and the projects' storage capacities will mean that in some projects, not all the inflow will be harnessed. In other words, the energy values calculated using these simplified procedures are preliminary, and the assessments will have to be repeated in detail in the simulations for the Final Studies (see item 5.3.2).

As the extent of the information available about the hydrology and topography of the river basin is still limited at the Preliminary Studies stage, simplified procedures must be used. This means that net withdrawals for multiple water uses and volumes allocated for flood control taken from the scenario built up for the basin can sometimes be disregarded. They should, however, be considered when they have a major impact on the energy definition/assessment of the different cascade options.

The procedures described in this item consider the most complex cases, where multiple water uses have to be taken into account, though the adaptation to simpler cases is straightforward. This method is included in the SINV system, and its use is recommended for the energy studies.

4.6.1 Firm Energy from a Project

The firm energy from each project can be calculated in the Preliminary Studies by the following equation:

$$Ef_i = 0.0088 \times Hlm_i \times Qlm_i \qquad (4.6.1.01)$$

where:

Ef_i	firm energy from project i, in average MW;
Hlm_i	net mean head of project i, in meters;
Qlm_i	mean stream discharge during the critical period for project i, in m^3/s; and
0.0088	coefficient corresponding to the specific mass of the water (1000 kg/m^3) multiplied by the turbine productivity (0.93), the generator productivity (0.97), the acceleration of gravity (9.81 m/s^2) and the factor 10^{-6}, which allows the energy to be expressed in average MW.

In order to determine Hlm_i and Qlm_i the following parameters must be known:

- **Maximum Normal Water Level** ($NAmxn_i$): this corresponds to the maximum water level in the reservoir under normal operating conditions. For reservoirs with flood storage capacity, the maximum normal level is the maximum volume of the reservoir minus the mean of the flood storage capacity allocated for each month of the project (VEm_i) throughout the critical period of the reference system.

- **Normal Downstream Water Level** ($NAjn_i$): this is the water level in the tailrace canal; in the Preliminary Studies, it can also be taken as the natural water level at the site, for a flow that is 10% higher than the mean flow in the critical period, or the NAmxn of the reservoir immediately downstream if this level is higher.
- **Maximum Gross Head** ($Hbmxn_i$): difference between $NAmxn_i$ and $NAjn_i$.
- **Maximum Drawdown** (di) and Live Storage (Vu_i): depending on the turbines' characteristics, the maximum drawdown of a project studied in the Inventory Studies should not in theory be greater than one third of the maximum gross head. The maximum drawdown should be set for each project using a process that maximizes its firm energy (item 4.6.4), while taking into account the capacity to replenish the reservoir's live storage (item 4.6.6).
- **Normal Minimum Water Level** ($NAmin_i$): this is the minimum water level of the reservoir during normal operating conditions; it is obtained by subtracting maximum drawdown from the maximum normal water level.
- **Minimum Gross Head** ($Hbmin_i$): difference between $NAmin_i$ and $NAjn_i$.
- **Mean Water Level** (NAm_i): the water level in the reservoir after a portion of its live storage, Vd_i, has been drawn down, given by:

$$Vd_i = \left(\frac{0.5 \times (Vu_i - Vesp_i)}{Vu_i - Vesp_i + 0.5 \times \sum_{k \in M_i} (Vu_k - Vesp_k)}\right) \times (Vu_i - Vesp_i) + Vesp_i \qquad (4.6.1.02)$$

where:

Vd_i	portion, corresponding to mean drawdown, to be subtracted from the live storage, in m^3;
Vu_i	live storage of project i, in m^3;
$Vu_i - Vd_i$	live storage, corresponding to the mean water level, in m^3;
$Vesp_i$	flood storage capacity at the start of the critical period of the reference system at project i;
M_i	group of projects upstream from project i, inclusive;
k	index of the project upstream from project i.

- **Mean Gross Head** (Hbm_i): difference between NAm_i and $NAjn_i$.
- **Maximum, Mean and Minimum Net Heads** ($Hlmxn_i$, Hlm_i, $Hlmin_i$): the net heads correspond to the maximum, mean and minimum gross heads minus the losses of hydraulic load in the headrace canals. In the Preliminary Studies, the hydraulic losses can be taken as 2% of the respective gross heads for short hydraulic conveyance facilities, and 3% for long hydraulic conveyance facilities.
- **Mean Net Discharge at the Critical Period** (Qlm_i): sum of the average natural flows at the project site during the critical period of the reference system, including the flows coming from the live storage of the reservoirs at the site and upstream, minus the evaporation loss and flood storage capacities for flood control corresponding to the beginning of the critical period, and the flows withdrawn for multiple water uses at the site and upstream from it.

$$Qlm_i = Qn_i - Qr_i + T^{-1} \sum_{k \in M_i} (Vu_k - Vesp_k - Evap_k \times Amed_k \times 10^6) \qquad (4.6.1.03)$$

where:

Qlm_i	mean net discharge for the critical period of the reference system at the site of project i, in m^3/s;
Qn_i	mean natural flow at the site of project i during the critical period of the reference system, in m^3/s;
Qr_i	mean volume of withdrawals for other water uses at the site of project i and upstream from it during the critical period of the reference system, in m^3/s;

T	number of seconds in the critical period of the reference system of the system;
Vu_k	live storage of project k, in m^3;
$Vesp_k$	flood storage capacity at the start of the critical period for project k, in m^3;
$Evap_k$	total net evaporation for k during the critical period, in m;
$Amed_k$	area of the reservoir at project k, corresponding t NAm_k , in km^2.

Net evaporation can be represented by a single vector that corresponds to the average monthly values. It is determined by the procedure set out in item 4.1.2.

4.6.2 Firm Energy from a Cascade

Once the head and mean net discharge have been set for each project, the preliminary firm energy (Ef) value of the whole cascade option can be defined, as:

$$Ef = \sum Ef_i = 0.0088 \sum Hlm_i \times Qlm_i \qquad (4.6.2.01)$$

In the Preliminary Studies, a cascade's firm energy calculated by the above formula corresponds to the firm energy entering the reference system from the set of projects in the cascade (item 4.6.3).

The firm energy of a cascade option and the individual project within it can be obtained from the "Firm Energy" function, "without simulation" option, in the SINV system. This function also provides the installed capacity of the plants in the cascade under analysis.

4.6.3 Firm Energy Contribution

During the Preliminary Studies, the evaluation of a project or group of projects' energy potential is an *approximate* calculation of the extra firm energy the project or projects can provide for the reference system, assuming that all the other projects in the cascade have already been built, called the last added contribution.

For a cascade as a whole, the firm energy contribution is estimated in the Preliminary Studies by the firm energy of the cascade, whose calculation is shown in item 4.6.2.

The firm energy contribution attributed to each project i in a cascade option can be calculated thus:

$$\Delta Ef_i = 0.0088 \times (Hlm_i \times Qlm_i + \frac{Vu_i - Vesp_i}{T} \times \sum_{k \in J_i} Hlm_k) \qquad (4.6.3.01)$$

where J_i is the group of projects from the reference system that are downstream from i.

The firm energy contribution of a group of projects S can be calculated as:

$$\Delta Ef_S = 0.0088 \times \left\{ \sum_{i \in S} \left(Hlm_i \times Qlm_i + \frac{Vu_i - Vesp_i}{T} \times \sum_{k \in J_i \setminus S} Hlm_k \right) \right\} \qquad (4.6.3.02)$$

where $J_i \setminus S$ is the group of projects downstream from project i that do not belong to the group of projects S.

4.6.4 Optimization of Live Storage

The live storage of the projects within a cascade should be determined by a process of optimization. First of all, the maximum drawdown levels are established for all the projects, corresponding to one third of the maximum gross head at each site. Next, a preliminary firm energy value for the cascade is determined using the procedure set out in the item above.

Once the preliminary value for firm energy has been established, the drawdown of the last reservoir downstream is reduced arbitrarily and its minimum water level is raised. This reduces the regulated flow and increases the mean net head of the group of projects. If this alteration increases the firm energy of the cascade, attempts to reduce the drawdown should be done successively as long as the level of firm energy from the projects increases. Once the live storage of the last reservoir downstream has been fixed, the process is repeated for the last project but one downstream from the group of project, and on successively until the project furthest upstream in the system. Experience from previous Preliminary Studies suggests that it is enough to carry out a single iteration of this kind; however, the analyst is ultimately responsible for deciding whether it is desirable to obtain greater precision in view of the nature of the data at his/her disposal.

The live storage of the reservoirs can be determined using the "Optimize Live Storage" function from the SINV system, which undertakes the process described above. Another choice is to use the "Energy Dimensioning" function, choosing the "without simulation" option from the SINV system to simultaneously dimension the live storage, installed capacity and reference head of the projects.

4.6.5 Installed Capacity

Once the live storage and firm energy have been estimated for each project, their installed capacity must be calculated so that the design and layout of the structures can be defined, as well as estimates of the corresponding costs.

Installed capacity is obtained as a function of the reference capacity factor and the firm energy of the project, as shown below:

$$P_i = \frac{Ef_i}{Fk} \qquad (4.6.5.01)$$

where:

P_i	installed capacity in MW;
Ef_i	firm energy in average MW; and,
Fk	reference capacity factor.

4.6.6 Reservoir Replenishment Time

Once the live storage and installed capacity have been calculated, it is important to check whether the reservoirs will be able to be replenished within 36 months from the end of the critical period. It is worth checking whether this criterion is satisfied. If the live storage does not fulfill this criterion, it should be reduced.

The flow from a reservoir during replenishment is defined as the mean flow of m months (m = 1,..., 36 months) after the end of the critical period, minus evaporation, mean volumes withdrawn for other uses, and the volumes retained for replenishing the reservoirs upstream from the project under investigation, including the project itself.

$$Qdefl_i = Qmed\_m_i - \sum_{k \in Mi} \left[\left(\frac{Evap_k x Amed_k}{2628} \right) + \left(\frac{Vu_k}{Nseg} \right) + Qret_k \right] \quad (4.6.6.01)$$

where:

M_i	group of projects upstream from i, inclusive;
$Qdefl_i$	mean flow from reservoir i during the replenishment period, in m³/s;
$Qmed\_m_i$	mean flow in months “m” after the critical period at project i, in m³/s;
$Evapmlt_k$	mean net evaporation of reservoir k, in mm;
$Amed_k$	area of the reservoir of project k corresponding to the level of reservoir k, minus half of its live storage, in km²;
Vu_k	live storage of project k, in m³;
$Qret_k$	mean flow withdrawn from the reservoir of project k for other uses, in m³/s;
Nseg	number of seconds in the “m” months.

The minimum outflow from a reservoir at the operational phase is defined as the higher of the following: the minimum outflow to meet socioenvironmental requirements, or the outflow required to operate just one generating unit.

$$Q\min\_opi = \max imum\left(Q\min\_ambi; \frac{Pi.F_i}{ni.Hlmi.0.0088} \right) \quad (4.6.6.02)$$

where:

$Qmin\_op_i$	minimum outflow during the operating period of project i, in m³/s;
$Qmin\_amb_i$	minimum outflow from project i for socioenvironmental needs, in m³/s;
P_i	installed capacity of project i, in MW;
F_i	factor that represents the minimum operating level of the turbine-generator set at project i (Francis turbine: F = 0.60 and Kaplan or Bulb turbine: F = 0.35);
n_i	number of units at project i (see item 5.8.2);
Hlm_i	mean net head of project i, in meters.

By definition, if the outflow of all the projects is equal to or greater than the minimum outflow (under any hypothesis, with m at anything between 1 and 36 months), the reservoirs can be replenished in the stipulated time period. This means that replenishment time is not a factor that can annul the live storage calculations done previously.

If not, the project that is furthest upstream from amongst the group of projects under study whose outflows are lower than the minimum outflow should have its reservoir's active storage reduced iteratively until that project's outflow reaches the minimum outflow for the operational period.

Next, the installed capacities of the projects should be redimensioned and the evaluation of the replenishment times should be recalculated. This process should be repeated until the outflows of all the projects are greater than their respective minimum outflows, taking into account the minimum flows from the projects that already exist. In the case of existing projects, their minimum flows can be assured by reducing the live storage of the projects that have not yet been built.

The capacity of the reservoirs in a cascade to be replenished in up to 36 months after a critical period can be verified using the “Live Storage Replenishment” function, without simulation, from the SINV system. This function processes the steps above exactly as they are described.

4.7 PROJECT LAYOUTS

In this item, the specific criteria and instructions are given for designing the layouts of the projects in the cascades formulated.

Having gathered and analyzed the local and regional data and information, as set forth in this chapter, a schematic layout should be planned for each project that will allow the approximate dimensions of the structures to be calculated in order to reach a cost estimate.

With the information gathered, it should be possible to prepare a planimetric and altimetric map of the construction site on a scale that is compatible with the space designated for the project. This map must be prepared and should represent the best possible estimate of the local morphology. It is often necessary to interpolate contour lines. At this stage, every effort should be made to characterize to as high a level of precision as possible the profile of the slopes of the abutments and the river valleys and any elevations at sites where headrace tunnels or canals could be built. The same map should also contain basic geological information, such as the thickness of the soil cover and the suitability of the foundations for concrete.

Once the topographic elements have been characterized, the lines and basic contours of the layout should be drawn on top of the map. When designing the layout, the guidelines set out in item 2.5 should be followed, broadly speaking. In order to do so, the approximate dimensions of the main structures must be calculated in order to estimate the associated costs (item 4.10). Once this has been done, the layout of the project should be drawn on the map and the main cross-sections and longitudinal sections should be drawn. At this stage, some adjustments are normally made to the layout to obtain the most suitable configuration, taking into account the degree of precision of the information available at this stage.

At the same time, the area of land to be flooded for the reservoir should be marked out on a map, which should also have any lands, towns or land developments identified.

4.8 EVALUATION OF NEGATIVE SOCIOENVIRONMENTAL IMPACTS PER PROJECT

These studies involve analyzing projects as to the negative socioenvironmental impacts they exert on each synthesis component, which consists of **identifying the impact processes** and **assessing the negative socioenvironmental impacts**.

The aims of these studies are:

- to supply information to help estimate the socioenvironmental costs of the projects;
- to attribute values to the negative socioenvironmental impacts of the projects by using impact indexes, which will serve for calculating the negative environmental indexes of the cascade options, as set forth in item 4.11.2;
- indicate the need to adjust the formulation of the cascades and the basic design of the projects, so as to minimize their negative socioenvironmental impacts;
- to identify the cumulative and synergistic effects throughout the sub-areas.

In the Preliminary Studies, the analyses should be undertaken for each project individually, without taking into account the broader context of the cascades they are part of. This is to prevent making the analysis overly complex and lengthy in view of the number of projects and cascades normally under consideration at this stage. For this reason, a more detailed analysis should only be done in the Final Studies once the most promising cascades have been selected for their economic and energy potential and socioenvironmental impact.

In order to make the evaluation feasible, **impact indicators** have been defined that collate the main impact processes that could affect each synthesis component when the projects are built. The indicators should provide the means to quantify and qualify the effects of pressures on the terrestrial and aquatic ecosystems and on the socioeconomic interactions brought about by the projects, also taking into account the uses of the land and waters in the river basin.

The particularities of the project layouts should also be considered. It is important, for instance, to be aware of any alterations to the streamflow downstream from the dam in the cases of projects that divert water through intake canals or tunnels.

Each impact indicator is associated to a set of **assessment elements**, which organize the information relative to the impact processes. The impact indicators and their respective assessment elements are presented in items 4.8.3 to 4.8.8. By doing this evaluation, it should be possible to qualify and quantify the indicators in the physical space and for future scenarios.

4.8.1 Identification of Impact Processes

The impact processes associated with each project should be identified. The following procedures are recommended:

- cross-referencing the results of the socioenvironmental diagnosis with the information about the prospective projects. It is useful to superimpose the maps that represent the synthesis components and have the divisions into sub-areas and areas of sensitivity marked out and classified with the maps of the project layouts resulting from the engineering studies;

- characterizing the main impact processes that emerge from the interaction between the project and the study area for each synthesis component. This analysis should be systematized per sub-area, since they highlight the processes that already exist in the study area that could be affected by the projects. The areas of sensitivity that are demarcated should also serve as a reference for the evaluation, since the impacts from processes that affect the areas of greater sensitivity should be more significant. This means that projects in the same sub-area will tend to bring about the same impact processes with similar profiles, differentiated by the specific features of the projects and, when pertinent, by any special features at their sites, such as areas identified as being of greater sensitivity;
- selecting the assessment elements capable of characterizing the impact processes identified on each synthesis component, which make the impact indicator capable of differentiating amongst the projects and, subsequently, amongst the different cascades. In selecting the assessment elements, different aspects from the study area and the projects themselves must be covered. A balance should also be struck between quantitative and qualitative assessment elements;
- undertaking interdisciplinary activities to assure the integration of the analyses undertaken for the different synthesis components. This makes it possible to incorporate the interrelationships between the impact processes of different synthesis components through their assessment elements;
- reviewing the characterization of impact processes per synthesis component and the selection of their respective assessment elements as a function of the integration of the analyses. This should give rise to a general description of the impact processes and the assessment elements adopted. At this point, those processes for which control, mitigation or compensation actions should be taken must be highlighted and translated into socioenvironmental costs to be incorporated into the implementation costs (item 4.10.1). Any variations in the layout of the projects that might ameliorate their negative socioenvironmental impacts should also be identified, as should any interference the environment could have on the projects, which will be used in the engineering project and cost estimates. It should be noted that this interference is not calculated for the assessment of socioenvironmental impacts.

4.8.2 Assessment of Negative Socioenvironmental Impacts

An estimate should be made of the intensity of the negative socioenvironmental impacts of each project on the sub-areas defined for each synthesis component, based on the impact indicators and their assessment elements. The impacts that need to be analyzed are those for which no control can be introduced and any other residual impacts left over after controls, mitigation or compensation have been provided. The following procedures are recommended:

a) analysis of the assessment elements for each project, with a view to building up the impact indicators adopted. The analysis should be made per synthesis component, seeking to strike a balance between qualitative and quantitative assessment elements. The assessment elements and the procedures used to build up each indicator should be listed, as should the criteria used for measuring the intensity of the impacts. Again, any areas of greater sensitivity in the sub-areas will be helpful in assessing the intensity of any given impact;

b) attribution of a negative impact index for each synthesis component per sub-area affected and for each prospective project. This activity consists of expressing the intensity of the negative socioenvironmental impact of a project as a numerical figure, which is based on the set of negative impact indicators adopted for each synthesis component. The indexes thus attributed are then aggregated per sub-area. This aggregation is achieved by ranking the indicators according to their importance to the sub-area and giving them relative weights.

The negative impact indexes should be attributed on a continuous scale from **zero** to **one**. **Zero** indicates the **absence of any impact**, while **one** means that the **entire process inherent to the synthesis component in question is compromised**. The intermediate values therefore represent the different degrees to which existing environmental processes are affected by the negative impact indicators chosen for each synthesis component. The criteria used for defining the values of the indexes should be reported.

An example is given in table 4.8.2.01 of the result of an impact assessment of a given synthesis component for a project.

The team that carries out the study should decide on the criteria to be used for attributing the levels of impact, seeking to reach a consensus as to the meaning of the intermediate values so that the results of the assessments of the different synthesis components can be compared amongst themselves. It is recommended that interdisciplinary activities be undertaken so that the criteria used for the river basin under study can be standardized. These should be justified and recorded so that any future reviews or updates of the studies can use the same criteria.

The maximum value on the evaluation scale (one) should not be relative; i.e. it is not the highest value amongst the impacts of the projects in the basin in question, but an absolute value of total impact that may or may not exist in the situation under study.

The criteria used for setting the values of the indexes should be recorded.

It is necessary to set values for the indexes of the projects per sub-area before the indexes of whole alternatives can be built up, which are based on the individual indexes of the projects that make them up, as shown in item 4.11.2.

Table 4.8.2.01– Impact Indexes per project
Example: Synthesis Component – Ways of Life

Sub-areas / Projects	I	II	III	IV	V	VI
A		0.10				
B		0.50	0.65	0.10		
C			0.85		0.35	
D			0.70			
E	0.05	0.05				
F		0.08				
G	0.10					
H	0.10				0.10	
I_1	0.30		0.10		0.30	
I_2	0.85		0.10		0.85	
J			0.60		0.30	
K	0.45				0.30	
L			0.75	0.40	0.60	
M	0.30					0.40
N	0.50		0.90			
O						0.88
P						0.40
Q_1						0.80
Q_2						0.95
R_1						0.90
R_2						0.95

c) analyses of the indexes of the projects per sub-area, which, as they represent the intensity of the negative impact on the synthesis component, supply important indications for the review of the projects' design in order to achieve a better socioenvironmental performance or, in extreme cases, to eliminate individual projects and/or cascades;

d) analysis of the repercussions on the study area of the impact processes in each sub-area, which will be combined to form the indexes for the different cascades, as described in item 4.11.2;

e) interdisciplinary discussions about the evaluations in order to analyze the results, identify any inconsistencies and minimize the degree of subjectivity involved in the judgments used for the different synthesis components. Depending on the outcome of these discussions, the levels of negative impact attributed to each project on the synthesis components may be reviewed.

The relative subjectivity inherent to these evaluations can only be minimized by standardizing the assessment criteria, assessment elements and procedures adopted in the methodology. The repeated application of the methodology and expansion of the database on the electricity sector with the results of monitoring activities are indispensable for future efforts to parameterize the assessment elements and make the values attributed to each of the environmental indexes more objective.

In items 4.8.3 to 4.8.8, the contents that need to be covered and procedures to be adopted for assessing the negative impacts on each synthesis component are set out in detail, with the respective impact indicators presented in tables. These tables are designed to cover the majority of situations encountered in different parts of Brazil, but should be adjusted and/or supplemented with information as necessary for each specific case under study. As Physical Processes and Features are taken to be a basic element for analyzing the synthesis components, as explained previously, they are incorporated into the evaluations of these components as assessment elements, in that they contribute to the impact processes identified.

4.8.3 Aquatic Ecosystems

The impact processes should be identified for each project and for each sub-area. Next, the assessment elements should be identified that best characterize these processes, with a view to estimating the extent to which the characteristics that are fundamental in preserving the biodiversity are affected (**impact indicators**). These elements should encompass aspects relating to the indicators of ecological significance identified in the diagnosis, which are used to identify the areas of sensitivity, as well as the characteristics of the projects, so that it can be estimated what changes the ecosystems are likely to suffer as a result of the planned interventions.

It should also be borne in mind that the assessment elements are used for the basic design of the projects and later when the socioenvironmental costs are estimated and subsequently incorporated into the implementation costs.

At this point, interdisciplinary activities should be carried out with a view to integrating the impact processes identified in one synthesis component with those from the others, drawing on information on the physical processes and features, so that the assessment elements effectively represent the interactions that exist.

The intensity of the socioenvironmental impact of a project on a given sub-area should be estimated by analyzing the degree to which the habitats which harbor biodiversity, migratory, endemic or exclusive species and exceptional situations are affected, and by how much other groups of vertebrates are also affected. In the analysis, the interference on the streamflow regime downstream from the reservoir should also be investigated. The following assessment elements are recommended:

- total area of the aquatic environment to be modified, which should be measured by summing the length of the channel of the main river and its tributaries (km) that is expected to change from being lotic to lentic, i.e. which will cease to be a stream and become a lake. This alteration has a major impact on the water systems, as many of the species will be unable to adapt to the new environment;
- relative stream order – this is the ratio between the stream order observed in the reservoir's drainage area and the maximum stream order observed in the sub-area, using the Strahler method;
- loss of ecologically strategic habitats – these habitats are: river pools, river beaches (used as breeding and feeding grounds for swamp-dwelling wildlife) and rivers with unique physiographic features, such as those that are meandering or braided. The loss of these habitats is assessed by the ratio of the area (in the case of lakes and beaches) or length (in the case of rivers) affected by the project to the total area/ length of these habitats in the sub-area;
- impact on migratory routes – the extent to which migratory routes used by rheophiles are affected by the project in each sub-area is assessed by their importance in drawing these rheophiles into the water basin;
- loss of habitats with high hydrodynamic energy – rapids and/or white water – the ratio between the length of the habitats with high hydrodynamic energy affected by the project and the total length of these habitats within the sub-area will give a good estimate of the degree to which species exclusive to these habitats will be impacted;
- loss of riparian forest – this can be estimated by the ratio between the length of riparian forest affected by the project against the total length of riparian forest in the sub-area;
- water quality of the prospective reservoirs – one of the greatest ecological, social and economic repercussions associated with the formation of artificial reservoirs is a phenomenon known as eutrophication. It is a process that reflects the hydraulic and morphometric characteristics of the project, and the hydrological, physiographical and land use aspects in the contributing basin and the area to be impounded. Given the ecological and economic significance of eutrophication, the impact assessment of this variable should be individualized, using simplified models for projecting the water quality in the future reservoirs. The variables that need to be assessed are: morphometry of the reservoir, average depth, residence time, phytomass of the flooded area, and land use in the drainage basin. At the end of the modeling, the different projects are assessed as to the degree to which they will contribute to maintaining the quality of the water in the river basin;
- alteration of natural streamflow regime – this happens with projects with flow regulation capacity. When a hydroelectric plant is in operation, there can be significant reductions in the flow immediately downstream from the dam. In these cases, the aquatic fauna and flora that depend on the natural course of these waters will be altered. The likelihood of this alteration is measured as the ratio between the reservoir's live storage and the average natural flow;
- diversion in the same river basin – this happens in certain cases when the water withdrawn for the power plant is preceded by channels or pipes that divert the flow of a river, leaving a section of the natural river bed with a reduced flow or completely dry. Once the water has gone through the turbines, it returns to its natural course. In order to assess this impact, the length of the section with a reduced flow and the reduction in the flow itself must be calculated; and

- diversion to a different river basin – this happens in certain cases when water which, prior to the building of the power plant, ran along the natural river bed is diverted after the impoundment to another river basin, leading to a reduction in the flow or complete drying up of the section immediately downstream from the dam, and a mixing of waters from different basins in the section of the basin that receives the diverted flow.

Table 4.8.3.01 shows the negative impact indicators and the assessment elements for this synthesis component.

Establishing Levels of Impact

A compilation of the assessment elements should be used in attributing a degree of negative impact for each project on each sub-area. These will be on a scale from **zero** to **one** to reflect the extent to which the features on which the continued biodiversity of the area is dependent will be compromised.

Finally, the repercussions on the study area of the impact processes in each sub-area should be analyzed, taking into account the different degrees of ecological significance and the areas of greater sensitivity identified in the diagnosis.

Table 4.8.3.01 – Impact indicators and assessment elements for the Aquatic Ecosystems synthesis component

Synthesis component	Impact indicator	Assessment elements
Aquatic Ecosystems	Interference in habitats that maintain the biodiversity, migratory, endemic or exclusive species (and other groups of vertebrates)	• Stream order (Strahler method); • Alterations to the total extent of the water environment to be modified; • Loss of Ecologically Strategic Habitats; • Migratory routes affected; • Loss of habitats of high hydrodynamic energy; • Alteration of riparian forest; • Water quality of future reservoirs – morphometric characteristics of the section of river affected; – volume of phytomass affected; – soil types affected; – mean depth; – residence time.
	Interference in the streamflow regime (effects downstream from the reservoir)	• Possibility of eutrophication of the reservoir; sites with a high concentration of heavy metals / with a possibility of biomagnification. • Occurrence of other vertebrates that could be impacted (aquatic mammals, reptiles). • Alteration of natural streamflows – Regulating capacity: mean natural flow to the reservoir and its live storage. • Diversion within same basin – Length of the section with reduced flow; – Reduction of flow. • Diversion to different basin – Mean flow diverted.

4.8.4 Terrestrial Ecosystems

The impact processes should be identified for each project and for each sub-area. Next, the assessment elements should be identified that best characterize these processes, with a view to estimating the extent to which the characteristics that are instrumental in maintaining the biodiversity are compromised. These elements should encompass aspects relating to the indicators of ecological significance and areas of sensitivity identified in the diagnosis, as well as the characteristics of the project, so that the change to the state of the biological systems to be expected from the future intervention can be estimated.

It should also be remembered that the assessment elements are used for the basic design of the projects and later for estimating the socioenvironmental costs, which are an integral part of the implementation costs.

At this point, interdisciplinary activities should be carried out with a view to integrating the impact processes identified with those from the other synthesis components, so that the assessment elements can effectively represent the interactions that exist.

The intensity of the socioenvironmental impact of a project on a given sub-area should be estimated by analyzing the degree to which the ecosystems and species are affected. The following assessment elements are recommended:

- land use in a project's direct and indirect areas of influence. Quantification (in km^2) of the urban areas, the areas occupied by farming and ranching, and the areas with patches of remaining natural vegetation, identifying the different stages of conservation (primary and secondary formations, classified by their different stages of regeneration);
- loss of riparian forest – this can be estimated by the ratio between the length of riparian forest affected by the project and the total length of riparian forest existing in the sub-area;
- loss of vegetation – this is assessed by the ratio between the area of forest affected by the project and the total area of forest existing in the sub-area;
- physiognomic exclusivity – the extent to which the phytophysiognomies exclusive to the sub-area are affected is estimated by the ratio between the surface area containing unique physiognomies affected by the project and the total surface area with unique physiognomies that exists in the sub-area;
- significance of fauna in the area affected – this is estimated as a function of the probability of species occurring whose natural population numbers are in some way under threat;
- conservation areas – direct or indirect interference in federal, state or municipal conservation areas.

Table 4.8.4.01 shows the negative impact indicators and assessment elements for this synthesis component.

Establishing Levels of Impact

A compilation of the assessment elements should be used in attributing a degree of negative impact for each project on each sub-area. These will be on a scale from **zero** to **one** to reflect the extent to which the features on which the continued biodiversity of the area is dependent will be compromised.

Finally, the repercussions on the study area of the impact processes in each sub-area should be analyzed, taking into account the different degrees of ecological significance and the areas of greater sensitivity identified in the diagnosis.

Table 4.8.4.01 – Impact indicators and assessment elements for the Terrestrial Ecosystems Synthesis Component

Synthesis component	Impact indicator	Assessment elements
Terrestrial Ecosystems	Interference in the features that determine the continued biodiversity of the area (impact on ecosystems and species)	• Loss of habitats with a higher degree of ecological integrity • Increased pressure on terrestrial ecosystems (deforestation, hunting, extraction of non-timber forest products, farming, ranching, illegal trade in animal and plant species, etc.) • Loss of riparian forest • Loss of vegetation • Interference in ecological corridors in the area of influence, priority areas for biodiversity conservation, conservation areas, buffer zones and all other protected areas. • Unique physiognomies, integrity of the terrestrial ecosystem. Loss of connectivity, increased fragmentation of the ecosystem • Significance of the fauna in the affected area

4.8.5 Ways of Life

The impact processes should be identified for each sub-area, meaning for each way of life affected by each project.

Once the impact processes have been identified per project and for each sub-area, the assessment elements should be identified that best characterize these processes, with a view to estimating their level of interference on the forms of social reproduction. These elements should encompass aspects relating to the pre-existing forms of social life and the features of the projects, so that the relationship between the vulnerability to change identified in the diagnosis and the kind of intervention to take place can be observed.

It should also be remembered that the assessment elements are used for the basic design of the projects and later for estimating the socioenvironmental costs, which are an integral part of the implementation costs.

At this point, interdisciplinary activities should be carried out with a view to integrating the impact processes identified with those from the other synthesis components, so that the assessment elements can effectively represent the existing interactions.

When estimating the intensity of the socioenvironmental impact of a project on a given sub-area, the following impact indicators should be considered, which will contribute towards the synthesis of the assessment elements:

- extent to which survival strategies are affected, which has to do with interferences in material conditions;
- extent to which the historically constructed society is affected, which has to do with interferences in socio-cultural conditions.

In order to assess the extent to which the *survival strategies* of each way of life in the area are affected, it is necessary to consider:

- changes to aspects that impinge on living conditions:
  - fall in consumption standards caused by a decline in labor conditions;
  - collective consumer goods affected (basic services: education, health, energy, communication, sanitation, transport and leisure);

- changes to basic traditional indicators of quality of life, such as health indicators;
- changes to the epidemiological profile, especially concerning water-borne diseases and/or diseases associated with unsanitary conditions.

- changes to the production systems of each Way of Life, observed by identifying the chances of recovery, based on the following:

- changes to the pre-existing state of capitalization/decapitalization, caused by changes to employment status, value of equity available and impacts on production. In some rural areas, this can most easily be perceived by observing the occupations of the workforce operating in partnership, income, tasks and/or wages in the labor undertaken by the workforce; this will usually include landless workers;
- changes/breakdowns in the networks of relationships which urban social groupings depend on and have at their disposal to assure their survival, and which may have to do with formal employment (formal labor market) or informal employment (part-time work, odd jobs);
- breakdown in the bonds of dependency between the rural and urban populations, which can mainly be seen from the perspective of labor relations (people who live in towns but depend on work in the countryside, or vice versa), as well as the situations in which food security/supply is dependent on rural output;
- changes to environmental factors, which can be seen from the loss of areas of floodplain which are important to riverside communities or the loss of areas suitable for agriculture, which often assure regional food security, as well as the subsistence economy. It is important to take into account any alterations to the streamflow regime in the river downstream from the dam.

The assessment of the extent to which the *historically constructed society* is affected, which expresses the social and cultural aspects of the Ways of Life identified, is carried out by observing the following:

- social ties affected:

- broken bonds of neighborliness or companionship;
- ethnic relations affected;
- aggravation of pre-existing conflict situations;
- breakdown of political relations that depend on the pre-existing social structure;
- effect on the extent of formal and informal organization of society.

- socio-cultural identity affected, and its spatial and temporal manifestations:

- loss of points of reference responsible for the social and cultural organization of the group (cultural manifestations, elements of historical and cultural heritage);
- loss of situations in which proximity to the river (e.g. flood dynamics) is instrumental in the socio-cultural organization.

Table 4.8.5.01 sets out the impact indicators and assessment elements for this synthesis component.

It is important to estimate and identify the number of people to be impacted. This information will be used in calculating the costs associated with the implementation of the projects. However, in this evaluation of the impacts, the aim is to qualify this group and assess the losses they would suffer, thus preventing a duplicate count of aspects already incorporated into the overall costs.

Establishing Levels of Impact

A compilation of the assessment elements should be used in attributing a degree of negative impact for each project on each sub-area. These will be on a scale from **zero** to **one** to reflect the extent to which the forms of social reproduction will be compromised.

Finally, the repercussions on the study area of the impact processes in each sub-area should be analyzed. This analysis should take into account the perception of representative situations which make questions that are apparently local become regional in nature, making the impact process have effects that extrapolate the boundaries of the sub-area, such as situations of contradictions and/or conflicts which are exacerbated by the intervention.

Table 4.8.5.01 – Impact indicators and assessment elements for the Ways of Life synthesis component

Synthesis component	Impact indicators	Assessment elements
Ways of life	Effect on survival strategies	• changes to aspects that structure the living conditions: – number of people or households affected (rural and urban) – collective consumer goods affected; – drops in consumption standards; – changes in indicators of quality of life; – changes to the disease profile; • changes to the production systems of each Way of life. – changes to the pre-existing capitalization/decapitalization conditions; – changes to the network of relationships that the urban social groups depend on to assure their survival; – breakdown of ties of dependency between towns and countryside; – changes to environmental factors.
	Effect on historically constructed society	• social ties affected; • changes to socio-cultural identity and its spatial and temporal manifestations. • interference in historical, cultural archaeological and other forms of heritage.

4.8.6 Territorial Organization

The impact processes should be identified for each sub-area affected by each project, with priority being given to characterizing those that are involved in breaking down flows of communication and the circulation of goods and people, and the land uses and occupations.

Once the impact processes have been identified per project and for each sub-area, the assessment elements should be identified that best characterize these processes, with a view to estimating the extent to which circulation and communication within the study area will be interrupted. This should encompass the elements that structure the way the land is organized and the features of the projects, so that the interference they will have on the degree of integration within each sub-area can be observed.

It should also be remembered that the assessment elements are used for the basic design of the projects and later for estimating the socioenvironmental costs, which are an integral part of the implementation costs. Some of these elements will be used in the assessment of positive impacts in the Final Studies.

At this point, interdisciplinary activities should be carried out with a view to integrating the impact processes identified with those from the other synthesis components, so that the assessment elements that best represent the existing interactions can be selected.

The intensity of the negative socioenvironmental impact of each project on the sub-areas affected should be estimated by analyzing the following impact indicators, which will help collate the different assessment elements:

- interference in the patterns of human settlements and mobility of the local population;
- interference in the flows of circulation and communication;
- interference in the political and administrative organization of the municipalities;
- interference in territorial management.

Interference in the patterns of human settlements and mobility of the local population

This indicator is instrumental in assessing any changes to the historically established settlement patterns within the area of the river basin, and the role any such changes would have in the territorial organization of the study area. Both qualitative and quantitative data should be used for this assessment. When communities need to be resettled, it is worth making comparisons with similar cases that may already have taken place in the same region as the study area.

The following assessment elements should be considered:

- number of people to be resettled and what proportion these are to the total population in the municipality and the river basin (estimate): this is a quantitative estimate of the number of people to be resettled, drawing a correlation with the total population within the study area and the total population living in the respective municipalities, followed by a qualitative analysis of the significance of the estimated resettlements. It could be helpful to make supplementary observations of the spatial location and internal characteristics of the population to be resettled, covering their original occupations (urbanized area, area in urban expansion, rural area, forest area) and original settlement patterns (residents of densely populated urban areas, suburbs, concentrated or widely-spaced settlements, isolated, sparse or clustered communities);
- number, location and features of villages affected, whether partially or totally;
- availability of land for planned resettlements;
- workers' village: planned location, estimated population associated with the works, relationship with the people living in the municipality and the whole study area.

Interference in flows of circulation and communication

The impact processes on flows of circulation and communication should be identified, characterized and qualified by looking at two aspects that sum up the interferences produced and the assessment elements associated with them: accessibility and reversibility of the interferences on occupations, circulation and communication.

Interference on accessibility can be estimated by the following items:

- production, consumption and service facilities affected: the production facilities affected should be identified, highlighting those involved in the processing of primary goods (agro-industrial, alcohol distilleries, etc.) and those that are labor intensive; depots, warehouses and silos; teaching and health establishments (rural and urban, serving people outside the local community); commercial establishments (especially accommodation and food establishments), transportation and communication. The features, spatial location and range of these facilities should be described, as well as the size, profile and location of the people concerned with them and the extent to which they will be affected;
- extent and functions of transportation infrastructure affected: the sections of roads, waterways and railroads affected should be described, mapped out and qualified so as to cover the following items: length, route, number of points of integration between different forms of transportation, towns directly connected to them, corridors and their areas of influence; goods and categories of users that are served by them; main origin and destination points of journeys;
- estimate of number of people affected per loss of transportation infrastructure (and services dependent on it), and proportion of these people to the total population in the municipality and the study area: once the interference in the towns and the transportation network has been identified, it should be estimated how many people will be affected by the loss of transportation infrastructure in such a way that their access to urban facilities will be restricted. This total should be correlated with the total number of residents in the study area and in the respective municipalities;

- estimate of users of water resources affected: the number of people affected should be estimated per use, such as supply, irrigation, fishing, trade, leisure, tourism; the nature and severity of the interference should be qualified per use. In order to use this estimate and the other data on this topic generated for the impact assessment, the information used here must be brought into line with the information on different water uses and the information and projections used in preparing the basic scenarios and alternatives for other water uses in the energy studies (item 4.2.5). Note that the impacts on this assessment element should not be taken into account for those projects/cascades where the pre-defined scenarios are observed. However, while in these studies the main uses considered are irrigation, flood control and navigation, the analysis is broader for the impact assessment, covering all actual and potential water uses;
- transportation integration affected: characteristics of the kinds of transportation integration and means of access affected and repercussions on the flows of circulation and communication.

The reversibility of the interference in the patterns of occupation, circulation and communication can be assessed by the following elements:

- alternatives to the functional relationships and flows of circulation and communication within the river basin that are interrupted – it should be investigated whether there are any alternative means of restoring the situations affected by the impact processes described in the above items. This would include: re-establishing flows interrupted by different elements of infrastructure; assurance of access by restoring interrupted flows or establishing new flows, etc.

The following items should be considered as part of these studies:

- the patterns of occupation that are characteristic in the basin and their expansion trends;
- any short- or medium-term large-scale projects or plans for investments in transportation infrastructure in the study area;
- the possibility of using different modes of transportation by the people affected;
- the availability of lands for relocating affected villages and providing infrastructure, by examining any land that is not answered for in the current occupation dynamics, and any environmental factors that may induce or restrict the new land occupation.

Interference in the political and administrative organization of the municipalities

- Role of the seats of municipal government and public, municipal, state and federal institutions affected: the administrative, political and institutional functions carried out by the institutions and the localities to be relocated should be defined. An essentially interpretative analysis should be made of the set of descriptive data gathered and collated to date.
- Loss of land: surface area and its proportion to the total land area in the municipality and the river basin: the quantitative data on and spatial visualization of the areas to be flooded are the main inputs for the qualitative analysis of the interference to be caused to the political and administrative entities. These interferences may lead to threshold situations, where the continued existence of a municipality may become unfeasible, with knock-on effects on other municipalities, their residents and services.
- Estimate of the total electorate to be resettled and their proportion to the total municipal constituency: this estimate relates to the assessment of the potential for changing the local constituencies.
- Loss of number of local representatives: it should be checked whether the number of representatives on the municipal council will be reduced as a result of the need to resettle local people, given that in some cases the number of people living in a municipality could be greatly altered.

Interference in Territorial Management

The incompatibilities and synergies with the policies, plans and programs for local and regional development should be analyzed with a view to assessing any potential negative or positive impacts or cumulative effects.

Table 4.8.6.01 shows the impact indicators and assessment elements for this synthesis component.

Attribution of Levels of impact

Negative impacts should be attributed per sub-area affected and for each prospective project. A compilation of the assessment elements should be used in attributing a degree of negative impact on a scale from **zero** to **one** to reflect the extent to which the flows of circulation and communication which are instrumental in organizing the land covered by the sub-area will be compromised.

Finally, the repercussions on the study area of the impact processes in each sub-area should be analyzed. This analysis should take into account the analyses from the diagnoses relating to the interactions between the processes in the sub-area and their contextualization in the study area, emphasizing those elements that are instrumental in making the impact processes in a given sub-area spill out beyond its boundaries and reverberating in others.

Table 4.8.6.01 – Impact indicators and assessment elements for the Territorial Organization synthesis component

Synthesis component	Impact indicator	Assessment elements
Territorial organization	interference in the patterns of human settlements and mobility of the local population	• number, location and characteristics of the settlements affected either partially or totally; • availability of land for the planned resettlements; • estimate of the number of resettled people (rural and urban); • workers' village: location, population during works, relationship with local population.
	interference in circulation and communication	**accessibility** • facilities for production, consumption and services affected; • extent and functions of transport infrastructure affected; • extent and functions of transport infrastructure expanded; • estimate of number of people affected by loss of transport infrastructure; • estimate of number of users of water resources affected; • transport integration affected. **reversibility of interferences in circulation and communication** • alternatives to functional relationships interrupted; • alternatives to the circulation and communications affected.
	interference in the political and administrative structure	• loss of land: (surface area and proportion of total are covered by municipality); • estimate of number of electors resettled and proportion of total municipal electorate; • loss of representatives; • role of municipal seats of government and public, municipal, state and federal institutions affected.
	interference in territorial management	• incompatibilities with policies, plans and programs for local or regional development; • synergies with policies, plans and programs for local or regional development.

4.8.7 Regional Economy

The impact processes should be identified for each sub-area affected, with priority being given to characterizing those processes that could reduce the production of wealth and public revenues or inflict damage on resources, as well as any potentialities, so that their consequences on the local and regional economy can be assessed.

Once the impact processes in each sub-area have been identified per project, the assessment elements should be selected that best characterize these processes with a view to estimating the extent to which the foundations of the economy in the study area will be affected. This should encompass the elements that structure the economy in the sub-areas and the characteristics of the projects, so that the interference they will have on each sub-area can be observed.

It should also be remembered that the assessment elements are used for the basic design of the projects and later for estimating the socioenvironmental costs, which are an integral part of the implementation costs. Some of these elements will be used in the assessment of positive impacts of the cascades in the Final Studies.

At this point, interdisciplinary activities should be carried out with a view to integrating the impact processes identified with those from the other synthesis components, so that the assessment elements that best represent the existing interactions can be selected.

The intensity of the negative socioenvironmental impact of each project on the sub-areas affected should be estimated by analyzing the following impact indicators, which will help synthesize the different assessment elements:

- interference in economic activities;
- interference in resources and potentialities;
- interference in municipal finances.

1) Interference in economic activities:

- number and type of establishments affected: when the object of the analysis is the formal sector of the economy, traditional indicators can be used. However, when the informal economy is prevalent, qualitative analyses must be undertaken. This is the case of simple activities for which no regular wages are paid, which are important because: (a) they represent alternative sources of employment and income, which are fundamental for an incipient urban economy; and (b) the impacts they suffer are not always recognized or acknowledged. They are therefore particularly important for assessing the impacts on the forms of labor that are not entirely subject to the capitalist dynamics of production and circulation;
- output affected per sector: this estimate is particularly important in cases where the analyses carried out in the diagnosis indicated the particular importance of one given sector. In these cases, the following should be considered: quantity of output per type of production and land area lost for this output (for the primary sector); value of transformation activities (for the secondary sector); details of the effect of lost income in trade and services on the standard of living (for the tertiary sector);
- river-related economic activities affected: so-called “riverside” economic activities affected, such as pottery works and floodplain agriculture, can be assessed by the value of their output and the number of jobs created;

- social and economic traits in the water basin: the economic activities affected should be characterized according to the role they play in the local and regional economies and their significance in assuring continued living standards;
- loss of employment and income: the number of jobs cut should be quantified and the way this is felt by the local population and number of people in employment should be qualified;
- markets affected: the sectors affected by the loss of output should be estimated, and the impact of this on the markets identified should be qualified;
- occurrence of conditions that sustain the reproduction of activities: the feasibility of resettling or restructuring the activities affected should be assessed in such a way that producers, consumers and land/property owners would not suffer any damages or loss.

2) Interference in resources and potentialities, especially concerning water uses:

- characteristics and size of the resources and potentialities in the river basin that are affected, such as mineral resources, arable areas, extraction of non-timber forest products, energy, tourism, areas of biological and genetic potential, and their relative importance within the study area;
- social and economic significance of the potentialities affected: qualification of their importance for maintaining living standards in the study area;
- existing/potential uses of the water resources affected or made unfeasible and number of people affected: the information used here must be brought into line with the information used for assessing the different water uses and the information and projections prepared for the scenarios of multiple water uses (item 4.2.2). It is mandatory to use this information. Note that the impacts on this assessment element should not be taken into account for those projects/cascades where the pre-defined scenarios are observed. This information will also be used in assessing the positive impacts in the Final Studies.

3) Interference in municipal finances:

- difference in tax revenues and revenue transfers: a fall in revenues is a direct consequence of a fall in tax revenues and monies distributed from shared revenues, which are directly related to the land size, the population size and their economic activities. The task is to estimate the effects arising if activities that generate taxes, transfers and draw-downs (from the Municipal Fund, *Fundo de Participações dos Municípios*) ceased, were made unfeasible or were disrupted;
- total revenues collected as financial compensation for municipalities affected and from ISS (VAT) during construction, for the assessment of positive impacts.

Table 4.8.7.01 shows the impact indicators and assessment elements for this synthesis component.

Attribution of levels of impact

Negative impacts should be attributed per sub-area affected and for each prospective project. A compilation of the aspects assessed using the criteria presented should be used in attributing a degree of negative impact a scale from **zero** to **one** to reflect the degree of interference to which the basis of the economy in the sub-area is subject.

Finally, the repercussions on the study area of the impact processes in each sub-area should be analyzed. This analysis should take into account the analyses from the diagnoses relating to the interactions between the processes in the sub-area and their contextualization in the study area, emphasizing those elements that are instrumental in making the impact processes in a given sub-area extrapolate its boundaries and reverberate in others.

Table 4.8.7.01 – Impact indicators and assessment elements for the Regional Economy synthesis component

Synthesis component	Impact indicator	Assessment elements
Regional economy	Interference in economic activities	• number and type of establishments affected; • value of output affected per sector; • river-related economic activities affected; • social and economic significance of the activities; • job cuts and income reduction; • jobs and income created; • markets affected; • existence of conditions to support the reproduction of activities.
	Interference in resources and potentialities, especially concerning water uses	• characteristics and size of the resources and potentialities in the river basin that are affected (mineral resources, areas suitable for agriculture, extraction of non-timber forest products, tourism, areas of biological and genetic potential); • opportunity for harnessing resources and potentialities; • social and economic significance of the potentialities affected; • existing/potential uses of the water resources affected / made unfeasible and number of people affected; • opportunities for new uses of water resources and number of people benefited.
	Interference in municipal finances	• difference in tax revenues and revenue transfers; • revenues from financial compensation and ISS (VAT).

4.8.8 Indigenous Peoples/Traditional Communities

The impact processes should be assessed for each indigenous land or each area occupied by a traditional community affected per project.

Once the impact processes have been identified, the assessment elements should be selected that best characterize these processes, with a view to estimating their level of interference in the forms of social reproduction present. These elements should encompass aspects relating to the pre-existing features of the communities and the characteristics of the projects, so that the relationship between the project and the thresholds of the pre-existing ethno-ecological conditions can be observed, which may contribute towards exacerbating conflicts, considering the specific history of each group and in view of their relationship with the rest of Brazilian society and the terms of preservation of their territory, which is often subject to invasions.

At this point, interdisciplinary activities should be carried out with a view to integrating the impact processes identified with those from the other synthesis components, so that the assessment elements can effectively represent the interactions that exist.

When estimating the intensity of the socioenvironmental impact of a project on a given indigenous land, the following impact indicators should be considered, which will help combine the assessment elements:

Exacerbation of Conflicts

This indicator has to do with interference in the socio-cultural structure and its interaction with the land. Taking the pre-existing status as previously characterized (pre-existing situations of conflict, trespassers on indigenous lands, legal protection of lands) and considering the ratio of flooded to available land, it can be assessed whether or not there is any negative impact on the political unit, the interethnic relationships (including integration with the market) and/or any historically constructed ties within and between groups. Also, as a final outcome of the assessments, it can be gauged whether the very survival of the group is put in jeopardy by the plans.

The following assessment elements should be used:

- pre-existing conflict situations;
- existence of territorial invasions;
- ratio between flooded/available land;
- legal protection of lands;
- impacts on political unity;
- impacts on interethnic relationships (including integration with the market);
- risk of extinction;
- impacts on ties within the groups and between different groups.

Interference in Ethno-Ecological Conditions

This indicator addresses the material structure of a group and the degree of its dependence on the land. The analysis of ethno-ecological aspects is integrated with the aspects relating to a group's material conditions for survival, as well as their cultural identity. It is therefore worth observing the ratio of flooded land to available land and prior knowledge about the importance of this land to a group's culture and survival to assess whether these factors will have an impact on its material reproduction, the degree of interference in sacred sites and/or sites of cultural and geomorphological heritage, and the ramifications this would have on the set of cultural traditions that give a group its specific cultural identity.

The following elements should be used:

- ratio between flooded land and available land: observe the capacity of the land to provide for a group's material reproduction;
- importance of the flooded land to a group – cultural and/or survival factors;
- interference in sacred sites and/or sites of cultural or geomorphological heritage (with an impact on cultural traditions).

Table 4.8.8.01 shows the impact indicators and assessment elements for this synthesis component.

Levels of impacts should be attributed to the study area for each prospective project. A compilation of the aspects assessed by the impact indicators should be used in attributing a degree of negative impact on a scale from zero to one to obtain an estimate of the extent of the interference in a group's forms of social and cultural reproduction.

Table 4.8.8.01 – Impact indicators and assessment elements for the Indigenous Peoples / Traditional Communities synthesis component

Synthesis component	Impact indicator	Assessment elements
Indigenous peoples / Traditional communities	Exacerbation of conflicts	• pre-existing conflict situations; • existence of territorial invasions; • ratio between flooded/available land; • legal protection of lands; • impacts on political unity; • impacts on interethnic relationships; • risk of extinction; • impacts on ties within the groups and between different groups.
	Interference in ethno-ecological factors	• ratio between flooded / available land (observe the capacity of the land to provide for a group's material reproduction); • significance of the flooded land to the group; • interference in sacred sites and/or sites of cultural or geomorphological heritage.

4.9 STANDARD ELETROBRAS COST ESTIMATE

4.9.1 Concepts

The methodology used in the Preliminary Studies for designing the structures should be simplified and the costs should be estimated in overall terms per structure, resulting in simplified cost estimates for the projects in the cascades formulated. The costs of the civil construction and equipment are grouped together so that the cost of a set of works, structures and services can be obtained more easily and an overall cost estimate can be drawn up quickly.

The main aim of the cost estimate at the Preliminary Studies stage is to give a quick, albeit approximate, idea of the costs of the projects, which will be used in the decision-taking process for selecting or proposing new cascade options.

The methodology used in this Manual generates costs using graphs and tables, and making use of parameters that are either preset or calculated by the user.

The cost tables and graphs and the recommendations in this Manual represent the average values obtained in studies already undertaken for a number of Brazilian hydropower stations. This study provides a synthesis of the overall civil construction and equipment costs for the main structures, using data from projects at the Feasibility Study stage and information from the 1997 Manual.

Any other kinds of structures not normally used in project design, any exceptional cases, or any items that fall outside the cost curves will need to be analyzed specifically by the project engineers.

4.9.2 Preliminary Cost Estimate

The cost estimates in the Preliminary Studies should be based on the main accounts from the Standard Eletrobras Cost Estimate (OPE) and should follow the summarized list below, which contains all the structures and equipment needed, including the indirect costs and interest payable during construction.

Account	Description
10.	land, resettlements, relocations and other socioenvironmental actions
11.	powerhouse (civil construction) and related land developments
12.16.	river diversion
12.17.	dam and dikes
12.18.	spillway
12.19.	intake
13.	turbines and generators
14.	auxiliary electrical equipment
15.	miscellaneous equipment
16.	roads, railroads and bridges
17.	indirect costs
18.	interest during construction

Annex C contains spreadsheet **49ope.xls**, which covers the cost estimate for the Preliminary Studies.

4.10 PROJECT DESIGN AND COST ESTIMATE

In this item the criteria and instructions for dimensioning and estimating the costs of projects are set out. The estimate assumes a preliminary definition of the structures and layouts, as well as the representative mean unit prices of the respective services and equipment.

The unit prices and graphs are expressed in Brazilian Reais and refer to the December 2006 database.

The preliminary cost estimate can be done using the information from spreadsheet **49ope.xls**, in Annex C, or using the tables and graphs presented in items 4.10.1 to 4.10.13.

4.10.1 Lands, Resettlements, Relocations and Other Environmental Actions (account.10)

The components that best express the socioenvironmental costs are identified so they can be included in the installation costs of the plants:

- lands and land developments for the reservoir, construction site, borrow areas and residential compound;
- resettlement of local residents, including indemnity, land acquisition and developments for rural resettlement projects and the relocation of towns and villages, including infrastructure works and social service facilities required in each case;
- recomposition or relocation of regional infrastructure.

The socioenvironmental costs to be estimated are set out below, along with the procedures to be used and the corresponding accounts in the Standard Eletrobras Cost Estimate (OPE).

Acquisition of urban lands and land developments (account .10.10.10)

- identification of the areas of land and land developments to be acquired based on the cartographic and topographic studies. When there is partial interference in towns or villages, it is recommended that the criteria concerning the physical quantities to be used for estimating the costs be defined in advance and set out clearly;
- survey of unit prices per m^2 of urban land, from information collected in the field;
- survey of unit prices per m^2 of urban land development, from secondary sources supplemented by field surveys. The estimate can be made from secondary sources by looking at statistics on the cost per m^2 of floor area, supplied by SINDUSCON and FIBGE, making any adjustments necessary to the specific situations at hand.

Acquisition of rural lands and land developments (account .10.10.11)

- determination of the areas of land to be acquired based on the cartographic and topographic studies;
- land prices from December 2006, expressed in Reais per hectare (R$/ha), can be obtained from secondary sources. Fundação Getúlio Vargas brings out statistics on monthly average prices per hectare for each unit of the federation, separated into four types of land use: land for crops, land for pasture, shrubland and woodland. Some states have more detailed statistics for their own regions. Independent of the particular circumstances, it is important to gather information from the field in the Preliminary Studies in order to calibrate the statistics, especially in regions where the land is known to be more expensive and where the surface area of the reservoirs will be larger. It is also a good idea to survey the

prices of land with and without land developments, given the difficulty of estimating quantities and prices of rural land developments in Inventory Studies. The most recent statistics should be used, and recourse to inflation adjustment should be avoided;

- determination and identification of the land developments to be acquired, using satellite images and aerial photographs to indicate the location and area, in m^2; and qualification on field trips of the most significant land developments by their type and features;
- surveys of prices of rural land developments from December 2006, which should be expressed in Reais/m^2. Given the acknowledged difficulty of obtaining information of this kind, there are two alternative ways of estimating these values. The first consists of estimating the price of rural land developments indirectly by the price of the land, as mentioned above. In this case, the procedure set out in the item above for surveying and identifying the rural land developments does not need to be detailed. The second alternative consists of making an estimate using secondary statistics on the cost per square meter of floor area, supplied by SINDUSCON and IBGE, and adopting this information to the specific situation.

Relocation of roads, railroads, bridges, pipelines, transmission lines and telephone lines (account .10.11)

- the unit costs for the relocation of roads, railroads and bridges are presented in tables B01, B02 and B03 of Annex B. Should any other relocations be required, their costs should be estimated according to the general criteria adopted.

Resettlement of households (account .10.11.20)

- if the environmental studies show clearly that rural households, towns, villages, indigenous communities or any other ethnic groups protected by law will have to be resettled, it is recommended that the socioenvironmental costs of the corresponding programs be estimated. The main cost components of these programs, such as the acquisition of lands and land developments, building of infrastructure, construction of buildings, individual and collective land developments, among others, can be calculated and have their costs estimated based on unit prices, using the same general criteria used for the cost estimates in the Inventory Studies.

Physical-Biota Programs (account .10.15.45)

- a minimum 0.5% of the total cost of the hydroelectric project should be set aside for these programs. CONAMA resolution 371/2006 states that this percentage should go towards programs designed to compensate for any damage to ecosystems, including the establishment of conservation areas. However, depending on the results of the environmental studies, it may be prudent to set aside more than 0.5%, especially for projects in the Amazon, where the cost of these programs tends to be higher.

Socioeconomic Programs (account .10.15.46)

- when the need for programs of this kind is indicated in the socioenvironmental studies, their cost should be estimated on a case by case basis or by setting aside a percentage of the total cost of the environmental programs.

Miscellaneous (account .10.27)

- in order to cover miscellaneous costs, 20% of the sum of account.10.10 (acquisition of lands and land development), account.10.11 (resettlement and relocation) and account.10.15 (other socioenvironmental actions) should be set aside.

Environmental Studies (account .17.22.40.54)

- in order to over these costs, 20% to 30% of the total cost estimated for the engineering studies should be set aside.

4.10.2 Powerhouse (civil construction) and Related Land Developments (account .11)

Installed capacity is calculated as shown in item 4.6.5. The value of the maximum net head is used in Graph 5.7.2.01 of the Final Studies to determine the type of turbine and its maximum power output for the head available. The highest power per unit should be used, assuming a minimum of two units.

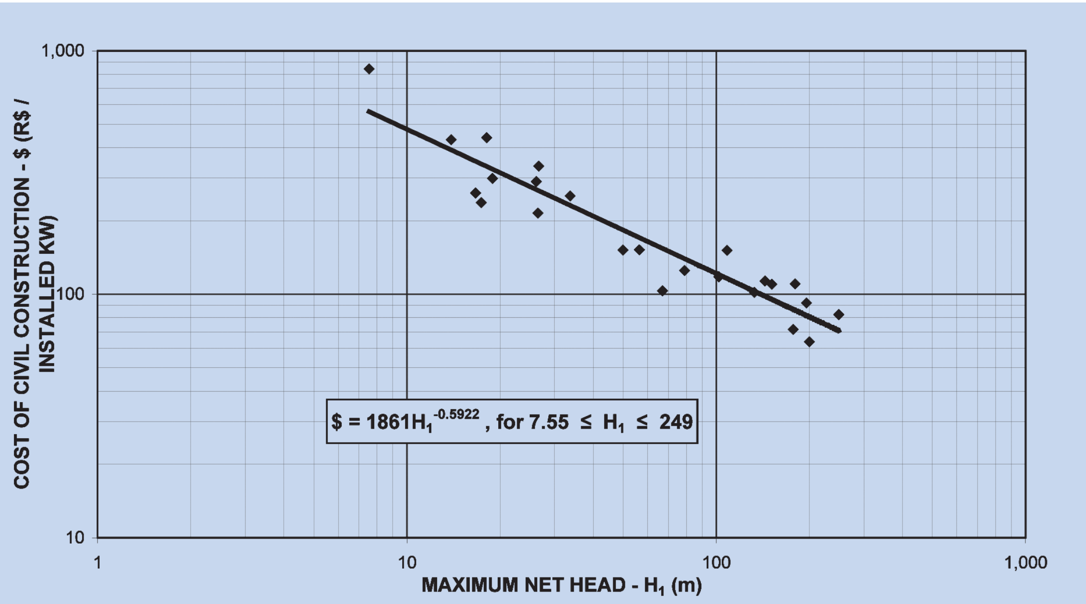
The total cost of this account is the sum of the construction cost of the powerhouse, the cost of the land development for the plant, and the cost of the operators' village.

The **total civil construction cost of the powerhouse** is calculated by multiplying the cost per kW installed by the total power output of the powerhouse. The cost per kW is provided by Graph 4.10.2.01 as a function of maximum net head and installed capacity.

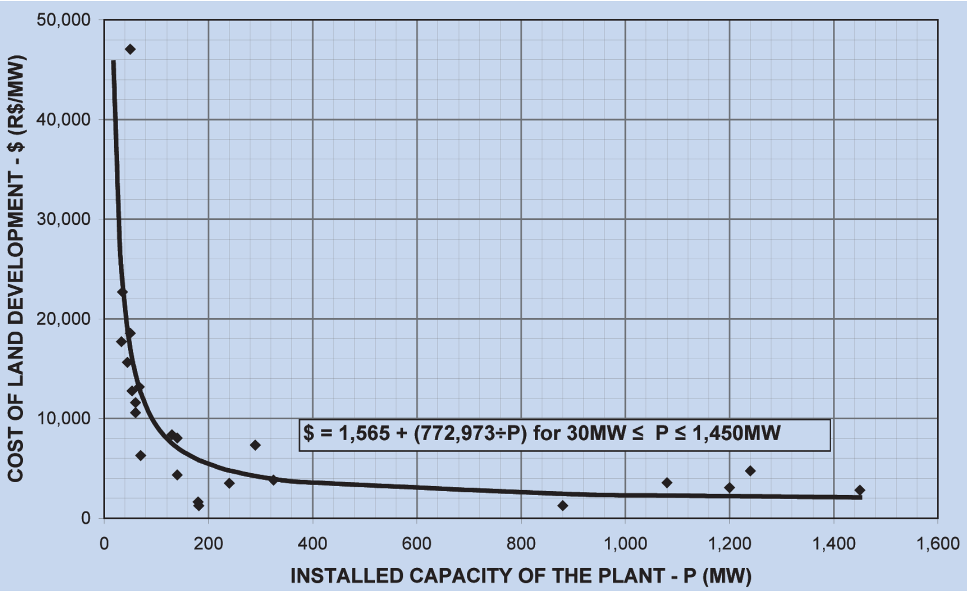
The cost of the **land developments for the plant area** is obtained from Graph 4.10.2.02, where the unit cost per MW is given as a function of the plant's installed capacity.

At this stage of the studies, the workers' village can be considered as part of the construction site, so in this case its costs will be included in the camp item (account.17), or else this cost can be included as a miscellaneous cost (account.11.27).

The amount to be set aside for miscellaneous costs should be 25% of the sum of accounts.11.12 (land development for the plant area) and .11.13 (powerhouse).



Graph 4.10.2.01 – Total cost of civil construction for outdoor powerhouse (PCE, 2007)

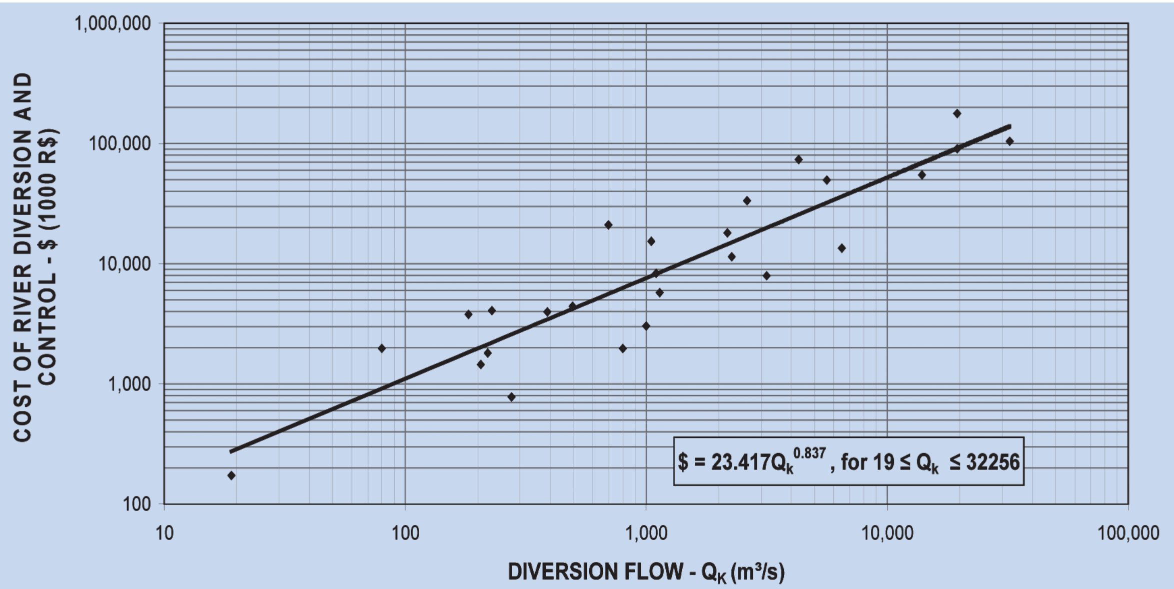


Graph. 4.10.2.02 – Cost of land development in the plant area (PCE, 2007)

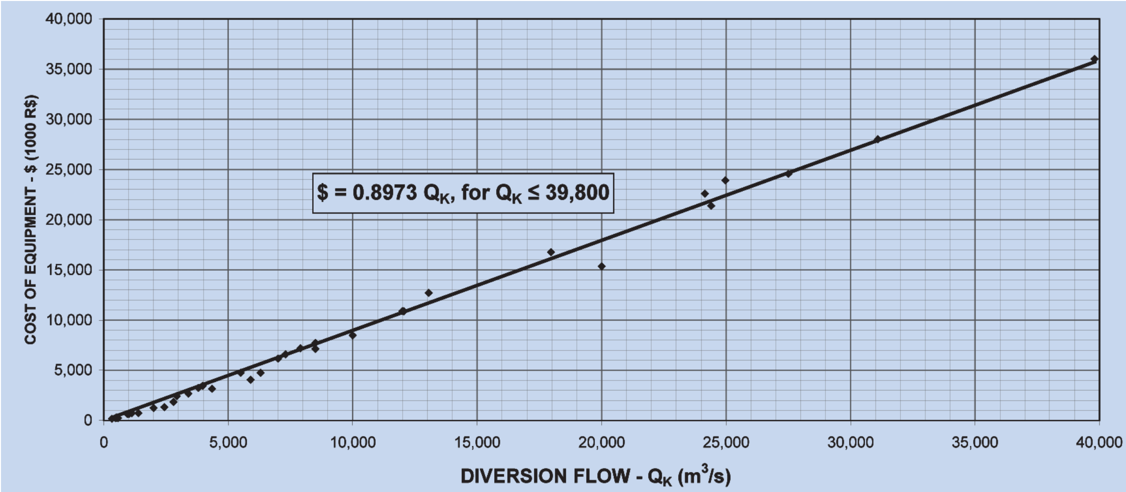
4.10.3 River Diversion (account 12.16)

The river diversion should be designed according to the layout defined for the project. In the Preliminary Studies, the solution does not have to be optimized, nor do details of the diversion scheme have to be established. The idea is to define the approximate size of the main structures used specifically for the diversion, which will have specific costs associated. The **overall cost of civil construction and services** for **river diversion and control** is obtained directly from Graph 4.10.3.01 as a function of the diverted flow.

The **cost of the hydromechanical equipment** is obtained from Graph 4.10.3.02 as a function of the diverted flow, and is valid for sluiceways, galleries and tunnels. An extra 20% should be set aside for miscellaneous costs in this account .12.16.



Graph 4.10.3.01 – Cost of civil construction and services for river diversion and control (PCE, 2007)



Graph 4.10.3.02 – Cost of hydromechanical equipment for river diversion (PCE, 2007)

4.10.4 Dams (account .12.17)

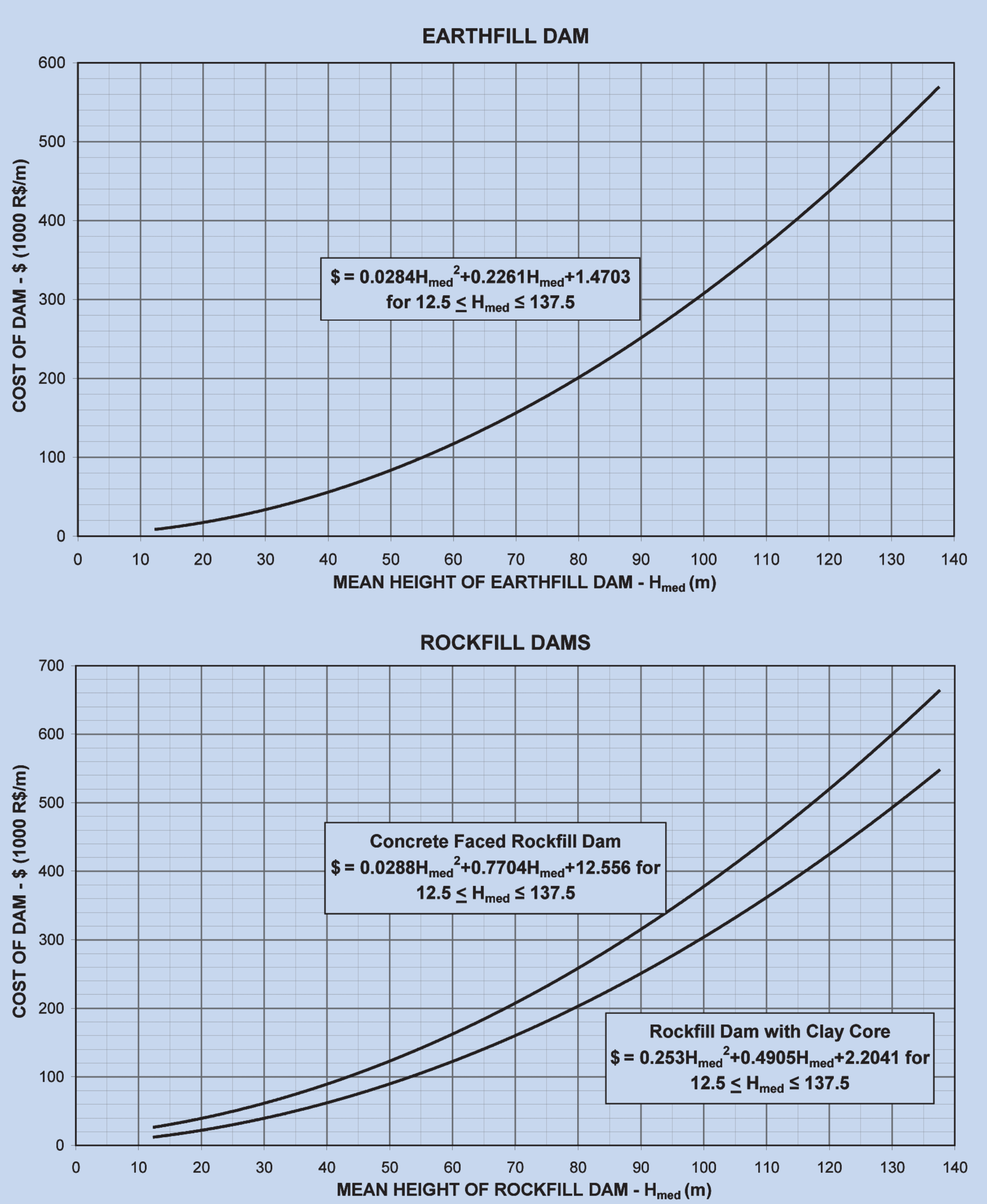
Based on the cross-section of the river valley, determine the mean height and length of the dam structure and obtain the cost corresponding to the kind of dam, using the following graphs:

- Graph 4.10.4.01 – embankment dams;
- Graph 4.10.4.02 – roller compacted concrete dams;
- Graph 4.10.4.03 – conventional concrete dams;
- Graph 4.10.4.04 – transition and retaining walls.

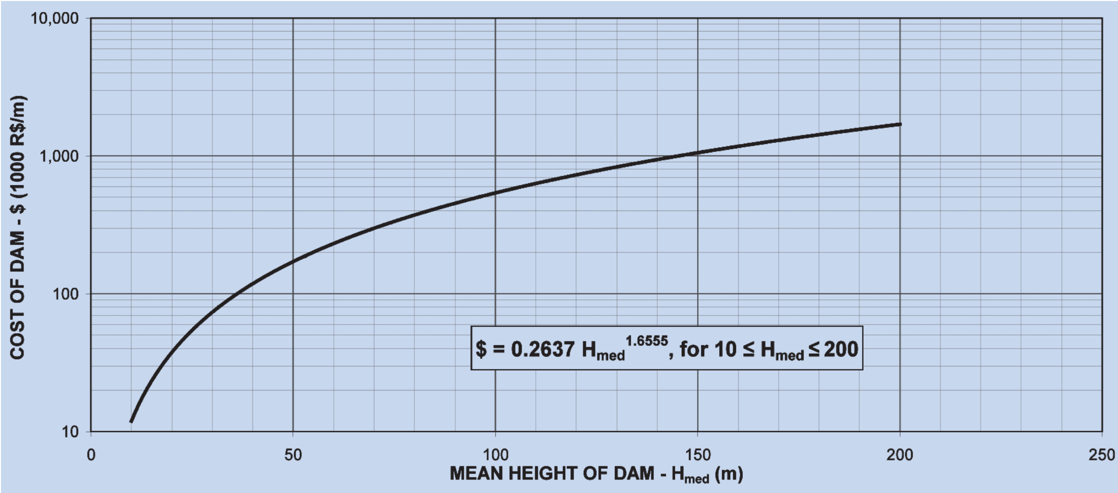
The mean height can be determined graphically by taking short sections of approximately constant height and calculating the weighted average of these values, or using computer methods.

For the concrete walls and transitions, the costs can be obtained using spreadsheet 584m.xls, prepared for the Final Studies, in Annex C. When the input data are not available they can be estimated.

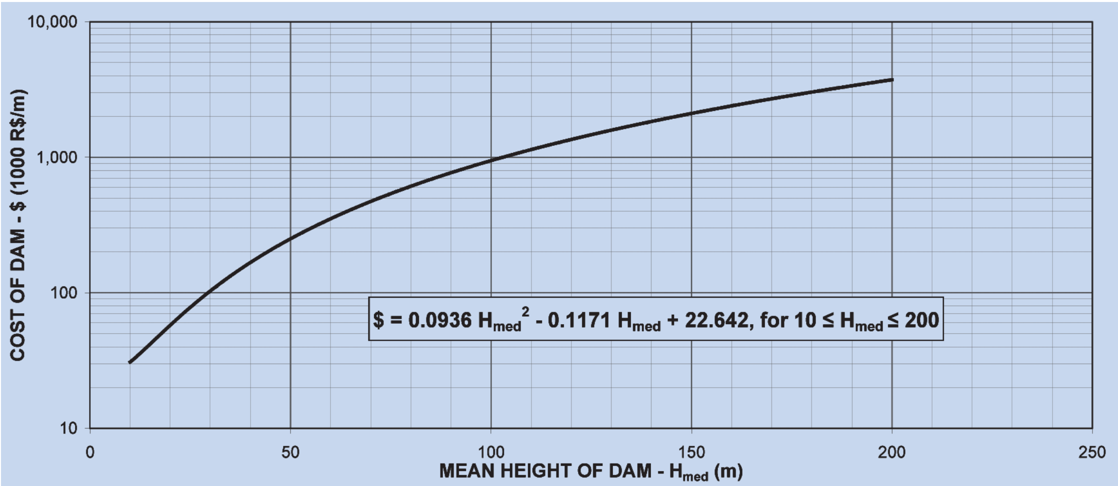
Miscellaneous costs should be calculated at 25% of the total of account .12.17.



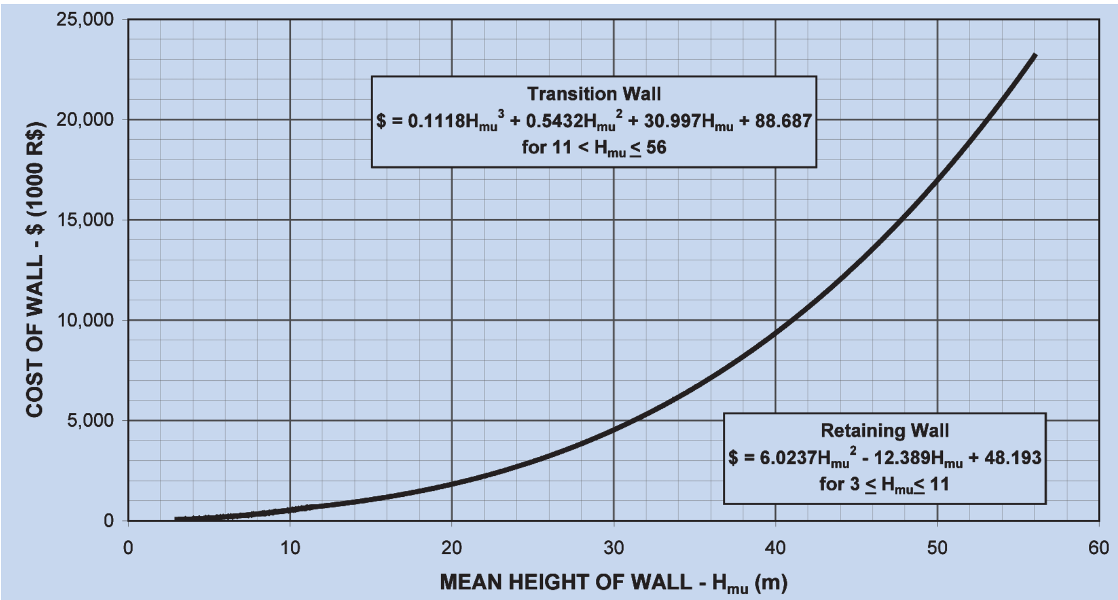
Graph 4.10.4.01 – Cost of embankment dams (PCE, 2007)



Graph 4.10.4.02 – Cost of roller compacted concrete dams (PCE, 2007)



Graph 4.10.4.03 – Cost of conventional concrete dams (PCE, 2007)



Graph 4.10.4.04 – Cost of concrete transition and retaining walls (PCE, 2007)

4.10.5 Spillway (account .12.18)

At this phase, only surface spillways are considered in the design and cost estimate of the civil construction and equipment.

For abutment spillways or low ogee spillways, the **civil construction cost** can be obtained directly from Graph 4.10.5.01, as a function of the flow through this structure.

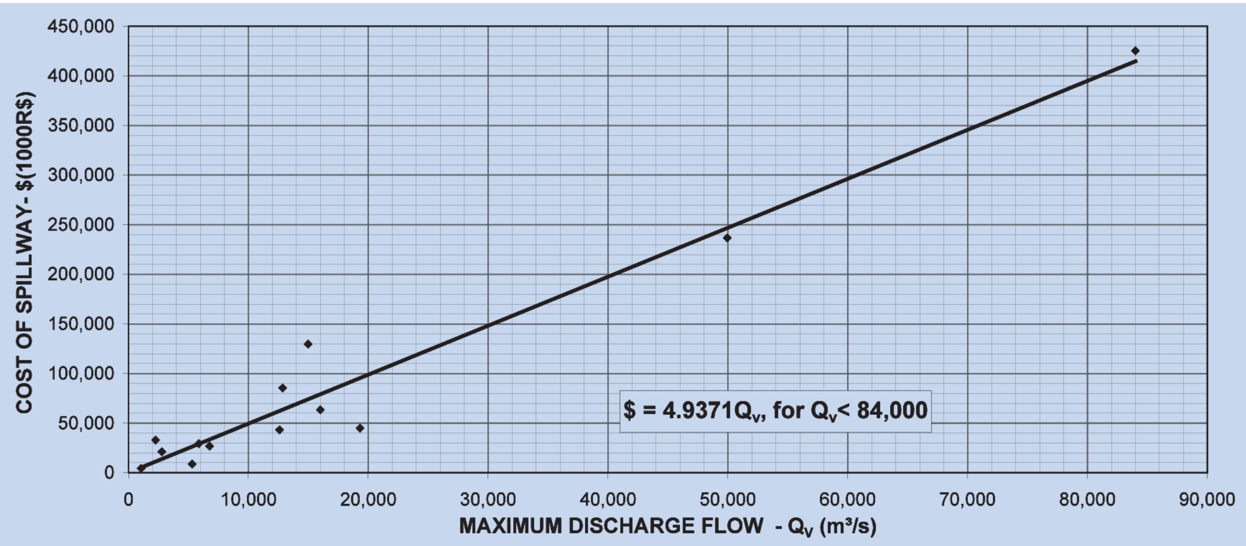
For high ogee spillways, the **civil construction cost** is a function of the maximum discharge capacity and the difference between the maximum water level in the reservoir and the maximum water level downstream, using the following graphs:

- Graph 4.10.5.02 - Civil construction cost of roller compacted concrete spillway with a high ogee crest;
- Graph 4.10.5.03 - Civil construction cost of a conventional concrete spillway with a high ogee crest.

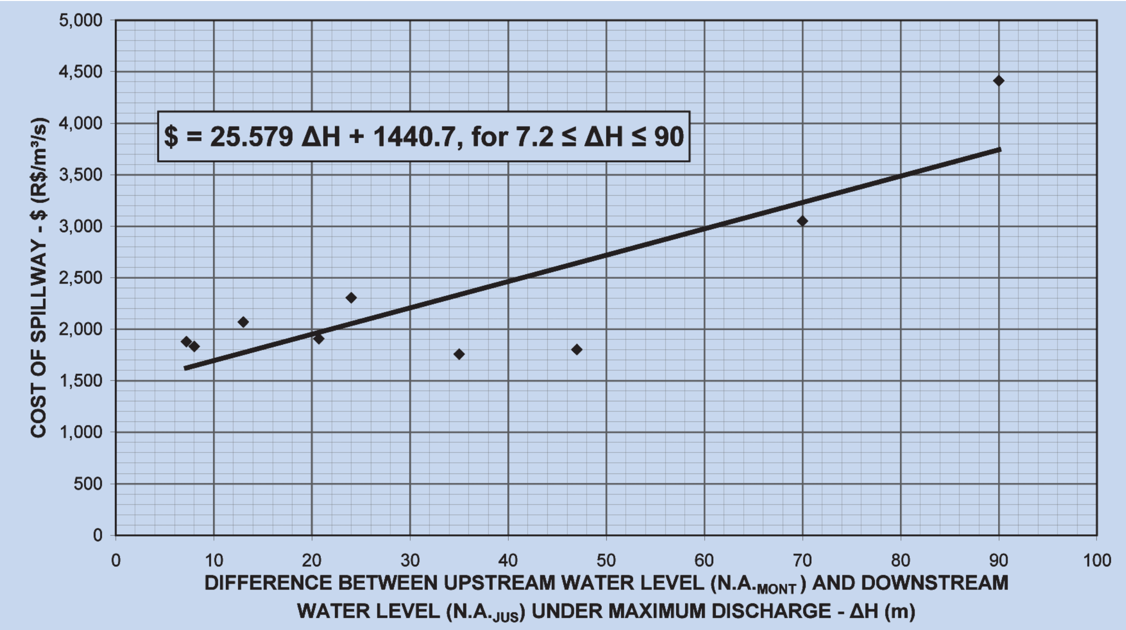
The overall civil construction cost of the spillway is obtained by multiplying the cost per m^3/s by the design flow (Graphs 4.10.5.01 to 4.10.5.03).

The **cost of the equipment** for the surface spillway is given by Graph 4.10.5.04, as a function of the spillway design flow (item 4.1.2).

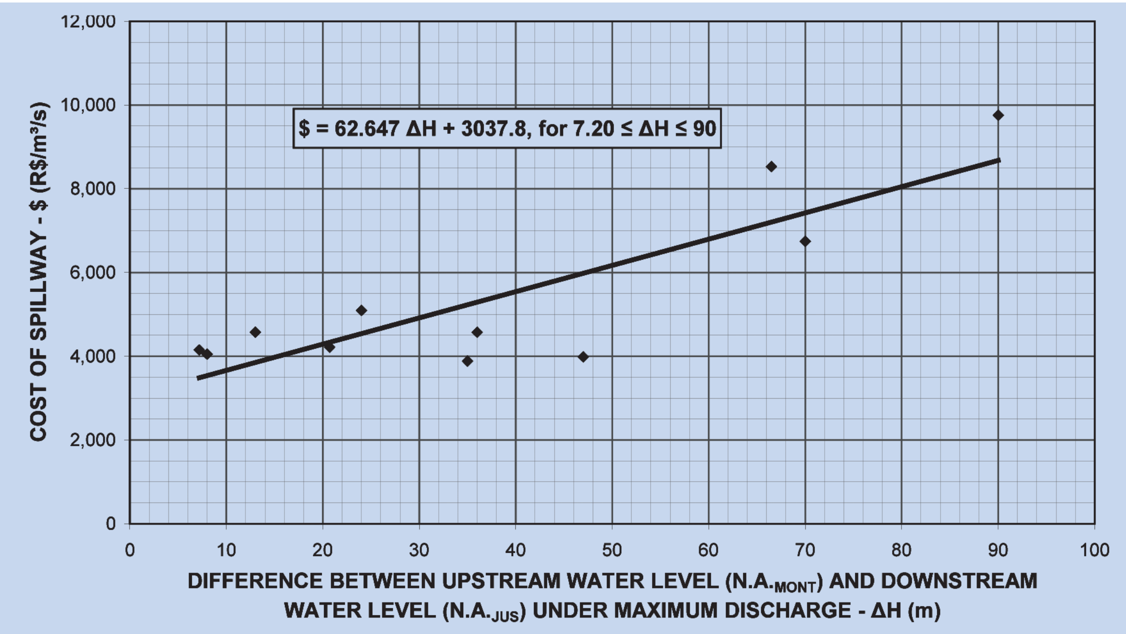
The amount to be set aside for miscellaneous costs should be 20% of the total of account.12.18.



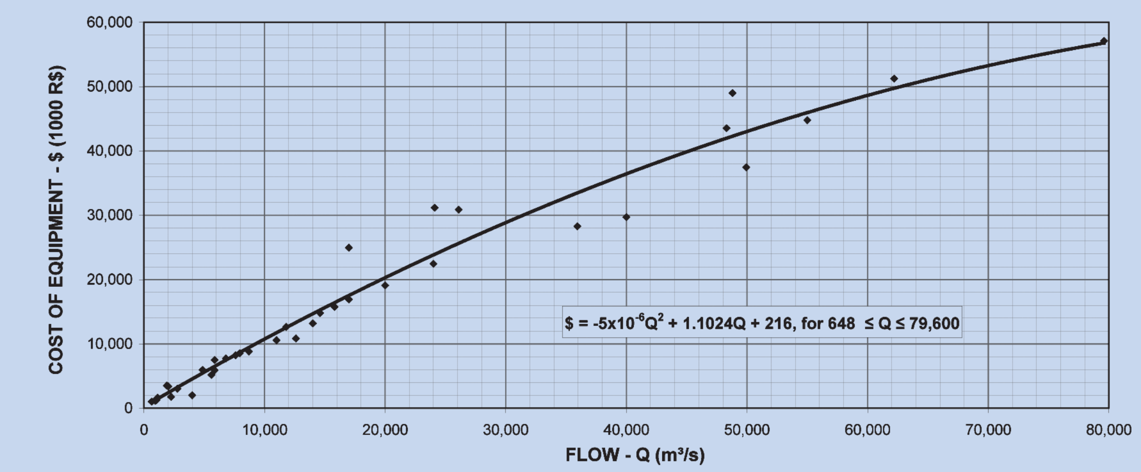
Graph 4.10.5.01 – Cost of civil construction work for spillway with a low ogee crest (PCE, 2007)



Graph 4.10.5.02 – Cost of civil construction work for roller compacted concrete spillway with a high ogee crest (PCE, 2007)



Graph 4.10.5.03 – Cost of civil construction work for conventional concrete spillway with a high ogee crest (PCE, 2007)



Graph 4.10.5.04 – Cost of hydromechanical and hoisting equipment for surface spillway (PCE, 2007)

4.10.6 Intake (account .12.19)

This account is the sum of the accounts for each of the following structures:

1) intake
2) headrace canal
3) intake penstock or tunnel
4) surge tank
5) pressure tunnels/penstocks
6) tailrace canal/tunnel

1) Intake (account .12.19.30)

The civil construction cost for a section of intake should be obtained directly from Graph 4.10.6.01, as a function of the parameter:

$$\frac{Q_a}{(H_t - d)^{\frac{1}{2}}}, \qquad (4.10.6.01)$$

where:

Q_a	maximum utilizable flow per opening of the intake, in m³/s;
Ht	height of a section of intake, in m; and
d	maximum drawdown, in m.

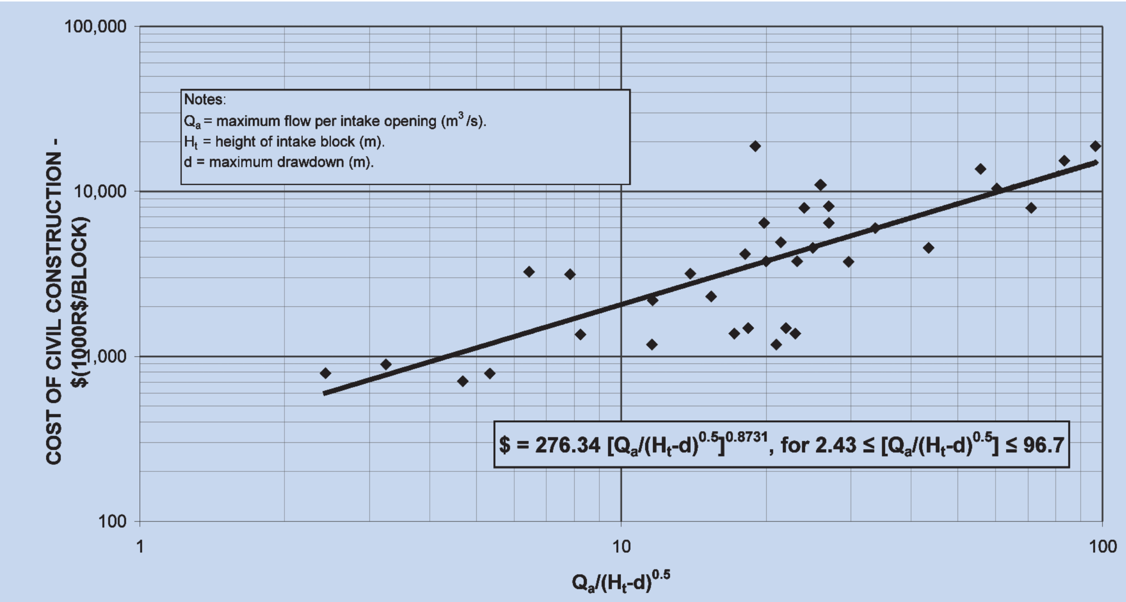
The total civil construction cost of the intake is obtained by multiplying the cost of one block by the total number of units.

The cost of the equipment for the intake is obtained from Graphs 4.10.6.02, 4.10.6.03 and 4.10.6.04, respectively, for use with semi-spiral concrete Kaplan turbines, Bulb turbines and other cases, as a function of the maximum utilizable flow per opening. Each graph has two curves. The cost obtained from curve "A" should be multiplied by the number of water intakes and added to the value obtained from curve "B". For each curve:

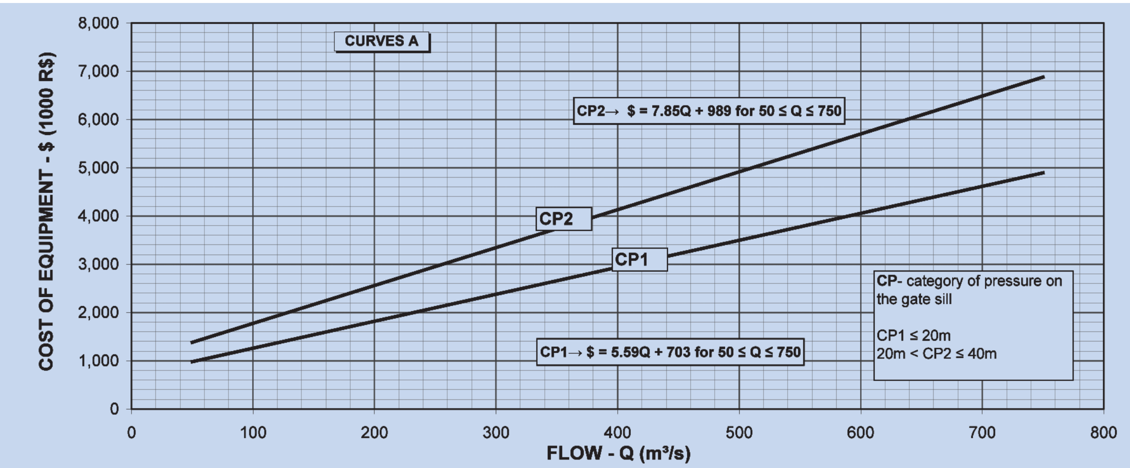
CP – Class of pressure on gate sill:

CP1 ≤ 20m   20m < CP2 ≤ 40m   CP3 > 40m

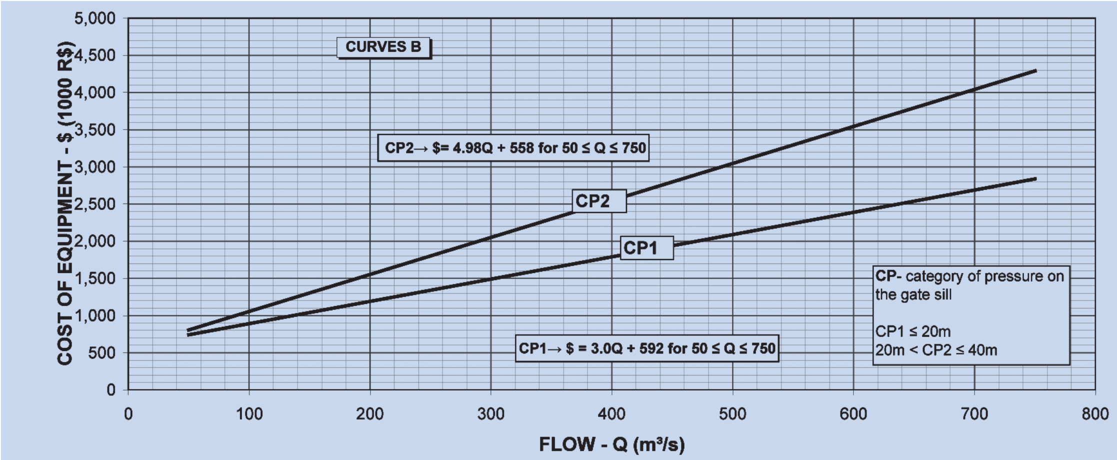
The pressure is obtained by subtracting the elevation of the intake sill from the maximum normal water level in the reservoir.



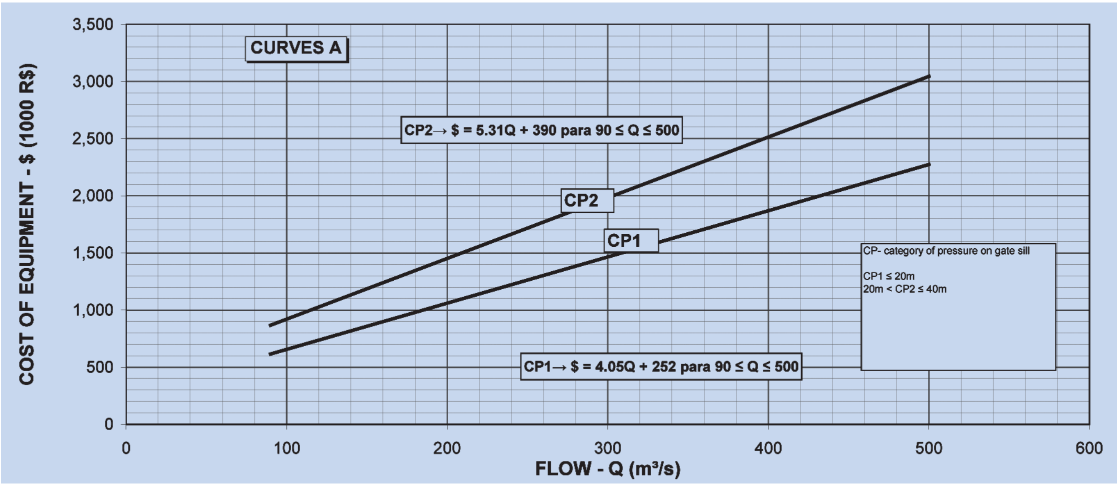
Graph 4.10.6.01 – Cost of civil construction work for intake (PCE, 2007)



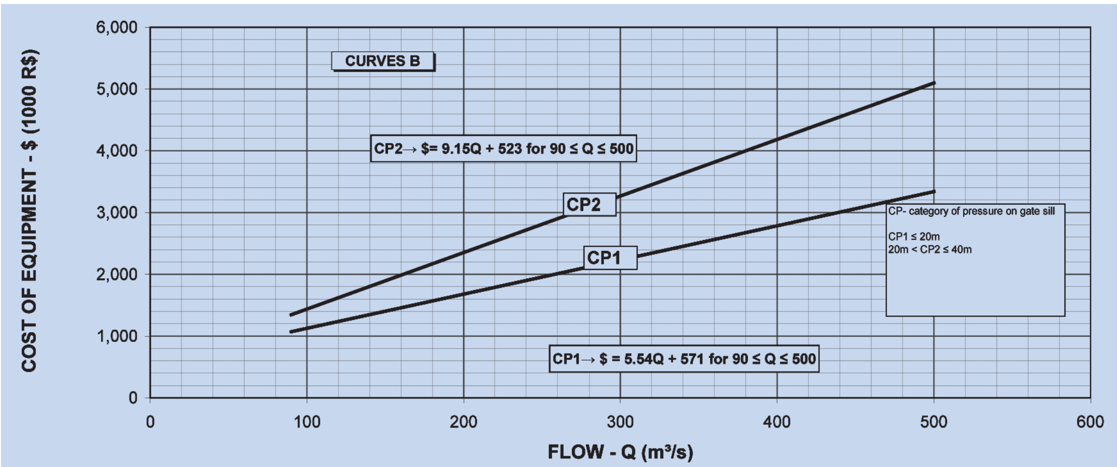
Graph 4.10.6.02 – Cost of intake equipment for Kaplan turbines with concrete semi-spiral casing (PCE, 2007)



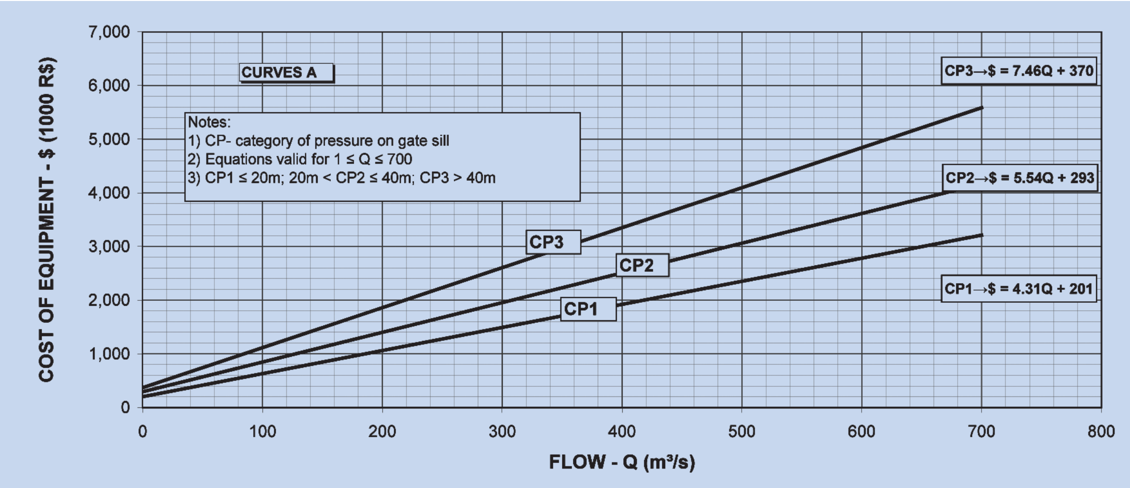
Graph 4.10.6.02a – Cost of intake equipment for Kaplan turbines with concrete semi-spiral casing (PCE, 2007)



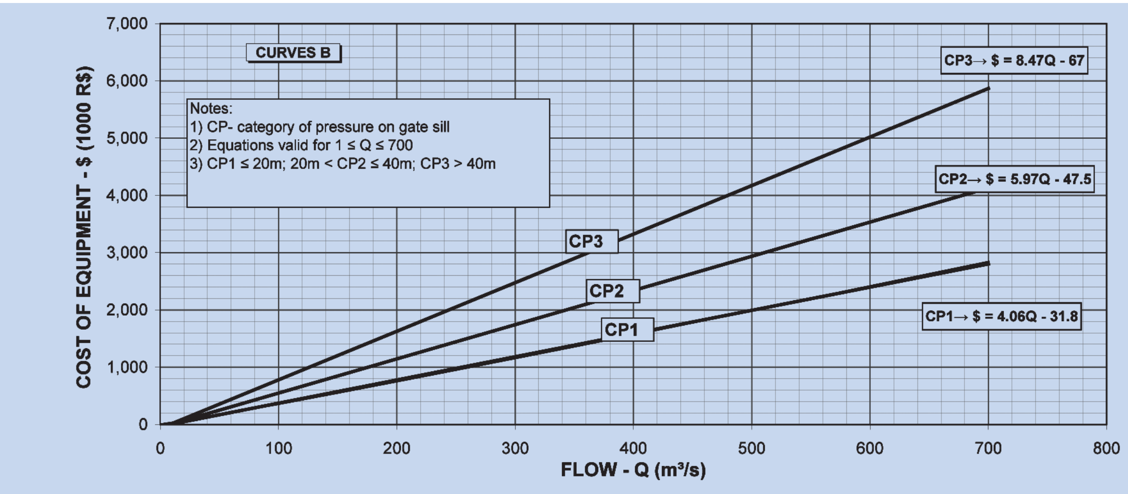
Graph 4.10.6.03 – Cost of intake equipment for Bulb turbines (PCE, 2007)



Graph4.10.6.03a – Cost of intake equipment for Bulb turbines (PCE, 2007)



Graph 4.10.6.04 – Cost of intake equipment for Francis or Kaplan turbines with steel spiral casing (PCE,2007)



Graph 4.10.6.04a – Cost of intake equipment for Francis or Kaplan turbines with steel spiral casing (PCE, 2007)

2) Headrace Canal (account .12.19.31)

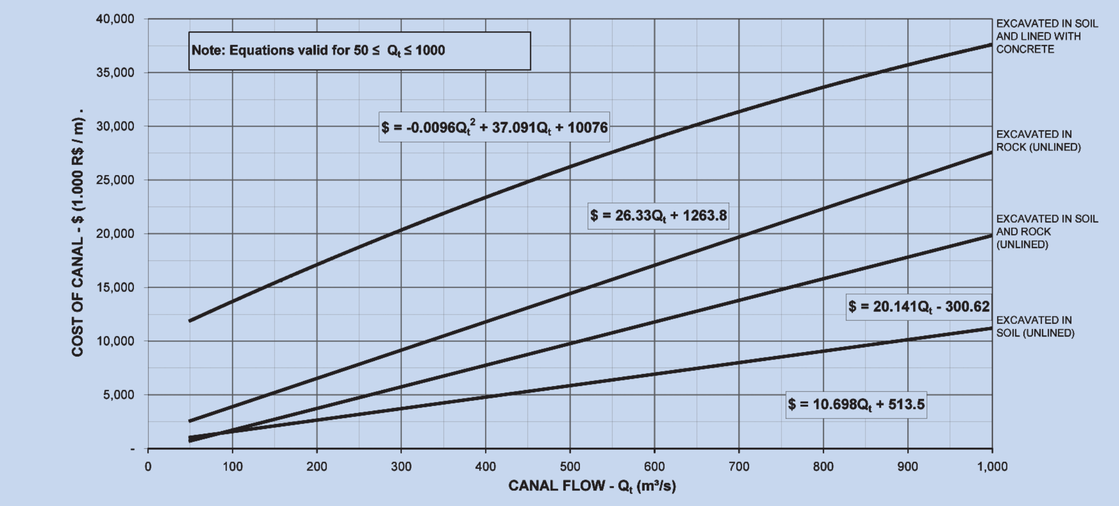
The cost per meter of the headrace canal can be obtained from Graph 4.10.6.05, as a function of the total maximum utilizable flow from the powerhouse. The overall cost is calculated on the estimate of the total length of the headrace canal and its cost per meter.

Total utilizable flow (Qt) is obtained by:

$$Qt = \frac{P}{0.0088 \times H_1} \qquad (4.10.6.02)$$

where:

P	installed capacity of the plant (MW)
H_1	maximum net head (m)



Graph 4.10.6.05 – Cost of the headrace canal (PCE, 2007)

3) Intake Penstock or Tunnel (account .12.19.32)

The cost per meter, both with and without lining, can be obtained from Graph 4.10.6.06, as a function of total maximum utilizable flow and the geological conditions at the site.

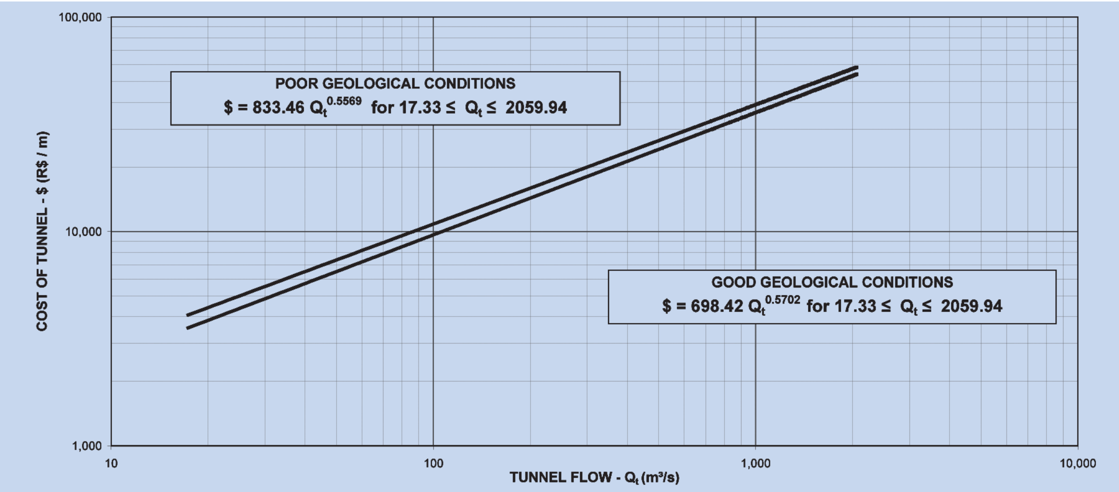
The total utilizable flow (Qt) is obtained by:

$$Qt = \frac{P}{0.0088 \times H_1} \tag{4.10.6.03}$$

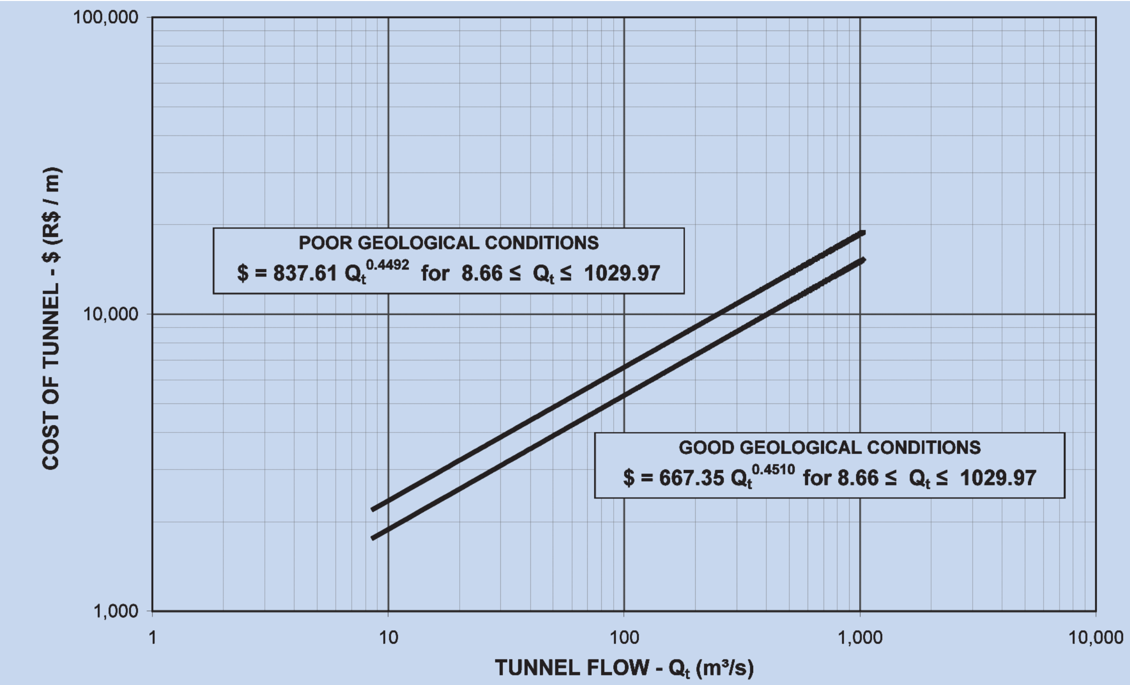
where:

P	installed capacity of the plant (MW)
H_1	maximum net head (m)

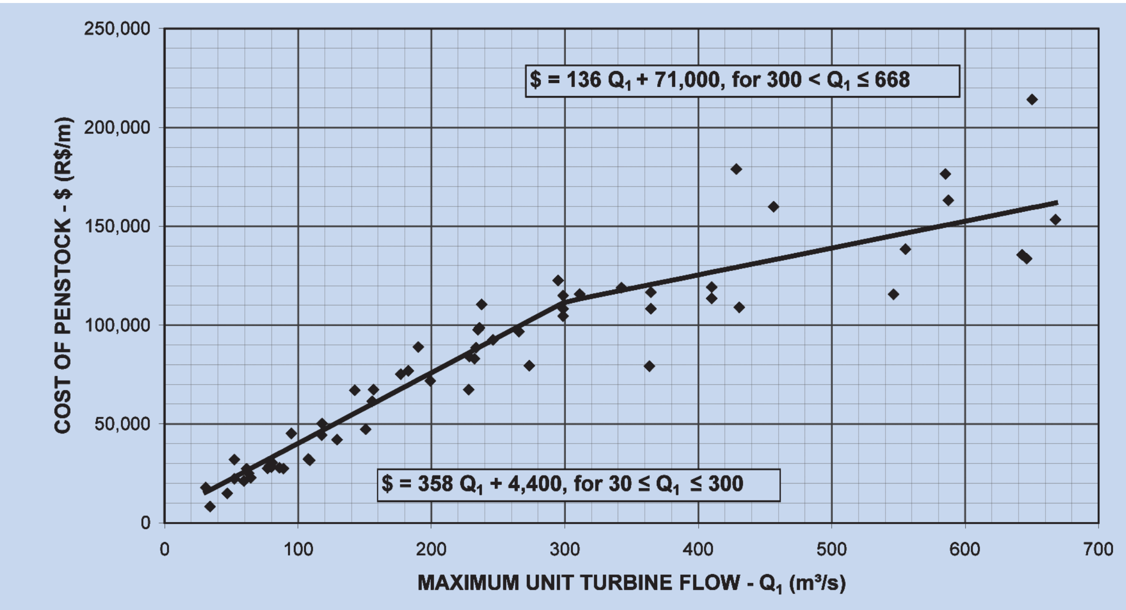
The overall civil construction cost of intake tunnels is obtained by multiplying the cost of the tunnel in Reais per meter by its total length.



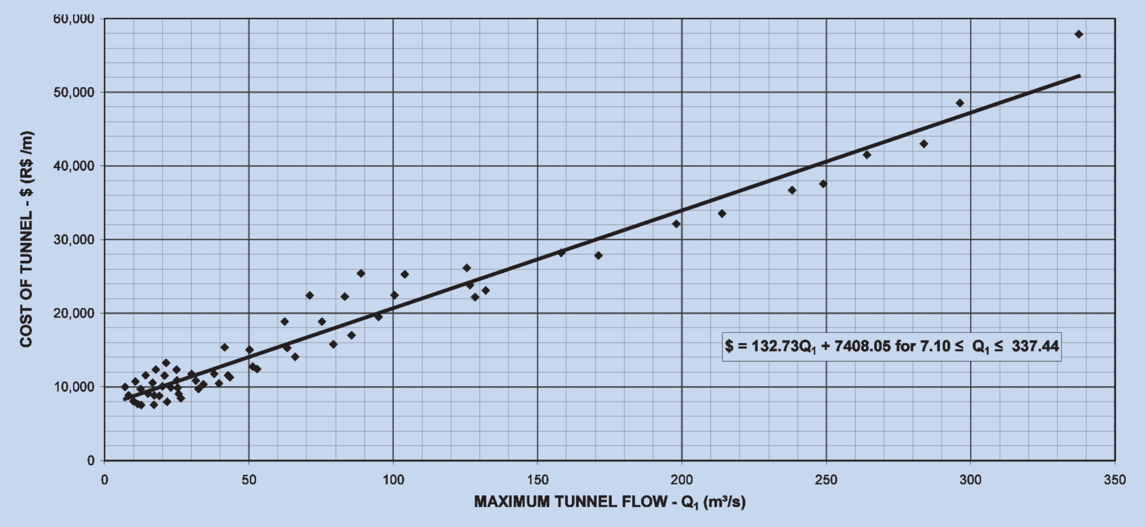
Graph 4.10.6.06 – Cost of lined headrace tunnels (PCE, 2007)



Graph 4.10.6.06a – Cost of unlined headrace tunnels (PCE, 2007)



Graph 4.10.6.07 – Cost of steel surface penstock (without valves) (PCE, 2007)



Graph 4.10.6.08 – Cost of pressure tunnel penstock (without valves) (PCE, 2007)

4) Surge Tank (account .12.19.33)

The cost of the surge tank can be determined using spreadsheet 586ch.xls prepared for the Final Studies, where the costs are obtained as a function of its diameter and maximum height.

5) Pressure Tunnel and/or Penstock (account .12.19.34)

The cost per meter length of the civil construction services and equipment for the pressure penstocks or pressure tunnels is obtained directly from Graphs 4.10.6.07 and 4.10.6.08, respectively, as a function of the turbine flow for each penstock or tunnel. These costs do not, however, cover the use of valves. The cost of this equipment should be estimated by the design engineer using the method presented in chapter 5, Final Studies.

6) Tailrace canal and/or tunnel (account .12.19.35)

The same methodology should be used as that used to estimate the cost of headrace canals and intake tunnels, where the cost is a function of the total flow from the powerhouse and the length of the tailrace canal or tunnel.

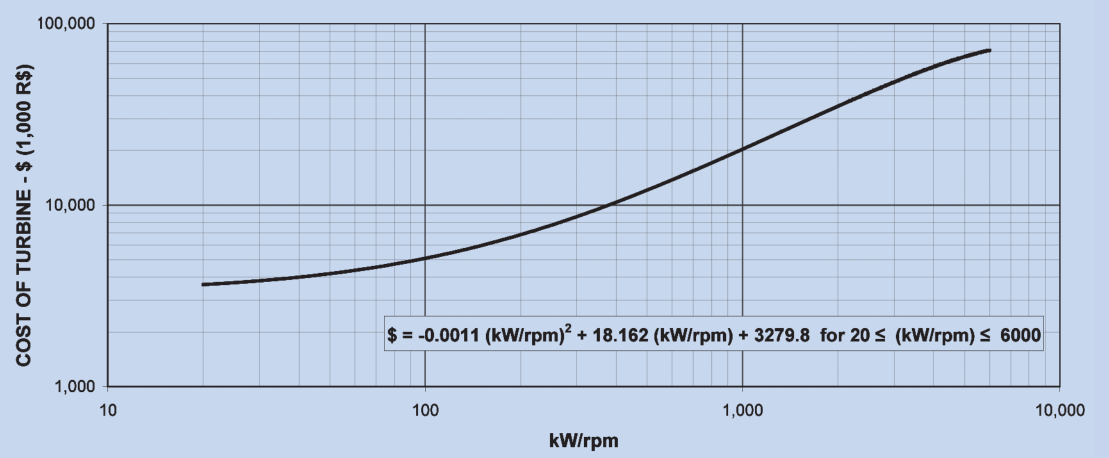
The amount to be set aside for miscellaneous costs should be 20% of the total of account .12.19.

4.10.7 Turbines and Generators (account .13)

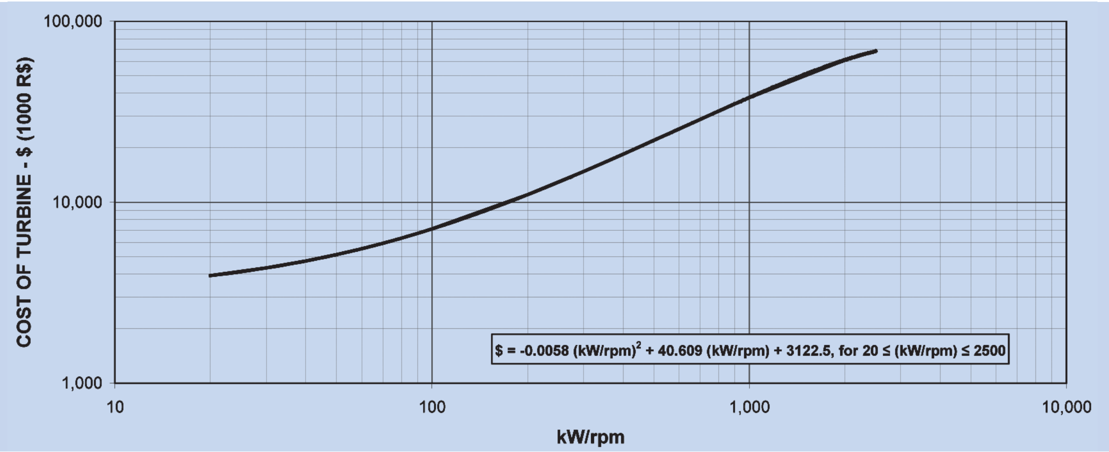
Graphs 4.10.7.01, 4.10.7.02, 4.10.7.03 and 4.10.7.04 give the costs of the turbines as a function of kW/rpm, respectively, for Francis or Kaplan turbines with a spiral casing, Kaplan turbines with a semi-spiral casing made of concrete, and Bulb turbines. The cost of Pelton turbines should be obtained from the manufacturer.

Graphs 4.10.7.05, 4.10.7.06 and 4.10.7.07 show the cost of the generators as a function of kVA/pole, respectively, for vertical axis and horizontal-axis generators, and horizontal-axis generators for Bulb turbines.

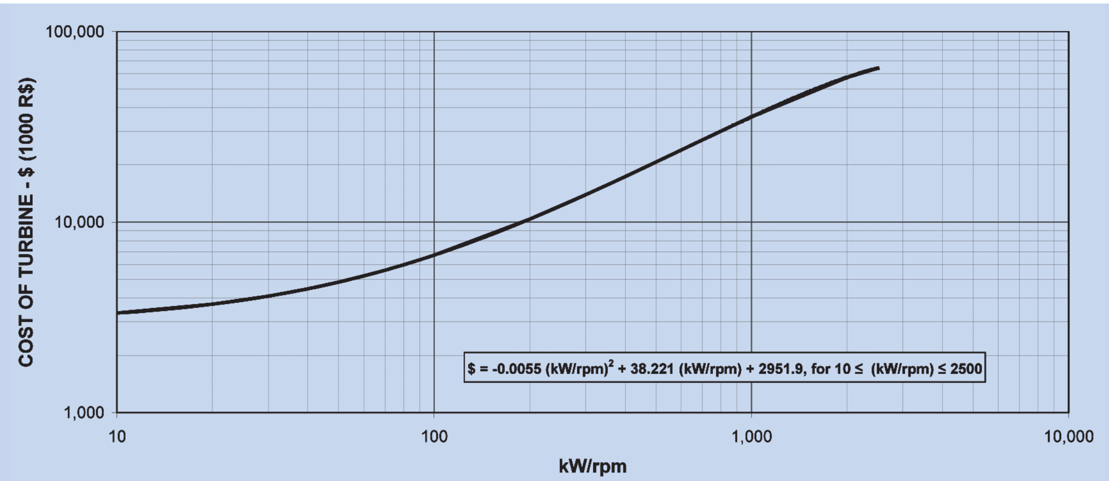
The amount to be set aside for miscellaneous costs should be 10% of the total of account .13.



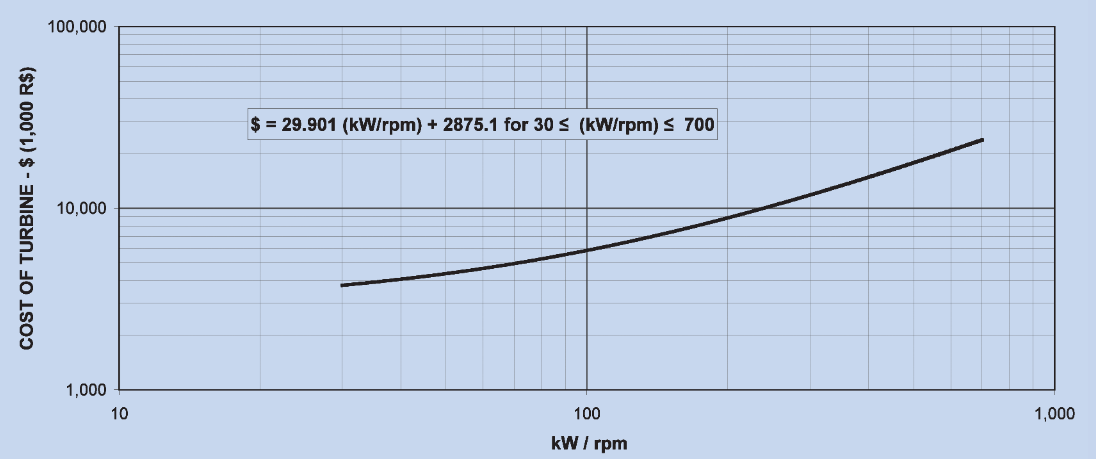
Graph 4.10.7.01 – Cost of Francis turbine (PCE, 2007)



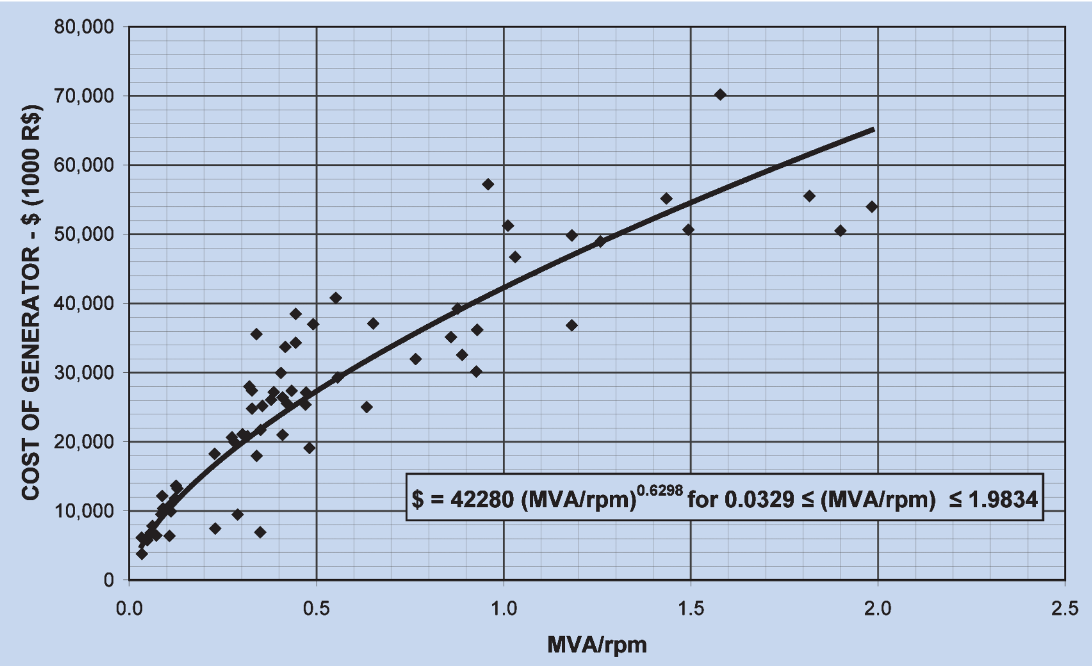
Graph 4.10.7.02 – Cost of Kaplan turbine with steel spiral casing (PCE, 2007)



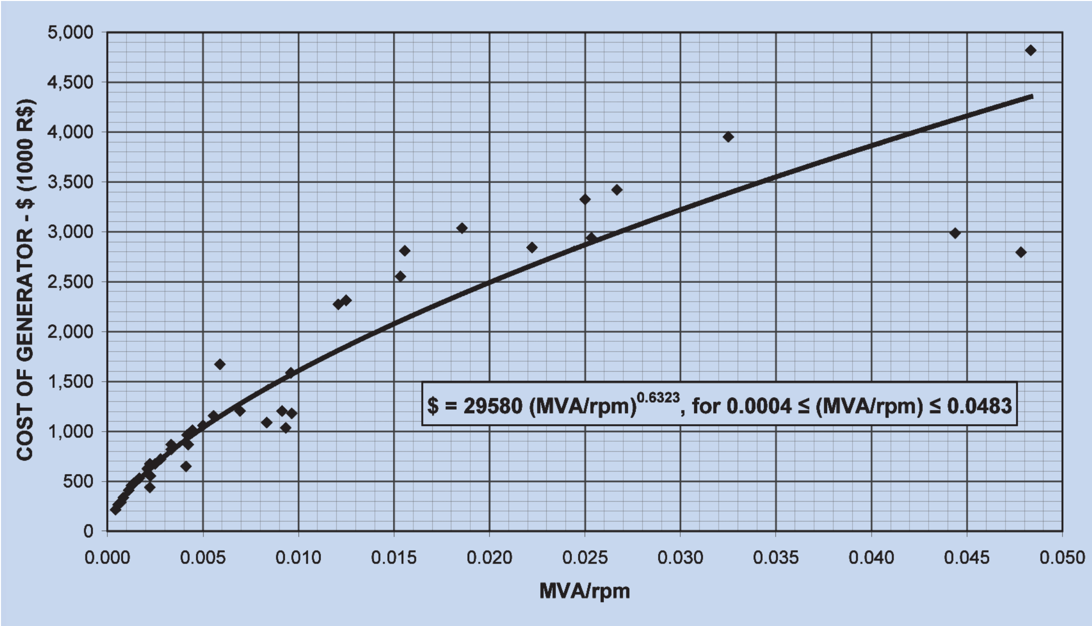
Graph 4.10.7.03 – Cost of Kaplan turbine with concrete semi–spiral casing (PCE, 2007)



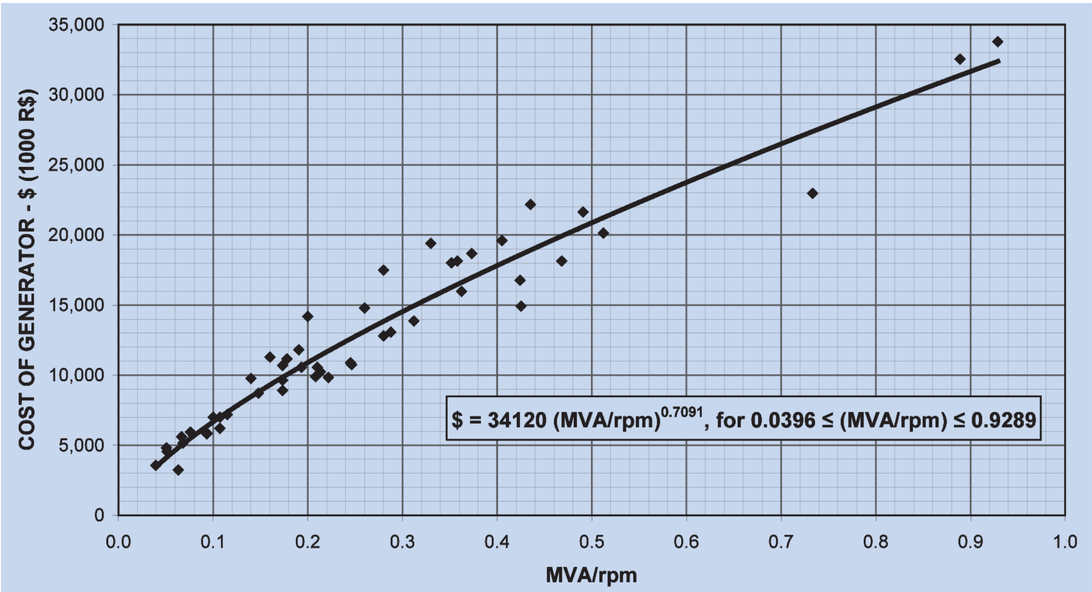
Graph 4.10.7.04 – Cost of Bulb turbine (PCE, 2007)



Graph 4.10.7.05 – Cost of vertical–axis generator (PCE, 2007)



Graph 4.10.7.06 – Cost of conventional horizontal-axis generator (PCE, 2007)



Graph 4.10.7.07 – Cost of horizontal-axis Bulb generator (PCE, 2007)

4.10.8 Auxiliary Electrical Equipment (account .14)

The overall cost of auxiliary electrical equipment for a plant is estimated at 18% of the overall cost of account .13, Turbines and Generators.

The amount to be set aside for miscellaneous costs should be 20% of the total of account.14.

4.10.9 Miscellaneous Plant Equipment (account .15)

The overall cost of the other equipment in the plant, including the cost of the main overhead crane and the hydromechanical equipment and draft tube crane, should be taken as 10% of the overall cost of account .13, Turbines and Generators.

The amount to be set aside for miscellaneous costs should be 20% of the total of account.15.

4.10.10 Roads, Railroads and Bridges (account .16)

The need to build any bridges or roads is identified by consulting road maps of the region and making reconnaissance trips, and their respective classes of construction should be decided on. Based on estimated lengths, the cost of the account can be estimated using the unit prices from tables 4.10.10.01, 4.10.10.02 and 4.10.10.03.

The amount to be set aside for miscellaneous costs should be 25% of the total of account.16.

Table 4.10.10.01 – Cost of Roads (R$/Km) (PCE, 2007)

DNIT CLASSIFICATION		UNIT COST (R$/km)					
		MAIN ARTERIAL ROAD (4-lane divided highway)	PRIMARY ARTERIAL ROAD (2-lane undivided highway)	SECONDARY ARTERIAL ROAD	PRIMARY COLLECTOR ROAD	SECONDARY COLLECTOR ROAD	LOCAL ROAD
TECHNICAL FEATURES	TOTAL LANE WIDTH (m)	14	7	7	6	6	6
	TOTAL WIDTH OF ROADWAY (m)	24	13	11	8	7	6
SOUTH, SOUTHEAST, CENTRAL WEST AND NORTHEAST REGIONS	PAVED	2,600,000	1,500,000	1,300,000	865,000	735,000	558,000
	UNPAVED	1,250,000	760,000	650,000	432,000	345,000	257,000
NORTH REGION, TO THE SOUTH OF THE AMAZON RIVER	PAVED	3,650,000	2,100,000	1,800,000	1,200,000	1,028,000	782,000
	UNPAVED	1,750,000	1,080,000	900,000	605,000	485,000	363,000
NORTH REGION, TO THE NORTH OF THE AMAZON RIVER	PAVED	4,700,000	2,700,000	2,320,000	1,550,000	1,320,000	1,005,000
	UNPAVED	2,240,000	1,400,000	1,160,000	778,000	625,000	463,000

Table 4.10.10.02 – Cost of Railroads (R$/Km) (PCE, 2007)

DESCRIPTION		COST DEC/06
INFRASTRUCTURE		(R$/km)
TYPICAL TOPOGRAPHY	FLAT	1,250,000
	HILLY	1,625,000
	MOUNTAINOUS	2,000,000
SUPERSTRUCTURE		(R$/km)
TYPES OF GAUGE	1,00 m	935,000
	1,60 m	1,100,000
	MIXED	1,265,000
SPECIAL CIVIL CONSTRUCTION		(R$/m)
CONSTRUCTION IN CONCRETE	BRIDGES	20,000
	VIADUCTS	25,000

Table 4.10.10.03 – Cost of Road Bridges (R\$/m²) (PCE, 2007)

	UNIT COST (R\$ / m²)			
	TYPE OF FOUNDATION			
REGION	DIRECT FOUNDATION	PILES	CONCRETE PILE FOUNDATION EXCAVATED WITHOUT USING COMPRESSED AIR	CONCRETE PILE FOUNDATION EXCAVATED USING COMPRESSED AIR
SOUTH, SOUTHEAST, CENTRAL WEST and NORTHEAST	1,200	1,500	1,700	1,900
NORTH, TO THE SOUTH OF THE AMAZON RIVER	1,700	2,000	2,400	2,700
NORTH, TO THE NORTH OF THE AMAZON RIVER	2,200	2,600	3,000	3,500

4.10.11 Total Direct Costs

The total direct cost is the sum of the accounts from items 4.10.1 to 4.10.10.

4.10.12 Indirect Costs (account .17)

The calculation of this cost, which represents the costs for the construction site and workers' village, engineering costs and the owner's administration costs, is based on percentages of the total direct costs, which vary from region to region:

40%	Amazon region to the north of the Amazon river
35%	Amazon region to the south of the Amazon river
30%	other regions

4.10.13 Total Cost Without Interest

The total cost without interest is the sum of the total direct cost and the indirect costs.

4.10.14 Interest during Construction (account .18)

When determining the interest during construction presented in item 5.7.9, annual interest rates of 10% and 12% were used, and the projects were differentiated according to the construction time, using standard curves of investment payments. The interest rate to be used for calculating interest during construction should be obtained from the concession-granting authority.

4.11 COMPARISON AND SELECTION OF CASCADES

In the Preliminary Studies, the analysis and comparison of cascades is designed not only to eliminate any uncompetitive options, but also to guide any reformulations needed, identifying the characteristics of the basin that are found to be instrumental in minimizing the cost/energy benefit ratio and the negative socioenvironmental impacts.

Based on the assessments, a shortlist of cascades will be selected to go through to the Final Studies that are the most competitive not only in terms of their cost/energy benefit ratio, but also from the perspective of the socioenvironmental impacts, for which it is hoped that the advantages achieved from a more detailed study will offset the effort of carrying it out. The cascades to be studied in greater depth in the Final Studies should be selected using the cost/energy benefit and negative socioenvironmental impact indexes, described in items 4.11.1 and 4.11.2.

4.11.1 Cost/Energy Benefit Index

The economic and energy analyses and comparisons carried out in Inventory Studies are based on cost/energy benefit indexes, which are expressed R$/MWh and are always calculated as the ratio of the cost of obtaining extra firm energy for the reference system and the value of this energy contribution. This index can be calculated individually for each project or for a group of projects in a cascade, and also for a cascade as a whole, as set out below.

Cost/Energy Benefit Index of Projects

The energy benefit from each project in a cascade is measured by the amount of firm energy the project will provide for the reference system, supposing that all the other projects in the cascade have already been built (last added firm energy contribution - item 4.6.3). The cost/energy benefit index for each project is defined as the ratio between its total annual cost and its energy benefit, and is calculated by:

$$ICB_i = \frac{CT_i}{8760 \times \Delta Ef_i} \quad (4.11.1.01)$$

where:

ICB_i	cost/energy benefit index of plant i, in R$/MWh;
CT_i	total annual cost of plant i, in R$; and
ΔEf_i	extra firm energy provided by the addition of plant i, in average MW, assuming that all the projects in the cascade have been built.

The total annual cost (CT_i) of each plant is calculated by:

$$CT_i = C_i \,.\, FRC + P_i \,.\, COM \,.\, 10^3 \quad (4.11.1.02)$$

where:

C_i	cost of project i, in R$, determined in the Preliminary Studies, including interest during construction; and
FRC	capital recovery factor throughout the project's useful life, based on the discount rate adopted, defined by the following expression:

$$FRC = \frac{j \times (1+j)^z}{(1+j)^z - 1} \tag{4.11.1.03}$$

where:

j	annual discount rate (item 2.6);
z	useful life of the plant, normally taken as 50 years (item 2.6);
COM	annual operating and maintenance costs of the plant, in R$/kW/year, set in the basic criteria (item 2.6).

Cost/energy benefit index of groups of projects

The energy benefit of a group of projects is measured by the firm energy contribution they would jointly make to the reference system, supposing that all the other projects in the cascade had already been built (item 4.6.3). The cost/energy benefit index for the group is then defined as the ratio between the annual cost of this group and its energy benefit.

Elimination of uncompetitive projects

Given that all the projects from a cascade should be economically advantageous, it is necessary to eliminate any that are uncompetitive. A comparison is made of the cost/energy benefit index of each project (ICB_i) using the reference unit cost (CUR), defined in item 2.6. A hydropower plant is only considered economically competitive if its cost/energy benefit index is lower than the reference unit cost.

When a project is eliminated from a cascade, the cost/energy benefit indexes of the other projects are affected. This means the elimination process should be iterative, ensuring that at the end only the economically competitive projects are included in the cascade under study.

In view of the interdependency of the projects, it is not enough just to check the individual economic-energy competitiveness of the projects in isolation; groups of uncompetitive projects should also be looked for, comparing their cost/energy benefit indexes with the reference unit cost. Groups of at least three projects should be checked, but there is no need to form groups of projects with no hydraulic connection.

The calculation of the cost/energy benefit index of the projects in one cascade option and the elimination of uncompetitive projects can be carried out using the “Eliminate” function, “without simulation” option, from the SINV system. This function gives the ICB of the projects and indicates those whose ICB is greater than their CUR, allowing these highlighted projects to be eliminated and giving rise to a new cascade option.

Review of Cascades

Once the process of assessing uncompetitive projects in a given cascade has been concluded, if any projects are indeed eliminated, it should be assessed whether it is technically and economically feasible to recover part of the head in the sections corresponding to the eliminated projects, raising the height of the dam downstream from the section and shifting downstream the dam axis of the project upstream from the section.

Cost/Energy Benefit Index of Cascades

Since each cascade will make a different firm energy contribution to the reference system, the comparison of the cascades amongst themselves can only be done once these values have been homogenized. This can be done by bringing the output of all the cascade options up to the same output level as the cascade option that makes the highest firm energy contribution, using the reference unit cost.

The cost/energy benefit Index for each cascade, which is the parameter by which they are evaluated, is given by:

$$ICB_a = \frac{CT_a + 8760 \times CUR \times (\Delta Ef^* - \Delta Ef_a)}{8760 \times \Delta Ef^*} \qquad (4.11.1.04)$$

where:

ICB_a	cost/energy benefit Index of cascade *a*, in R$/MWh;
CUR	reference unit cost, in R$/MWh;
ΔEf^*	firm energy contribution of the cascade with the highest production from the group under analysis, in average MW;
ΔEf_a	firm energy contribution of cascade *a*, in average MW; and
CT_a	total annual cost of cascade *a*, after all the economically unfeasible projects have been eliminated, in R$.

The firm energy contribution of a cascade should not be calculated as the sum of the last added firm energy contributions of the projects it is made up of, as this would mean summing the benefits of projects more than once. In the Preliminary Studies, the firm energy contribution of a cascade should be calculated as shown in item 4.6.3.

The SINV system has a function called "Energy/Economic Assessment", which determines the ICB of the different cascades in a given group, using the procedures described in 4.6. In order to use this, choose the "without simulation" option.

4.11.2 Negative Socioenvironmental Index

The negative socioenvironmental index of a cascade should express the magnitude of the negative impact brought about by the set of projects that comprise it on the study area. In the Preliminary Studies, the aim of this index is to rank the cascades as a function of the extent to which they meet the objective of **minimizing negative socioenvironmental impacts**, providing an input for the selection of those that will go through to the Final Studies.

The negative socioenvironmental index of a cascade is calculated in two stages:

- negative impact index of a cascade on each synthesis component (relative to the aggregate indexes of the projects into an index for the cascade in question);
- negative impact index of a cascade on the socioenvironmental system (relative to the aggregate negative impact indexes for all the synthesis components).

Negative impact index of a cascade on each synthesis component (IAC)

The negative socioenvironmental impact index of a cascade on each synthesis component should represent the impact of the set of projects it contains on the synthesis component in the study area, assuming that all the projects from the cascade have been built. As such, it is necessary not only to consider the impact processes from each project in isolation, but also the cumulative and synergistic effects between the projects which affect a given sub-area and the effects between the impact processes in the different sub-areas.

However, these factors would make the analysis extremely complex, given the number of cascades considered in the Preliminary Studies. Instead, a simplification of the cumulative and synergistic effects is used in order to draw up a shortlist of the cascades that merit more detailed analysis in the Final Studies.

At the Preliminary Studies stage, the following procedures are used to calculate the **negative impact index of a cascade on each synthesis component (IAC)**:

- Calculation of the negative impact index of a cascade on the synthesis components in each sub-area

At this stage, a simplified procedure[11] can be used to calculate the cumulative impact index for each sub-area, where **n** is the number of projects in the cascade under analysis that impact on sub-area **j**. Based on the negative impact indexes of each of these projects in isolation $I_{SA}(j,i)$, i =1,..,n, the cumulative impact of all the projects is given by the following iteration:

$$I^c_{SA}(j,i) = I^c_{SA}(j,i-1) + \left[\left(1 - I^c_{SA}(j,i-1)\right)^* I_{SA}(j,i)\right], \qquad i = 1, \ldots, n \tag{4.11.2.01}$$

where:

$I_{SA}(j,i)$	impact on sub-area j when only the ith project in the cascade is built;
$I^c_{SA}(j,i)$	cumulative impact on sub-area j when projects 1, 2, ..., i are built in the cascade;

where $I^c_{SA}(j,0) = 0$ the initial value of the cumulative impact.

After all the iterations have been done (considering all the projects in the cascade in isolation), the cumulative impact on sub-area **j** is given by:

$$I^c_{SA}(j) = I^c_{SA}(j,n) \tag{4.11.2.02}$$

In the case of the Indigenous Peoples synthesis component, as the unit of analysis is the entire study area the same procedures is used as that for a single sub-area.

- Aggregation of indexes for the sub-areas into a negative impact for the cascade on a synthesis component in the study area.

The negative impact index of a cascade on a synthesis component in the study area **(IAC)** is given by the weighted sum of the impact indexes for the sub-areas:

$$IAC = \sum_j I^c_{SA}(j) P(j) \tag{4.11.2.03}$$

where:

P(j)	weighting factor for each sub-area i

The weighting factors for the sub-areas, defined at the diagnosis stage, are used to provide relative impact indexes for the sub-areas within the context of the study area. In order to keep the **IAC** between zero and one, the weights **P(j)** should also be attributed on a continuous scale from zero to one, such that the sum of the weights of all the sub-areas will equal one.

In order to calculate the index, it is suggested that a table be prepared as shown below.

In the case of the Indigenous Peoples synthesis component, as the unit of analysis is the entire study area, no weighting is needed.

11 Adapted from EPE/CNEC/Arcadis Tetraplan – "Avaliação Ambiental Integrada dos Aproveitamentos Hidrelétricos na Bacia do Rio Tocantins – Relatório P3 - Avaliação Ambiental Distribuída e Conflitos", March 2007.

Table 4.11.2.01 – Negative impact index of a cascade on the Way of Life synthesis component

Projects \ Sub-areas (Weights)	I (0.07)	II (0.08)	III (0.18)	IV (0.12)	V (0.25)	VI (0.30)	
A		0.10					
B		0.50	0.65	0.10			
C			0.85		0.35		
F		0.08					
G	0.10						
H	0.10				0.10		
I	0.30		0.10		0.30		
M	0.30					0.40	
N	0.50		0.90				
Q_2						0.95	
$I^c_{SA}(j)$	0.801	0.586	0.990	0.10	0.590	0.970	IAC
$I^c_{SA}(j)\,P(j)$	0.056	0.047	0.179	0.012	0.148	0.291	0.733

- Negative Impact Index of a Cascade on the Socioenvironmental System

The **negative impact of a cascade on the socioenvironmental system (IA)** should express its total negative impact on the study area, i.e. it should consider the impacts caused by the cascade on all the synthesis components.

This index is obtained by the weighted sum of the **negative impact indexes of the cascade on the synthesis component (IAC)**, as calculated previously.

$$IA = \Sigma\, IAC_i \times P_{ci} \qquad (4.11.2.04)$$

where:

P_{ci}	weighting factor for each synthesis component

In order to keep the **IA** between zero and one, the weights $\mathbf{P_{ci}}$ should also be attributed on a continuous scale from zero to one, such that the sum of the weights of all the synthesis components will equal one.

The weighting factors are used to attribute relative impact socioenvironmental indexes for the cascade on the synthesis components in the study area. These weights should represent the relative importance of the impact processes on each synthesis component on the socioenvironmental system, which can be measured by the repercussions of these processes on the other components.

In order to assess this repercussion, the analyses should be considered of the interactions between the synthesis components relative to the socioenvironmental status of the study area carried out in the diagnosis.

This activity is undertaken by the technical team responsible for the studies, which should also consider the views of the different interest groups identified in the region throughout the studies.

Given the interdisciplinary nature of this activity and the great margin for subjectivity involved, it is necessary to systematize the procedures for weighting the synthesis components using specific methods and techniques. These methods should make it possible to represent the subjective assessments in a system of weights, as well as combining the opinions of different assessors.

The "Calculate Negative Impact" function from the SINV system can be used to determine negative socioenvironmental impact indexes for the different cascades per synthesis component, and the negative socioenvironmental impact index of the different cascades on a river basin.

Analytic Hierarchy Process

We present below an adaptation of the Analytic Hierarchy Process model proposed by Saaty (1980), which ranks the synthesis components by comparing them against each other. This is a useful method because it is easy to use and there is a computer system available to apply it.

In order to determine the relative importance of the impact processes from each synthesis component on the socioenvironmental system, it is necessary to set up an indirect measurement scale. The aim of this is to establish, from subjective assessments by the team members, a system of weights for the synthesis components as a function of the relative importance of their impact processes.

The pairwise comparison process proposed by Saaty (1980) enables the relative importance (or priority) of the different synthesis components being compared to be established, in view of their impact processes on the socioenvironmental system. All the components should be compared with all the others.

The weights (or priorities) are attributed by setting up a matrix of the same size as the number of components to be compared, where the elements in the matrix are values on a standard comparative scale which represent the relative importance that the decision makers taking part in the process give to the component on each line (i) against the component in each column (j). Saaty (1980) recommends the comparative scale presented below, though it should be understood that the values are absolute magnitudes and not simple ordinal numbers.

Table 4.11.2.02 – Scale of comparative importance between the synthesis components

Importance of preferred element	Definition	Importance of non-preferred element	Definition
1	equally important	1	equally important
3	slightly more important	1/3	slightly less important
5	more important	1/5	less important
7	much more important	1/7	much less important
9	absolutely more important	1/9	absolutely less important
2,4,6,8	intermediate values	1/2, 1/4, 1/6, 1/8	intermediate values

In the pairwise comparison, the attribution of relative importance implies in a(i,j) = 1/ a (j,i), which means the matrix is reciprocal. In other words, the preferred element is given a number on a scale from 1 to 9 and the non-preferred element is given the reciprocal number. In this case in particular, the attribution of importance should take into account the repercussions of the impact processes from one given synthesis component on the rest.

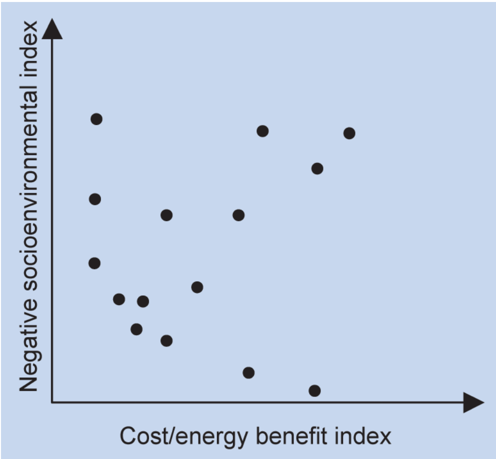
Having drawn up the matrix, calculations are done to obtain the autovector of the highest value which, after it has been normalized, corresponds to the "priority vector", expressing the relative weights between the synthesis components being compared. The synthesis components associated with the highest values are those whose impact processes are deemed to be of the greatest importance.

The priority matrix should represent the general opinion of the decision makers in this process. For cases in which no consensus can be reached, a matrix can be built up of the averages given by the decision makers, where each element is the geometric average of the values attributed by each person. In this case, each decision maker will first build up their own matrix, which will then be inputted into the final matrix representing the group's opinion, obtaining a single "vector of priorities".

As the values are attributed subjectively, it is a good idea to apply consistency tests to the judgments made to assure the reliability of the methods used. If the consistency index is less than 0.1, the judgments can be considered satisfactory (Saaty,1980).

4.11.3 Selection of the cascades

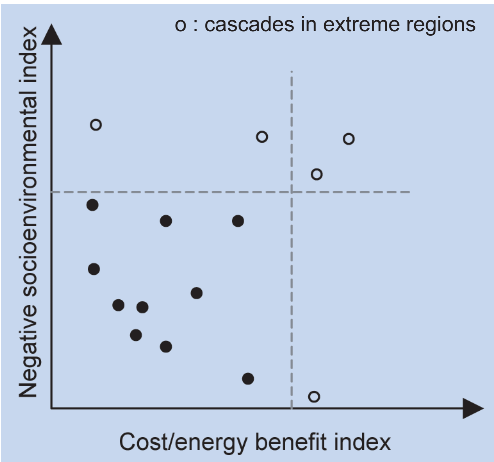
The cascades to be examined in greater depth during the Final Studies should be selected in view of their respective cost/energy benefit index (item 4.11.1) and negative socioenvironmental index (item 4.11.2). They should be compared graphically, where one of the axes indicates the cost/energy benefit index and the other, the negative socioenvironmental index, as shown below.



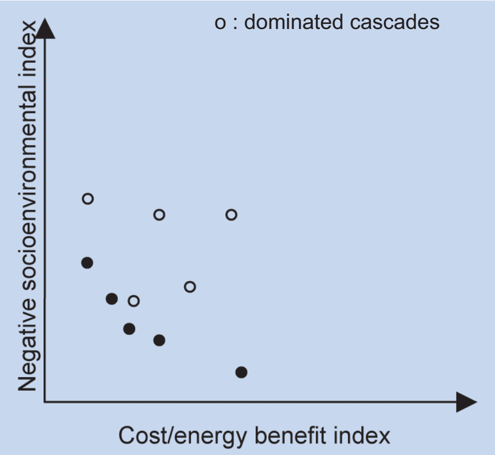
Generally speaking, the cascades to be selected are those that are represented by the points nearest to the bottom left-hand side of the graph, which corresponds to the lowest values for both indexes. The number of cascades selected to go through to the Final Studies will depend greatly on the morphology and extent of the river basin under study. If there is a significant natural head that is distributed evenly throughout the rivers, there may be more cascade options than in cases where there is one pronounced head concentrated in a short stretch of river. Generally speaking, in studies of this nature, there are normally no more than ten basic cascades that go through to the Final Studies. It is reasonable to expect between five and ten.

The following concepts must be used to choose the cascades:

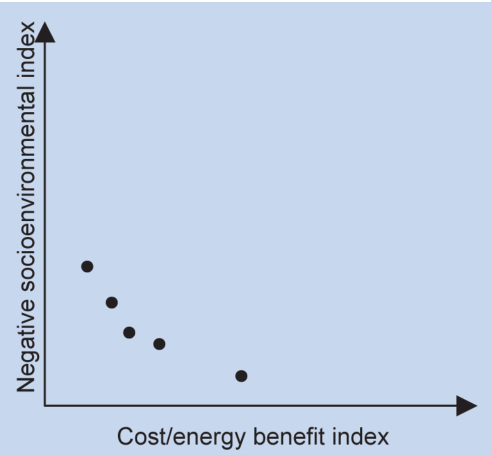
- **Extreme Regions** – The cascades located at the extremities of the graph, representing a high cost/energy benefit index and/or negative socioenvironmental index, should be ruled out.



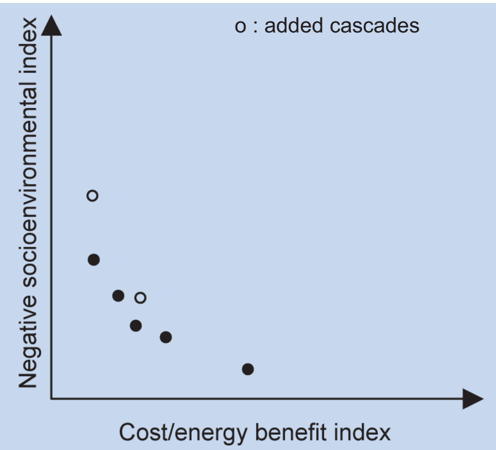
- **Dominated Cascades** – A cascade is dominated (uncompetitive) if there is another one with lower energy cost/benefit and socioenvironmental indexes. All uncompetitive cascades should be ruled out.



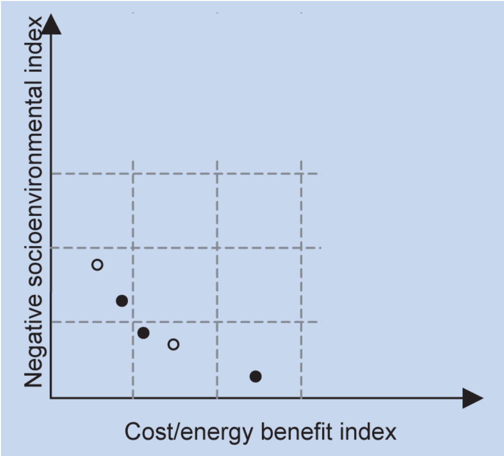
- **Non-Dominated Cascades** – Once the cascades from the extreme areas and those that are dominated have been ruled out, there should just be a few cascades left, which are called the non-dominated or Pareto optimum cascades. These are the cascade options that should go through to the Final Studies.



However, if there is too small a set of non-dominated cascades, a new set of non-dominated cascades can be looked for from the set of dominated ones, always seeking to reformulate them in such a way that their indexes are improved, then adding them to the set of cascades selected.



On the other hand, when the set of undominated cascades is too large, the graph can be split into a grid, and cascade options can be selected according to their relationship with the other cascade options in the same square of the grid.



The SINV system can be used not only to calculate the cost/energy benefit index and negative socioenvironmental impact index of the different cascades, but it also gives a graphic view of the extreme cascades and those that are dominated and non-dominated, making it easier to select which cascades should go into the Final Studies. Therefore, the use of this system is recommended not just for calculating cost/energy benefit and negative socioenvironmental impact indexes, but also for selecting the cascades to advance to the Final Studies.

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chapter 5

Final Studies

CAPÍTULO 5

The basic aim of the Final Studies is to choose the cascade and associated structures and equipment that will develop all the hydropower potential of the river basin that can be economically and environmentally harnessed. At this phase, the data and studies from the previous phase are collated, and any studies required to supplement the data on the projects included in the cascades short-listed for the Final Studies are undertaken. This involves making in-depth energy studies, reviewing layouts, drafting designs and making cost estimates of the projects involved. An economic/energy index, negative socioenvironmental impact index and positive socioenvironmental impact index are attributed to each cascade, providing parameters for them to be compared amongst themselves and for the final decision on which cascade to select.

The main aim of the field studies and engineering studies is to improve the detail of the data and undertake basic studies for this stage, in order to:

- make it feasible to design more consistent final layouts for the projects and make any adjustments necessary to the cascades under study;
- supply more accurate information for the purposes of quantifying and estimating the cost of each project in the cascades under study; and
- attribute a cost/benefit index to each cascade option so they can be ranked according to the extent to which they fulfill the precondition of **maximizing power generation and minimizing costs**, furnishing a valuable input for the final selection of one cascade.

The main aim of the socioenvironmental studies is to:

- supplement the socioenvironmental information deemed of importance gathered in the Preliminary Studies in order to design the final layouts of the projects and make any adjustments necessary to the cascades under study;
- supply more accurate information for the purposes of estimating the socioenvironmental costs associated to each project and each cascade;
- include in the socioenvironmental assessment an analysis of the cumulative and synergistic effects arising from the introduction of the group of projects that make up each cascade option;
- make it possible to attribute a negative socioenvironmental impact index to each of the cascades, ranking them according to the extent to which they fulfill the precondition of minimizing negative socioenvironmental impacts;
- attribute a positive socioenvironmental impact index to each cascade, furnishing a valuable input for the final selection of one cascade.

The procedures for undertaking the socioenvironmental studies during this stage are set out throughout this chapter in combination with the procedures for the energy, engineering and multiple water use studies. There are three phases of studies: consolidation of the socioenvironmental diagnosis, assessment of the negative and positive socioenvironmental impacts of the cascades, and attribution of a negative socioenvironmental impact index and a positive socioenvironmental impact index to the cascade options.

During the consolidation of the diagnosis, the issues identified in the Preliminary Studies as being of most importance are supplemented and all systemic processes must be highlighted. The guidelines and recommendations for these are presented in items 5.1 and 5.2. The results and analyses are used to review the maps of the synthesis components, which are then used to make the final adjustments to the cascades.

In addressing the negative socioenvironmental impacts, the studies follow the same basic structure as in the Preliminary Studies, albeit in greater depth and more detail. At this stage, the impact analysis is done for groups of projects rather than for individual projects, so that the cumulative and synergistic effects between the different projects in a given cascade can be incorporated into the impact assessment. This means that the studies for identifying, predicting and assessing negative impacts and attributing impact indexes per synthesis component are done for groups of projects with an impact on the same sub-area. The information produced on negative impacts is taken into account in the review of the socioenvironmental cost estimate and the final layouts of the projects.

The analysis of positive socioenvironmental impacts is very important at this stage, and is undertaken directly on groups of projects in each cascade option. The process for identifying, predicting and assessing the positive impacts and the respective attribution of impact indexes for each of the socioenvironmental aspects selected is also described in item 5.4.

The procedures for obtaining the negative and positive socioenvironmental impact indexes for each cascade are described in items 5.8.2 and 5.8.3.

5.1 DATA CONSOLIDATION AND SUPPLEMENTARY INVESTIGATIONS

The starting point for the Final Studies is the cascade options selected in the Preliminary Studies. The information obtained for each cascade must be supplemented, confirmed or adjusted until it is deemed sufficient to provide a consistent characterization of the total costs of the corresponding construction work, structures and equipment.

In each case, it is up to the analyst to decide the scope of the additional data required based on his/her understanding of the extent to which this will influence the final cost estimate.

5.1.1 Cartography and Topography

The cartographic services undertaken at the final phase of the studies also depend greatly on the features of the river basin, such as its dimensions, slope, vegetation, the extent of human occupation and, obviously, all the data collected and surveys undertaken in the Preliminary Studies.

At this stage, the cartographic data from the Preliminary Studies should be reassessed. Definitive and/or supplementary surveys must be carried out where any fragility is identified.

The following items in particular must be established definitively:

- the longitudinal profiles of the water courses, indicating the water levels at the pre-selected sites and the elevation of any existing civil engineering structures (bridges, tunnels, etc.);
- surveys of saddles around the rim of the reservoir;
- elevation-area and elevation-volume curves for each reservoir;
- topographic features of the preselected sites, including all the elements of all the structures in the layout, as shown in item 2.5, in compliance with class VIPA, from Brazilian standard ABNT 13.133;
- topographic survey of the river bed along the dam axis;
- longitudinal sections along the hydraulic conveyance facilities, the river diversion, the spillway and navigation system and fish passage system (for navigation and migratory species);
- installation of two topographic marks at each project site with their respective azimuth marks, with planimetric and altimetric coordinates linked to the Brazilian Geodetic System;
- planimetric and altimetric coordinates for the geological, geotechnical, hydrometric and environmental investigations undertaken.

Technical Specifications

As for the technical specifications of the georeferenced data, the drawings or images that involve cartographic coordinates will be submitted to the relevant entities following the standards set out in item 4.1.1 of the Preliminary Studies.

Outputs

The following items must be submitted:

- list of official geodetic marks (benchmark and vertex) used as a basis for coordinate transport;
- report on the geodetic marks installed at the project sites identified in the cascade selected and used in the field support services, including photographs, maps of access routes, codes, geographic

coordinates and UTM coordinates corresponding to the datum used and any other relevant technical information;

- descriptive records including: description of the services, equipment used, level of precision, computer programs used;
- schematic drawings on A3 size paper and in a suitable scale of the longitudinal profiles of the rivers under study, indicating their main tributaries (include information about the cascade options studied, operational water levels and all the areas of interest, e.g. indigenous lands, towns, bridges, conservation areas, water withdrawals, etc.);
- planimetric/altimetric plan of the preselected sites obtained using the methodology recommended in item 4.1.1, on A3 paper and using a suitable scale;
- drawings of the topographic cross-section of the river and longitudinal section at the locations of the project structures (including the tailrace canal) at all the sites included in the shortlisted cascades;
- digital file of the planimetric/altimetric plans of the reservoirs;
- electronic spreadsheet used as the basis for preparing the elevation-area-volume curves for the projects in the different cascades;
- technical records of the services used to allocate planimetric and altimetric coordinates to the geological, geotechnical, hydrometrical and environmental studies;
- specific *Anotações de Responsabilidade Técnica* (ARTs) for the services undertaken (field and office).

5.1.2 Hydrometeorology

In the Final Studies, the breadth and depth of the information obtained in the Preliminary Studies should be reviewed. The basic studies to be undertaken at this stage are:

- determination of the rating curve in the tailrace canal at each site selected: field studies should be continued to measure the water levels and flows in order to determine the rating curves at each site; if possible and whenever necessary, staff gauges linked to the benchmark system mentioned in item 4.1.1 should be installed. When it is impossible to make direct measurements at a particular site, the rating curves or some of their points should be estimated by the methods generally used to correlate levels and flows with known values from other staff gauges;
- mean monthly natural flows: all the studies needed to determine the flow records at the dam sites should be reviewed and completed in the light of the new information obtained from the field and other sources;
- determination of extreme flows: data from the Preliminary Studies is reviewed and supplemented; the annual maximum daily mean flows should be determined and analyzed statistically; the flood flow-recurrence time curve should be plotted. Other information than the recorded flows can be used, such as flood marks and the testimony of residents of the area to supplement or complete the historical records.

5.1.3 Consolidation of the Scenario of Multiple Water Uses in the River Basin

The studies that were started in the preliminary phase of the Inventory Studies should be reviewed, supplemented and collated so that a long-term scenario can be built up for the purposes of analyzing the cascade options and making a final selection.

5.1.4 Geology and Geotechnics

In the Final Studies, the focus should be on the selected sites and the areas of the reservoirs which may present specific problems.

At the project sites under study, the following basic information should be supplemented:

- type and geotechnic features of the foundations for the structures; and
- existence, type, characteristics and approximate size of deposits of soil, sand, gravel and rock that could be used as natural building materials.

Taking the data from the Preliminary Studies and the preliminary layout of the structures for the projects, **supplementary surveys** should be undertaken to better define the state of the foundations and the natural building materials and to obtain comparative parameters, such as a basic geological and structural model of the study area, and the results of any known geomechanical tests on the different kinds of bedrock in the foundations. The following are suggestions of programs to be undertaken:

- geological surface mapping;
- manual boreholes, trenches and auger borings; and
- geophysical surveys.

At this phase, it is not normally necessary to drill any mechanical boreholes (percussion or rotary drilling) except in special cases or if the land has a particularly complex geological make-up. In these cases, the investigations, which should be designed according to the dimensions of the construction work and the geological complexity, should determine the thickness of the soil cover on the bedrock, the characteristics of these materials, the depth of the water table and the general characteristics of the foundations so that the type of structure to be built on the site can be chosen and the costs of the processes required can be calculated.

As part of the socioenvironmental studies, the geological aspects should be investigated in greater depth, especially if there are any minerals that could be influenced by the reservoirs (check data from DNPM) and if there are any areas of greater susceptibility to landslides around the reservoir banks. A preliminary assessment should also be made of the silting potential of the reservoir and regional seismicity.

5.1.5 Environment

The aim of this activity is to gather the information required not only to gain a greater understanding of the socioenvironmental issues identified as being of most significance during the Preliminary Studies, but also to identify and characterize the cumulative and synergistic processes that would arise if the group of projects in the study area were built.

The survey of data and information on each synthesis component and each aspect selected for the analysis of the positive socioenvironmental impacts should supplement the analysis from the Preliminary Studies.

Field surveys should be undertaken to supplement the secondary data gathered.

5.2 CONSOLIDATION OF THE SOCIOENVIRONMENTAL DIAGNOSIS

The aim of the studies to be undertaken at this phase is to:

- supplement the socioenvironmental information of significance with a view to designing the final layouts of the projects and make any adjustments needed to the different cascades, while also making an analysis of the positive socioenvironmental impacts;
- provide reference data for assessing positive and systemic impact processes and the cumulative and synergistic effects arising from the interaction between the projects in any given cascade.

The studies should therefore be designed to consolidate knowledge about the study area, by studying in greater depth the issues identified as being of most significance during the Preliminary Studies, while also taking into account the results of the technical meeting held to present the results of those studies, as described in item 2.9. As the analyses at this stage are of groups of projects or even of whole cascade options, emphasis is put on systemic issues and the conditions that would bring about cumulative and synergistic effects between the impact processes of the projects from any given cascade.

The spatial representation of the information on each synthesis component and each aspect selected for the analysis of the positive socioenvironmental impacts should be reviewed in light of the knowledge acquired during the Preliminary Studies and the more in-depth knowledge gained on the study area.

When analyzing the negative socioenvironmental impacts, the segmentation of the study area into sub-areas for each synthesis component should also be reassessed, providing suitable reference data for the analysis of the impact processes concerning the different cascades or groups of projects from a given cascade. The weights attributed to each sub-area should also be reviewed, taking into account the outcome of the technical meeting mentioned above.

A description should be annexed to the map prepared for each synthesis component, highlighting the aspects that were instrumental in defining the boundaries of each sub-area, and placing it within the broader context of the study area as a whole, setting up its relationship with the other sub-areas. The description should also highlight any aspects of note, areas of sensitivity and potentialities in the region, and any existing or potential conflicts that could influence the assessment of the negative or positive impacts of the different cascade options. Those aspects that would trigger any negative or positive cumulative or synergistic interactions between the projects in a given sub-area should also be highlighted. In some cases it may be worth preparing specific maps to detail the aspects to be analyzed in the assessment of positive impacts described in item 5.4.2.

At the end of the diagnosis, the analyses of all the synthesis components and socioenvironmental aspects selected for analyzing the positive socioenvironmental impacts should be collated. Using an interdisciplinary dynamic, the interactions between the processes identified in the synthesis components should be identified and comprehended, building up a picture of the socioenvironmental status of the study area. All these interactions should be represented spatially on a single map.

5.3 ENERGY STUDIES

5.3.1 Simulation of Operation

In the Preliminary Studies the energy potential of each cascade option is estimated using simplified procedures. This is justified by the very limited nature of the information available at that stage on the hydrology and topography of the river basin.

These procedures assume that all the hydropower potential from the natural river basin would be harnessed during the critical period of the reference system, plus the reservoirs' live storage, minus not only evaporation losses, but also water withdrawals for other uses and volumes allocated for flood control at the beginning of the critical period, whenever these significantly influence the analysis and selection of cascade options and the dimensions of the projects. However, when the system is actually operating, the restrictions imposed by the turbine-generator sets and the storage capacities of the projects will mean that some of the hydropower potential calculated in the Preliminary Studies cannot be harnessed. In other words, the firm energy as calculated in the Preliminary Studies can be seen as no more than an index for measuring the relative merit of the energy contributed by the different cascades under analysis.

During the Final Studies, when there are fewer cascades under analysis and more accurate hydrological and topographic information is available, a more accurate estimate of the energy contributed by each cascade must be carried out. This requires using mathematical models of the plant and reservoir systems and the determination of firm energy using simulations of operation. The whole set of plants in the reference system (item 2.1.1.) should be represented in the model, and all the plants in the reference system that are hydraulically connected to the projects under study must be included in the simulation for it to be effective.

When using these models, it is recommended that the head losses in the hydraulic conveyance facilities of the projects being studied should be factored in by using the values determined in the structure designs (item 5.7.6). The water withdrawals for multiple uses calculated in the scenario (item 4.2.2) and consolidated in item 5.1.3 should be subtracted from the natural flows into the projects. The flood storage capacities should be subtracted from the projects' live storage. Other restrictions, such as the minimum outflows and minimum water level for multiple water uses in the river basin used in the scenarios of multiple water uses should also be factored into the simulation.

Firm Energy – Initially, an estimate of firm energy from the simulated system is used, which is calculated approximately using procedures similar to those used in the Preliminary Studies, such as a target market that is constant throughout the critical period of the reference system. The simulation is set up to operate the system following preset rules which are designed to meet this market. The energy generated each month is accrued, which means there may be a deficit or surplus with regard to the target market (in the case of spilled discharge, the system could generate a surplus). The mean energy generated during the simulation can be taken as the firm energy from the simulated system if the amount of residual energy stored at the end of the simulation is negligible. However, if there is a significant amount of residual energy stored at the end of the simulation, it should be used up, one way being to increase the target market.

The target market is, then, increased by the amount of residual energy stored divided by the number of months in the critical period, and a new simulation is run. This increase may not be enough to use up the residual energy (cases where there is spilled discharge or the reservoirs are totally depleted in some month during the critical period). Alternatively, the mean energy generated could decrease rather than increase, because from a certain point onwards the drop-off in generation caused by excessive reservoir drawdown will outweigh the residual energy available.

For this reason, an iterative process is used, where the average energy generated, the residual energy and the additional energy generated for the target market are calculated successively. This process stops either when the residual energy is negligible or when the mean energy generated decreases. The highest mean energy value generated in the iterations is taken as the firm energy for the simulated system.

Aside from calculating the firm energy for the system as a whole, the simulation also gives the firm energy and energy spilled for each project, as well as the mean net head (average of the net heads at the beginning of the month).

Firm Energy Contribution – A project's or group of projects' firm energy contribution is the difference between the mean energy generated in the simulations of the system with and without the projects. Likewise, the firm energy contribution of a cascade is the difference between the mean energy generated in simulations of the system with and without all the plants from the cascade in question.

The firm energy of the projects that make up the cascade options and that of the cascades themselves can be obtained using the Firm Energy function in the SINV system. In order to obtain these values using a simulation of operation, select the "with simulation" option. At the end of this function, the firm energy of the cascade and the individual projects that comprise it is determined, as well as the installed capacity of each of the projects.

5.3.2 Calculating Live Storage

The live storage of the reservoirs should be established by optimizing the drawdowns in order to maximize the firm energy from a cascade taken as a basis for the energy dimensioning. In order to define the live storage of the projects in the Final Studies, the same iterative process as described in item 4.6.4 should be followed. However, in the Final Studies the firm energy from the cascade at each iteration should be obtained by the simulated operation of the system, as described in item 5.3.1.

In order to determine the live storage of the projects in a cascade option, the Optimize Live Storage function from the SINV system can be used, selecting the "with simulation" option. Another way is to use the Energy Dimensioning function, with simulation, which will calculate the live storage, the installed capacity and the reference head iteratively. Both functions mirror the procedures described in items 4.6.4 and 5.3.1.

If the sedimentology studies indicate that there is a major silting issue, the mean sediment volume that is expected to occupy part of the reservoir during the project's useful life should be subtracted from the live storage.

5.3.3 Effective Installed Capacity

The installed capacity of each project should be recalculated in the Final Studies using the firm energy and mean net head values derived from the simulation. The same formulas are used for this calculation as those used in the Preliminary Studies (item 4.6.5.). As the energy generation values obtained in the simulation models depend on the installed capacity of the different projects in the system, this calculation should be done using an iterative process. The projects are redimensioned with each iteration, using the firm energy and mean net head values obtained for them in the previous iteration. In order to do this, the use of the SINV system is recommended, selecting the Energy Dimensioning function, “with simulation”, which will calculate live storage, installed capacity and reference head iteratively.

5.3.4 Reservoir Replenishment Time

As in the Preliminary Studies, once the live storage and installed capacity have been calculated, it is important to check whether the time needed to replenish the reservoirs will exceed 36 months as of the end of the critical period. This can be done by simulating the operation for three years after the critical period of the reference system. If any of the projects exceeds this maximum replenishment time, its live storage should be reduced and the simulation should be redone. This iterative process can be done automatically using the SINV system, Live Storage Replenishment function, with simulation.

5.4 ASSESSMENT OF THE SOCIOENVIRONMENTAL IMPACTS OF THE CASCADE OPTIONS

5.4.1 Assessment of Negative Socioenvironmental Impacts

These studies involve analyzing the cascade options for their negative socioenvironmental impacts on each synthesis component, reviewing the impact processes relative to the projects in isolation and identifying and assessing the impact processes caused by groups of plants.

The aim of these studies is to:

- provide information for any adjustments to the design of the final layouts of the projects and composition of the cascades, with a view to improving their capacity to meet the objective of minimizing their negative socioenvironmental impacts;
- provide information for a more accurate estimate of the projects' socioenvironmental costs;
- consider in the analysis the systemic impact processes per sub-area and those that arise from cumulative and synergistic interactions between the projects from a given cascade;
- attribute negative socioenvironmental impact indexes to the cascades for each sub-area, per synthesis component. These indexes are used when calculating the negative socioenvironmental impact indexes of the cascade options, as set forth in item 5.8.2.

The analysis should be conducted using the synthesis components defined in the Preliminary Studies (item 4.3) adding any issues identified as requiring more in-depth study, as described in item 5.2.

Likewise, the **impact indicators** and their respective **assessment element** defined for each synthesis component in the Preliminary Studies (item 4.8) apply to the analysis in the Final Studies. As the analysis of the negative impacts is undertaken on groups of projects or even on entire cascades, adjustments must be made to the way the information on the assessment elements is systematized, in order to translate the analysis scale from the level of an individual project to the level of groups of projects within individual cascades, based on the specific nature of the synergic processes identified.

Review of Impact Processes relating to Individual Projects

This review should be undertaken whenever a significant amount of more detailed information on the socioenvironmental reality is added during the consolidation of the diagnosis, and/or when the boundaries of the sub-areas are adjusted, or in the studies where the scenario of multiple water uses is taken into account.

The purpose of the review is to provide inputs for the design of the final layouts of the projects, which could generate new environmental restrictions or attenuate restrictions previously identified. It should also provide more precise information for the estimate of socioenvironmental costs. It should be conducted so as to systematize the information on the impact processes, in line with references suitable for making an integrated analysis of groups of projects.

Even though negative socioenvironmental impact indexes are attributed to groups of projects in the Final Studies, it is important to review the negative socioenvironmental impact indexes attributed to the individual projects. This is so that records of the information are available for future comparisons between different projects.

Identification of Impact Processes caused by Groups of Projects

This should be based on the sub-areas marked out on the maps of each synthesis component and their respective areas of sensitivity, since multiple projects affecting a single sub-area may bring about impact processes with similar profiles which would be likely to act synergically. By grouping the projects in this way, it is hoped that their cumulative and synergistic processes can be used in assessing the cascades in the Final Studies, considering the repercussions of groups of projects arranged in different ways.

The suggested procedures for each cascade under analysis are as follows:

a) identify the projects that impact on each sub-area in order to identify the combined impact of groups of projects on each synthesis component;

b) characterize the main impact processes arising from the interaction between the groups of projects identified and the study area, for each synthesis component, highlighting those of a permanent nature and with a broader scope, which will be more subject to cumulative processes. Also, any impact processes which extrapolate the boundaries of one sub-area should by identified. It may be helpful to superimpose the maps of the synthesis components reviewed in the Consolidation of the Diagnosis containing the areas of sensitivity with the maps and layouts of the cascades selected for the Final Studies.

The following items should be addressed: the impact processes inherent to each project in particular; the interaction between these processes; any new processes arising from the joint action of the projects on the sub-area in question, and their repercussions on the other sub-areas.

It is recommended that grids and interaction networks be used to help identify the interactions, synergies and cumulative effects between the processes[1].

As an example, table 5.4.1.01 shows some negative impact processes that could be potentially cumulative, noting that the relevance of these processes to each study will depend on the characteristics and interactions observed in the region under study.

Table 5.4.1.01 – Examples of negative cumulative and synergistic impact processes

Synthesis Component	Impact
Aquatic Ecosystems	– alteration of streamflow regime – alteration of sediment transport – alteration of water quality – interruption of migratory routes – interference in habitats that sustain biodiversity
Terrestrial Ecosystems	– loss, fragmentation or isolation of habitats – interference or pressure on protected areas (conservation areas, protected areas and indigenous lands) – loss of vegetation – pressure on threatened species
Ways of Life	– pressure on living conditions by attracting new people – number of people affected (rural and urban) – loss of river-dependent subsistence activities – alteration of epidemiological profile – loss of archaeological, historical and cultural heritage – exacerbation of conflicts
Territorial Organization	– interference in patterns of settlement – interference in the circulation of people, goods and services – loss of land by municipalities

1 Recent studies undertaken by the electricity sector have stressed the issue of cumulative and synergistic processes, especially in those conducted by EPE for the Integrated Environmental Assessment of the Uruguai, Parnaíba, Paranaíba, Doce and Paraíba do Sul river basins.

Synthesis Component	Impact
Regional Economy	– loss of productive lands – loss of resources (mining, fishing, tourism, agriculture, etc.)
Indigenous Peoples / Traditional Communities	– pressure on socio-cultural relations – pressure on ethno-ecological conditions

Source: EPE, 2007 (AAI Tocantins, AAI Doce), CEPEL, 2002.

c) select the **assessment elements** capable of characterizing the impact processes identified on each synthesis component, ensuring that the impact indicator is capable of highlighting differences between the cascades under comparison;

d) carry out interdisciplinary activities to ensure the integration of the analyses undertaken for the different synthesis components. Methods such as grids, networks and superimposing maps can be used to ensure the integration of the data. Having done this, it will be possible to incorporate the interrelations of the impact processes from different synthesis components using their assessment elements. It will also be possible to find out in which synthesis components the greatest repercussions of each process are felt, and identify the most suitable assessment elements for representing these interrelations;

e) review the characterization of the impact processes relating to the groups of projects per synthesis component in view of the integration of the analyses. The outcome of this should be a general description of the impact processes considered and the assessment element adopted;

f) the processes for which control, mitigation or compensation actions can be designed must be highlighted, as this will be used in the review of the estimate of socioenvironmental costs, which are part of the total construction costs (items 5.6 and 5.7). Any adjustments to projects or the layout of cascades that would improve their socioenvironmental performance should also be identified.

Whenever the nature of the impact processes means that the comparison of the cascades can be carried out within the context of the study area as a whole without the need for breaking it down into sub-areas, the procedures put forward here still apply but will be used directly on the cascade as a whole rather than on groups of projects.

Assessment of Cumulative and Synergistic Negative Socioenvironmental Impacts

The intensity of the impact of a group of projects on the sub-areas defined for each synthesis component should be estimated on the basis of the **impact indicators** and their **assessment elements**. As already mentioned, the impacts that must be assessed are those for which no control can be introduced, or those which leave residual impacts after control, compensation or mitigation actions have been introduced. The following procedures are recommended:

a) analyze the assessment element for each group of projects with an impact on a given sub-area. This analysis should be done for each synthesis component and should take into consideration the interrelationships identified between the components;

b) attribute a negative socioenvironmental impact index to a group of projects that reflects their impact on a synthesis component per sub-area affected (I_{SAi}) as shown in the table below. This is done using the impact indicators and is based on the assessment element selected as a function of the specific nature of the impact processes. The criteria used for attributing the indexes should be stated and explained. The index should be attributed to groups of projects in a sub-area in such a way as to incorporate any cumulative processes amongst the effects of the projects.

For instance, in the case of the Terrestrial Ecosystems synthesis component, if four projects have some interference in sub-area II, as shown in Table 5.4.1.02, reducing the quantity of vegetation, the assessment element should calculate the result of the relationship between the wooded surface area affected by the four projects and the total wooded surface area in the sub-area. A similar procedure should be adopted for all the assessment elements selected to represent cumulative impacts. In other words, in order better to portray cumulative effects, the impact indexes should not be based on the individual impact indexes attributed to the each project in the Preliminary Studies and aggregated for the sub-area, but should be based on the assessment elements for each impact process relative to the projects that affect each sub-area. The same procedure is used for the assessment of positive impacts.

As more than one indicator is often used, made up of several assessment elements, it is necessary to aggregate the indexes relative to each indicator per sub-area to assess the impact on each synthesis component. This can be done by ranking the indicators according to their importance to the sub-area and by attributing relative weights.

Negative socioenvironmental impact indexes should be attributed on a continuous scale from zero to one. Zero indicates no impact, while one represents a situation where the processes inherent to the synthesis component in question are totally compromised. The intermediate values should therefore represent the extent to which the pre-existing environmental processes are compromised, according to the assessment criteria set for each synthesis component.

Table 5.4.1.02: Cumulative and synergistic impact index per sub-area for the Terrestrial Ecosystems synthesis component

Projects \ Sub-Areas	I	II	III	IV	V	VI
A		x				
B		x	x	x		
C			x		x	
F		x				
G	x					
H	x				x	
I	x	x	x	x	x	
M	x					x
N	x		x			
Q_2						x
I_{SAi}	0.65	0.55	0.95	0.20	0.40	1.00

As already mentioned, the members of the team carrying out the study should seek to reach a consensus as to the precise meaning of these intermediate values so that the assessments of different synthesis components can be compared amongst themselves. In order to do so, it is recommended that interdisciplinary activities be carried out so that the assessment criteria of the indicators can be standardized, taking into account the situations in the river basin under study. The maximum value on the scale (one) is not a comparative value, meaning it is not the highest value from amongst the projects in the river basin under study. Rather, it should represent an absolute or hypothetical situation where the process in question is totally compromised, which may or may not be the case.

Values must be attributed to the indexes for the groups of projects per sub-area before indexes can be attributed to the whole cascade in question using the procedures set out in item 5.8.2. However, whenever pertinent, indexes can be attributed directly to entire cascades provided the comparability of the judgments and the selectivity of the indicators are not affected.

c) hold interdisciplinary discussions on the assessments undertaken in order to integrate the results, identify any inconsistencies and minimize the subjectivity of the assessments made for the different synthesis components. The impact indexes attributed to each group of projects may be reviewed in the light of these discussions.

The relative subjectivity inherent to these assessments can only be reduced by standardizing the assessment criteria, assessment elements and other procedures adopted in the methodology. The repeated use of this methodology and expansion of the electricity sector database with the results of monitoring activities are central to the future effort to parametrize the assessment elements and consequently refine the values given to the socioenvironmental indicators.

5.4.2 Assessment of Positive Socioenvironmental Impacts

These studies involve analyzing socioeconomic factors, for which any favorable alterations must be assessed and translated into a positive socioenvironmental impact index to be used when the final selection of one cascade option is made. The local and regional positive socioeconomic impacts on the following items should be addressed:

- local labor market;
- municipal revenues;
- road infrastructure;
- efficient use of water resources.

The aspects highlighted above for analyzing positive impacts are the ones that appear most often in socioenvironmental studies for hydroelectric projects and about which there is less uncertainty as to the benefits they would bring to local and regional development.

However, as the analyses are being undertaken, other aspects which the building of the projects could benefit by boosting local and regional socioeconomic development may be identified, in which case they should be added to this assessment.

It should be understood that environmental compensation (Act 9.985/2000) should not be regarded as a positive impact.

The characterization elements and assessment elements needed for the analysis of positive impacts are already part of some of the synthesis components used for analyzing the negative impacts. However, they take on different functions when they are used to analyze the processes that make a positive contribution to the development of the region in question.

The gathering of information pertaining to these aspects should be started in the diagnostic studies undertaken in the Preliminary Studies (item 4.3) and supplemented as necessary when they are consolidated in the Final Studies (item 5.2).

The positive impact analyses are carried out on the groups of projects that make up each cascade and include the following stages:

- identification of the main positive impacts to be considered in the analyses and of the assessment elements for each project and group of projects that make up each cascade. This analysis should be done for each element in the environmental system selected;
- analysis of assessment elements per sub-area, considering the impact indicators selected;
- attribution of a positive socioenvironmental impact index for each cascade relative to each element in the environmental system selected.

The indexes should be attributed on a scale of **zero** to **one**, where **zero** represents the absence of any positive impact and **one** represents an extremely significant positive impact for the region.

As assessments of positive impacts are referenced to given elements used to characterize and assess the synthesis components used in the negative impact analysis, the sub-areas considered for the Way of Life, Territorial Organization and Regional Economy synthesis components should serve as a basis for these assessments. However, it is worthwhile making a detailed analysis of the sub-units of analysis to ensure they are suitable for this assessment. A review must be made of the relative weights of the sub-areas for the synthesis components to be taken as a reference.

The contents of the impact indicators and assessment elements are presented in table 5.4.2.01 and described in the items below.

Table – 5.4.2.01 – Impact indicator and assessment element for analyzing positive impacts

Element from the Environmental System	Impact Indicator	Assessment Element
Local labor market	– ratio of number of jobs created to economically active population (EAP)	– economically active population (EAP) – number of direct jobs created – municipalities benefited directly by the projects
Municipal revenues	– percentage increase in municipal revenues	– estimated energy generated per project – estimated value of financial compensation – estimated area flooded per municipality – estimated distribution of funds per municipality – budget of each municipality – survey of costs of services during the construction work for the projects – estimated ISS (service tax) levied per municipality
Road infrastructure	– length of roads to be built for the projects	– length, in kilometers, of the roads existing in the river basin – length, in kilometers, of the roads considered for the cost calculation of the projects – details of the roads to be built (if they will link up municipal seats of government to centers of major regional influence, or two or more highways of importance to the flux of people and cargos in the region; estimated number of people benefited by the new roads)
Efficient use of water resources	– percentage contribution made by the projects to the water uses planned for the river basin	– existing and potential uses of the water resources – uses of the water resources included in the Water Resource Plan or in sector/regional plans

Local Labor Market

Any boost to the local labor market would arise from the economic activities undertaken as a result of a hydropower project being built, which would create new indirect and direct jobs and enhance activity in the retail and service industries. This is a temporary impact, but it can be very significant. Additionally, the effects on the local economy may be more or less long-lasting depending on a number of factors, one of which is the mobilization of different agents (state, private sector, business sector, community).

This impact, which is positive in nature, should not be confused with other negative impacts that may be related to it, such as increased demand for public services as a function of the influx of people, the unplanned development of towns as the labor force is swelled, and the slowing down or shrinkage of local economies, with unemployed people staying on and the workforce being laid off. Impacts such as these should be addressed in the assessment of negative impacts.

In order to appreciate the dynamics of the labor market qualitatively, one acceptable simplification is to consider it as being directly proportional to the number of direct jobs created. As the economically active population (EAP) is one of the most important measures for characterizing a local labor market, this is the variable that has been selected for comparing the number of direct jobs created in a municipality, information on which can be obtained from the IBGE Demographic Census (Censo Demográfico). The indicator for the potential to boost the local labor market is, therefore, the **ratio between the number of direct jobs created and the EAP**.

Other variables and indicators can be used to enrich the characterization of the local labor market, such as the urban EAP, the rural EAP, the rate of employment/unemployment, etc.

The distribution of ways of life in the river basin should also be considered to weight the positive impact in areas where the predominant activities are not taken into account in modern, capitalist labor relations.

The following information should be available before the methodology described below can be used:

1) list of the municipalities in each of the sub-areas;
2) identification of the municipalities benefited directly by the projects, per sub-area;
3) number of direct jobs created at the peak of the construction work, per project and per sub-area;
4) EAP of the municipalities in the river basin and total per sub-area.

In this context, the "municipalities benefited directly by the projects" are those where the support bases are located while the projects are being built.

The sub-areas defined for the Regional Economy synthesis component are recommended to be used for this purpose, just altering the relative weights of the sub-areas as a function of the different analysis objectives. The information about the municipalities where the construction will be concentrated and which will consequently have jobs created at this time and the estimate of the number of jobs created per project should be obtained from the engineering studies team.

The following steps should be taken in order to attribute an impact index to each cascade:

a) calculate the percentage of direct jobs created at the peak of the construction work per sub-area as a proportion of the sum of the economically active population in the municipalities in the sub-area. The reference sub-area for any given project is the one which has the municipality that is directly benefited by this impact. If more than one municipality will have a support base for the construction of a project and these are in different sub-areas, the engineering studies team should be approached with a request for the estimated number of direct jobs to be created in the different municipalities to be benefited, so that the data can be distributed amongst the different sub-areas.

$$\text{Indprel} = \frac{\sum \text{Emp}_{\text{sub}}}{\sum \text{PEA}_{\text{sub}}} \times 100 \qquad (5.4.2.01)$$

where:

Indprel	preliminary indicator;
Emp_{sub}	direct jobs created by the projects in a given sub-area; and
PEA_{sub}	EAP of a given sub-area.

b) attribute a positive impact value to each sub-area associated to the percentage calculated. It has been decided that a 30% increase in the EAP of a given sub-area would represent a very significant boost for the labor market and even the local economy. This is, then, the parameter used to indicate the maximum positive impact on a sub-area, and takes the maximum value (1.0). Intermediate values will be attributed proportionally to increases in the EAP of between 0 and 30%, while any increase of over 30% in the EAP in the sub-area will still be given the maximum value (1.0);

c) compose the positive impact index of the potential boost to the local labor market for each cascade. The final index is the weighted sum of the impact values for the sub-areas, using the reviewed weights for the different sub-areas.

$$IPMT=\sum peso_{sub} \times Ind_{sub} \qquad (5.4.2.02)$$

where:

IPMT	positive impact index relative to the boost in the labor market;
$peso_{sub}$	weight of the sub-area; and
Ind_{sub}	indicator per sub-area.

Increased Municipal Revenues

An increase in municipal revenues is a positive impact of importance in the construction and operation of hydropower plants. It arises mainly from the payment of financial compensation for the exploitation of the water resources for the purposes of generating electricity, and from the amount of service tax (ISS) levied by the municipalities on the services rendered during construction.

Financial compensation was first introduced by Act 7990 of 1989, which states that a portion of revenues from a hydropower plant should be passed on to the states and municipalities whose land was impounded for the reservoir. The legislation that existed at the time this Manual was published put the monthly financial compensation for municipalities at 45% of 6% of the power generated by the plant per month, multiplied by the reference power rate (Tarifa Atualizada de Referência, or TAR). If more than one municipality is affected, this sum must be divided in proportion to the percentage of the area flooded in each municipality.

In a given river basin, some projects will be benefited by extra energy thanks to the existence of regulating reservoirs upstream. In order to calculate financial compensation in this case, the provisions of ANEEL resolution 88/2001[2] should be consulted, which concern the transfer of part of the financial compensation to the states and municipalities affected by these reservoirs as a proportion of the energy increase.

In order to estimate the service tax (ISS) to be levied by the municipalities where the construction sites are located, it can be assumed that most of the services rendered for building a hydropower project are for civil construction and equipment assembly, which account for around 60% of the total cost of construction[3]. The percentage of service tax levied varies from one municipality to another.

The suggested indicator only takes into account the benefits granted to the municipalities affected, since the assessment of positive impacts is designed to give precedence to local effects, as this is where the main negative impacts are felt. The indicator used to assess this positive impact is therefore the **increase in municipal revenues calculated as the ratio between the estimated benefits to be paid to the municipalities affected and their municipal revenues.**

The following steps should be followed in order to attribute a positive impact to each cascade option:

a) choice of year of reference: a year should be chosen for the purposes of comparing the values. It is best to use the most recent year for which all the data on municipal revenues can be gathered;

b) estimate of total energy generated annually from one project, considering extra energy from regulating reservoirs upstream, as set forth in ANEEL resolution 88/2001.

Use the following formula:

$$GT(k)=\sum_{i \in J(k)} g(i,k) \qquad (5.4.2.03)$$

2 Consult current industry regulations.

3 Source: EPE – Estudos de AAI (2007)

where:

GT (k)	total energy generated by project k, in MWh;
g (i, k)	annual firm energy generated by project i due to project k, in MWh; and
J(k)	group of planned and existing projects in the cascade downstream from project k.

In order to estimate the value for g(i,k), simulations of operations can be run using the SINV system.

c) estimate of the financial compensation to be distributed annually to the municipalities per project:

$$CFA(k) = 0.06 \times 0.45 \times GT(k) \times TAR \qquad (5.4.2.04)$$

where:

CFA(k)	financial compensation for one project distributed to the municipalities affected (R$);
GT(k)	total generation from project k (MWh);
TAR	reference power rate (tarifa atualizada de referência); the TAR must be from the reference year; and
factor of 0.45	proportion due to municipalities.

d) calculation of the proportion of land flooded for reservoirs:

A reservoir for a hydropower plant occupies land that may belong to more than one municipality. For each municipality affected directly by the reservoir of a project included in a cascade option, the value of the proportion of flooded land can be calculated using the following equation:

$$PMA(m,k) = \frac{AMA(m,k)}{A(k)} \qquad (5.4.2.05)$$

where:

PMA(m, k)	proportion of area flooded in municipality **m** by project **k;**
AMA(m,k)	area of municipality **m** flooded by project k; and
A(k)	total area of the reservoir for project k.

The result gives the proportion of the financial compensation paid for project k that the municipality will receive.

The recommended way for calculating AMA(m,k) and A(k) is to use geoprocessing. By superimposing the drawing of the area occupied by the reservoir on the map of the municipalities, it is possible to calculate the areas (km^2) to be flooded in each municipality affected.

e) estimate of the total financial compensation to be received by each municipality for a project in the cascade under analysis:

$$CFMA\,(m,k) = CFA(k) \times PMA(m,k) \qquad (5.4.2.06)$$

where:

CFMA(m,k)	financial compensation that municipality **m** will receive for project **k.**

f) estimate of the total financial compensation that each municipality will receive:

$$CFM(m) = \sum_{j \in AP} CFMA\,(m,k) \qquad (5.4.2.07)$$

where:

CFM(m)	total financial compensation received by municipality **m;**
AP	group of projects that make up the cascade under analysis.

g) estimate of how much financial compensation each sub-area will receive:

For each of the positive impacts arising from increased municipal revenues, the same division of sub-areas as that used for assessing the negative impacts on the Regional Economy synthesis component should be used. Generally speaking, this subdivision will be the same as the political division of the municipalities, meaning that each sub-area will correspond to a set of municipalities that represent a level or category of economic development.

$$CFS(s) = \sum_{i \in MS(s)} CFM(i) \quad (5.4.2.08)$$

where:

CFS(s)	total financial compensation received by sub-area **s**;
CFM(i)	financial compensation received per municipality affected in the sub-area; and
MS(s)	group of municipalities affected in sub-area **s**.

h) estimate of service tax (ISS) levied per municipality benefited by each project:

In order to carry out this estimate, the mean percentage of ISS levied by the municipalities where the services will be rendered while the projects covered in the inventory study are being built should be calculated. It is also necessary to gather data on the total cost of services (CTS) for the construction of each project, which can be obtained from the cost estimates presented.

A list of the municipalities that will levy service taxes while each project is being constructed should be drawn up together with the engineering studies team. Each municipality benefited is calculated as receiving a proportion **p** of the total ISS levied. The two items that account for the majority of the construction work for a hydropower plant are the dam and the powerhouse, so it is reasonable to assume that the municipalities where these two structures are built should receive the most benefits. Therefore, when it is not decided which municipalities will be benefited, the estimated ISS revenues should be split in equal parts between the municipalities where the main dam axis is to be (which is normally where the powerhouse is also built).

The ISS collected by municipality **m** can be obtained by:

$$ISS(m,i) = CTS(i) \times p(m,i) \times AL(m) \quad (5.4.2.09)$$

where:

ISS(m,i)	total ISS collected by municipality **m** during the construction of project **i**;
CTS(i)	total cost of services for the construction of project **i**;
p(m,i)	estimated proportion of ISS collected during the construction of project **i** that should go to municipality **m**;
AL(m)	estimated rate of ISS levied by municipality **m**.

i) distribution of ISS funds throughout the useful life of a hydropower plant:

Most of the revenues from ISS service tax are collected by the municipalities while the power plants are being built, which normally takes three to six years. However, municipalities also receive financial compensation over a different time frame, which begins when the plants are commissioned and lasts until the end of their useful life.

In order to assess the accrued benefit derived from the increased revenues for the municipalities directly affected by the projects covered in the Inventory Studies, assuming that the main sources of revenues will be service tax (ISS) and financial compensation (CF), the values must be compared over the same time frame. The method recommended for doing this is to transform the total ISS revenues during the construction of a project into an equivalent annual revenue throughout its useful life.

$$ISSV(m,i) = \frac{ISS(m,i) \times \left[(1+a)^t \times a\right]}{(1+a)^t - 1} \quad (5.4.2.10)$$

where:

ISSV(m, i)	equivalent of the ISS collected by municipality **m** during the construction of project **i** transformed into annual revenues for municipality **m** throughout the useful life of project **i**;
a	interest rate; and
t	useful life of the power plant (years).

The interest rate and useful life used in this calculation should be the same as those used in the engineering studies.

j) estimate of total ISS collected by the municipalities affected in each sub-area:

$$ISS(s) = \sum_{i \in MS(s)} ISSV(i) \tag{5.4.2.11}$$

where:

ISS(s)	sum of the ISSV received by the municipalities in sub-area **s**; and
MS(s)	set of municipalities in sub-area **s**.

k) calculation of the total benefit per sub-area:

The total benefit for a given sub-area **s** derived from increased revenues is the sum of the financial compensation and ISS in that sub-area, as shown below:

$$BT(s) = ISS(s) + CFS(s) \tag{5.4.2.12}$$

where:

CFS(s)	total financial compensation received by sub-area **s**; and
ISS(s)	total ISS received by sub-area **s**.

l) calculation of municipal revenues and sum of revenues in the sub-area:

Data on the municipal budgets of all the municipalities directly affected by the projects in the Inventory Studies for the reference year chosen can be obtained from the website of the National Treasury, Ministry of Finance (Secretaria de Tesouro Nacional do Ministério da Fazenda)[4].

By definition, R(s) equals the sum of the municipal revenues collected in the reference year by all the municipalities that make up the sub-area. R(s) should be calculated for all the sub-areas.

$$R(s) = \sum_{i \in MS(s)} RM(s) \tag{5.4.2.14}$$

RM(s)	municipal revenues of the municipalities that make up each sub-area; and
R(s)	total revenues for the sub-area.

m) positive impact per sub-area:

The percentage increase of revenues for the municipalities benefited in each sub-area is obtained as the ratio between the total revenue increase BT(s) in the sub-area and the sum of the municipal revenues in the sub-areas R(s).

An increase of around 30% means that the municipalities in a given sub-area will obtain additional revenues of this amount throughout the useful life of the projects constructed there. This is regarded as a very significant increase for a municipal economy and will have positive knock-on effects on the region. This is the percentage recommended to represent a satisfactory value (VS), which is the ratio between the total revenues BT(s) and the revenues from the sub-areas R(s) considered ideal. The

4 http://www.stn.fazenda.gov.br/estados_municipios/index.asp

positive impact for this percentage (30%) should be taken as the maximum value (1.0), with all other percentages corresponding proportionally to intermediate values.

The positive impact index for a sub-area can be given by:

$$I(s) = \mathrm{Min}\left\{1, \frac{BT(s)}{R(s)} \times \frac{1}{VS}\right\} \tag{5.4.2.15}$$

n) calculating the positive impact index of a cascade:

The final index is given by the weighted sum of the impact indexes of the sub-areas, using the reviewed weights for the sub-areas:

$$I_{AM\ alt} = \sum_{s} I(s) \times p(s) \tag{5.4.2.16}$$

where:

$I_{AM\ alt(s)}$	positive impact index from municipal revenues in each sub-area **s**; and
p(s)	weight of each sub-area **s**.

Road Infrastructure

This element takes into account all improvements made to the area around the plant and any connections made with adjacent municipal seats of government and neighboring areas when the projects are built, especially the building of roads and bridges, given their importance for improving accessibility and circulation throughout the region. In the description of the Territorial Organization synthesis component, it states that one of the ways of estimating changes to accessibility includes describing, mapping out and qualifying the road infrastructure affected, including the length of roads, route, occurrence of connections, corridors and their areas of influence, among others. Based on this information and the map prepared of the region's roads, experts can identify per project what infrastructure-related elements could improve regional integration and access to certain places. The divisions established for the Territorial Organization synthesis component can be used for this element, with weights being attributed to each sub-area as a function of the density of the transportation infrastructure and its local and regional importance, including any expansion needs.

The indicator suggested for this is the **length of roads, in kilometers, to be considered for calculating the costs of the projects**, which is information that is covered in the Standard Eletrobrás Cost Estimate (Orçamento Padrão da Eletrobrás, or OPE, account .16), considering only the length of road that is added to the road network. Other elements can be used as factors that leverage this variable, highlighting important characteristics for the calculation of the benefits:

- particular features of the roads to be built (if they will link up with seats of municipal government, centers of regional influence or two or more highways of importance for the circulation of people and goods in the region; estimate of the number of people benefited by the new road, etc.);
- the bridges included in the OPE should be assessed according to their local importance.

The number of kilometers per project and per sub-area should be identified, as well as plans to build bridges and other factors that could leverage this indicator. For this element, the sub-areas from the Territorial Organization synthesis component should be used.

Based on an understanding of regional dynamics, the analysts should establish a maximum value for the indicator, which would represent the number of kilometers of road added to the existing road network in the region that would bring about a major positive alteration. This will be the maximum value (1.0) on the scale of impacts. Zero will represent situations where no roads are added. If there are other factors that improve road transportation, these can be taken into account, with the impact value being raised accordingly. This is how each sub-area is attributed a value for this indicator.

The impact index for a project will be given by the weighted sum of the indexes of the sub-areas, using the reviewed weights for the sub-areas.

Efficient Use of Water Resources

Opportunities for positive impacts related to multiple water uses should be identified from the analysis of expected changes to water uses in the river basin which is undertaken when the Scenario of Multiple Water Uses is prepared, as described in item 4.2.2.

This analysis gives a long-term view that is compatible with the time frame of the National Plan for Water Resources (20 years). For each section of river in the basin under study, it gives the flows and heads required for other uses that will restrict energy generation, but which could be seen as opportunities for positive impacts, such as irrigation for agriculture, flood control, navigation, water supply, aquaculture, and even tourism in some cases. If the Scenario of Multiple Water Uses predicts that specific volumes should be set aside for regulating reservoirs, for instance, it can be deduced that a positive impact will arise from this in the form of flood control. Likewise, if the scenario identifies a net loss of flow because of withdrawals, a positive impact can be deduced from this in the form of increased access to water in the region under study. Whenever there is any kind of navigation system, or when building the reservoir makes navigation feasible, by, for instance, assuring the flows required for operating the locks, this could also indicate the potential for a positive impact, in this case benefiting small-, medium- or large-scale navigation.

Based on the analysis of trends presented in the Scenario of Multiple Water Uses, the level of positive impacts should be estimated for the region, considering each cascade option and the multiple uses of the waters in the river basin.

The positive impact indicator suggested for this is designed to measure how much each of the projects that make up each cascades will contribute to the aims of the existing river basin plans, regional plans or sector plans by expanding the area suitable for irrigation, providing more extensive waterways for navigation, introducing more flood control mechanisms, etc.

The following steps should be taken:

a) Attribution of a positive impact to each of the uses per cascade option

Depending on the nature of the water uses, assessments should be made per sub-area, using the division established for the most representative synthesis component in each case (e.g. for irrigation, the Regional Economy synthesis component could be used; for water supply, the Ways of Life synthesis component would be more appropriate). For navigation, the whole study area can be adopted as the unit of analysis, without sub-dividing into sub-areas. When the impact is assessed per sub-area, the total impact of the cascade should be calculated afterwards by summing the weighted impacts of the different sub-areas.

The level of positive impact for each of the uses should be attributed on a scale from zero to one. If there is no contribution to any of the plans, the positive impact is zero. Otherwise, the greater the contribution to the plan analyzed arising from the expansion provided by the projects, the greater the positive impact and the closer the value comes to one.

b) The final positive impact arising from the improvement each cascade option would bring to the efficient use of water resources should be determined by compiling the impact for each water use (irrigation for agriculture, flood control, navigation, water supply), weighted by their relative importance, considering the context of the river basin under study. Equation 5.4.2.17 shows how this should be done.

$$\sum_j \left(Ia_j \times p_j\right) = \left(Ia_{irr} \times p_{irr}\right) + \left(Ia_{cc} \times p_{cc}\right) + \\ + \left(Ia_{aq} \times p_{aq}\right) + \left(Ia_{nf} \times p_{nf}\right) + \left(Ia_{as} \times p_{as}\right) + \left(Ia_{tr} \times p_{tr}\right) \quad (5.4.2.17)$$

where:

Ia_j	positive impact of the cascade option on use j;
p_j	relative importance of use j;
irr	represents irrigation;
cc	represents flood control;
aq	represents aquaculture/fish farming;
nf	represents navigation;
as	represents water supply; and
tr	represents tourism.

The weighting of the importance (p_j) of each use should be based on the priorities set in the Water Resource Plan (Plano de Recursos Hídricos, PRH) taken as a reference for the Scenario of Multiple Water Uses (item 4.2.2). The reference for the Scenario of Multiple Water Uses may vary as shown below:

a) scenario based on a Water Resource Plan for the river basin;

b) scenario based on one or more sector plans or regional development plans;

c) scenario prepared without any reference plans.

When the scenario is based on a Water Resource Plan for the river basin (a), the weighting of each water use in the final assessment of the positive impact in question should be based on the priorities of use established in the Water Resource Plan (PRH) for the river basin.

When the scenario is based on one or more sector or regional development plans (b) the weighting of each water use in the final assessment of the positive impact in question should be based on the information presented in the diagnosis and in the Scenario of Multiple Water Uses. In this case, the final positive impact should also be reduced by a factor of 0.5 because in the absence of a PRH there is no guarantee that greater efficiency of the water uses will be achieved.

When the scenario is prepared without any reference plans (c), the opportunity for enhancing the efficient use of the water resources should be taken as zero, and this dimension should not be addressed when compiling the final positive impact index.

Indicators and methodologies are proposed below for attributing the level of impact to each of the water uses addressed.

a) Irrigation

Periods of drought are major factors in the development of agriculture in some regions. The building of reservoirs can be an opportunity for developing irrigation projects, bringing major benefits. If the Scenario of Multiple Water Uses mentions this kind of activity, the following steps should be taken to make the analysis:

- check whether there is a Water Resource Plan for the river basin or some plan for future irrigation projects with a time frame that is compatible with the Water Resource Plan. Taking this plan as a reference, check the location and intended area of irrigation (hectares) and the contribution that the new hydropower projects could make to these irrigation projects;
- should the Plan indicate the need for flow regulation and the project be designed to have a regulating reservoir, the positive impact can be calculated as the percentage of the area to be irrigated (according to the plan) that could benefit from this flow regulation compared to the total irrigated area set out in the plan;

- should a minimum water level be required for withdrawals, the positive impact can be calculated as the percentage of the area to be irrigated (according to the plan) in the area surrounding the reservoir compared to the total irrigated area set out in the plan;
- the level of impact can be calculated by locating the areas identified for irrigation, the total withdrawals for irrigation in the plan, and the areas benefited by each cascade option. The sub-areas from the Regional Economy synthesis component should be used. The positive impact assessment should be presented in terms of percentages for each sub-area impacted by each cascade, based on the **ratio between the area of land to be irrigated as a result of the projects from the different cascade options and the area suitable for irrigation described in the plan analyzed**, using equation 5.4.2.18 below:

$$Ia_{irr}^{x,y} = \frac{\sum_{n}\left(Airr_{n}^{x,y}\right)}{\sum_{m}\left(Airrplan_{m}^{x}\right)} \qquad (5.4.2.18)$$

where:

$Ia_{irr}^{x,y}$	positive impact on irrigation in sub-area x brought about by the projects in cascade y;
$Airr_{n}^{x,y}$	area n (in hectares) that could become suitable for irrigation thanks to the projects from cascade option y in sub-area x; and
$Airrplan_{m}^{x}$	area m (in hectares) of expansion of irrigation planned for sub-area x.

- next, the percentage thus obtained is transformed into a value on a scale from zero to one;
- the total impact of each cascade on irrigation will be given by the weighted sum of the values obtained for each sub-area, considering the reviewed weights for each sub-area.

b) Flow Regulation / Flood Control

When the projects are built, the flows and water levels may be regulated to a greater or lesser extent, which helps with flood control or may improve the flood alert system.

In order to assess this positive impact, the Scenario of Multiple Water Uses should be consulted to see if there is any need for flood control in the municipalities where the sub-areas under study are. If so, the following steps may be followed:

- check for the existence of plans for flood control measures in the Water Resource Plan for the river basin or in the Macro-Drainage Plan, with a time frame that is compatible with that of the Water Resource Plan. Check where the rural and urban populations that should be protected according to the plan are concentrated;
- select which synthesis component to use as the basis for the analysis of sub-areas, taking into account the features of the region and what aspects will be most benefited from flood control (Ways of Life, Regional Economy and Territorial Organization);
- estimate the area that will cease to be flooded by floodwaters or the improvements to the flood alert system, considering each sub-area from the synthesis component adopted as a reference, i.e. the ratio between the area subject to flooding before the measures in the plan are taken and after they are put into place, per sub-area;
- the assessment of the positive impact should be based on the total area to be protected in the basin by the projects in each cascade option. This should give the **ratio between the area benefited by the projects and the total area protected by the flood control measures in the plan for each sub-area**, as shown in equation 5.4.2.19.

$$Ia_{cc}^{x,y} = \frac{\sum_n \left(Acc_n^{x,y}\right)}{\sum_m \left(Accplan_m^x\right)} \qquad (5.4.2.19)$$

where:

$Ia_{cc}^{x,y}$	positive impact on flood control in sub-area x brought about by the projects in cascade y;
$Acc_n^{x,y}$	area n (in hectares) benefited by the flood control brought about by the construction of the projects that comprise cascade y in sub-area x; and
$Accplan_m^x$	area m (in hectares) planned to have flood control in the sub-area.

- Afterwards, the percentage obtained should be transformed into a number from zero to one.
- The total impact of the cascade in terms of flood control is given by the weighted sum of the values obtained for each sub-area, considering the reviewed weights for the sub-areas.

c) Navigation

Waterfalls or other obstacles prevent rivers from being used for navigation. When reservoirs are built, rivers can be made usable as waterways, which is a potentially significant positive impact for regions with intense or potential flows of people and/or goods. This effect is considered to be a positive impact when navigation is dependent on the construction of the future reservoir.

In order to assess the opportunity for positive impacts of this nature, the Scenario of Multiple Water Uses must be checked to see if it mentions this kind of activity. If it does, the following steps should be taken:

- check for the existence a Water Resource Plan for the river basin or a navigation sector plan covering a time frame that is compatible with the Water Resource Plan. If there is a river basin or sector plan that includes the construction and/or improvement of a waterway (independent of the size), the positive impacts to be considered are those improvements that the construction of the project will bring about;
- determine the length and location of sections of river where the building of the reservoirs would make it feasible to introduce or expand navigation;
- the positive impact assessment for navigation is based on an assurance of the minimum water depths required for such, or the construction of some system of locks for vessels, cargos or passengers using the waterway, depending on the project design. Whenever the building of the reservoir gives rise to plans for new locks for the site or allows for navigation on the river, it can be assumed that there is a potentially major positive impact;
- the level of positive impact is determined as the **ratio of the length of sections of river made navigable by building the projects –** provided they are suited to the size and number of vessels expected for the area, as described in the Water Resource Plan or sector plan **– to the total length set out in the plan** (equation 5.4.2.20). The information needed for this calculation is the number of kilometers and location of the sections benefited by each cascade option, and the number of kilometers and location of the sections included in the plan.

$$Ia_{nf}^{x,y} = \frac{\sum_n (nf_n^{x,y})}{\sum_m (nf_m^x)} \qquad (5.4.2.20)$$

where:

Ia_{nf}^{xy}	positive impact on navigation in sub-area x caused by the projects in cascade y;
$nf_n^{x,y}$	section made navigable by the construction of the projects in cascade y in sub-area x; and
nf_m^x	section m planned to be made navigable in sub-area x.

- Afterwards, the percentage obtained should be transformed into a number from zero to one. Whenever necessary, the analyst may make some adjustment to take into account the statistics on flows of people and goods in the region;
- In this case, as the assessment is not made by sub-areas, a total impact for each cascade will be given.

d) Public Water Supply

When reservoirs are built, it is possible to provide more direct water withdrawals, increased flows for withdrawal, and serve a broader geographical area and number of people in the municipalities that make up the sub-areas from the Ways of Life synthesis component.

In order to assess the potential for this kind of positive impact, the Scenario of Multiple Water Uses should be checked for plans to expand public water supplies to the people living in the region and the places where withdrawals would be made. If this subject is addressed, the following steps can be taken:

- check for the existence a Water Resource Plan for the river basin or a public water supply plan or Municipal Master Plans covering a time frame that is compatible with the Water Resource Plan that includes expansions of water supplies to the people living in the region, and the respective locations of the withdrawals;
- use the sub-areas from the Ways of Life synthesis component as the basis for the analyses and review the relative weights of the sub-areas in view of the objective of this assessment;
- the assessment of positive impacts brought about by the new hydroelectric projects on public water supply should be based on the extent to which they contribute to expanding supply to the local population;
- should the plan indicate the need for flow regulation and the project be designed to have a regulating reservoir, the positive impact can be calculated as the percentage of the (inhabited) area covered in the plan to be benefited by this flow regulation compared to the total area intended to be served according to the plan;
- should the water level have to be maintained for withdrawals, the positive impact can be calculated as the percentage of the (inhabited) area covered in the plan to be served in the area surrounding the reservoir compared to the total area to be supplied, as set out in the plan, in much the same way as is proposed for irrigation. In order to do this, the inhabited areas that need to be supplied and the withdrawals contained in the plan must be located, as well as the people to be benefited by each cascade option;
- the positive impact can be assessed as a percentage for each sub-area impacted by each cascade option, based on the **ratio between the area where water supply will be expanded as a result of the projects in the cascades, and the area covered by the plan**, using equation 5.4.2.21 below.

$$Ia_{as}^{x,y} = \frac{\sum_{n}\left(Nabs_{n}^{x,y}\right)}{\sum_{m}\left(Nabsplan_{m}^{x}\right)} \tag{5.4.2.21}$$

where:

$Ia_{as}^{x,y}$	positive impact on water supply in sub-area x brought about by the projects in cascade y;
$Nabs_{n}^{x,y}$	withdrawals and villages planned to receive a water supply that would be benefited by the construction of the projects in cascade y in sub-area x; and
$Nabsplan_{m}^{x}$	withdrawals and villages planned to receive a water supply in sub-area x.

- Afterwards, the percentage obtained for each sub-area should be transformed into a number from zero to one. The total impact of the cascade is given by the weighted sum of the values obtained for each sub-area, considering the reviewed weights for the sub-areas.

e) Aquaculture

Once the reservoirs have been built, their use can be shared with fishing activities, and the volume of water in the lakes can be used to farm fish species suitable for aquaculture, which can be considered a positive impact.

In order to assess the positive impact on aquaculture, it must first be checked whether this kind of activity already exists in the municipalities from the sub-areas in the Regional Economy synthesis component, or if the Scenario of Multiple Water Uses covers this kind of use. The location of activities of this kind should be identified, along with yield forecasts (t/ha). If fishing exists in the area, the following steps should be taken:

- check whether the Scenario mentions a Water Resource Plan for the basin, or a fish farming or aquaculture industry plan whose time frame is compatible with that of the Water Resource Plan;
- assess the expansion forecast in the plan for these activities in the municipalities from the sub-areas. This assessment can be expressed in terms of the areas and yield (t/ha) forecast in the plan that would be benefited by each cascade option;
- the positive impact assessment for aquaculture should be based on the contribution that building the reservoirs would make to creating new areas or expanding existing areas for fish farming within the sub-areas, expressed as a percentage of the total areas forecast in the plan. (Equation 5.4.2.22).

$$Ia_{aq}^{x,y} = \frac{\sum_{n}(Caq_{n}^{x,y})}{\sum_{m}(Caqplan_{m}^{x})} \qquad (5.4.2.22)$$

where:

$Ia_{aq}^{x,y}$	positive impact on aquaculture in sub-area x brought about by the projects from cascade y;
$Caq_{n}^{x,y}$	aquaculture capacity benefited by the construction of the projects from cascade y in sub-area x; and
$Caqplan_{m}^{x}$	planned aquaculture capacity expansion in the plan analyzed for sub-area x.

- Afterwards, the percentage obtained for each sub-area should be transformed into a number from zero to one. The total impact of the cascade is given by the weighted sum of the values obtained for each sub-area, considering the reviewed weights for the sub-areas.

f) Tourism

In some regions, reservoirs can provide new leisure and vacation options in the study area. These could include recreation activities such as angling and bathing, landscape enhancement and others that did not exist prior to the project.

In places where the potential for this kind of activity has already been identified and where a river basin or sector plan exists that mentions the development of tourism and/or leisure associated to the reservoir (e.g. holiday homes, hotel infrastructure, introduction of lake-based water sports, etc.), there can be considered to be potential for positive impacts.

It should be assessed what contribution the building of **run-of-river projects** could make to the tourism activities addressed in the plans analyzed. It is therefore necessary to locate the tourist areas included in the plans and the benefits brought about by each cascade. In the case of reservoirs with drawdown capacity, no positive impact can be assumed for tourism.

In order to assess the potential for positive impacts, it should be checked in the Water Resource Plan for the river basin or in the tourist industry plan mentioned in the Scenario of Multiple Water Uses if there are plans to expand tourism in the area. If there are, the following steps can be taken:

- for each sub-area impacted, measure the feasibility of expanding tourist areas and increasing the number of people benefited as a result of building the reservoirs (e.g. bathing in the reservoirs);
- the positive impact assessment must be expressed in percentage terms for each sub-area impacted by each cascade, based on the **ratio between the areas to be benefited by the projects from each cascade option to the areas included in the plan consulted**, as shown in equation (5.4.2.23).

$$Ia_{tr}^{x,y} = \frac{\sum_{n}(Atr_{n}^{x,y})}{\sum_{m}(Atrplan_{m}^{x})} \tag{5.4.2.23}$$

where:

$Ia_{tr}^{x,y}$	positive impact on tourism in sub-area x brought about by the projects in cascade y;
$Atr_{n}^{x,y}$	expansion of tourist activities brought about by the construction of the projects from cascade y in sub-area x; and
$Atrplan_{m}^{x}$	planned expansion of tourist activities in sub-area x.

- Afterwards, the percentage obtained for each sub-area should be transformed into a number from zero to one. The total impact of the cascade is given by the weighted sum of the values obtained for each sub-area.

5.5 FINAL LAYOUT OF PROJECTS

5.5.1 Introduction

In this item, guidelines are presented for designing the overall layout of projects and their structures, and information is provided on the criteria to be used for dimensioning these structures and equipment and for quantifying the construction work required.

At this phase of the Inventory Studies, the structures and equipment that make up the hydroelectric project do not have to be defined in detail because not enough is generally known about the local topographic, hydrological and geological conditions for a high level of detail to be achieved. The process of defining the layout of the structures and equipment consists of selecting from amongst the typical standard solutions that are most commonly used, based on current experience, the ones that would best suit the physical characteristics of the site under study, using conservative criteria and assumptions.

Some examples of typical layouts for hydropower plants are presented in Figures 5.5.1.01, 5.5.1.02 and 5.5.1.03.

Below, some *typical layouts* that can be used for designing the projects are presented and defined, alongside criteria that will determine their selection and the procedures for quantifying them for cost estimate purposes. However, it should be understood that the set of solutions presented here is not exhaustive and the conditions for their use and procedures for their design can be adapted.

Some specific calculation procedures, such as those used for determining the volume of excavation required for the approach and downstream channels, are approximate, and should only be used when the data from the field is not detailed enough for more accurate procedures to be used.

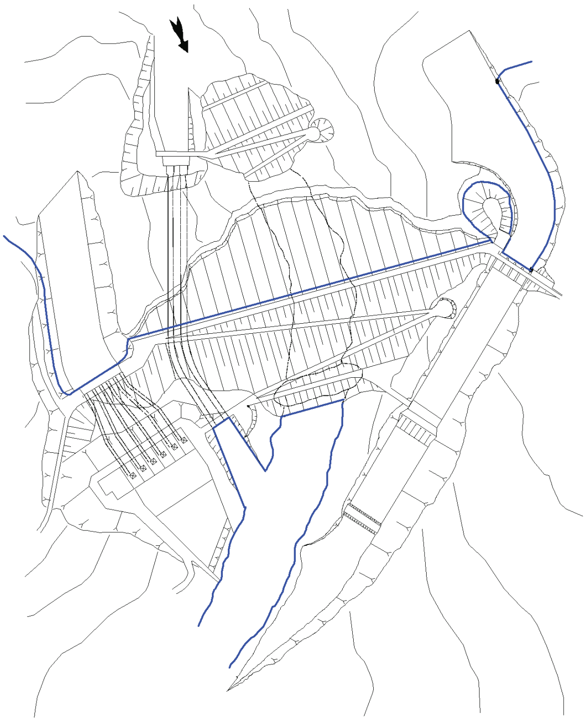


Fig. 5.5.1.01 – Typical layout for a medium-wide river valley (Gov. Bento Munhoz da Rocha Neto hydropower plant – Foz do Areia, Iguaçu River, South of Brazil).

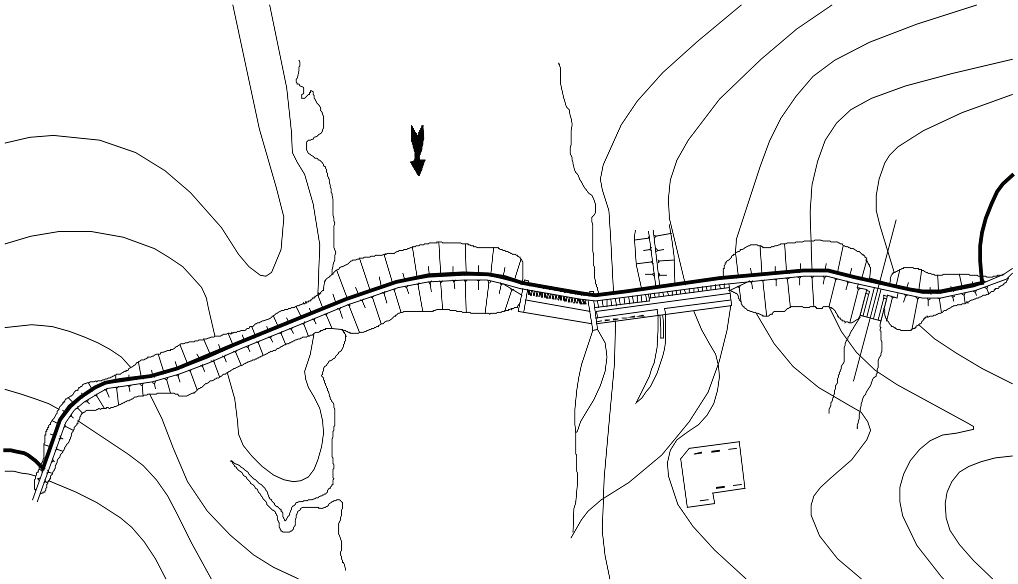


Fig. 5.5.1.02 – Typical layout for a very wide river valley (Tucuruí hydropower plant, Tocantins River, Noth of Brazil).

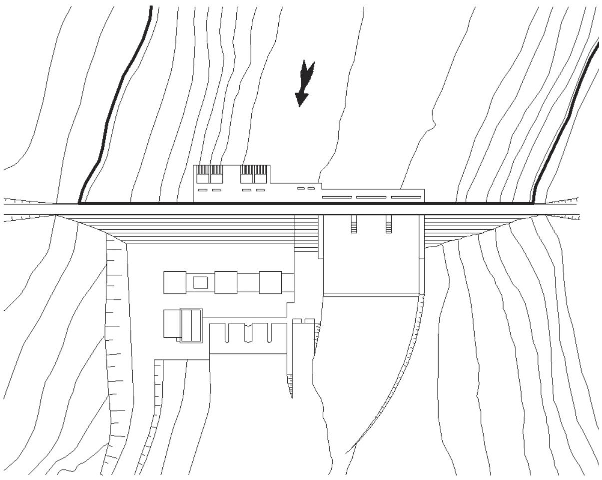


Fig. 5.5.1.03 – Typical layout for a narrow river valley.

5.5.2 Hydraulic Conveyance Facilities

Hydraulic conveyance facilities can comprise the following structures:

- headrace canal;
- forebay;
- intake;
- intake penstock or tunnel;
- surge tank;
- pressure penstock or tunnel;
- powerhouse; and
- tailrace canal or tunnel.

The dimensions of the hydraulic conveyance facilities are determinant factors for the design of a plant's overall layout. The structures used for hydraulic conveyance should be arranged so as to provide the shortest route possible, resulting in the lowest construction volumes.

The layout of the hydraulic conveyance facilities will depend primarily on the topographic and geological features at the site, the maximum turbine flow and the maximum reservoir drawdown. Some typical layouts for the hydraulic conveyance facilities are described below.

- Hydraulic conveyance facilities for projects in which the difference in water level is essentially caused by the dam, with the powerhouse located at the foot of the dam:

- projects with a low head, without any pressure penstocks and with the intake and powerhouse integrated into the same structure, using Kaplan turbines with a semi-spiral concrete casing or Bulb turbines (Fig. 5.5.2.01); and
- projects with a medium or low head, with a gravity intake making up part of the dam, and with pressure penstocks that are partially or fully embedded into the concrete of the intake (Fig. 5.5.2.02).

In this case, the powerhouse equipped with Kaplan turbines with a steel spiral casing or Francis turbines can be on the river bed or not. In projects with a low head or concrete dams, the powerhouse is generally on the river bed. In projects with few generating units and a medium head, or earthfill and concrete dams, the intake is generally in one of the abutments in order to reduce the volume of concrete required.

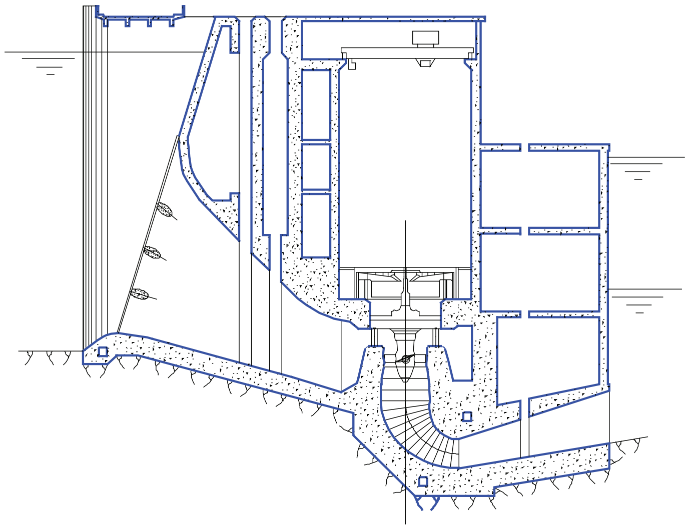


Fig. 5.5.2.01 – Project with integral intake powerhouse (Esperança hydropower plant).

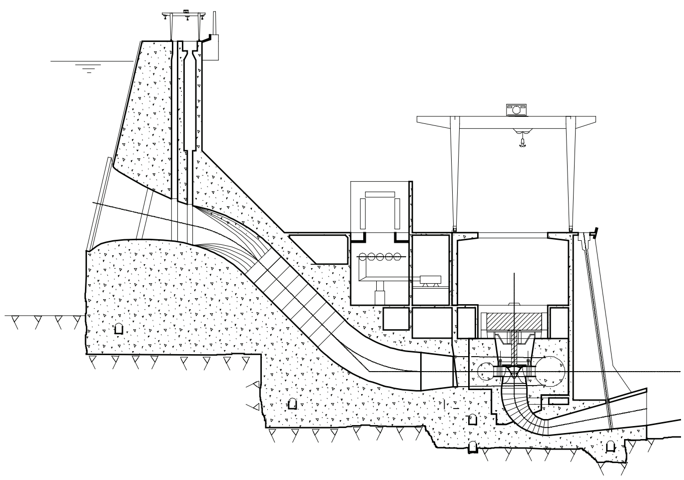


Fig. 5.5.2.02 – Compact project with pressure penstocks (Água Vermelha hydropower plant).

- Hydraulic conveyance facilities for projects where the river is diverted permanently to a new course:
- projects with the permanent river diversion through a canal (Fig. 5.5.2.03), made up of a headrace channel, intake, pressure penstock or tunnel, powerhouse and tailrace canal; and
- projects with the permanent river diversion through a penstock (Fig. 5.5.2.04), made up of a headrace canal, intake, low pressure intake penstock, surge tank, valve houses, pressure penstock or tunnel, underground or surface powerhouse and tailrace canal or tunnel.
  In both these kinds of projects, the powerhouse is at a distance from the dam and is generally equipped with Pelton or Francis turbines.

The choice between a canal and a low pressure penstock depends on the economic analysis, and should also consider the potential use of excavated material in building earthfill dams. Generally speaking, diversions through canals are recommended for projects with small reservoir drawdowns and when the topography is flat. When a penstock is used, the maximum turbine flow will be low and this is almost always recommended when the shortest distance between the reservoir and the powerhouse goes through hilly terrain. When a tunnel is used, the rock cover should be more than three times the diameter of the tunnel.

Projects with diversions through canals may require an extra control structure at the canal inlet, while diversions through tunnels may require a surge tank and valves.

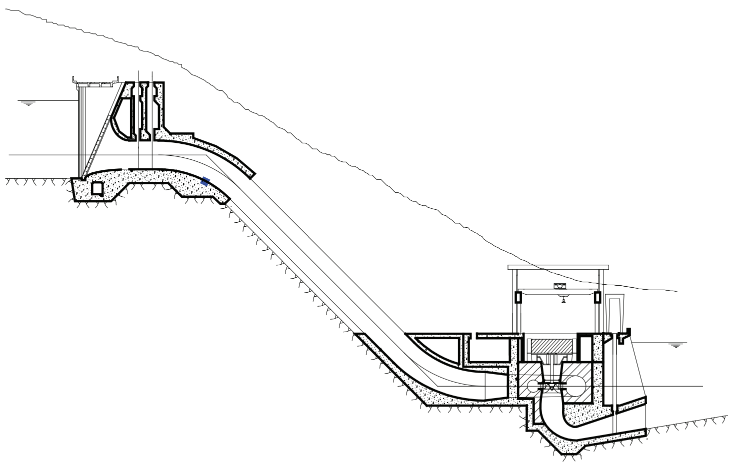


Fig. 5.5.2.03 – Project with river diversion through an open channel (Erveira hydropower plant).

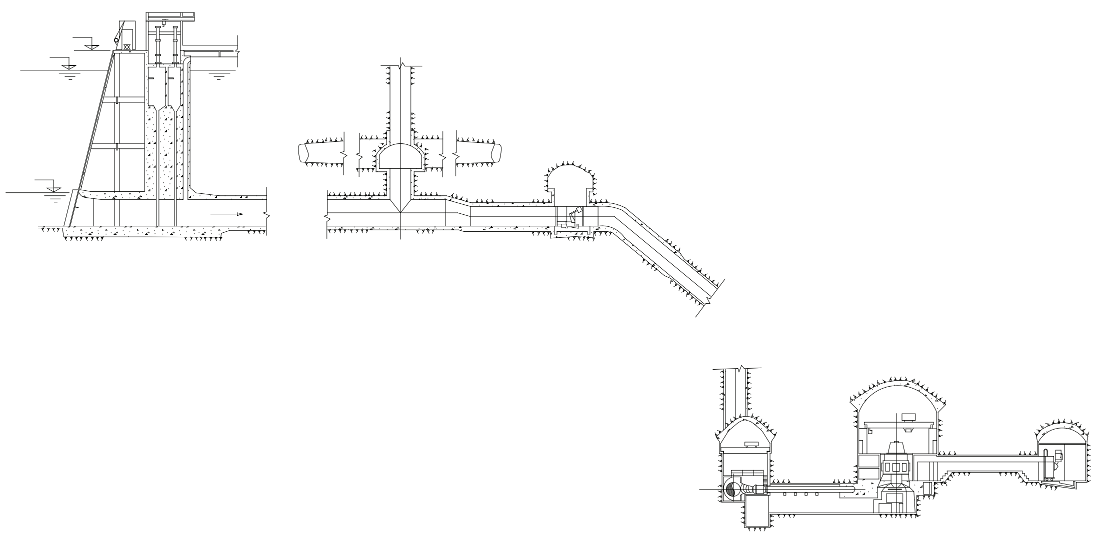


Fig. 5.5.2.04 – Project with diversion through a penstock (Capivari Cachoeira hydropower plant).

The preliminary estimates of *head loss* consider the sum of the head loss at the intake and pressure penstocks – along the penstock, at the transition, at bends and where the diameter is reduced, and at the bifurcation and valves, when applicable – plus the head loss in the headrace and tailrace canals, when this is significant, and in the headrace tunnel and surge tank when applicable. In low-head projects these losses should be calculated more carefully and include losses at the draft tube outlet.

The head loss should be adjusted in the energy studies once the hydraulic conveyance facilities have been designed whenever there is a big discrepancy with the original value.

HEADRACE CANAL (ACCOUNT.12.19.31)

The headrace canal can be classified as:

- short: no need to be specially designed; the velocity of discharge should only be checked if it is greater than the minimum, around 1.0 to 1.5 m/s; and
- long: normally linking two points on the same river, normally keeping to contour lines and excavated in an abutment and in some cases with a side embankment.

Basic Design and Recommendations

The profile of a *long headrace canal* will depend on the local topographic and geological conditions and the general layout of the other structures. Generally speaking, it will follow contour lines in order to minimize excavation requirements.

The breadth of the canal bottom should ideally be constant. The side walls of canals excavated in soil should be inclined at 1V:1.5H; if they are in rock, they should be 1V:0.25H.

In some cases, the closing of the final section of the headrace canal will require the construction of concrete walls or dikes at a right-angle to the intake.

When the canal is very long (over 3.0 km), at the end of it, near the intake, a forebay should be built to supply or store water should the turbines be started or stopped suddenly. In these cases, a lateral spillway can be built.

In every case, a flow control structure should be designed for the inlet to the canal.

For short headrace canals, which are defined by the excavation needed to access the intake, there is normally no need to determine the profile of the water line. It should be enough to check the maximum velocity and assume a loss at the inlet of about 20% of the velocity head.

Criteria and procedures for dimensioning and quantification

The procedures for designing a long headrace canal are set out in item 5.7.6. – Intake – Headrace Canal. Use spreadsheet 576cn.xls for calculating dimensions, quantifying volumes and estimating costs.

The design takes into account at least three cases for the minimum hydraulic cross-section of the canal:

- in soil and rock;
- in rock; and
- in soil.

Basic data are used to calculate the flow depth, width of the canal and volumes of excavated material and concrete.

The headrace canals should be designed for the total maximum turbine flow of the plant and for the reservoir at its minimum normal level.

When necessary, for watertightness or structural reasons, concrete lining measuring 0.1 m thick should be planned for flows of less than 100 m^3/s, or 0.2 m for all other cases.

The average velocity of discharge should be around 1.0 m/s in canals excavated in soil and with a lining to protect them against erosion. For canals excavated in rock or lined with concrete, the velocity can be around 1.8 m/s.

INTAKE

The most usual kinds of intake are:

- intake tower;
- gravity; and
- integral intake powerhouse.

Intake towers (Fig. 5.5.2.05) are normally used in projects where the diversion tunnel or gallery is also part of the intake.

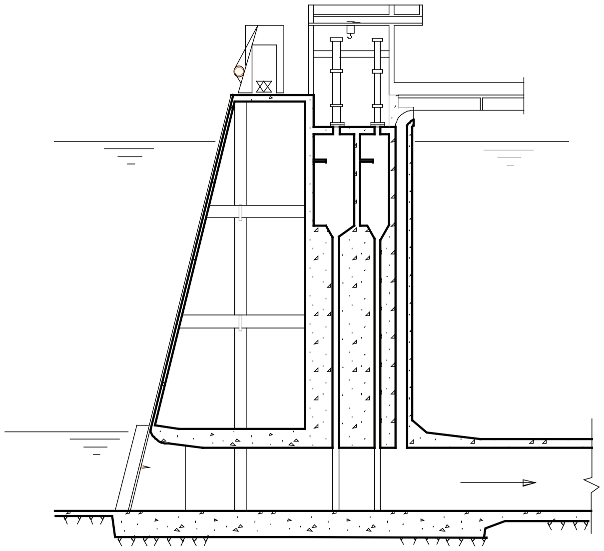


Fig. 5.5.2.05 – Intake Tower.

Gravity intakes can be integrated or not into the dam structure and make use of surface penstocks (Fig. 5.5.2.06). These intakes are used in projects using Pelton, Francis or Kaplan turbines with a steel spiral casing.

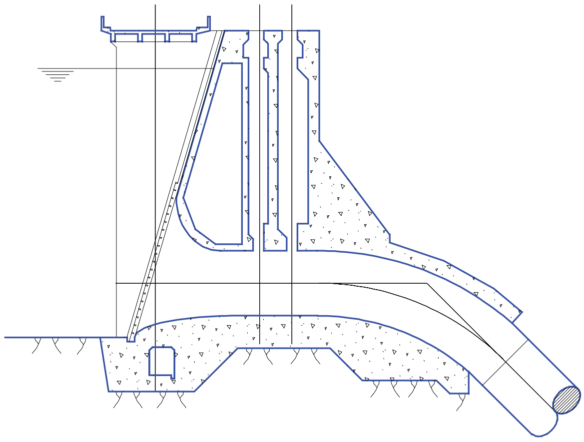


Fig. 5.5.2.06 – Gravity intake with surface penstocks.

One variant is the reduced gravity option (Fig. 5.5.2.07), which normally sits on the bedrock. This kind of intake uses pressure or non-pressure tunnels. The spacing between the units is increased to assure the stablility of the underground excavation. These intakes are used in projects equipped with Pelton or Francis turbines, or occasionally with Kaplan turbines with a steel spiral casing.

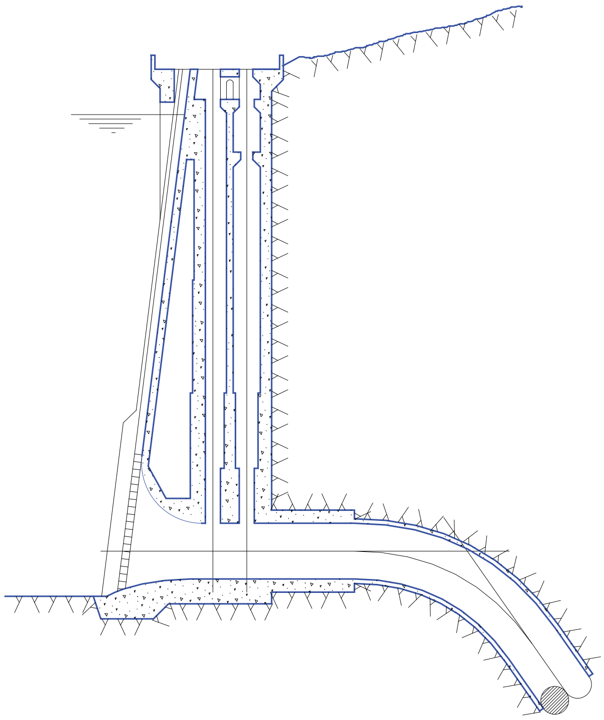


Fig. 5.5.2.07 – Reduced Gravity Intake.

Integral intake powerhouses are recommended for projects using Bulb turbines (Fig. 5.5.2.08) or Kaplan turbines (Fig. 5.5.2.01) with a semi-spiral concrete casing. For this kind of structure, fixed-wheel gates should be used upstream from Kaplan turbines or downstream from Bulb turbines.

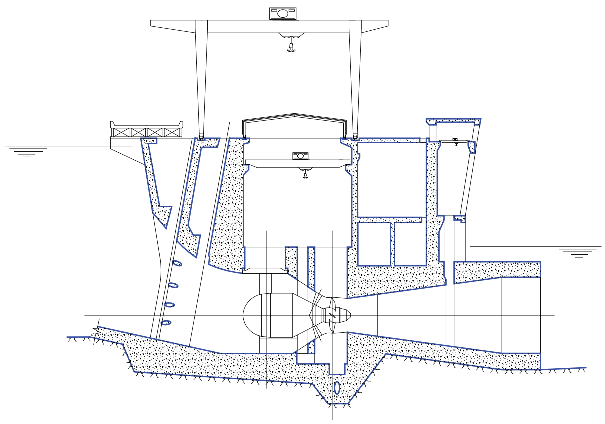


Fig. 5.5.2.08 – Integral Intake Powerhouse.

Basic Design and Recommendations

The location of the *gravity intake* in projects with a powerhouse at the foot of the dam will depend on the position of the powerhouse. When the gravity intake has a long headrace canal, the intake should be shifted as far downstream as possible along the intake route, lengthening the canal and shortening the penstock. The intake must be equipped with one fixed-wheel gate per unit, which should be near the upstream face of the structure, immediately downstream from the inlet transition.

The *reduced gravity intake* is positioned along the profile of the intake at the point where the tunnel can be excavated (with rock cover measuring at least three times the diameter).

In either of these kinds of structure, any of the following kinds of optimizations are avoided in Inventory Studies:

- narrowing in the area of the gate while simultaneously increasing its height to reduce the volume of concrete and cost of the gates; and
- installation of intermediate pillars at the inlet to reduce the cost of trash racks and/or gates.

The position of the *integral intake powerhouse* obviously depends on where the powerhouse is built. When the powerhouse is equipped with Bulb turbines, this kind of intake can be designed without an intermediate pillar or with one pillar to reduce the gap for the stoplogs in the case of high turbine flows; the emergency gates are positioned inside the draft tube. One or two pilars can be built for Kaplan turbines.

The structure should be as low as possible, but should respect the need for submergence and the required elevation of the foundations. The minimum submergence is determined using the method developed by Gordon (1970), which recommends using an expression as a function of the water velocity at the gate, the height of the gate and the shape of the headrace canal.

Whatever the kind of intake, the position of the gate should be decided on with minimum submergence, which is understood as being the minimum vertical distance between the upper horizontal edge in the section of the gate and the minimum normal water level in the reservoir. The height of submergence is designed to eliminate or minimize the formation of vortexes.

Just upstream from the gates, there should be stoplogs with the same dimensions.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning the intake are described in item 5.7.6. – Intake – Gravity Intake. Use spreadsheet 576TG.xls for calculating dimensions, quantifying volumes and estimating costs of gravity intakes.

Intakes for intake tunnels should be designed using the same criteria as those used in intakes for pressure penstocks.

For the purposes of estimating the construction work using the electronic spreadsheets, the area of the intake is defined by the ends of the concrete structures, limited downstream by the outside face of the wall.

The volume of concrete can be obtained from a theoretical curve developed by COPEL (1980), which is a function of the height of the structure and the diameter of the penstock that the intake is connected to. The volumes of concrete for gravity and reduced gravity intakes of the same height and diameter are considered to be equivalent. It is acceptable to assume that the volume of the buttress for the reduced gravity intake will offset the increased volume due to the greater spacing between the units.

The size of the trash racks for the intake should not be estimated at this moment. In this manual, a low enough velocity is assumed (around 1.0 m/s) to keep head loss at acceptable levels in the section with trash racks.

In intakes with up to 10 units, two stoplogs should be planned, so that two units can be closed simultaneously. For the other units, only fixed and embedded parts can be used. When an intake has over 10 units, it should be planned in such a way that three units can be shut down simultaneously.

The gantry crane for operating the intake stoplogs is normally a bridge crane running along tracks fixed to the crest of the structure. There should be a crane of this type no matter how many units there are. When the layout is such that the same gantry crane can be used to operate the spillway stoplogs, too, this is acceptable. The capacity of the crane is defined as a function of the heaviest weight to be manouvered and the cost should be allocated to the structure with the heaviest stoplog.

INTAKE PENSTOCKS (ACCOUNT.12.19.32)

Intake penstocks can be on the surface (fiberglass, concrete, steel, etc.) or underground (in tunnels) and always operate at low pressure.

Intake penstocks are only normally on the surface when the maximum turbine flow and pressure are very low, when the lining does not have to be very thick.

This manual only provides the criteria and procedures for dimensioning underground penstocks.

Basic Design and Recommendations

The same intake tunnel may be used for more than one generating unit.

The *profile of the intake tunnel* will depend on the local topographic and geological conditions and the general layout of the project. The profile should take the tunnel to the beginning of the pressure penstock along straight sections, prioritizing areas with the most coverage, keeping the total length to a minimum and avoiding any faults identified in the general geological studies. However, depending on the length and the construction methods used, it may be worth defining the profile so that intermediate openings can be created for construction purposes so that the distance for transporting excavated material can be optimized.

The *longitudinal profile* of the tunnel should in theory be almost horizontal, with a slope of 0.5% and a rectangular arc cross-section.

The maximum and minimum *mean velocity of discharge* will depend on whether the tunnels will be lined with concrete.

The diameter of the excavation section should be at least 3.0 m and no more than 15.0 m. In tunnels whose diameter is over 15.0 m, the mean velocity of discharge should be increased to the limit and if necessary it should be lined with shotcrete to raise the velocity limit, or else have the number of tunnels increased. If the diameter results in values that are lower than the minimum, the mean velocity can be reduced to maintain this limit or else the possibility of replacing one section with a canal or surface penstock should be investigated.

Criteria and procedures for dimensioning and quantification

The procedures for designing intake tunnels are set out in item 5.7.6 – Intake – Intake Tunnels. Use spreadsheet 576CA.xls for calculating dimensions, quantifying volumes and estimating costs.

The diameter of the tunnel, head losses and construction volumes can be calculated from the total maximum flow and by adopting a mean velocity of discharge.

If the geological data indicate that the tunnel will probably be excavated through good quality bedrock, and if the rock cover will be greater than the pressure head, the section will not need to be lined. Tunnels should be lined in the following situations:

- in sections where the rock cover is greater than 50% of the piezometric level, a shotcrete lining measuring an average of 7.5 cm thick should be used;
- in sections where the rock cover is less than 50% of the piezometric level or the geological conditions are not favorable, a structural concrete lining should be used whose thickness should be defined as a function of the diameter, service pressure and geological features.

When the diameter is greater than 5 m and there are sections of tunnel with rock cover that is less than three times the inner diameter of the tunnel, rock anchors or rock bolts should be used.

Head losses due to changes in the cross-section and bends are determined as a function of how many there are and of what kind.

SURGE TANK (ACCOUNT.12.19.33)

The purpose of surge tanks is to stabilize the pressure variations resulting from partial or total variations in the turbine flow at start-up, when there are load variations or if there is generator load rejection.

Basic Design and Recommendations

In projects with an intake tunnel, a surge tank should be built at the end section of the tunnel:

$L_{ca} \geq 6\ H_{b1}$	in plants with an installed capacity of up to 100 MW; and
$L_{ca} \geq 4\ H_{b1}$	in plants with an installed capacity of over 100 MW.

where L_{ca} is the length of the intake tunnel upstream from the surge tank and H_{b1} is the maximum gross head.

If a plant with a river diversion through a tunnel needs a surge tank, this should be positioned at the downstream end of the intake tunnel. In Inventory Studies there is no need to design complex structures for surge tanks. Simple solutions such as those shown in Figure 5.5.2.09 can be used, which will give the volumes of excavated material and concrete, which are the largest cost components. This means that surge tanks excavated in rock and lined with concrete should be prioritized.

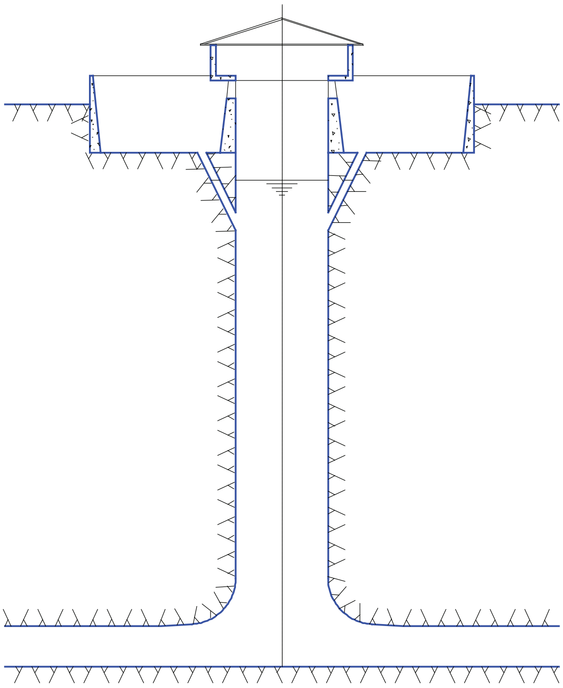


Fig. 5.5.2.09 – Surge Tank.

Pressure penstocks downstream from the surge tank can be on the surface or underground.

When the water level in the surge tank is higher than the top of the bedrock, a 1 m thick concrete wall can be built on the upper part.

The elevation of the bottom of the surge tank should be lower than the elevation of the intake sill.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning a surge tank are set out in item 5.7.6. – Intake – Surge Tank. Use spreadsheet 576Ch.xls for calculating dimensions, quantifying volumes and estimating costs.

The area of the cross-section of the surge tank and the height and oscillation of the water level in it can be calculated from the basic data obtained from the dimensions of the intake tunnel and water levels previously defined.

The Thoma formula is recommended for calculating the minimum area of a surge tank's cross-section, then adopting an area that is 25% greater to assure stability of oscillation.

The volume excavated in rock should be calculated assuming it is underground.

The freeboard of the surge tank is set at 1 m.

The thickness of the concrete lining for the section excavated in rock will depend on the diameter of the surge tank.

The whole area where concrete and rock come into contact with each other should be cleaned, and grout holes should be bored to consolidate the lined area.

PRESSURE PENSTOCKS (ACCOUNT.12.19.34)

A pressure penstock is the structure that links the intake to the powerhouse, and operates under pressure.

Pressure penstocks can be on the surface or in tunnels. The selection of the kind of penstock will depend on the local topographic and geological conditions and the costs involved.

Basic Design and Recommendations

Pressure penstocks are generally the structure that allow for greater optimization. Generally speaking, the layout should be designed in such a way that the penstocks are short.

Hydraulic valves should be incorporated into the hydraulic conveyance facilities in the following circumstances:

- when each generating unit has to be isolated individually, in cases where a single pressure penstock feeds into more than one turbine; and
- when it is advisable to avoid totally empty long intake tunnels or penstocks very often for maintenance of the generating unit.

In high head projects, which are generally over 250 m, or when the intake tunnel and/or pressure penstock is long (generally three times the maximum gross head), it may be necessary to have an emergency valve inside the powerhouse just upstream from the turbine.

When the *pressure penstock* is on the surface and the maximum turbine flow of each generating unit is high, there should be one unit per penstock.

Surface penstocks can be adapted to fit the topography, respecting any geological constraints. Although there is no impediment as to the number of vertical or horizontal deflections – points where the flow changes direction – it is worth keeping them to a minimum as they increase head loss, and anchor blocks, heavily reinforced concrete structures and rock anchors are required to make the penstock stable. As the methodology presented in this manual does not involve designing anchor blocks for changes in horizontal direction, they should be considered as having the same volume as a block designed for vertical deflection.

For *pressure tunnels*, it is normal to have more than one generating unit per tunnel, which will contain a two- or three-way bifurcation shortly upstream from the turbines. When the total maximum turbine flow is low, it is common practice to use just one tunnel until the manifold. The diameter after the bifurcation should be big enough to maintain the same velocity as before the bifurcation.

The profile of the tunnel is independent of the topography but must respect the geological conditions. However, it is normally composed of three straight sections: two horizontal end sections and one intermediate section at a slope of approximately 40º.

The cross-section of the tunnel is circular and is lined with concrete, and also has steel lining in the final section. This steel section can also sit on a concrete berth and in this case the tunnel should be 2 m larger to allow for inspections and maintenance. However, the calculations in spreadsheet 576TF.xls assume the same diameter and filling the gap with concrete.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning pressure penstocks on the surface or pressure tunnels are described in item 5.7.6 – Intake – Pressure Penstocks. Use spreadsheet 576TF.xls for calculating dimensions, quantifying volumes and estimating costs of pressure tunnels, and 576CF.xls for surface pressure penstocks.

The inner diameter of pressure penstocks or pressure tunnels along the section with steel reinforcement should be determined using the method described by Sarkaria (1979), based on the unit capacity of the turbine and the upstream and downstream water levels.

In order to respect the maximum surge pressure in the pressure penstock or tunnel and the maximum overvelocity allowed for the generating unit and the associated WD^2, it may be necessary to increase the diameter of the penstock/tunnel, in which case the diameter will be different from that suggested by Sarkaria.

It can be assumed that the maximum dynamic surge pressure resulting from the sudden shutting down of the turbine distributor can be up to 30% of the maximum gross head, and it can also be assumed that this surge pressure will be exerted at the turbine, with linear variation up to the intake or the surge tank.

The methods for determining the mean velocity of discharge, the length of each section, the surge pressure, the head losses, the thickness of the steel plate and the construction volumes are set out below.

Head losses are calculated using a simplified method. Localized losses are determined as a percentage of the velocity head and continuous head using Manning's formula.

The following criteria should be used to select the kind of valve in the range of applications common to butterfly and spherical valves (nominal diameter ≤ 3.0 m and design pressure – static head + surge pressure – between 200 and 300 meters of water head):

- cost difference between both options – a spherical valve will normally be more expensive than the equivalent butterfly valve;
- localized head loss – the head loss at a butterfly valve will normally be greater than the head loss at an equivalent spherical valve;
- safety – guarantee of watertightness.

A butterfly valve can be assumed to have the same diameter as the pressure penstock/tunnel, while a spherical valve should have the same diameter as the inlet to the turbine spiral casing.

The radius of the pressure penstock/tunnel can be taken as being four times the internal diameter of the penstock.

For *pressure penstocks on the surface*, the first and last sections should be horizontal and be long enough to fit the curve. The first section can be slightly sloping if the penstock has been designed to have five sections with different slopes.

The methodology was developed for designing penstocks/tunnels with four sections, but it can be used for ones with three or five sections.

- in penstocks/tunnels with three sections – classic, compact style, one section at a 45º angle, for example, and two short sections to fit the curves at the intake and powerhouse – use the spreadsheet and assume sections 2 and 3 as having the same slope, and set the length of section 2 at 1 m; and
- in long penstocks/tunnels with five sections, use the spreadsheet but eliminate the first horizontal section and start with the sloping section, which should slope only slightly (e.g. less than 7º).

For the purposes of calculating the construction volumes, the pressure penstock should be assumed to be limited upstream by the external face of the downstream wall of the intake.

The volumes of common excavation and excavated rock should be determined by the layout and the geological information.

Wherever there are to be concrete blocks, the foundations should be cleaned.

The average spacing between the saddle blocks should be 1.6 times the diameter of the penstock.

The methodology for determining the volumes of concrete in the saddle blocks was developed for medium-sized plants with diameters of between 4 and 8 m. For smaller diameters, this volume could be overestimated.

The relative pressure inside the whole of the penstock should be positive in order to prevent it from collapsing. This should be checked taking into acount the minimum piezometric line, which is obtained by assuming a maximum negative pressure from the minimum water level of the reservoir. This happens if the distributor is opened suddenly when the reservoir is at its minimum level.

The thickness of the steel plates should be calculated to withstand the maximum dynamic pressure.

The relationship between the diameters of the *pressure tunnel* in the parts with steel reinforcement and concrete lining is defined by assuming the same continuous head loss per meter length. The result is a 10% greater internal diameter in the part lined with concrete.

The volume of rock excavated underground is estimated to include the concrete section. There is no need to plan for or dimension drainage galleries.

The whole area of the foundations where the concrete and rock come into contact with each other should be cleaned, as well as contact grouting and consolidation.

The thickness of the concrete lining is defined as a function of the internal diameter of the tunnel, the geological conditions and the average hydrostatic head.

There should be steel lining in the sections where the rock cover is less than 70% of the static pressure plus maximum surge pressure. The method for determining the thickness of this lining is the same as that used for penstocks on the surface, while assuming that it will withstand half the maximum dynamic pressure. This conservative estimate is used in the spreadsheet.

POWERHOUSE (ACCOUNT.11.13)

There are two kinds of powerhouse used in the overall layout of the projects:

- underground (Fig. 5.5.2.10); and
- surface.

They can also be classified according to their superstructure as:

- indoor: when they have a complete superstructure with permanent cover; the heavy parts are moved by a bridge crane (Fig. 5.5.2.11);

- semi-outdoor: when the superstructure is high enough for an auxiliary bridge crane to be used, heavy parts are moved using an external gantry crane through movable covers (Fig. 5.5.2.12); and
- outdoor: when there is no superstructure; the gantry crane operates on the level of the generator floor and the equipment is protected by movable covers (Fig. 5.5.2.13).

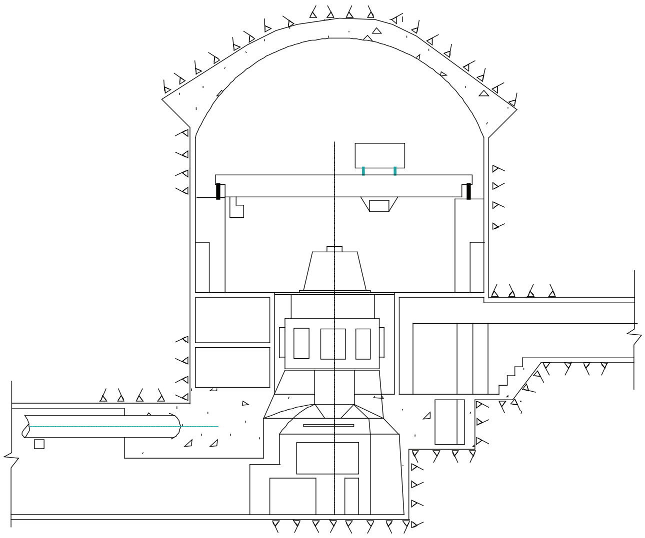


Fig. 5.5.2.10 – Underground Powerhouse – typical cross-section.

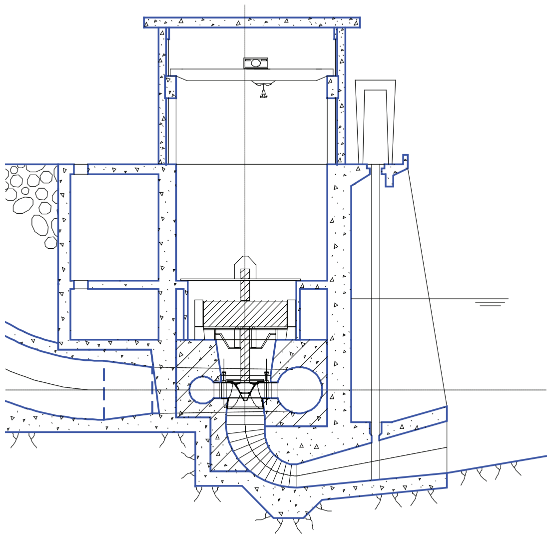


Fig. 5.5.2.11 – Indoor Powerhouse – typical cross-section.

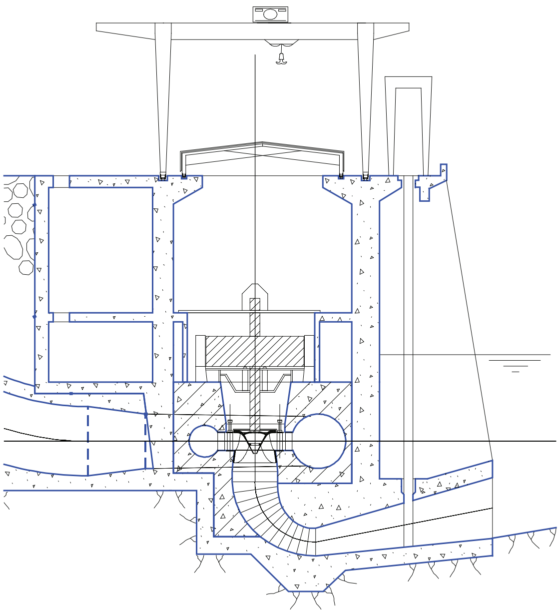


Fig. 5.5.2.12 – Semi-Outdoor Powerhouse – typical cross-section.

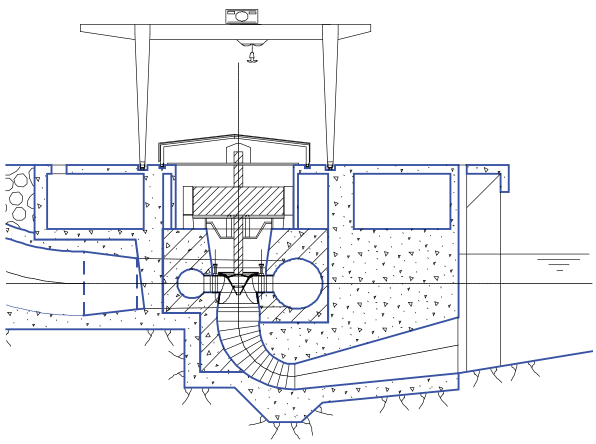


Fig. 5.5.2.13 – Outdoor Powerhouse – typical cross-section.

Basic Design and Recommendations

The kind of superstructure to be used depends basically on the layout of the generating unit, the variations of the water level in the tailrace canal, the kind of turbine to be used, the climatic conditions in the region, the costs, the accessways, the design of the outlet to the power lines, and ease of construction and maintenance.

For the purposes of Inventory Studies, the following kinds of *turbine* are considered: Bulb, Kaplan, horizontal- and vertical-axis Francis and Pelton turbines with unit capacities varying between 5 MW and the capacity limits of each type, as set out in Graph 5.7.2.01 (item 5.7.2).

The points plotted on Graph 5.7.2.01 represent the limits based on current experience using Bulb, Kaplan, horizontal- and vertical-axis Francis and Pelton turbines with unit capacities greater than 5 MW. The limiting curves for each kind of turbine define their application or indicate the upper limit to be respected when using this manual, be it technolgocial for manufacture or physical for transportation.

When the net head is such that more than one kind of turbine can be used, the decision should be based on the installation and operational factors at play and the costs and benefits associated with each option.

In the absence of more accurate data, Francis turbines should be preferred over Kaplan or Pelton turbines when they are suitable. Likewise, Bulb turbines should be chosen over Kaplan turbines whenever productivity and overall implementation costs (civil construction plus electromechanical costs) indicate the choice of a Bulb turbine.

The location of the powerhouse is normally chosen assuming that the total available head will be harnessed.

The turbines should be positioned in relation to the minimum water level in the tailrace canal in such a way that cavitation effects can be minimized without significantly raising the cost of the powerhouse. For this, the suction head should be taken, i.e. the distance between the line from the center of the distributor and the minimum water level in the tailrace canal, as shown in Figures 5.7.2.05, 5.7.2.07, 5.7.2.11, 5.7.2.14 and 5.7.2.17 (see formula for calculation of σ and h_s).

For applications that are compatible with the scope of this manual, the use of *velocity multipliers* is not covered.

The position of *service galleries* will depend on the kind of powerhouse and the overall layout. Normally, galleries are positioned upstream from the powerhouse over the pressure penstocks.

A load handling system should be designed, especially for the assmbly and maintenance of the generation equipment, using *bridge and/or gantry cranes* with a high enough capacity to move the heaviest component.

The *assembly area* should ideally be located at one end of the powerhouse, through which the equipment can be introduced. The should be a covered area within the range of the bridge or gantry crane to be used for assembling the equipment, especially the generators. For powerhouses with up to three units, there should be an equipment assembly area measuring the equivalent of 1½ blocks of a unit that is wide enough to unload the largest equipment (large-scale plants). For powerhouses with four or more units, the width should be the equivalent of 2.25 blocks of a unit.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning powerhouses are described in item 5.7.2. – Powerhouse. Use spreadsheets 572KP.xls, 572FV.xls, 572FH.xls, 572KA.xls, 572KC.xls or 572B.xls for calculating dimensions, quantifying volumes and estimating costs.

The dimensions of the powerhouse and its equipment will depend on the type, number and capacity of the turbines, the topographic and geological features, the overall layout of the project and any other pertinent information.

Only the number of generator poles presented in table 5.7.2.01 should be selected. It is advisable to consult the generator manufacturers before adopting the number of poles marked in bold in this table.

For vertical-axis Francis turbines with a maximum unit turbine flow rate greater than 20 m^3/s and for other turbine applications, if the initial velocity is lower than 300 rpm for a system at 60 Hz or lower than 250 rpm for 50 Hz, select the number of poles corresponding to the synchronous velocity that is immediately higher.

For vertical-axis Francis turbines with a maximum unit turbine flow rate of over 20 m^3/s or for Pelton turbines, if the initial velocity is 300 rpm or greater for a system at 60 Hz, or 250 rpm for 50 Hz, select the number of poles corresponding to the synchronous velocity that is immediately lower when the velocity calculated is between the immediately lower synchronous velocity and a velocity corresponding to 75% of the difference between the immediately higher and immediately lower synchronous velocities, plus the lowest synchronous velocity. From this point on, select the number of poles corresponding to the synchronous velocity that is immediately higher.

For vertical-axis Francis turbines and with a maximum unit turbine flow rate of 20 m^3/s or lower, or for horizontal-axis Francis turbines, select the number of poles corresponding to the synchronous velocity that is immediately lower than the velocity calculated.

For the purposes of quantifying services, a surface powerhouse is defined:

- upstream and downstream by its length;
- sideways by its width, including the assembly area and also an access area for heavy vehicles; and
- downwards, to the bottom of the draft tube.

When the powerhouse is underground, the volume of rock excavated is defined:

- upstream and downstream by the length of the block of the unit, including the valve house, when applicable;
- sideways by its width, including the assembly area and also an access area for heavy vehicles;
- downwards, by the bottom of the draft tube; and
- upwards, by the height needed to operate the bridge crane.

Also, for underground powerhouses, it is necessary to:

- plan a space for the valves; and
- not calculate the access tunnel in this account.

The volume of concrete for *surface powerhouses* is calculated as the sum of the volumes in the assmbly area, the powerhouse per se and the galleries for electric cables.

In the absence of more accurate information, the volumes necessary for surface powerhouses can be estimated by:

- the volume of the powerhouse per se obtained from a statistical curve (COPEL, 1981b);
- for the volume of concrete for the assembly area for powerhouses with up to three units, assume the volume of concrete to be the equivalent of half the volume of the block of a unit and for other powerhouses consider it to be double plus one quarter; and
- the volume of concrete for the service galleries is included in the volume for the powerhouse, even when they are located upstream.

The volume of concrete for underground powerhouses can be estimated as shown below, in the absence of more accurate information:

- shotcrete with an average thickness of 0.1 m for the walls and vault;
- for the volume of concrete for the assembly area for powerhouses with up to three units, assume the volume of concrete to be the equivalent of half the volume of the block of a unit and for other powerhouses consider it to be the double;

- the volume of concrete for the service galleries is included in the volume for the superstructure; and
- for the valve house, estimate it to be 10% of the volume for the infrastructure.

In powerhouses with up to 10 units, it should be possible to simultaneously shut the draft tube for up to two units using stoplogs. For the other units, it is enough to have just fixed and embedded parts. When a plant has over 10 units, it should be planned in such a way that three units can be shut down simultaneously.

There are three methods presented for calculating the costs of generators: one for generators with Bulb turbines, one for conventional horizontal-axis generators and one for vertical-axis generators. In order for the last method to encompass the whole range of generatorrs that exist, one more parameter is introduced, magnetic torque (λ), as well as one coefficient, μ. The power coefficient of the generators used for this adjustment was between 6 and 7.5, for which reason a mean value of 7.2 is adopted.

The expressions used to estimate the volumes were established for medium-sized plants with rotors measuring 2.5 to 6 m in diameter. For plants with smaller rotors, the volumes may be overestimated.

In Inventory Studies, there is no need to quantify *installation and final works* (account .11.13.00.15), as this cost can be obtained from a graph.

Likewise, there is no need to design the *auxiliary electrical equipment* (account .14) or to determine the quantity of *miscellaneous equipment* (account .15.00.00.23.31).

TAILRACE TUNNEL AND/OR CANAL (ACCOUNT 12.19.35)

Depending on the kind of layout, the flow through the turbines can be returned to the river through:

- an open chute, when there is a surface powerhouse;
- a tunnel with free surface flow, whenever there is an underground powerhouse equipped with Pelton turbines and also optionally when there is a Francis turbine; and
- a pressure tunnel, when there is an underground powerhouse equipped with Francis turbines.

Basic Design and Recommendations

When the water diversion for the power plant is short or when there is no diversion, the tailrace canal can be a simple channel with an almost flat bottom. When the canal has to be longer, it should be designed to operate with a small head loss.

The return of the waters from the powerhouse and spillway to the river should be organzed in such a way that they do not interfere with each other, so that they do not affect the operation of the generating units and cause oscillations in the water level downstream.

The design of tailrace tunnels takes into account the minimum and maximum water level at the point where the tunnel flows into the river. A surge tank may be needed at the beginning of the tailrace tunnel to buffer the pressure variations arising from the operation of the turbine.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning the tailrace canal are set out in item 5.7.6. – Intake – Tailrace Canal. Use spreadsheet 576Fu.xls for calculating dimensions, quantifying volumes and estimating costs.

The width and elevation of the bottom of the canal can be calculated as a function of the flow depth, assuming a mean velocity of 1.5 m/s.

For tailrace canals over 3 km in length, a lower velocity should be adopted, and the head loss should be calculated and taken into consideration when the canal is designed.

LAND DEVELOPMENT IN THE PLANT AREA (ACCOUNT .11.12)

This account covers the civil construction work in the plant area that is not specifically for other structures or the operators' village, including storage facilities, workshops, roads, walkways, water, sewage, power and lighting systems, landscaping and especially the accessways linking structures, including the accessway that links the entrance booth to the village. The only item that should be quantified as a function of the overall layout is the access, especially when the powerhouse is underground.

OPERATORS' VILLAGE (ACCOUNT .11.14)

In Inventory Studies, the operators' village is taken as a proportion of the workers' camp, whether it is integrated into a town or purpose built.

5.5.3 River Diversion (account .12.16)

Introduction

The river is diverted in one or more stages using cofferdams, to allow for the construction of the different structures for the project.

The diversion scheme is linked to the overall layout of the project, in that it influences its design and is dependent on it. Generally speaking, for any given site the diversion scheme will depend on the following factors, above all:

- regional topographic features;
- local geological features;
- streamflow regime of the river;
- characteristics of the definitive structures to be constructed, especially the maximum height and kind of dam; and
- assessment of the risks that are permitted at the site and downstream.

When a minimum discharge is required downstream when the river bed is being narrowed, a structure must be designed to assure residual flow.

The choice of the kind of river diversion structure will depend on the features of the dam:

- in concrete gravity dams, it is worth diverting the river through the dam itself using sluiceways; and
- embankment dams will require an auxiliary structure, such as galleries, sluiceways or tunnels.

The schemes should be conservative in nature. Ideally, the solutions should fit into one of the following typical schemes or a combination thereof:

Type	Diversion Scheme
1	River diversion through tunnels excavated in one of the abutments, completely excluding water from the construction area using cofferdams built upstream and downstream. Closure of the tunnels by means of a gate (Fig. 5.5.3.01)
2	River diversion through galleries built under the dam. Closure of the galleries by means of a gate or cofferdams, depending on the hydrological conditions (Fig. 5.5.3.02).

Type	Diversion Scheme
3	River diversion in several stages. 1st phase of diversion: partial closure of the river using a longitudinal cofferdam so the concrete structures can be built – spillway, dam and/or intake – in the dried area. 2nd phase of diversion through provisional sluiceways or passageways through the concrete structures that have been partially or completely built, while the construction of the rest of the section protected by second phase cofferdams is completed. Final closure of the concrete structure by means of a gate or other device (Fig. 5.5.3.03).
4	River diversion over the top of alternate lowered blocks of a concrete dam. Can be used for small flows and where this kind of dam is used (Fig. 5.5.3.04).

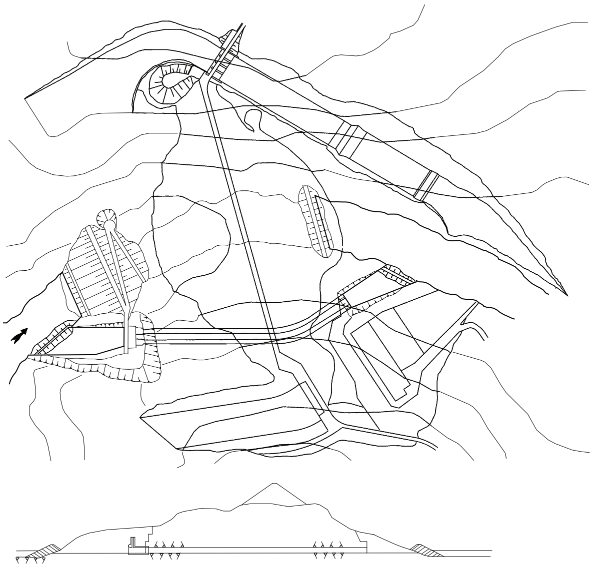


Fig. 5.5.3.01 – River diversion through tunnels in an abutment – plan and cross section.

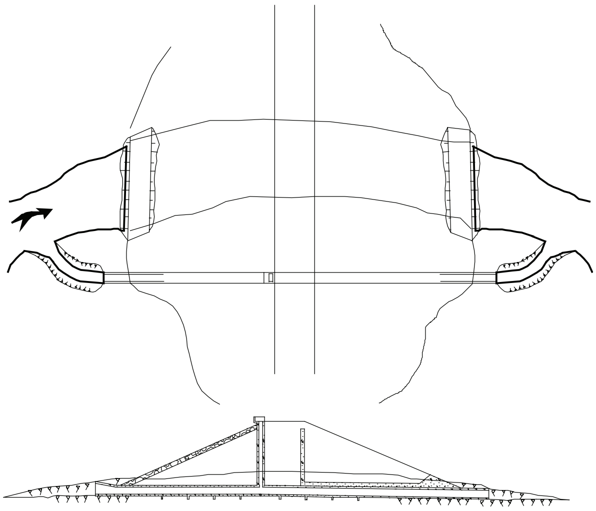


Fig. 5.5.3.02 – River diversion through a gallery under the dam – plan and cross section.

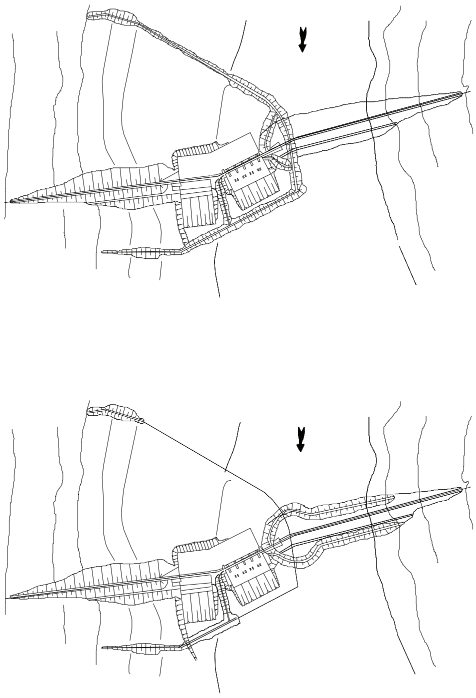


Fig. 5.5.3.03 River diversion through sluiceways.

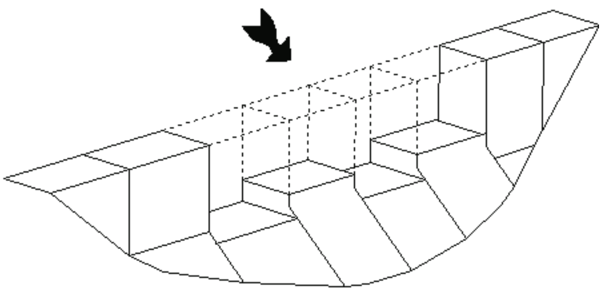


Fig. 5.5.3.04 – River diversion over the top of alternate lowered blocks in a concrete dam.

Recurrence Time

The flows used for designing the diversion construction work should be defined for each stage as a function of the risk of flooding the dried area while it is being used to divert the waters.

This risk, defined as the likelihood of flooding during the period of exposure to this risk, should be calculated on the basis of the diversion schemes:

Diversion Scheme	Risk
through tunnels or galleries in layouts with earthfill dams	3%
through tunnels or galleries in layouts with rockfill dams	5%
through sluiceways in layouts with earthfill dams:	
– first stage: by narrowing the river bed	5%
– second stage: through sluiceways	2%
through sluiceways in layouts with rockfill dams:	
– first stage: by narrowing the river bed	5%
– second stage: through sluiceways	3%
when the structures at risk are made of concrete	10%

When a diversion scheme results in the formation of provisional reservoirs upstream from the cofferdams and could create flood waves downstream if the cofferdams were to fail, putting inhabited areas or instalations and constructions of significant value in jeopardy, the risk should be regarded as being twice the percentage quoted above.

COFFERDAMS (ACCOUNT .12.16.22)

Cofferdams are provisional structures that exclude water from given areas for the structures required for the plant to be constructed.

There are many kinds of cofferdam. The most common are made of rock, earth and rock, and concrete.

The cofferdams used for diverting rivers or excluding water from a section of river are mostly earth or rockfill.

Whenever possible, earth and rockfill cofferdams should be used, even if this will increase the length of the diversion conduits. In special cases, when the cost of this soluion is too high or unrealistic, other kinds of cofferdam, such as cellular sheet piling or concrete cofferdams, can be used.

EARTH AND ROCK COFFERDAMS (ACCOUNT .12.16.22.19)

Basic Design and Recommendations

When the diversion is through tunnels or galleries, two cofferdams are built, one upstream and one downstream from the dam area, crossing the whole of the river valley, as shown in Fig. 5.5.3.01 and 5.5.3.02.

Fig. 5.5.3.05 shows a typical section of cofferdam built across the water flow. The impervious material is placed on the face that is in contact with the water.

When the diversion is through definitive concrete structures, it can be done in two steps. In the first step, the river bed is narrowed so the diversion structure – generally sluiceways – can be built. Next, the narrowed section is closed so the dam can be built, while the river flows through the diversion structure.

The typical cross-section should be different for first-stage cofferdams that are longitudinal to the flow. A design such as shown in Fig. 5.5.3.06 should be used, where the impervious material is in the core of the cofferdam to prevent its being eroded, and a layer of rip-rap is placed on the face of the cofferdam in contact with the water. It is harder to build. In the other sections and for the second-stage cofferdam, use the design shown in Fig. 5.5.3.05.

Many layouts require cofferdams to be built so that the approach or downstream channels can be excavated for the diversion, spillway, intake and powerhouse.

Details such as deflector baffles at the outlet of the diversion channel should not be considered.

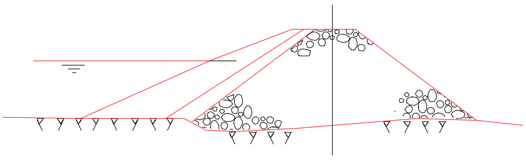


Fig. 5.5.3.05 Cofferdam across a river.

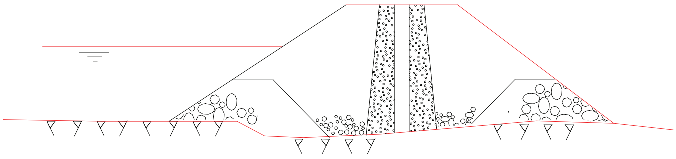


Fig. 5.5.3.06 – Cofferdam along a river.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning cofferdams are described in item 5.7.3 – River Diversion – Rock and Earth Cofferdams. Use spreadsheets 573ERT12.xls and 573ERT3.xls for calculating dimensions, quantifying volumes and estimating costs.

The dimensions of the cofferdam and the construction volumes can be obtained from the water level during the river diversion, which is defined previously when the diversion structures are designed.

For any kind of diversion, be it through conduits, galleries or tunnels, cofferdams must have a freeboard of 2 m above the maximum water level.

The part of the cofferdam incorporated into the dam should be subtracted from the total volume of the dam.

The quantities are a function of the mean height squared and the length of the cofferdam, for both kinds of cofferdam.

REMOVAL OF COFFERDAMS (ACCOUNT .12.16.22.21)

The cofferdams should be removed totally or partially to allow the other structures to be built or to allow the river to flow during the other stages of the diversion.

The calculation of the quantity to be removed can be estimated by calculating the percentage removed as a proportion of the length of the sections to be removed to the total length, or to be more precise, adopting the same methodology as employed to determine the cofferdam quantities.

When cofferdams are used for diversions through tunnels or galleries, the only structure to be removed will be the pre-cofferdams used to protect the excavation of the approach and downstream channels, diversion gallery or tailrace or headrace canals.

DEWATERING AND OTHER COSTS (ACCOUNT .12.16.22.22)

The dewatering cost will depend on the area from which water must be pumped and the length of time for which it must last. In the absence of more accurate information, a percentage of the cost of the cofferdams can be used.

DIVERSION TUNNEL (ACCOUNT .12.16.23)

Diversion tunnels are used for constructions in narrow river valleys where the geological conditions are favorable and when the dam height makes it unfeasible to construct a high ogee spillway for the sluiceways to be built in. Generally speaking, tunnels are more expensive than sluiceways.

Basic Design and Recommendations

Fig. 5.5.3.07 shows a typical structure for the inlet to a diversion tunnel.

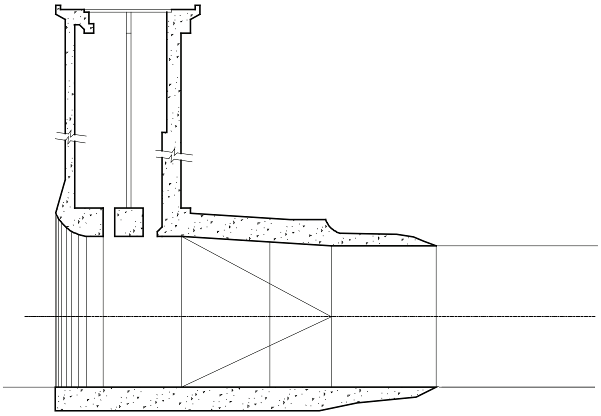


Fig. 5.5.3.07 Typical inlet to a diversion tunnel.

The overall layout of the structures will impinge on the *arrangement of the diversion tunnels* in one or both of the abutments, and the position of the cofferdam axis upstream and downstream.

The *diameter and number* of tunnels are a function of the project flow and the mean velocity of discharge permitted. Meanwhile, this will depend on whether the tunnels have a concrete lining or not. The diameter of the excavation should be at least 3.0 m and no more than 15.0 m.

The *spacing between the axes* of two parallel tunnels should be at least twice their diameter for good geological conditions.

The inlet and outlet should be positioned to assure *rock cover* for unlined tunnels of at least twice their diameter under good geological conditions.

When the *layout* is being defined, ideally the outlet of the tunnel should be under water enough to ensure that the outlet is submerged for the diversion design flood – define the elevation of the sill at the outlet so as to have at least 95% of its diameter below the water level –, and thus ensure that the inlet is also underwater, and allow for a slope of 0.5%.

However, when the tunnel is in a section of rapids or cuts through a bend in the river, the above recommendation could result in a very deep inlet structure for the tunnel. In this case, the restriction of the minimum elevation should be about one diameter below the normal water level in the region of the inlet to the approach channel.

The downstream water level is controlled by conditions that are independent of the construction. For the purposes of the diversion construction work, this level should be estimated for the design flood of the construction work in question. When a project is being constructed after the one immediately downstream from it has been built, the water level in its reservoir will determine the level in the tailrace channel. The water level in the tailrace channel should be estimated by field reconnaissance, as a function of local observations, in the same way as is done for the energy studies and hydrometeorological studies.

It is preferable not to have *bends*, but if they must be used, their radius should be over five times the tunnel diameter.

A concrete structure should be planned for the *inlet* to house the devices that will close the diversion gates and incorporate the transitions at section changes. Normally, the flow in diversion conduits is not controlled by gates, except in special cases where the diversion tunnels also have a permanent function, such as in bottom outlets, or when a minimum discharge must be maintained while the reservoir is being filled. In these cases, the control structure is generally installed in the middle of the tunnel and the inlet structure is merely a transition.

The inlet block can be estimated as having a minimum width that is twice the diameter of the tunnel. Its minimum height should correspond to the difference in level between the elevation of the crest of the upstream cofferdams and the bottom of the approach channel. Its length along the tunnel axis should be twice the diameter.

At the *outlet* it is normal to plan a concrete outlet structure with the aim of assuring the stability of the excavation work.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning diversion tunnels are described in item 5.7.3 – River Diversion – Diversion Tunnels. Use spreadsheet 573TD.xls for calculating dimensions, quantifying volumes and estimating costs.

The diameter of the tunnels and their total head loss are calculated as a function of the number of tunnels and the kind of lining, including continuous losses, losses at the inlet structure, at bends and at the tunnel outlet, as well as the construction volumes.

The water level upstream, which determines the height of the corresponding cofferdam, equals:

- the water level downstream plus the head loss if the outlet is submerged; and
- the elevation of the sill of the control structure, plus the hydrostatic head, if the discharge is controlled.

The head loss at the tunnel inlet includes the hydraulic loss when the diameter gets smaller and in the tunnel where there are any changes in the direction of the flow, and other localized losses caused by the geometry of the inlet structures. The head loss coefficient at the inlet depends fundamentally on the geometry of the structure. The continuous loss throughout the tunnel should be calculated using Manning's roughness coefficient. If the tunnel bends significantly, the corresponding losses should also be estimated.

At the outlet of the diversion structure, all the velocity head is normally is dissipated, whether the discharge outlet is submerged or free-flowing.

In special cases, when a gradual transition between the tunnel and the river is planned, part of the velocity head can be recovered, but considerations of this nature are not taken into account in this version of the manual.

Normally, tunnels used exclusively for diversion are not lined with structural concrete. When the recommended criteria for construction cannot be followed or when the quality of the rock is questionable, or whenever a tunnel's diameter is greater than 8 m, the use of rock anchors and/or rock bolts on the roof arch should be considered, as well as shotcrete.

In order to estimate the quantity of shotcrete required, consider the whole surface except the sill. The estimate of the volume of concrete for the lining is a function of the real thickness of concrete and the length of the section to be lined.

Diversion structure outlets are normally closed by stoplogs, one per opening, which are capable of closing under flow with the help of a construction crane. There should also be an emergency gate, which can be used if there is any problem during closing.

The gates should be designed for the greatest load they will experience, which is for the maximum normal water level of the reservoir.

DIVERSION CHANNEL (ACCOUNT .12.16.24)

The diversion channel is the structure normally used for narrowing the river bed using cofferdams, or is formed by excavating a canal per se in one of the abutments.

One of its purposes is to divert the flow of the river so that the main structures can be built – part of the dam, walls, spillway, powerhouse or any other concrete structure – on the river bed. This results in a shorter construction schedule and reduces costs.

Basic Design and Recommendations

The basic design of the overall layout of the construction will define how the diversion channel will be. There is a classic method of narrowing the river bed that is carried out when it is quite wide. A canal is excavated when the river is not wide enough for it to be narrowed and where the construction of a cofferdam across the river from bank to bank would be advantageous.

Generally speaking, a canal excavated in rock is a solution adopted in the Feasibility Studies when there is more detailed information available about the construction, such as the building schedule and the availability of excavated rock for the structure.

Initially, the axis of the first-stage cofferdam is decided upon, which will border the area from which water is to be excluded. In this process, it is the width rather than the length of the narrowed area that is the critical dimension. When a canal is excavated in one of the abutments, the first-stage cofferdam will cross the whole river and the width to be analyzed is that of the canal itself.

When river beds have to be narrowed and canals are excavated in soil, it is a good idea to have the hydraulic control section at the outlet, to assure a lower mean velocity of discharge in the canal and therefore reduce the risk of erosion of the cofferdam or of the side slopes of the channel. The flow control should be at the channel outlet, ensuring that this is its narrowest section, while the cofferdam should be built on a reinforced promontory that can even be made of concrete if necessary. When canals are excavated in rock this is not a concern.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning diversion channels are described in item 5.7.3 – River Diversion – Diversion Channels. Use spreadsheet 573C.xls for calculating dimensions, quantifying volumes and estimating costs.

The water level upstream from the cofferdam can be calculated from the diversion flow, the width of the chute and the elevation of the bottom of the chute in the outlet section.

The water levels at the upstream cofferdam and along the canal are calculated using simplified methods. First, the flow regime in the canal must be determined: whether it is subcritical or supercritical.

The flow regime in the canal is subcritical when the mean slope of the canal bottom is less than the critical discharge slope. Under these circumstances, the canal may or may not be submerged by the natural discharge from the river. The canal will be submerged when the energy head of the river under natural conditions is greater than the energy head inside the canal for uniform streamflow regimes.

When the canal outlet is controlled, the water level at the upstream cofferdam can be taken as equal to the critical energy head at the outlet plus the energy loss along the canal. The water level along the canal can be assumed as equal to the mean level in the canal. This mean water level can also be used to determine the head loss along the canal.

For uncontrolled discharge, the water level at the upstream cofferdam is the natural water level at the outlet plus its energy head and the head loss along the canal. The water level along the canal can be assumed to be the natural water level. The head loss along the canal is determined as a function of the mean depth of the water column in the canal.

When the discharge is supercritical, the water level at the upstream cofferdam can be assumed as equal to the critical energy head at the canal inlet. The water level along the canal can be calculated as a variable between the critical water level at the inlet and outlet.

For the procedures proposed, the width of the diversion channel and the river are the average dimensions and not the free surface area.

The head loss along the canal is determined by Manning's formula. A mean value should be taken for Manning's roughness coefficient, taking into account the river or canal banks and bed, giving precedence to the features of the river bed.

The cost of diversion channels can rise significantly if the hillsides need to be protected against erosion in the section in question. Different mean velocity limits should be considered for the different terrains and linings.

In the calculation procedures adopted, a rectancular cross-section is assumed with a horizontal bottom.

Water column profile along the canal

In exceptional cases, it may be necessary to determine the water column profile along the canal with greater accuracy.

The following are recommended:

- for uncontrolled subcritical channel flows, determine the depth at constant channel flow for the same energy head of the natural streamflow and vary it with the energy gradient;
- for subcritical channel flows that are controlled at the outlet, use the Direct Step Method (Chow, 1959) presented below; and
- for supercritical channel flows, determine iteratively the depth under a uniform streamflow regime that results in the energy gradient being equal to the mean slope of the river bottom.

In the Direct Step Method, the depth is fixed and the position of the section is determined, rather than defining the position of the section then determining the depth. This methodology also employs some simplifications:

- trapezoidal cross-section (can be retangular);
- horizontal bottom along any given section;
- unvarying width along the canal;
- unvarying slope of the canal bottom;
- unvarying inclination of the sides along the canal and the inclination for both sides (a general average value should be adopted); and
- one Manning's roughness coefficient for the whole of the canal and the same for both banks and the bottom.

The water column profile along the canal is obtained in two parts. In the first, the critical discharge characteristics of the first section of calculation are determined for the canal outlet: the depth of the water column (by iterations, without requiring great precision), the specific energy head and the water level. In the second part, the approximate water levels are determined successively from one section to another, as follows:

- set the depth of each section as slightly greater than it was for the previous section;
- determine the specific energy and the energy gradient in this section;
- determine the mean energy gradient;
- the distance between these sections is obtained by the ratio between the difference in specific energy between the sections and the difference of the slope of the river bottom to the mean energy gradient;
- determine the elevation of the river bottom for this section; and
- the water level in this section is determined.

The mean velocity limit should be respected. If the limit is exceeded, the mean velocity in the canal must be reduced by increasing the narrowed area. The velocity restriction in the narrowed section can be overcome by protecting the surface with larger blocks of rock or by lining with concrete.

DIVERSION GALLERY (ACCOUNT .12.16.24)

A diversion gallery is a concrete conduit that generally has a rectancular cross-section.

Concrete galleries are recommended for low flows through the river diversion and when low-cost structures are planned. They are built under embankment dams and are used in layouts with abutment spillways. They are not dependent on the geological conditions.

Basic Design and Recommendations

Fig. 5.5.3.08 shows the longitudinal section of a diversion gallery under a dam.

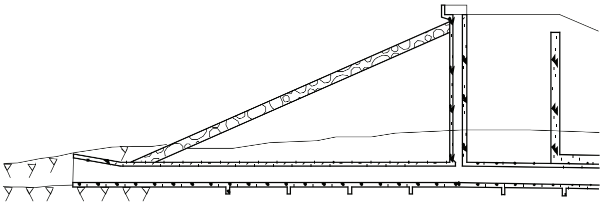


Fig. 5.5.3.08 Diversion Gallery.

Diversion through concrete galleries should be avoided in major projects whenever tunnels can be used. This is because galleries are almost always a vulnerable point in a dam, which means the studies for their design are complex, as is the process of eliminating uncertainty from the corresponding cost estimates.

Galleries should ideally be seated on solid foundations. If they are not, they should be constructed in segments linked by dilatation joints, which will allow the structure to adjust to the differential settlements.

When the gallery design shows the need for more than one unit, they should ideally be positioned side by side, resulting in a single block with individual gates placed in structures built specially for this purpose.

The following criteria should also be used when designing a diversion gallery:

- the vertical and horizontal profile must be straight;
- the gallery should be constructed perpendicular to the dam axis and near the bottom of the valley, where the best foundation conditions are predicted to be; and
- the construction should be in a trench, and should be distanced as far as possible from the river channel in order to minimize the cost of water control during its construction.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning diversion galleries are described in item 5.7.3 – River Diversion – Diversion Galleries. Use spreadsheet 573GA.xls for calculating dimensions, quantifying volumes and estimating costs.

The dimensioning procedure consists of determining the number of gallery passages and the dimensions necessary for the design flood at the desired hydrostatic head, represented by coefficient k_Q.

First, the number of passages is defined by an algorithm (COPEL, 1996) as a function of the maximum dimensions (a maximum width of 3.3 m is set) and the upper velocity of discharge limit, then the dimensions of the gallery's cross-section are obtained, taking into account the flow through the gallery. The initial recommended value for k_Q is 3.8. Higher values will result in galleries with smaller dimensions and higher upstream water levels. When the upstream water level needs to be altered, the calculation can be done iteratively by changing the value of k_Q.

The upstream water level, which determines the height of the corresponding cofferdam, equals:

- the downstream water level plus the head loss if the gallery outlet is submerged; or
- the elevation of the sill of the gallery plus the hydrostatic head for a free flowing outlet.

The head loss at the inlet, including the loss at the embedded parts, can be assumed to be 20% of the velocity head. The continuous head loss can be determined using Manning's formula.

When the outlet is free flowing, the hydrostatic charge upstream from the sluiceways can be estimated as a function of the flow and the dimensions of the sluiceways, provided there are no gates.

The hydraulic design should take the following criteria into account:

- for construction reasons, the cross-section should be rectangular, with the height of the rectangle being 1.0 to 1.5 times its width (a ratio of 1:2 has been set);
- minimum cross-section of 1.5 x 1.9 m;
- the coefficient k_Q is set at a value that allows the hydrostatic head upstream from the structure to be almost twice the height of the opening; and
- a mean velocity is limited to 15 m/s.

Normally, the flow through galleries is not controlled, except in the case of the final closing of the river in order to maintain a minimum flow while the reservoir is being filled. The hydraulic control is usually placed a little upstream from the crest. One flow control option is to use a bottom outlet.

The excavation volumes are estimated as a function of the thickness of the layer of soil and the dimensions of the galleries (item 5.7.3.).

The estimate of the total volume of concrete is a function of the thickness and dimensions of the galleries, obtained from item 5.7.3.

DIVERSION SLUICEWAY (ACCOUNT .12.16.23)

Diversion sluiceways are openings in the form of rectangular conduits left in some concrete structures to allow the river to flow through while it is being diverted.

Basic Design and Recommendations

Sluiceways are generally built into the body of high ogee spillways or concrete gravity dams. Diversion through sluiceways in the spillway is recommended for low or medium-height dams.

Figures 5.5.3.09 and 5.5.3.10 show the longitudinal section of a diversion sluiceway in a high ogee spillway and a sluiceway in a dam, respectively.

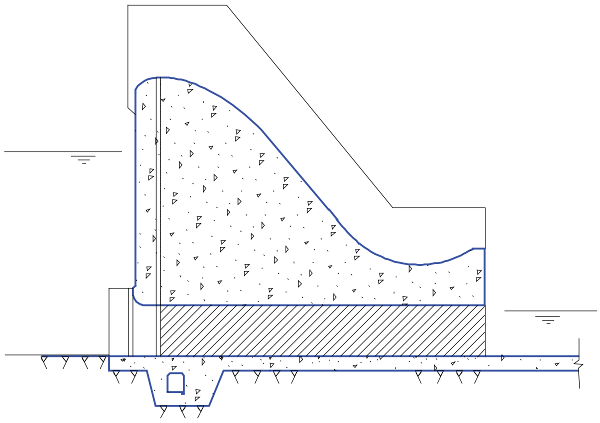


Fig. 5.5.3.09 Longitudinal section of a diversion sluiceway in a high ogee spillway.

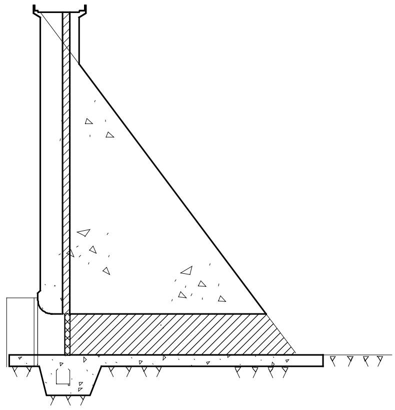


Fig. 5.5.3.10 Longitudinal section of a sluiceway in a concrete gravity dam.

Sluiceways can be positioned in structures on the river bed or in abutments.

For structures on the river bed, the *number of openings* can be increased to allow for lower cofferdams. When the structures are in abutments, it is important to be aware that sluiceways will normally imply in increased concrete and excavation volumes. It is therefore acceptable to install fewer sluiceways with larger dimensions.

The *elevation of the sill* of sluiceways in structures on river beds is generally defined by the elevation of the river bed. When the structures are in abutments, the elevation of the sill of the sluiceway should be defined by taking into account the hydraulic issues involved in closing off the river and the cost, among other aspects. When the sill is much higher than the water level, it could make closing off the river for the diversion more difficult, depending on the characteristics of the river, and make it necessary to raise the cofferdam elevation. Meanwhile, a very low sill will require extra excavation volumes and more concrete. The elevation of the sill of the sluiceway should be as high as possible, bearing in mind both constraints mentioned above. It is recommended that it be lower than the water level of the river at the beginning of the approach channel under normal conditions to make it easier to close.

The structure's design will depend on the channel flow inside the sluiceway, which is mainly defined by its position in relation to the water level in the downstream channel.

When the sluiceway is higher than the water level, it will usually operate as an opening in a thick wall with discharge in a free flow. In other cases, especially when there is a great variation in the water level of the river, the sluiceway outlet will be submerged.

Criteria and procedures for dimensioning and quantification

When diversion sluiceways are built into gated surface spillways, their width is restricted by the width of the gates. Meanwhile, for ungated spillways and concrete dams, the dimensions and number of openings can be chosen freely.

The *upstream water level*, which determines the height of the corresponding cofferdam, equals:

- the downstream water level plus the head loss if the sluiceway outlet is submerged; or
- the elevation of the gallery sill plus the hydrostatic head for free-flowing outlets.

The head loss at the inlet, including the loss at fixed parts, can be assumed to be 20% of the velocity head. The continuous head loss can be determined using Manning's formula.

The hydrostatic head upstream from the sluiceways, for free-flow outlets, can be estimated from the abacus as a function of the sluiceways' dimensions, provided it is free flowing.

The mean velocity of discharge is limited to 15 m/s. In order to respect this velocity limitation, the number of sluiceways must be increased or coefficient k_Q must be decreased.

All *the civil construction quantities* relating to the sluiceways should be allocated to the spillway or dam.

The *volume of concrete* corresponds to the added volume of the walls upstream from the dam or the ogee, and the reduced volume at the sluiceway inlets. When calculating the volume of concrete, it can be assumed that the quantities of cement and steel reinforcement will be greater than those for the ogee crest of the spillway. These volumes corespond to the sill, the walls and a slab for the roof.

Normally, there is no *flow control* through sluiceways, with the exception of the control corresponding to the final closing of the river in order to maintain a minimum discharge while the reservoir is being filled. The flow control structure is usually the inlet transition. One flow control option is to have a bottom outlet.

Below, the different criteria and procedures for two different sluiceway designs are described:

- in gated surface spillways;
- in concrete dams and ungated surface spillways.

DIVERSION SLUICEWAYS THROUGH GATED SURFACE SPILLWAYS

The procedures for dimensioning diversion sluiceways through gated surface spillways are described in item 5.7.3. – River Diversion – Diversion Sluiceways. Use spreadsheets 575COBD.xls and 575COSE.xls for calculating dimensions, quantifying volumes and estimating costs.

The dimensioning methodology consists of determining the necessary height for the design flood at the desired hydrostatic head, represented by coefficient k_Q, once the number and width of passages has been decided on.

The design proposed in the manual does not impose building diversion sluiceways throughout the length of the spillway. The number of sluiceways should be around three quarters the number of gates. This is a particularly attractive option when not all the spillway structure is inside the river channel.

The ratio of the height to the width of the sluiceways should not exceed 3:1.

The thickness of the walls between sluiceways is the same thickness as the walls for the spillway, and the width of the sluiceways should equal half the size of the remaining gap.

The height of a sluiceway is given by an algorithm (COPEL, 1996), which takes into account the flow through the sluiceway. Initially, k_Q can be taken as 3.2. Higher values will result in lower sluiceways and higher upstream water levels. When the upstram water level needs to be altered, the calculation can be done iteratively by modifying k_Q.

The number of sluiceways or coefficient k_Q can be increased in order to keep the height of the sluiceway within the limit imposed.

The total width of the sluiceways for the purposes of quantiyfing the volume will be the sum of the width of the openings and walls, including the ends.

DIVERSION SLUICEWAYS THROUGH CONCRETE DAMS OR UNGATED SURFACE SPILLWAYS

The procedures for dimensioning diversion sluiceways through concrete dams or ungated surface spillways are described in item 5.7.3. – River Diversion – Diversion Sluiceways. Use spreadsheets

574CCGAD.xls, 574CCRAD.xls, 57COBD.xls and 575COSE.xls for calculating dimensions, quantifying volumes and estimating costs.

The ratio between the height and width of the sluiceways in dams or ungated spillways should be 2:5.

The methodology consists of defining the number of passages and the dimensions required for the design flood at a desired hydrostatic head, represented by coefficient k_Q, as was the case for the galleries.

First, the number of passages is defined by an algorithm (COPEL, 1996) as a function of its maximum dimensions and the upper dischage velocity, then the dimensions of the cross-section are obtained for a sluiceway in a concrete dam or ungated spillway, taking into account the flow through the sluiceway. The initial recommended value for k_Q is 3.2. Higher values will result in sluiceways with smaller dimensions and higher upstream water levels. When the upstream water level needs to be altered, the calculation can be done iteratively by changing the value of k_Q

5.5.4 Dams and Dikes

Selecting a dam

The only kinds of dams considered in this manual are earthfill, rockfill or conventional or roller compacted concrete gravity dams. Other kinds of dams, such as arch, buttress or double curvature arch dams, should only be used under exceptional circumstances, since their use depends on more accurate geological information that is not normally available at this stage of the studies and also because they are associated with a level of optimization that is more appropriate during the Feasibility Studies.

The choice of the kind of dam will primarily depend on the existence of material suitable for its construction, the geological and geotechnical conditions, and the topography at the dam site. Other equally important factors are:

- the availability of soil or rock from excavations of the right quality and quantity and at a pace that is compatible with the construction of the proposed layout;
- nature of the foundations: rockfill and concrete dams should only be built on rock foundations, while earthfill dams can also be built on earth; and
- climatic conditions: the existence of relatively long rainy seasons will make the construction of compacted earthfills or clay cores overly expensive because they will affect the progress of the construction work.

A site can be deemed appropriate for constructing a homogeneous *earthfill dam* when the field reconnaissance indicates that the rock is at a great depth at the area under study. This kind of dam requires a more inclined slope for the upstream and downstream faces, which results in greater volumes. This is why it is used for small and medium-height dams.

Sites can be deemed appropriate for building a *rockfill dam* with a clay core or with a concrete face if the field reconnaissance indicates that there is sound bedrock of good quality at a shallow depth along the dam axis. This kind of dam does not need special foundation conditions. Large volumes of excavated rock for the powerhouse, canals and spillways are a good indicator of the potential for this kind of dam. If there are rainy seasons or very high levels of humidity that will make it hard to build the clay cores or if there is any difficulty getting suitable material for the core, the best choice is to build a concrete face.

A site can be deemed appropriate for building a *concrete dam* when the field reconnaissance indates that there is sound bedrock with low compressibility at a shallow depth along the dam axis, as this kind of dam exerts greater pressure on foundations. Stability is primarily provided by the forces of gravity. Generally speaking, only solid concrete gravity dams should be considered.

Among the potential choices of dams, the different types of construction should be analyzed to identify the most economical solution.

Basic Design and Recommendations

The choice of the axis position will depend on the kind of dam and the location of the other structures. Generally speaking, narrower sections will be chosen, especially in the deepest part of the river valley. Once the choice is made, field reconnaissance should be undertaken to visually confirm the downstream water level and make an expeditious investigation of the foundation conditions.

Typical, standard cross-sections should be used, as shown in Fig. 5.5.4.01, 5.5.4.02, 5.5.4.03, 5.5.4.04, 5.5.4.05 and 5.5.4.06, since generally speaking the extent of knowledge about the foundations and building materials available will not allow the cross-section to be optimized.

Generally speaking, the excavations required for the structures should be balanced against the need for rockfill and earthfill. However, as this balance depends on the real construction work schedule, there could be the need to stock material temporarily or to use additional deposits. This will increase the cost and distort the original estimates. It is therefore worthwhile developing a flexible layout and factoring in a loss of around 20% for the use of the material from the excavations required, depending on the size of the construction work, for losses and the use of rock excavated for internal accessways. Additionally, space for stockpiling the excavated material should be factored in, which may vary from 25% to 30% for rock and around 15% for earth.

Foundation treatment is particularly important when assessing the cost of embankment dams, despite the well-known difficulty of characterizing them in Inventory Studies. For this reason, for the purposes of estimating quantities, the criteria set out below can be used as guidelines, with the recommendation that a conservative penalty should be applied to any cases where the costs determined from their specific application are considered insufficient. These criteria are presented in simplified terms for the cost estimate and do not necessarily represent the recommended solutions for specific cases.

It is recommended that for the purposes of calculating civil construction services, the sections should be positioned along the longitudinal axis of the dam where there are major changes in cross-section, such as at the bottom of saddles, the top of hills or the banks of the river, and must also be at any points of interruption, such as intakes or spillways. It should be remembered that the elevation of the foundation should represent the mean value per section.

EARTHFILL DAMS (ACCOUNT .12.17.25)

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning earthfill dams are described in item 5.7.4. – Dams and Dikes – Earthfill Dams. Use spreadsheet 574T.xls for calculating dimensions, quantifying volumes and estimating costs, and for the typical cross-section presented in Fig. 5.5.4.01.

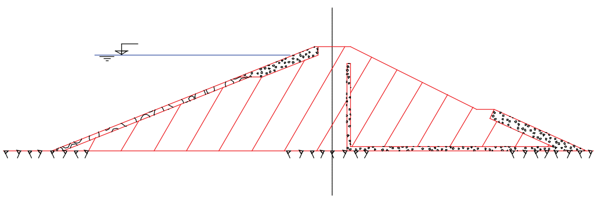


Fig. 5.5.4.01 – Typical cross-section of an earthfill dam.

The calculation procedure adopted for determining the construction quantities is that of finite differences. It consists of determining quantities per section, between two cross-sections of the dam axis, and the final sum.

For each section, the average quantity per meter of dam at the end sections is determined as a function of the height of the dam in that section, weighted by the length of the section.

The number of sections or the distance between them can vary greatly depending on the size of the dam. It is recommended that between 15 and 40 sections be defined, with an average distance of 20 m to 100 m between each section.

The height of the dam is defined as the distance between the crest and the foundation, which corresponds to the mean elevation of the land minus the excavation of topsoil.

The freeboard is basically defined as a function of the risk of overtopping and the damage this would cause. Overtopping could be caused by the incorrect operation of the spillway, or by wind-induced waves. More accurate criteria should be adopted in the Feasibility Studies. The value adopted for the freeboard is 4.0 m.

The slopes of the upstream and downstream faces of the dam are a function of the kind of building material available for the dam and its maximum height. However, in this manual the values are set at 3.0H:1V upstream and 2.5H:1V downstream.

The width of the dam crest is set at 10 m.

The average thickness of the layer of soil to be removed in the dam area should be defined by inspecting the area around the abutments but not necessarily along the entire length of the dam. The average thickness to be removed from the river bed in particular may be different from that in the abutments, and may often be zero.

A 1 m layer of topsoil can be removed. The volume of common excavation will include a 10 m strip beyond the offset of the dam.

The volume of any cofferdams incorporated into the dam should be subtracted from the total calculation of volumes.

The erosion protection of the upstream face is provided by a 1.5 m thick layer of riprap in the area corresponding to the drawdown going to a maximum of 4.0 m below the minimum level in the reservoir. The erosion protection of the downstream face can be provided by grass along the entire face, with the exception of the area at the foot of the slope.

With the exception of the removal of topsoil, foundation cleaning is only required at the base of the cut-off when it reaches the top of the sound bedrock.

If the material on the rock is impermeable, such as compact clay or fine silt, there is practically no need to treat the foundation beyond cleaning its surface.

If the material on the bedrock is permeable, a cutoff trench with a trapezoidal cross-section should be provided for, which should be excavated to the level of the impermeable foundation and filled with compacted clay to a maximum depth of 15 m. The inclination of the sides of the cutoff trench should be 1H:1V and for construction reasons the minimum width at its base should be 6 m.

When the top of the bedrock is reached, the foundation treatment includes removing loose material, excavating 1.5 m and applying a layer of leveling concrete measuring an average of 0.5 m at the base of the cutoff.

If the thickness of the permeable layer is greater than 15 m, an impermeable blanket should be positioned upstream and relief drainage wells should be built downstream.

The upstream impermeable blanket should reach up to a distance ten times the height of the water column in the reservoir and its thickness should be 10% of this value. The whole area under the impermeable blanket should be cleaned in the same way as the surface under the main earthfill. The depth of the relief wells should be the same as the height of the dam in the corresponding section and they should be spaced 10 m apart.

A vertical filter that is 2 m wide and 1.5 m thick should be used.

Other costs are estimated as a function of the length of the crest of the dam.

ROCKFILL DAMS WITH A CENTRAL OR INCLINED CLAY CORE (ACCOUNT .12.17.25)

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning rockfill dams with a central or inclined clay core are described in items 5.7.4 – Dames and Dikes – Rockfill Dam with Central Clay Core and 5.7.4 – Dams and Dikes – Rockfill Dam with Inclined Clay Core. Use spreadsheets 574ENAV.xls and 574ENAI.xls, respectively, for calculating dimensions, quantifying volumes and estimating costs.

A typical cross-section of a dam with a central core is shown in Fig. 5.5.4.02. The cross-section of a dam with an inclined core is shown in Fig. 5.5.4.03.

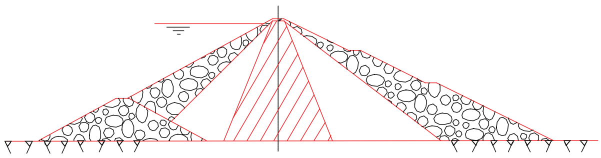


Fig. 5.5.4.02 – Typical cross-section of a rockfill dam with a central clay core.

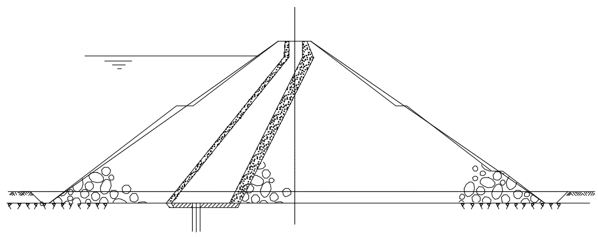


Fig. 5.5.4.03 – Typical cross-section of a rockfill dam with an inclined clay core (Salto Osório hydropower plant).

The calculation procedure adopted to determine the civil construction quantities is identical to that described previously for earthfill dams, with the exception of the average distance between sections, which is recommended to be between 20 m and 80 m.

The height of the dam is defined as the distance between the crest and the foundation, corresponding to the elevation of the land minus the topsoil excavated.

The freeboard is basically defined as a function of the risk of overtopping and the damage arising from this. Overtopping could be caused by the incorrect operation of the spillway, or by wind-indued waves. More accurate criteria should be adopted in the Feasibility Studies. The table below shows some suggested values for the freeboard.

H_{bl} (m)	for
3.0	dam with a maximum height of less than 20 m and a reservoir of less than 50 km^2
4.0	all other cases

The average inclination of the upstream and downstream faces (m) is defined as a function of the building material available for the dam and its maximum height, and will vary according to the table below.

m	for
1.3	low dam in area with favorable geological conditions, with no intermediate berms
1.7	very high dam in area with poor geological conditions, and with intermediate berms

The width of the crest of the dam is set at 10 m.

The average thickness of the layer of soil to be removed in the dam area should be defined by inspecting the area around the abutments but not necessarily along the entire length of the dam. The average thickness to be removed from the river bed in particular may be different from that in the abutments, and may often be zero.

In order to calculate volumes, subtract the volume of the cofferdams incorporated into the dam.

The volume of common excavation includes a 10 m section beyond the offset of the dam.

The use of a 4-meter-wide transition layer and a 2-meter vertical filter is assumed.

The whole area where the clay core comes into contact with the foundation should be cleaned.

If the field reconnaissance activities identify any sign that the bedrock is greatly altered, the foundation treatment should include not only cleaning until satisfactory material is reached, but also the construction of a trapezoidal cross-section cutoff excavated down to the level of the sound bedrock and filled with compacted clay. The inclination of the sides of the cutoff should be 1H:1V, the minimum width at the base of the cut-off trench should be 6 m and the maximum depth should be 15 m. A grout curtain should also be used.

The foundation treatment includes removing loose material, excavating 1.5 m deep into the rock and applying a layer of leveling concrete (dental concrete) measuring an average of 0.5 m thick over the area of the clay core.

Other costs will be estimated as a function of the length of the crest of the dam.

CONCRETE FACED ROCKFILL DAMS (ACCOUNT .12.17.25)

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning concrete faced rockfill dams are described in item 5.7.4. – Dams and Dikes – Concrete Faced Rockfill Dams. Use spreadsheet 574EFC.xls for calculating dimensions, quantifying volumes and estimating costs.

A typical cross-section of a concrete faced rockfill dam is shown in Fig. 5.5.4.04.

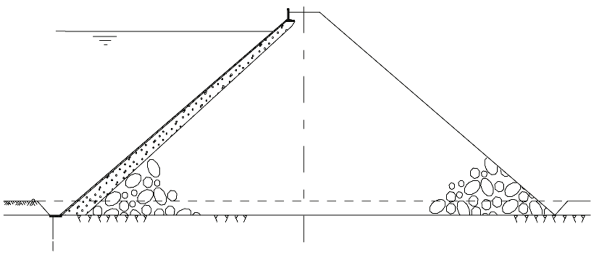


Fig. 5.5.4.04 – Typical cross-section of a concrete faced rockfill dam (Foz do Areia hydropower plant – Gov. Bento Munhoz da Rocha).

The calculation procedure adopted for determining the construction quantities is identical to the procedure described for earthfill dams, except that the average distance between the sections is recommended to be between 20 m and 80 m.

The height of the dam is defined as the distance between the crest and the foundation on sound bedrock, which corresponds to the elevation of the land minus the excavation height.

The freeboard is basically defined as a function of the risk of overtopping and the damage arising from this. Overtopping could be caused by the incorrect operation of the spillway, or by wind-induced waves. For this kind of dam, as with embankment dams with a clay core, the freeboard is only counted up to the crest, so the guardrail is not taken into consideration. More accurate criteria should be adopted in the Feasibility Studies. The table below shows some suggested values for the freeboard.

H_{bl} (m)	for
3.0	dam with a maximum height of less than 20 m and a reservoir of less than 50 km^2
4.0	all other cases

The average inclination of the upstream and downstream faces (m) is defined as a function of the building material available for the dam and its maximum height, and will vary according to the table below.

m	for
1.3	low dam in area with favorable geological conditions
1.5	very high dam in area with poor geological conditions

The width of the crest of the dam is set at 10 m.

The average thickness of the layer of soil to be removed in the dam area should be defined by inspecting the area around the abutments but not necessarily along the entire length of the dam. The average thickness to be removed from the river bed in particular may be different from that in the abutments, and may often be zero.

For the purposes of volume calculations, the volume of the downstream cofferdam should be subtracted when it is incorproated into the dam.

The volume of common excavation includes a 10 m section beyond the offset of the dam.

The purpose of the layer of crushed rock is to provide a bed for the concrete slab to sit on the rockfill.

In this manual, a plinth with average dimensions is used which is valid for any dam height.

The thickness of the concrete slab must increase at a rate of 0.5 m per 140 m dam height.

The whole area of the dam foundations must be cleaned, including the plinth.

The foundation treatment includes rock anchors and a grout curtain along the plinth, as well as a layer of leveling concrete under the plinth, which is included in the concrete calculation.

Other costs are estimated as a function of the length of the dam crest.

CONCRETE GRAVITY DAMS (ACCOUNT .12.17.26)

Concrete dams are also used to complement earthfill and rockfill dams to close off river valleys between concrete structures or between concrete structures and an abutment.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning conventional or roller compacted concrete dams are described in 5.7.4 – Dams and Dikes – Roller Compacted Concrete Dams and 5.7.4 – Dams and Dikes – Conventional Concrete Dams and Concrete Gravity Dams. Use spreadsheets 574CCG.xls and 574CCGAD.xls (without and with sluiceways, respectively) for calculating dimensions, quantifying volumes and estimating costs of conventional concrete dams, and spreadsheets 574CCR.xls and 574CCRAD.xls (without and with sluiceways, respectively) for roller compacted concrete dams.

If the river diversion is through sluiceways in the dam, the estimate of civil construction quantities should take into account any additions due to the sluiceways, as set out in item 5.7.3.

A typical cross-section of a conventional concrete dam is shown in Fig. 5.5.4.05. For roller comparcted concrete dams, the most typical cross-section can be seen in Fig. 5.5.4.06.

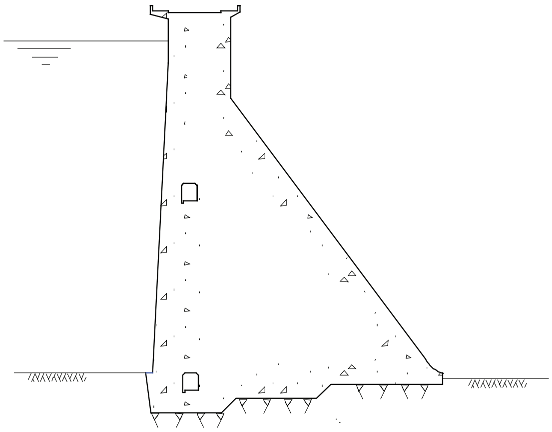


Fig. 5.5.4.05 – Typical cross-section of a conventional concrete dam.

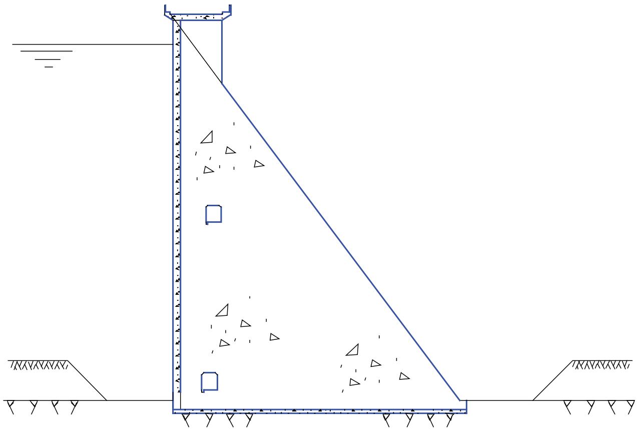


Fig. 5.5.4.06 – Typical cross-section of a roller compacted concrete dam.

The calculation procedure adopted for determining civil construction quantities is that of finite differences.

As in the previous cases, for each section, the averages of the quantities per meter of dam in the end sections are determined as a function of the height of the dam in that section, and weighted by the length of the section.

The quantity of sections or the distance between them may vary greatly depending on the size of the dam. It is recommended that between 15 and 40 sections be defined with an average distance of between 15 m and 60 m between the sections.

The height of the dam is defined as the distance between the crest and the foundations. The level of the foundations is the result of removing the layer of soil and excavating to 1.5 m below the top of the bedrock.

As the damage that could be caused by overtopping a concrete dam is less than that of an earthfill or rockfill dam, greater risks are normally taken and lower values can be adopted for the freeboard. The table below presents some suggested values.

H_{bl} (m)	for
2.0	dam with a maximum height of less than 20 m and a reservoir of less than 50 km^2
3.0	other cases

The average inclination of the downstream face (m) is defined as a function of the foundations and will vary according to the table below

m	for
0.75	dam in area with favorable geological conditions
0.8	dam in area with normal geological conditions

The thickness at the crest of the dam is 8 m, and the width of the crest is 10 m.

The average thickness of the layer of soil to be removed in the dam area should be defined by inspecting the area around the abutments but not necessarily along the entire length of the dam. The average thickness to be removed from the river bed in particular may be different from that in the abutments, and may often be zero.

The volume of excavated soil will include a 10 m stretch beyond the offset of the dam.

The whole area where the dam comes into contact with the foundations should be completely cleaned.

The foundation treatment includes removing loose material, excavating 1.5 m deep into the rock and installing a drainage curtain near the upstream face immediately downstream from a grout curtain.

The foundation treatment for roller compacted concrete dams includes applying a 0.5 m layer of leveling concrete (dental concrete) to the whole foundation.

Also for roller compacted concrete dams, a 1.0 m layer of conventional concrete is applied to the crest, a 1.2 m wide section on the upstream face and 2.5 m^2 in the area of the conventional concrete parapet.

Other costs are estimated as a function of the length of the crest of the dam.

TRANSITIONS AND CONCRETE WALLS (ACCOUNT .12.17.27)

These are concrete structures used to link or provide a transition between earthfill structures – embankment dams and dikes – and concrete structures – spillways, intake or concrete dams.

This Manual recommends the basic wall types presented in Figures 5.5.4.07 and 5.5.4.08.

RETAINING WALLS

Basic Design and Recommendations

Retaining walls are structures built perpendicular to the dam axis upon which the dam is supported. The dam should have a graviy cross-section as shown in Fig. 5.5.4.07.

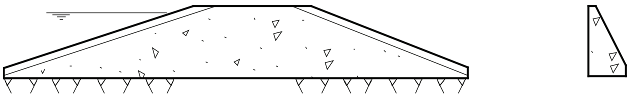


Fig. 5.5.4.07 – Typical cross-section of a retaining wall

Retaining walls are generally recommended if the height of the transition section is less than 30 m.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning retaining walls are described in item 5.7.4. – Dams and Dikes – Transitions and Concrete Walls. Use spreadsheet 574m.xls for calculating dimensions, quantifying volumes and estimating costs.

The height of the dam is defined as the distance between the crest and the top of the sound bedrock.

The freeboard is defined for the dam.

The mean inclinations of the upstream and downstream faces are the same as those defined for a concrete dam.

The volume of common excavation is already included in the dam calculations, except for earthfill dams.

In any type of transition, the foundations must be made of rock. The complete cleaning of the foundation is required, as explained previously for the dams, except for earthfill dams. No foundation treatment is required.

The volume of concrete for the wall of the spillway and for concrete dams is subtracted from the calculation of concrete required.

TRANSITION WALLS

Basic Design and Recommendations

Transition walls are structures with a gravity profile, as shown in Fig. 5.5.4.08, whose axis should be the same as that of the dam. The earthfill is built around the wall. The length of the crest of the transition should be such that it penetrates 10 meters into the earthfill at the elevation of the crest.

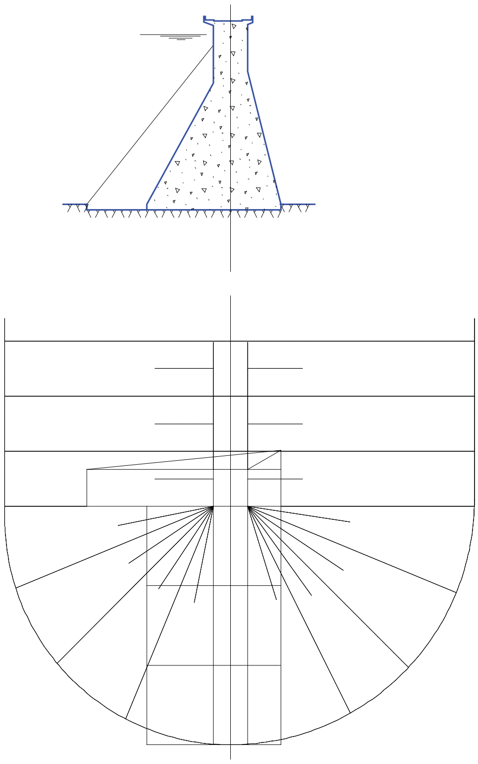


Fig. 5.5.4.08 – Typical cross-section of a transition wall

Transition walls are normally recommended if the height of the transition section is 30 meters or more. However, their use is recommended for all transitions to intakes, no matter how high the section.

The transition wall recommended in this manual is specially dimensioned for rockfill dams with an inclined clay core, but can be used for other types of dam.

When the freeboards of the structures to be connected is different, there should be a ramp along the crest of the wall to adjoin them.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning transition walls are described in item 5.7.4. – Dams and Dikes – Transitions and Concrete Walls. Use spreadsheet 574m.xls for calculating dimensions, quantifying volumes and estimating costs.

The height of the dam is defined as the distance between the crest and the top of the sound bedrock.

When it is connecting to a concrete dam, the clay core can often seal against the end face of the transition wall.

The volume of common excavation is already included in the dam calculations, except for earthfill dams

In any type of transition, the foundations must be made of rock. The complete cleaning of the foundation is required, as explained previously for the dams, except for earthfill dams. The same kind of foundation treatment should be used as is recommended for concrete dams.

5.5.5 Spillways (account .12.18)

Spillways are designed to discharge floodwaters and maintain the water in a reservoir at the desired level. A design flood should be adopted for a recurring period of 10,000 years, which corresponds to a 1% risk of its being equalled or exceeded during an estimated useful life of 100 years.

Spillways can be classified as *tunnel* or *surface* spillways.

Tunnel spillways can have bottom outlet gates, galleries or conduits with fixed cone valves. Tunnel spillways should only be used if there are discharge requirements downstream which cannot be met by a surface spillway.

There is also another kind of outlet which is temporary and whose main purpose is to assure a minimum flow downstream while the reservoir is being filled.

A fixed-cone valve is normally used when locks are used for navigation, to ensure a constant flow downstream from the dam.

Surface spillways can be gated or ungated. They can have a high ogee crest, a low ogee crest or be built on an abutment.

Ungated spillways are typically used in run-of-the-river plants, where the dam can have one free spillway crest. They raise the water level in the reservoir higher. The other kinds of ungated spillway, like siphon or glory hole spillways, are little used and when they are the spillway flow is normally low.

Gated spillways are recommended for projects with reservoirs with drawdown capacity.

Except in certain cases, the layouts investigated in Inventory Studies should only use gated or ungated surface spillways (free spillway crest).

Emergency spillways should be avoided — fuse or others — with a view to reducing the required capacity of the main spillways. Likewise, it is not good practice to position the spillway between generation units in the powerhouse or over it.

The choice of what kind of spillway to use and where to position it will depend on the overall design of the project, the kind of diversion used and the local geological features.

High ogee spillways have a high free spillway crest, with or without tainter gates and an energy dissipator. They are normally used for projects with medium-height dams and also serve to divert waters through sluiceways in their structure.

Low ogee spillways have a low free spillway crest, with or without tainter gates and an energy dissipator. They are generally used for low dams and can also be used for river diversion.

Abutment spillways have a low free spillway crest followed by a chute and an energy dissipator. They can be gated or not. They are normally used for high earthfill dams closing off the whole of the river valley and provide river diversion through tunnels or galleries. They are built on one of the abutments or occasionally in a saddle, and may or may not make use of a bend in the river.

This kind of spillway has:

- approach channel;
- crest structure and control equipment;
- chute and sidewalls;
- stilling basin and energy dissipator; and
- downstream channel leading to the river bed.

The *energy* of the discharge through the spillway is very high and must be efficiently dissipated in the shortest space possible, especially to prevent damage to structures of the project per se.

Hydraulically speaking, the difficulty of dissipating energy depends on the specific flow per meter width of the spillway and the way to minimize it is to increase the dissipation structure or even to reduce the height of the gates while correspondingly increasing their width.

There are many kinds of energy dissipators. Their selection should be made taking into account the kind of spillway, the hydraulic parameters of the project and the local geological conditions. This manual recommends the use of a ski jump or stilling basin.

A stilling basin is recommended where the geological conditions are poor.

A ski jump, which is where the energy is dissipated by the impact of a jet on the impact basin, requires more resistant material in the basin to minimize regressive erosion (downstream to upstream) from the impact point of the jet. In this kind of dissipator, the effect of the impact of the jet can be minimized by reducing the specific flow or producing good dispersion and aeration of the jet.

Experience in projects for these devices shows that the cost of constructing spillways using ski jumps is much lower (MAGELA, G.P, CBGB, Publicação 03/96 – Erosão em bacias de lançamento).

GATED SURFACE SPILLWAYS WITH A LOW OGEE CREST

Basic Design and Recommendations

Spillways of this kind can also be used as a provisional *river diversion* structure. The diversion may be done over the ogee, which can be partially or totally lowered and then concreted later.

The *location* of the spillway will depend on the overall layout, but it is normally near or within the river bed adjacent to one of the banks if the river is wide enough, so as to minimize the excavation volumes and make it easier to use as a diversion structure.

The whole concrete structure should stand on sound rock foundations.

The hydraulic performance of this kind of spillway is normally inferior to that of other kinds because its downstream outlet is submerged. This means the area of the gates must be greater.

The river bed itself normally serves as both the approach and downstream channels. When this is not the case, the spillway's *discharge axis* should be straight and the angle between this axis and the direction of the river in the downstream channel should be no more than 45º. A bend is only allowed in the approach channel in low velocity regions.

The elevation of the bottom of the *approach channel* to the spillway is defined whenever possible so as to allow for a good hydraulic performance. This elevation is often set as the same elevation as the river bed.

When spillways are entirely built on one of the abutments, *cofferdams* should be built, one upstream from the approach channel and another downstream from the downstream channel, so that the excavation work can be undertaken. When spillways are totally or partially on the river bed, a first-stage cofferdam will be needed before the construction of the spillway can begin.

It is a good idea to avoid using very large gates as they are hard to operate, even if this would result in a lower investment.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning gated spillways with a low ogee crest are basically the same as those for designing gated spillways with a high ogee crest, described in item 5.7.5 – Spillways. Use spreadsheet 575cobd.xls for calculating dimensions, quantifying volumes and estimating costs of spillways with a stilling basin, and 575cose.xls for spillways with a ski jump. In the latter case, replace the volume of concrete for the deflector baffle with the volume for a protection slab.

Spillways should be designed to discharge the design flood without raising the maximum normal water level in the reservoir or reducing the design flood through the reservoir.

The height of the gates is taken as the difference between the normal maximum water level in the reservoir and the elevation of the ogee crest, and is selected as a function of the design flood of the spillway and the number of gates required, as well as other factors. Item 5.7.5. gives a suggestion for choosing the height of the gate.

The dimensions of the *gate openings*, which is a function of the design flood of the spillway, the height of the gates and the discharge coefficient, should respect the following limits:

minimum number	2
maximum height	21.0 m
maximum width	20.0 m
minimum proportions	width ≥ 70% of height
maximum proportions	width ≤ height

The spillway dimensions and construction volumes can be calculated from the design flood, the elevation of the approach channel, the maximum normal water level in the reservoir, the water level in the downstream channel, the topography and the height and number of gates.

Fig. 5.5.5.01 shows a typical cross-section of a gated spillway with a low ogee crest.

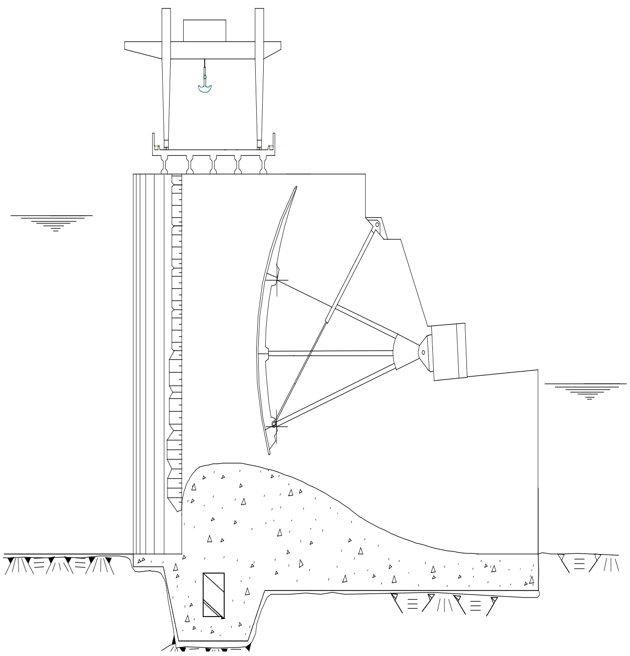


Fig. 5.5.5.01 – Typical cross-section of a gated surface spillway with a low ogee crest.

The US Corps of Engineers (1971) equation was used to define the profile of the ogee.

The discharge coefficient will depend on the geometry of the crest, the height of the ogee crest and the hydrostatic head on the ogee crest (Bureau of Reclamation, 1977), and should be corrected in order to take into account submergence downstream.

The width of the chute should only be corrected for the contraction of the jet at the end of the pillars.

The depth of the *stilling basin* is determined iteratively and based on the Froude number at the inlet to the basin for the 100-year flood flow.

Initially, the elevation of the bottom of the stilling basin is set, and the suitability of this value is checked by calculating the velocity, the depth of flow in the chute and the Froude number before the hydraulic jump, the depth of flow in the chute after the jump and, finally, the elevation of the bottom of the basin. If this value is different from the value set initially, the calculations should be redone until the required degree of precision is reached.

The Froude number should be betwen 4.5 and 9.0, because the hydraulic jump will be stable, better defined and less sensitive to variations in the downstream water level in this range (Chow, 1959). One way of raising the Froude' number to 4.5 would be to lower the bottom of the stilling basin beyond the result given by the calculation above. Likewise, one way to reduce the Froude number to 9.0 would be to reduce the width of the stilling basin, whenever economically feasible.

The length of the stilling basin is determined as a function of the depth of flow in the chute after the hydraulic jump.

Fig. 5.5.5.02 shows a typical cross-section of a stilling basin.

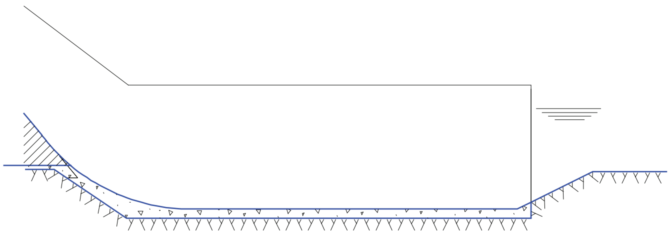


Fig. 5.5.5.02 – Typical cross-section of a stilling basin.

The use of a ski jump is not recommended for this kind of spillway because the low height makes it difficult to form an efficient jet. When the geological conditions are good, the flow can be released directly into the river or chute without using any energy dissipation structure apart from a concrete slab to protect the concrete structure against erosion.

The width of the energy dissipation structure is the same as that of the chute, except when the stilling basin is planned to have a Froude number greater than 9.0.

The height of the *sidewalls* along the downstream face and the ski jump should be 1.6 times the depth of flow in the chute to offset air entrainment in the water column. In the stilling basin the height is set above the depth downstream from the hydraulic jump. These walls are either gravity walls or are anchored to the rock and should be at least 1.0 m thick with an external slope of 0.5H:1V when they are not set into the rock.

The volume of common excavation is determined as a function of the average layer of soil in the area of the structure.

The volume of excavated rock is determined as a function of the mean elevation of the rock surface and the elevation of the foundation's structure.

The whole area of the foundation should be cleaned. The foundation treatment entails a drainage line immediately downstream from the grout curtain.

The volume of concrete for the spillway is determined as a function of its geometry.

GATED SURFACE SPILLWAY WITH A HIGH OGEE CREST

Basic Design and Recommendations

This kind of spillway can also be used as a provisional *river diversion* structure through sluiceways within its structure.

The *location* of the spillway will depend on the overall layout, the kind of dam and the use of the excavated material in the spillway area for building the dam:

- when it is incorporated into a conventional or roller compacted concrete dam, it should be near or on the river bed if this is wide enough, so as to minimize the amount to be excavated and to make it easier to use as a diversion structure;
- when it is used with rockfill dams it should be inside the abutment so as to minimize the length of concrete walls needed to link the dam and the spillway, but without requiring excessive amounts of excavation. In this case, the material from the excavation can be used for the cofferdam and the dam;
- when it is used with earthfill dams, the cost of the excavation will have to be weighed up against the cost of the concrete, since there will be much less rock needed.

The whole concrete structure should stand on foundations of sound bedrock.

The spillway's *discharge axis* should be straight and the angle between it and the direction of the river in the downstream channel should be no more than 45º. A bend is only allowed in the approach channel in areas of low velocity.

The profile of the *approach channel* should be designed to minimize head losses and allow the homogeneous distribution of the flow throughout the control structure. The elevation of the bottom of the spillway approach channel is defined in order to ensure good hydraulic performance. This elevation is often set as the same as the river bed or the bottom of the approach channel to the diversion sluiceways. When it is in one of the abutments in a part with no sluiceways, the elevation is defined in such a way that the height of the ogee crest will be around half the height of the gates. The flow rate in the approach channel should not exceed 2 m/s.

The *downstream channel* is simply excavated in rock and its dimensions should be such that the velocity will be below the maximum allowable for the local geological conditions.

When spillways are entirely built on one of the abutments, *cofferdams* should be built, one upstream from the approach channel and another downstream from the downstream channel so that excavation work can take place. When spillways are totally or partially on the river bed, a first-stage cofferdam will be needed before the construction of the spillway can begin.

It is a good idea to avoid using very large gates as they are hard to operate, even if this would result in a lower investment.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning gated spillways with a high ogee crest are described in item 5.7.5. – Spillways – Gated Surface Spillways with a High Ogee Crest. Use *spreadsheet* 575cobd.xls for calculating dimensions, quantifying volumes and estimating costs of spillways with a stilling basin, and 575cose.xls for spillways with a ski jump.

The same criteria as those presented for designing surface spillways with a low ogee crest can be used for this kind of spillway.

Fig. 5.5.5.03 shows a typical cross-section of a gated high ogee spillway.

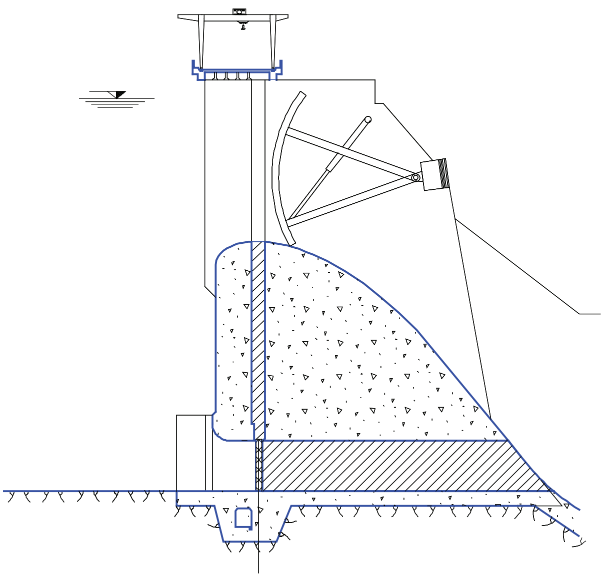


Fig. 5.5.5.03 – Typical cross-section of a gated surface spillway with a high ogee crest.

The US Corps of Engineers (1971) equation was used to define the profile of the ogee. When ogee crests are higher and when the slope of the downstream face reaches 133%, this slope must be maintained.

The same considerations for the discharge coefficient and sidewalls as those set out for low ogee crests are valid for this kind of spillway.

When the structure has a *ski jump*, its cross-section should be a circular curve with a radius that is three times the depth of the water column, at a tangent to the base and ending at a 25.8º angle to the horizontal.

The elevation of the sill of the ski jump can be higher than the maximum water level in the downstream channel for the 100-year flood flow.

GATED SURFACE ABUTMENT SPILLWAY

Basic Design and Recommendations

When this kind of spillway is used, an independent *river diversion* structure will be needed.

This kind of spillway should be *built* on one of the abutments next to the dam, in saddles or low points provided by the local relief. The use of excavated material in the area of the spillway for building the dam is normally a major consideration in defining its location.

The whole concrete structure must have sound rock foundations.

The spillway's *discharge axis* should be straight and the angle between it and the direction of the river in the downstream channel should be no more than 45º. A bend is only allowed in the approach channel in areas of low velocity.

The profile of the *approach channel* should be designed to minimize head losses and allow the homogeneous distribution of the flow on the ogee crest. The elevation of the bottom of the approach channel is defined in order to ensure that the ogee crest of the spillway is not too high. Crests whose height is between 25% and 40% of the height of the gate can be tried initially.

The profile of the spillway will depend on the local topographic and geological conditions.

The cross-section of the chute should be rectangular and have a constant width, with the bottom made of a concrete slab. The longitudinal slope of the chute should also be constant whenever possible. When this solution results in excessively high excavated volumes, a gentle slope of about 3-5% can be used, then increasing to about 20-30%.

The downstream channel should simply be excavated in rock and its dimensions should be such that the velocity is limited to the maximum possible for the local geological conditions.

A *cofferdam* should be built downstream from the downstream channel for the excavation work to be carried out.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning gated abutment spillways are described in item 5.7.5. – Spillways – Gated Surface Abutment Spillways. Use *spreadsheet* 575coenb.xls for calculating dimensions, quantifying volumes and estimating costs of spillways with a stilling basin, and 575coens.xls for spillways with a ski jump.

The same design criteria can be used as those used for the surface spillway with a low ogee crest.

Fig. 5.5.5.04 shows a typical cross-section of a gated abutment spillway.

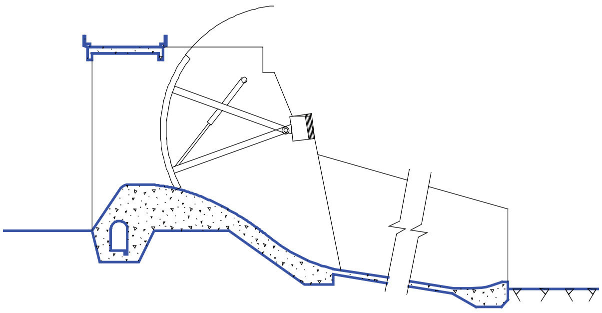


Fig. 5.5.5.04 – Typical cross-section of a gated abutment spillway.

The US Corps of Engineers (1971) equations are used to calculate the profile of the ogee. The design of the downstream profile of the ogee should aim to improve the hydraulic performance of the spillway. The radius of curvature adopted for the outlet of the ogee crest is the same as the height of the gates.

The inclination of the upstream face of the ogee crest is defined as a function of the hydraulic performance intended and the height of the crest. In this manual, three options are presented: 1H:1V, 0.67H:1V and 0.33H:1V. An acceptable hydraulic performance can be achieved with a reasonable volume of concrete by varying the inclination of the upstream face and the height of the ogee. A gentler inclination and higher ogee crest will improve the hydraulic performance but increase the amount of concrete.

The discharge coefficient will depend on the geometry of the crest, the height of the ogee crest, the hydrostatic head on the ogee, the inclination of the upstream face of the spillway and the water level downstream (Bureau of Reclamation, 1977).

For abutment spillways, in order to better approximate the discharge flow, the contraction of the jet at the end pillars is not taken into account.

The other design criteria are the same as those presented for a gated surface spillway with a high ogee crest.

The concrete slab should be estimated at being 0.70 m thick, for the purposes of calculating quantities.

UNGATED SPILLWAY WITH A HIGH OGEE CREST

Basic Design and Recommendations

Spillways of this kind can also be used as a provisional *diversion* structure, with sluiceways in their structure.

The *location* of the spillway will basically depend on the use of the excavated material in the spillway area for building the dam:

- when it is incorporated into a conventional or roller compacted concrete dam, it should be near or on the river bed if this is wide enough, so as to minimize the amount to be excavated and to make it easier to use as a diversion structure;
- when it is used with rockfill dams it should be inside the abutment so as to minimize the length of concrete walls needed to link the dam and the spillway, but without requiring excessive amounts of excavation. In this case, the material from the excavation can be used for the cofferdam and the dam;
- when it is used with earthfill dams, the cost of the excavation will have to be weighed up against the cost of the concrete, since there will be much less rock needed.

The whole concrete structure should stand on foundations of sound rock.

The spillway's *discharge axis* should be straight and the angle between it and the direction of the river in the downstream channel should be no more than 45º. A bend is only allowed in the approach channel in areas of low velocity.

The elevation of the bottom of the spillway *approach channel* is defined in order to ensure good hydraulic performance. This elevation is often set as the same as the river bed or the bottom of the approach channel to the diversion sluiceways. When it is in one of the abutments in a part with no sluiceways, the elevation is defined in such a way that the height of the ogee crest will be around half the height of the gates. The flow rate in the approach channel should not exceed 2 m/s.

The *downstream channel* is simply excavated in rock and its dimensions will be such that the velocity will be below the maximum allowable for the local geological conditions.

When spillways are entirely built on one of the abutments, *cofferdams* should be built, one upstream from the approach channel and another downstream from the downstream channel, so that excavation work can take place.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning ungated high ogee spillways are described in item 5.7.5. – Spillways – Ungated Surface Spillways with a High Ogee Crest. Use *spreadsheet* 575lobd.xls for calculating dimensions, quantifying volumes and estimating costs of spillways with a stilling basin, and 575lose.xls for spillways with a ski jump.

The spillways should be designed to discharge the design flood without reducing the peak flood flow by storing part of the flood flow in the reservoir.

The maximum energy head on the crest is taken to be the difference between the maximum maximorum water level and the normal maximum water level in the reservoir that coincides with the elevation of the ogee crest. The maximum maximorum water level is selected as a function of the restrictions on the rising of the water level in the reservoir, the design flood of the spillway and the width of crest required, among other factors.

The spillway dimensions and construction volumes are calculated from the design flood, the elevation of the approach channel, the normal maximum water level in the reservoir, the water level in the downstream channel and the local topography.

Fig. 5.5.5.05 shows a typical cross-section of an ungated high ogee spillway.

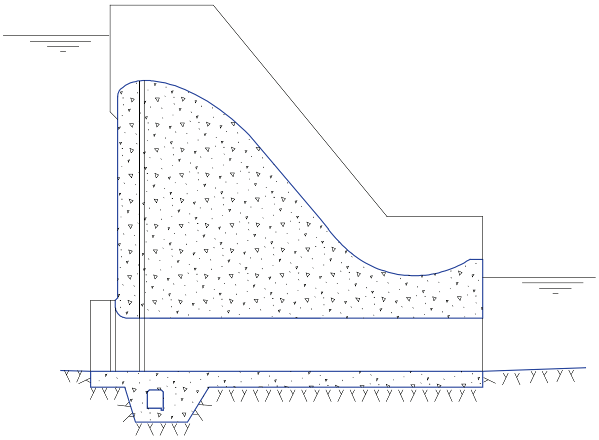


Fig. 5.5.5.05 – Typical cross-section of an ungated surface spillway with a high ogee crest.

The US Corps of Engineers (1971) equation was used to define the profile of the ogee and When the slope of the downstream slope reaches 133%, this slope must be maintained.

The discharge coefficient will depend on the geometry of the crest, the height of the ogee and the hydrostatic charge on the ogee (Bureau of Reclamation, 1977), and should be corrected in order to take into account the submergence downstream.

The discharge coefficient should be corrected to take account of the contraction of the jet near the end pillars.

The depth of the *stilling basin* is determined iteratively and based on the Froude number at the basin inlet for the 100-year flood flow.

Initially, the elevation of the bottom of the stilling basin is set, and the suitability of this value is checked by calculating the velocity, the depth of flow in the chute and the Froude number before the hydraulic jump, the depth of flow in the chute after the jump and, finally, the elevation of the bottom of the basin. If this value is different from the value set initially, the calculations should be redone until the required degree of precision is reached.

The Froude number should be betwen 4.5 and 9.0, because the hydraulic jump will be stable, better defined and less sensitive to variations in the downstream water level at this range (Chow, 1959). One way of raising the Froude number to 4.5 would be to lower the bottom of the stilling basin beyond the result given by the calculation above. Likewise, one way to reduce the Froude number to 9.0 would be to reduce the width of the stilling basin, whenever economically feasible

Fig. 5.5.5.02 shows a typical cross-section of a stilling basin.

When the structure has a *ski jump*, its cross-section should be a circular curve with a radius that is three times the depth of the water column, at a tangent to the chute and ending at a 25.8º angle to the horizontal.

The elevation of the sill of the ski jump can be higher than the maximum water level in the downstream channel for the 100-year flood flow.

Fig. 5.5.5.03 shows a typical cross-section for a ski jump.

The width of the energy dissipation structure is the same as that of the chute, except when the Froude number of the stilling basin needs to be greater than 9.0.

The height of the *sidewalls* along the downstream face and the ski jump should be 1.6 times the depth of the channel flow to offset air entrainment in the water column. In the stilling basin the height is set above the depth downstream from the hydraulic jump. These walls are either gravity walls or are anchored to the rock and should be at least 1.0 m thick with an external face of slope 0.5H:1V when they are not set into the rock

The volume of common excavation is determined as a function of the average layer of soil in the area of the structure.

The volume of excavated rock is determined as a function of the mean elevation of the rock surface and the elevation of the foundation's structure.

The whole area of the foundation should be cleaned. The foundation treatment entails a drainage line immediately downstream from the grout curtain

The volume of concrete for the spillway is determined as a function of its geometry.

UNGATED ABUTMENT SPILLWAY

Basic Design and Recommendations

When this kind of spillway is used, an independent *river diversion* structure will be needed.

This kind of spillway should be *built* on one of the abutments next to the dam, in saddles or low points provided by the local relief. The use of excavated material in the area of the spillway for building the dam is normally a major consideration in defining its location.

The whole concrete structure must have sound rock foundations.

The spillway's *discharge axis* should be straight and the angle between it and the direction of the river in the downstream channel should be no more than 45º. A bend is only allowed in the approach channel in areas of low velocity.

The profile of the *approach channel* should be designed to minimize head losses and allow a homogeneous distribution of the flow on the ogee. The elevation of the bottom of the approach channel is defined in order to ensure that the ogee of the spillway is not too high. Ogee crests whose height is between 25% and 40% of the height of the gate can be tried initially

The profile of the spillway will depend on the local topographic and geological conditions.

The cross-section of the chute should be rectangular and have a constant width, with the bottom made of a concrete slab. The longitudinal slope of the chute should also be constant, whenever possible. When this solution results in overly large excavated volumes, a gentle slope of about 3-5% can be used, then increasing to about 20-30%.

The *downstream channel* is simply excavated in rock and its dimensions will be such that the velocity will be below the maximum allowable for the local geological conditions.

A *cofferdam* should be built downstream from the downstream channel for the construction work to be done.

Criteria and procedures for dimensioning and quantification

The procedures for dimensioning gated abutment spillways are described item 5.7.5. – Spillways – Gated Surface Abutment Spillways. Use *spreadsheets* 575loens.xls for calculating dimensions, quantifying volumes and estimating costs of spillways with stilling basins, and 575loenb.xls for spillways with a ski jump.

Spillways should be designed to discharge the design flood without reducing the peak flood flow by storing part of the flood flow in the reservoir.

The maximum energy head on the crest is taken to be the difference between the maximum extreme water level and the normal maximum water level in the reservoir that coincides with the elevation of the ogee crest. The maximum extreme water level is selected as a function of the restrictions on the rising of the water level in the reservoir, the design flood of the spillway and the width of ogee required, among other factors.

The spillway dimensions and construction volumes are calculated from the design flood, the elevation of the approach channel, the normal maximum water level in the reservoir, the water level in the downstream channel and the local topography.

Fig. 5.5.5.06 shows a typical cross-section of an ungated abutment spillway.

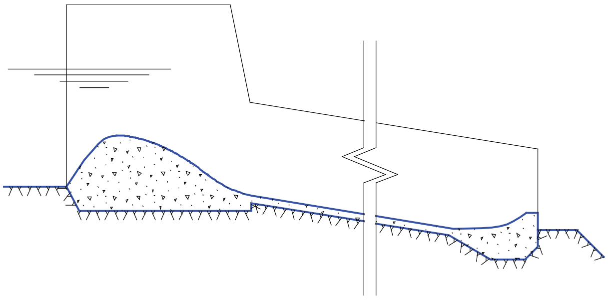


Fig. 5.5.5.06 – Typical cross-section of an ungated surface spillway with a low ogee crest.

The US Corps of Engineers (1971) equation was used to define the profile of the ogee. The downstream profile of the ogee should be such that the hydraulic performance of the spillway is maximized. The radius of curvature of the ogee outlet is taken as being the same as the height of the gates.

The slope of the upstream face of the ogee crest is defined as a function of the hydraulic performance intended and the height of the crest. In this manual, three options are presented: 1H:1V, 0.67H:1V and 0.33H:1V. An acceptable hydraulic performance can be achieved with a reasonable volume of concrete by varying the slope of the upstream face and the height of the crest. A gentler inclination and higher ogee crest will improve the hydraulic performance but increase the amount of concrete.

The discharge coefficient will depend on the geometry of the crest, the height of the ogee and the hydrostatic charge on the ogee (Bureau of Reclamation, 1977), the inclination of the upstream face of the spillway and the water level downstream (Bureau of Reclamation, 1977).

For abutment spillways, in order to better approximate the discharge flow, the contraction of the jet at the end pillars is not taken into account.

The depth of the *stilling basin* is determined iteratively and based on the Froude number at the inlet to the basin for the 100-year flood flow

Initially, the elevation of the bottom of the stilling basin is set, and the suitability of this value is checked by calculating the velocity, the depth of flow in the chute and the Froude number before the hydraulic jump, the depth of flow in the chute after the jump and, finally, the elevation of the bottom of the basin. If this value is different from the value set initially, the calculations should be redone until the required degree of precision is reached.

The Froude number should be betwen 4.5 and 9.0, because the hydraulic jump will be stable, better defined and less sensitive to variations in the downstream water level at this range (Chow, 1959). One way of raising the Froude number to 4.5 would be to lower the bottom of the stilling basin beyond the result given by the calculation above. Likewise, one way to reduce the Froude number to 9.0 would be to reduce the width of the stilling basin, whenever economically feasible.

The length of the stilling basin is a function of the depth of the channel flow after the hydraulic jump.

Fig. 5.5.5.02 shows a typical cross-section of a stilling basin.

When the structure has a *ski jump* (Fig. 5.5.5.03), its cross-section should be a circular curve with a radius that is three times the depth of the water column, at a tangent to the chute and ending at a 25.8º angle to the horizontal.

The elevation of the sill of the ski jump can be higher than the maximum water level in the downstream channel for the 100-year flood flow.

A width of the energy dissipation structure is the same as the chute, except if the Froude number for the stilling basin should be greater than 9.0.

The height of the *sidewalls* along the downstream face and the ski jump should be 1.6 times the depth of flow in the chute to offset air entrainment in the water column. In the stilling basin the height is set above the depth downstream from the hydraulic jump. These walls are either gravity or are anchored to the rock and should be at least 1.0 m thick with an external face of slope 0.5H:1V when they are not set into the rock.

The volume of common excavation is determined as a function of the average layer of soil in the area of the structure.

The volume of excavated rock is determined as a function of the mean elevation of the rock surface and the elevation of the foundation's structure.

The whole area of the foundation should be cleaned. The foundation treatment entails a drainage line immediately downstream from the grouting curtain.

The volume of concrete for the spillway is a function of its geometry.

The concrete slab should be estimated at being 0.70 m thick, for the purposes of determining quantities.

5.5.6 Roads, Railroads and Bridges (account .16)

The services for the construction or improvement of roads, railroads, bridges, service bridges and airports required to provide access to the power plants that make up part of the cascade options should be estimated by the length of the connections from the plant to the region's transportation network.

The roads to the operators' village and the powerhouse and the roads interconnecting the different structures are not part of this account.

Access roads must be able to withstand the normal flow of vehicles during the construction work and the occasional transportation of equipment to the power plant. The kind of paving to be used, the infrastructure pattern, and the width and category of the construction methods will depend on the needs and choice of the owner. The costs for this account are calculated using the unit prices for each category multiplied by the length or area to be built.

The airport must meet the minimum requirements for access to the construction site, which will normally imply building a small runway or using other airfields near the site. In Inventory Studies, it must be borne in mind that the projects are part of a cascade, which means that not all of them should necessarily incorporate the cost of building a landing strip. In this manual, a short landing strip is recommended, which is normally all that will be needed to meet the needs of the projects.

5.5.7 Indirect Costs (account .17)

Indirect costs cover all construction of a provisional nature and general services required for constructing the project, such as:

- construction and maintenance of the construction site and workers' camp;
- engineering services and environmental studies; and
- owner's administration expenses.

The construction of the construction site and workers' camp involves constructing the provisional structures and land developments required, which will be removed or abandoned once the construction work has been finished.

This cost is calculated as a function of the volume of services required, translated by a factor F, as shown in the calculation described in item 5.7.8 – Indirect Costs. Use spreadsheet 57ope.xls to estimate these costs.

The engineering services cover all the design and technical consultancy services rendered by the owner and/or contractor, such as the Feasibility Study, basic design, executive design, field and laboratory engineering services (topographic surveys, aerophotogrammetric surveys, geotechnical studies, hydrotechnical studies, hydraulic models) and environmental studies and plans.

The engineering costs are estimated as a percentage of the total direct cost of the project under study.

The owner's administration expenses cover all the control services, administrative support and consultancy provided by the owner or contractor that are directly connected to the construction work.

The owner's administration expenses are estimated as a percentage of the total direct cost of the project under study.

5.5.8 Interest During Construction (account .18)

Interest during construction covers the appropriation or forecasting of financial revenues from own capital (financial revenues) and capital held by third parties (financial expenses) during the construction of the project, according to the disbursement schedule. The calculation is made using a standard interest rate and is capitalized annually during the construction period.

In order to determine the interest during construction presented in item 5.7.9, an annual interest rate of 10-12% is assumed, and the projects are differentiated by the construction time, using standard investment curves. The interest rate must be obtained from the concession-granting authority.

When projects cannot use the standard investment curves presented, the interest during construction can be calculated using the procedure set out below.

The length of time required for the construction and likely construction schedule are needed.

The construction schedule can be drawn up based on the overall layout and quantities calculated using the procedure set out below, making any adjustments deemed necessary:

- determine the time taken to divert the river. If a diversion tunnel is to be used, set aside two months for preparation, six months for excavating the accessways and outlet of the tunnels, and a maximum rate of 100 meters per month per tunnel. Should the diversion be on the river bed, once the construction work has been prepared, assume the first-stage cofferdam will be built at a maximum rate of 100,000 m^3/month;

- depending on the layout, determine the likely starting date of concreting operations, assuming a minimum of six months for preparation and installation of the concrete and rock crusher plants, as well as the availability of the space required for the concreting work;
- make an approximation of the time needed to start the concreting of a surface powerhouse, which will normally require at least 50% of the volume of the tailrace canal to have been finished, as well as all the excavations for the powerhouse per se and the partial treatment of the foundations for the powerhouse area. When excavation is done in restricted areas, assume 100,000 m^3/month for common excavation and 70,000 m^3/month for excavation in rock. When the powerhouse is underground, assume 80 meters/month/face for the progress of horizontal tunnels, 40 meters/month/face for long tunnels at an slope of over 45º, and 20,000 m^3/month for excavation in the machine hall;
- allow 6-12 months after beginning the concreting of the powerhouse (assembly area and block of the first unit) before the first unit can start to be assembled, depending on the volume and estimated work conditions;
- allow 24-30 months for the assembly of the first unit for Francis or Kaplan turbines, and 18-24 months for assembling Pelton turbines. In both cases, add a further 3 to 4 months for commissioning and testing;
- make a qualitative examination of the layout and the size of the main quantities in order to check whether the activities mentioned above are actually factors of restriction for the schedule. When there are large volumes of concrete and excavated material in restricted areas, it may mean that very high productivity rates are required to achieve the volumes established, which may be incompatible with the availability of space and access constraints.

5.6 STANDARD ELETROBRAS COST ESTIMATE

The Standard Eletrobrás Cost Estimate (Orçamento Padrão Eletrobrás, or OPE) is recommended for cost estimates and budgeting purposes. It is a standard document that can be used at any stage of development of a hydropower project. The OPE accounts spreadsheet is in line with the requirements established by the Brazilian Ministry of Mines and Energy. Its definitions and descriptions can be found in *Descrições e Instruções para Aplicação do Orçamento Padrão Eletrobrás de Usinas Hidroelétricas* (Descriptions and Instructions for Using the Standard Eletrobrás Cost estimate for Hydropower Plants).

In this phase of the Inventory Studies, the costs of different structures are estimated individually as a function of the amounts of civil construction, services and equipment required. This is done in a simplified way, without entering into great detail but with great enough precision for the estimate to be a good approximation of the real cost of the structure in question. The OPE is, then, detailed to a level that is appropriate for this phase of the Inventory Studies.

Table 5.6.01 presents the cost estimate spreadsheet for the Final Studies of Inventory Studies, containing adaptations as necessary, including socioenvironmental items.

The interest rate to be used for account 18 must be obtained from the concession-granting authority (item 2.6).

Table 5.6.01 – Cost Estimate Spreadsheet for the Final Studies

ACCOUNT	ITEM	UN.	QUANT.	UNIT PRICE R$	COST R$ 10^3
.10.	**LANDS, RESETTLEMENTS, RELOCATIONS AND OTHER SOCIOENVIRONMENTAL ACTIONS**				**0**
.10.10	**LAND ACQUISITIONS AND LAND DEVELOPMENTS**				0
.10.10.10	URBAN REAL ESTATE	gl			0
.10.10.10.10	Reservoir	ha			0
.10.10.10.11	Construction site, workers' camp, borrow areas, etc.	ha			0
.10.10.10.40	Conservation Areas and Permanent Preservation Areas	ha			0
.10.10.10.43	Towns and villages	gl			
.10.10.10.44	Isolated social and economic infrastructure	gl			
.10.10.10.17	Other costs	gl			
.10.10.11	RURAL REAL ESTATE	gl			0
.10.10.11.10	Reservoir	ha			0
.10.10.11.11	Construction site, workers' camp, borrow areas, etc.	ha			0
.10.10.11.40	Conservation Areas and Permanent Preservation Areas	ha			0
.10.10.11.41	Rural resettlement	ha			0
.10.10.11.42	Indigenous peoples and other ethnic groups	ha			0
.10.10.11.43	Towns and villages	gl			
.10.10.11.44	Isolated social and economic infrastructure	gl			
.10.10.11.17	Other costs	gl			
.10.10.12	ACQUISITION AND LEGAL EXPENSES	%	15	0.00	0
.10.10.13	OTHER COSTS	gl			0
.10.11	**RESETTLEMENTS AND RELOCATIONS**				0
.10.11.14	ROADS	km			0
.10.11.15	RAILROADS	km			0
.10.11.16	BRIDGES	m²			0
.10.11.18	TRANSMISSION AND DISTRIBUTION SYSTEM	gl			
.10.11.19	COMMUNICATIONS SYSTEM	gl			
.10.11.20	POPULATION RESETTLEMENT	gl			0
.10.11.20.41	Rural Resettlement	gl			

.10.11.20.42	Indigenous peoples and other ethnic groups	gl			
.10.11.20.43	Towns and villages	gl			
.10.11.20.44	Isolated social and economic infrastructure	gl			
.10.11.20.17	Other costs	gl			
.10.11.21	OTHER RESETTLEMENTS / RELOCATIONS	gl			
.10.11.13	OTHER COSTS	gl			
.10.15	**OTHER SOCIOENVIRONMENTAL ACTIONS**				0
.10.15.44	SOCIOENVIRONMENTAL COMMUNICATION	gl			
.10.15.45	PHYSICAL AND BIOTIC ENVIRONMENT	gl			0
.10.15.45.18	Reservoir Cleaning	ha			0
.10.15.45.40	Conservation Areas and Permanent Preservation Areas	ha			0
.10.15.45.45	Conservation of flora	gl			
.10.15.45.46	Conservation of fauna	gl			
.10.15.45.47	Water quality	gl			
.10.15.45.48	Recuperation of degraded areas	gl			
.10.15.45.17	Other costs	gl			
.10.15.46	SOCIOECONOMIC AND CULTURAL ENVIRONMENT	gl			0
.10.15.46.42	Indigenous peoples and other ethnic groups	gl			
.10.15.46.49	Basic sanitation and healthcare	gl			
.10.15.46.50	Housing and education infrastructure	gl			
.10.15.46.51	Salvaging of cultural heritage	gl			
.10.15.46.52	Support for municipalities	gl			
.10.15.46.17	Other costs	gl			
.10.15.47	LICENSING AND INSTITUTIONAL MANAGEMENT	gl			0
.10.15.47.53	Licensing	gl			
.10.15.47.55	Institutional Management	gl			
.10.15.47.17	Other Costs	gl			
.10.15.48	MULTIPLE USES	gl			
.10.15.13	OTHER COSTS	gl			
	Subtotal of account .10				0
.10.27	**MISCELLANEOUS ITEMS FROM ACCOUNT .10**	%	20	0.00	0

.11.	**POWERHOUSE (CIVIL CONSTRUCTION) AND RELATED LAND DEVELOPMENTS**				**0**
.11.12	**LAND DEVELOPMENTS IN THE PLANT AREA**	gl			
.11.13	**POWERHOUSE**				0
.11.13.00.12	Excavation	gl			0
.11.13.00.12.10	common	m³		7.60	0
.11.13.00.12.11	surface rock	m³		21.00	0
.11.13.00.12.12	underground rock	m³		0.00	0
.11.13.00.13	Foundation cleaning and treatment	gl			
.11.13.00.14	Concrete	gl			0
.11.13.00.14.13	cement	t		348.00	0
.11.13.00.14.14	concrete without cement	m³			0
.11.13.00.14.15	reinforcement steel	t		4,327.00	0
.11.13.00.15	Installations and final works	gl			
	Subtotal of Account .11				0
.11.27	**MISCELLANEOUS ITEMS FROM ACCOUNT .11**	%	20	0.00	0

.12.	**DAMS AND INTAKES**				**0**
.12.16	**RIVER DIVERSION**				0
.12.16.22	COFFERDAMS	gl			0
.12.16.22.14	Concrete for deflector baffle	gl			
.12.16.22.19	Earth-rock cofferdam	m³			0
.12.16.22.20	Special cofferdams	gl			
.12.16.22.21	Removal of cofferdams	m³			0
.12.16.22.22	Dewatering and other costs	%	15	0.00	0
.12.16.22.56	Service bridge	gl			
.12.16.23	DIVERSION TUNNEL	gl			0
.12.16.23.12	Excavation	m^3			0
.12.16.23.12.10	common	m³		7.60	0
.12.16.23.12.11	surface rock	m³		21.00	0
.12.16.23.12.12	underground rock	m³		0.00	0
.12.16.23.13	Foundation cleaning and treatment	gl			

.12.16.23.14	Concrete	gl			0
.12.16.23.14.13	cement	t		348.00	0
.12.16.23.14.14	concrete without cement	m^3			0
.12.16.23.14.15	reinforcement steel	t		4,327.00	0
.12.16.23.23	Gates and related closing equipment	gl			0
.12.16.23.23.16	Gates without cranes	un			0
.12.16.23.23.56	Other embedded parts	gl			
.12.16.23.23.17	Stoplog	un			0
.12.16.23.23.56	Other embedded parts	gl			
.12.16.23.17	Other costs	gl			
.12.16.24.	DIVERSION CHANNEL OR GALLERY / SLUICEWAY	gl			0
.12.16.24.12	Excavation	m^3			0
.12.16.24.12.10	common	m^3		7.60	0
.12.16.24.12.11	surface rock	m^3		21.00	0
.12.16.24.13	Foundation cleaning and treatment	gl			
.12.16.24.14	Concrete	gl			0
.12.16.24.14.13	cement	t		348.00	0
.12.16.24.14.14	concrete without cement	m^3			0
.12.16.24.14.15	reinforcement steel	t		4,327.00	0
.12.16.24.23.	Gates and related closing equipment	gl			0
.12.16.24.23.16	Gates without cranes	un			0
.12.16.24.23.56	Other embedded parts	gl			
.12.16.24.23.17	Stoplog	un			0
.12.16.24.23.56	Other embedded parts	gl			
.12.16.24.23.17	Downstream stoplog	un			0
.12.16.24.23.56	Other embedded parts	gl			
.12.16.24.17	Other costs	gl			
.12.17	**DAMS AND DIKES**				0
.12.17.25	EARTHFILL AND ROCKFILL DAMS AND DIKES	gl			0
.12.17.25.12	Excavation	m^3			0
.12.17.25.12.10	common	m^3		7.60	0
.12.17.25.12.10	borrow area	m^3			0
.12.17.25.12.11	surface rock	m^3		21.00	0
.12.17.25.12.11	quarry	m^3			0
.12.17.25.13	Foundation cleaning and treatment	gl			0
.12.17.25.14	Concrete face	gl			0
.12.17.25.14.13	cement	t		348.00	0
.12.17.25.14.14	concrete without cement	m^3			0
.12.17.25.14.15	reinforcement steel	t		4,327.00	0
.12.17.25.24	Compacted earthfill	m^3		2.69	0
.12.17.25.25	Rockfill	m^3			0
.12.17.25.26	Clay core	m^3		11.10	0
.12.17.25.29	Transitions / drains	m^3		10.80	0
.12.17.25.32	Protection of dam faces	gl			0
.12.17.25.32.18	Upstream face	m^3		12.90	0
.12.17.25.32.19	Downstream face	m^2		5.90	0
.12.17.25.17	Other costs	%	2	0.00	0
.12.17.26	CONCRETE DAMS	gl			0
.12.17.26.12	Excavation	m^3			0
.12.17.26.12.10	common	m^3		7.60	0
.12.17.26.12.11	surface rock	m^3		21.00	0
.12.17.26.13	Foundation cleaning and treatment	gl			
.12.17.26.14	Conventional concrete	gl			0
.12.17.26.14.13	cement	t		348.00	0
.12.17.26.14.14	concrete without cement	m^3			0
.12.17.26.14.15	reinforcement steel	t		4,327.00	0
.12.17.26.14	Roller-compacted concrete	gl			0
.12.17.26.14.13	cement	t		348.00	0
.12.17.26.14.14	concrete without cement	m^3			0
.12.17.26.17	Other costs	%	2	0.00	0
.12.17.27	CONCRETE TRANSITION AND RETAINING WALLS	gl			0
.12.17.27.12	Excavation	m^3			0

.12.17.27.12.10	common	m³		7.60	0
.12.17.27.12.11	surface rock	m³		21.00	0
.12.17.27.13	Foundation cleaning and treatment	gl			
.12.17.27.14	Concrete	gl			0
.12.17.27.14.13	cement	t		348.00	0
.12.17.27.14.14	concrete without cement	m³			0
.12.17.27.14.15	reinforcement steel	t		4,327.00	0
.12.17.27.17	Other costs	gl			
.12.18	**SPILLWAYS**				0
.12.18.28	SURFACE SPILLWAYS	gl			0
.12.18.28.12	Excavation	m^3			0
.12.18.28.12.10	common	m³		7.60	0
.12.18.28.12.11	surface rock	m³		21.00	0
.12.18.28.13	Foundation cleaning and treatment	gl			
.12.18.28.14	concrete	gl			0
.12.18.28.14.13	cement	t		348.00	0
.12.18.28.14.14	concrete without cement	m³			0
.12.18.28.14.15	reinforcement steel	t		4,327.00	0
.12.18.28.23	Gates and related closing equipment	gl			0
.12.18.28.23.16	Gates and cranes	un			0
.12.18.28.23.56	Other embedded parts	gl			
.12.18.28.23.17	Stoplog	un			0
.12.18.28.23.56	Other embedded parts	gl			
.12.18.28.23.20	Crane	un			0
.12.18.28.17	Other costs	%	2	0.00	0
.12.18.29	GLORY HOLE, TUNNEL SPILLWAY, ETC.	gl			0
.12.18.29.12	Excavation	m^3			0
.12.18.29.12.10	common	m³		7.60	0
.12.18.29.12.11	surface rock	m³		21.00	0
.12.18.29.13	Foundation cleaning and treatment	gl			
.12.18.29.14	concrete	gl			0
.12.18.29.14.13	cement	t		348.00	0
.12.18.29.14.14	concrete without cement	m³			0
.12.18.29.14.15	reinforcement steel	t		4,327.00	0
.12.18.29.23	Gates and related closing equipment	gl			0
.12.18.29.23.16	Gates and cranes	un			0
.12.18.29.23.17	Stoplog	un			0
.12.18.29.23.56	Other embedded parts	gl			
.12.18.29.23.20	Crane	un			0
.12.18.29.17	Other costs	gl			
.12.19	**INTAKE**				0
.12.19.30	INTAKE	gl			0
.12.19.30.12	Excavation	m^3			0
.12.19.30.12.10	common	m³		7.60	0
.12.19.30.12.11	surface rock	m³		21.00	0
.12.19.30.13	Foundation cleaning and treatment	gl			
.12.19.30.14	concrete	gl			0
.12.19.30.14.13	cement	t		348.00	0
.12.19.30.14.14	concrete without cement	m³			0
.12.19.30.14.15	reinforcement steel	t		4,327.00	0
.12.19.30.23	Gates and related closing equipment	gl			0
.12.19.30.23.16	Gates and cranes	un			0
.12.19.30.23.17	Stoplog	un			0
.12.19.30.23.56	Other embedded parts	gl			
.12.19.30.23.20	Crane	un			0
.12.19.30.23.21	Trash racks and trash rack cleaners	gl			
.12.19.30.17	Other costs	%	2	0.00	0
.12.19.31	HEADRACE CANAL	gl			0
.12.19.31.12	Excavation	m^3			0
.12.19.31.12.10	common	m³		7.60	0
.12.19.31.12.11	surface rock	m³		21.00	0
.12.19.31.13	Foundation cleaning and treatment	gl			0

.12.19.31.14	Concrete	gl		0
.12.19.31.14.13	cement	t	348.00	0
.12.19.31.14.14	concrete without cement	m³		0
.12.19.31.14.15	reinforcement steel	t	4,327.00	0
.12.19.31.17	Other costs	gl		
.12.19.32	PENSTOCK	gl		0
.12.19.32.12	Excavation	m^3		0
.12.19.32.12.10	common	m³	7.60	0
.12.19.32.12.11	surface rock	m³	21.00	0
.12.19.32.12.12	underground rock	m³	0.00	0
.12.19.32.13	Foundation cleaning and treatment	gl		
.12.19.32.14	Concrete	gl		0
.12.19.32.14.13	cement	t	348.00	0
.12.19.32.14.14	concrete without cement	m³		0
.12.19.32.14.15	reinforcement steel	t	4,327.00	0
.12.19.32.17	Other costs	gl		
.12.19.33	SURGE TANKS	gl		0
.12.19.33.12	Excavation	m^3		0
.12.19.33.12.10	common	m³	7.60	0
.12.19.33.12.11	surface rock	m³	21.00	0
.12.19.33.12.12	underground rock	m³	0.00	0
.12.19.33.13	Foundation cleaning and treatment	gl		
.12.19.33.14	Concrete	gl		0
.12.19.33.14.13	cement	t	348.00	0
.12.19.33.14.14	concrete without cement	m³		0
.12.19.33.14.15	reinforcement steel	t	4,327.00	0
.12.19.33.23	Equipment	gl		0
.12.19.33.23.23	Steel lining	t		0
.12.19.33.17	Other costs	gl		
.12.19.34.	PRESSURE PENSTOCK AND/OR TUNNEL	gl		0
.12.19.34.12	Excavation	m^3		0
.12.19.34.12.10	common	m³	7.60	0
.12.19.34.12.11	surface rock	m³	21.00	0
.12.19.34.12.12	underground rock	m³	0.00	0
.12.19.34.13	Foundation cleaning and treatment	gl		
.12.19.34.14	Concrete	gl		0
.12.19.34.14.13	cement	t	348.00	0
.12.19.34.14.14	concrete without cement	m³		0
.12.19.34.14.15	reinforcement steel	t	4,327.00	0
.12.19.34.23	Gates and related closing equipment	gl		0
.12.19.34.23.23	Steel lining	gl		
.12.19.34.23.24	Butterfly valve	gl		
.12.19.34.23.24	Spherical valve	gl		
.12.19.34.17	Other costs	gl		
.12.19.35	TAILRACE CANAL AND/OR TUNNEL	gl		0
.12.19.35.12	Excavation	m^3		0
.12.19.35.12.10	common	m³	7.60	0
.12.19.35.12.11	surface rock	m³	21.00	0
.12.19.35.12.12	underground rock	m³	0.00	0
.12.19.35.13	Foundation cleaning and treatment	gl		0
.12.19.35.14	Concrete	gl		0
.12.19.35.14.13	cement	t	348.00	0
.12.19.35.14.14	concrete without cement	m³		0
.12.19.35.14.15	reinforcement steel	t	4,327.00	0
.12.19.35.17	Other costs	gl		
.12.20	**SPECIAL CONSTRUCTIONS**			0
.12.20.36	LOCK AND/OR PORT	gl		0
.12.20.36.12	Excavation	m^3		0
.12.20.36.12.10	common	m³	7.60	0
.12.20.36.12.11	surface rock	m³	21.00	0
.12.20.36.13	Foundation cleaning and treatment	gl		0
.12.20.36.14	Concrete	gl		0

.12.20.36.14.13	cement	t		348.00		0
.12.20.36.14.14	concrete without cement	m³				0
.12.20.36.14.15	reinforcement steel	t		4,327.00		0
.12.20.36.23	Gates and related closing equipment	gl				0
.12.20.36.23.25	Lock equipment	gl				
.12.20.36.17	Other costs	gl				
.12.20.37	OTHER SPECIAL CONSTRUCTIONS	gl				
	Subtotal for construction work					0
	Subtotal for equipment					0
.12.27.98	MISCELLANEOUS ITEMS FROM ACCOUNT .12 civil constructi	%	20	0.00		0
.12.27.99	MISCELLANEOUS ITEMS FROM ACCOUNT .12 equipment	%	15	0.00		0
.13.	**TURBINES AND GENERATORS**					**0**
.13.13.00.23.17	Stoplog	un				0
.13.13.00.23.20	Crane	un				0
.13.13.00.23.28	Turbines	un				0
.13.13.00.23.29	Generators	un				0
.13.13.00.23.56	Extra embedded parts	gl				
	Subtotal of account .13					0
.13.27	**MISCELLANEOUS ITEMS FROM ACCOUNT .13**	%	10	0.00		0
.14.	**AUXILIARY ELECTRICAL EQUIPMENT**					**0**
.14.00.00.23.30	Auxiliary electrical equipment	%	18	0.00		0
	Subtotal of account .14					0
.14.27	**MISCELLANEOUS ITEMS FROM ACCOUNT .14**	%	20	0.00		0
.15.	**MISCELLANEOUS EQUIPMENT FOR THE PLANT**					**0**
.15.13.00.23.20	Bridge crane	un				0
.15.13.00.23.20	Gantry crane	un				0
.15.00.00.23.31	Miscellaneous equipment	%	6	0.00		0
	Subtotal from account .15					0
.15.27	**MISCELLANEOUS ITEMS FROM ACCOUNT .15**	%	15	0.00		0
.16.	**ROADS, RAILROADS AND BRIDGES**					**0**
.16.00.14	ROADS	km				0
.16.00.15	RAILROADS	km				0
.16.00.16	BRIDGES	m²				0
.16.00.17	AIRPORT	gl				
	Subtotal from account .16					0
.16.27	**MISCELLANEOUS ITEMS FROM ACCOUNT .16**	%	20	0.00		0
	TOTAL DIRECT COSTS					0
.17.	**INDIECT COSTS**				**#NÚM!**	
.17.21	**CONSTRUCTION SITE AND WORKERS' CAMP**				#NÚM!	
.17.21.38	CONSTRUCTIONS FOR CONSTRUCTION SITE AND WORKI	gl				0
.17.21.39	MAINTENANCE AND OPERATION OF CONSTRUCTION SITI	gl				#NÚM!
.17.22	**OWNER'S ENGINEERING AND ADMINISTRATION**					0
.17.22.40	ENGINEERING	gl				0
.17.22.40.36	Basic Engineering Project	%	3.5	0.00		0
.17.22.40.37	Special Engineering Services	%	1.0	0.00		0
.17.22.40.54	Environmental Studies and Projects	%	0.5	0.00		0
.17.22.41	ADMINISTRATION BY OWNER	%	12	0.00		0
	Subtotal of account .17				#NÚM!	
.17.27	**MISCELLANEOUS ITEMS FROM ACCOUNT .17**	%	20	#NÚM!	#NÚM!	
	DIRECT AND INDIRECT COSTS				#NÚM!	
.18.	**INTEREST DURING CONSTRUCTION**	**%**		**#NÚM!**	**#NÚM!**	
	TOTAL COST INCLUDING INTEREST DURING CONSTRUCTION				**#NÚM!**	
	Installed Capacity	kW				
	Cost in R$ x 1,000/kW					#NÚM!

5.7 DESIGN AND COST ESTIMATE OF PROJECTS

This item sets out the procedures for dimensioning the structures and equipment and quantifying the civil construction services. Most of the criteria are presented in item 5.5, alongside the guidelines for the basic design of the overall layout.

5.7.1 Lands, Rights of Way and Socioenvironmental Actions

The procedures set out in item 4.10.1. should be used for estimating the socioenvironmental costs. The quantities, unit prices and recommended criteria should be reviewed in order to incorporate the findings of the fieldwork and other studies undertaken. The depth and detail of these estimates should be compatible with the general guidelines established for each inventory study, considering the features of the river basin under study and the socioenvironmental interferences identified.

When the studies are more complex, involving larger river basins with bigger-scale projects, a higher degree of precision should be aimed for in the estimates of socioenvironmental costs. Likewise, studies for river basins in special ecosystems or ecosystems that are protected by law, especially those within the Amazon, should be undertaken with particular care.

5.7.2 Powerhouse

This item is organized as follows:

- general, covering the common aspects of powerhouse design;
- powerhouse equipped with Pelton turbines;
- powerhouse equipped with vertical-axis Francis turbines;
- powerhouse equipped with horizontal-axis Francis turbines;
- powerhouse equipped with Kaplan turbines with a steel spiral casing;
- powerhouse equipped with Kaplan turbines with a semi-spiral casing made of concrete; and
- powerhouse equipped with Bulb turbines.

In order to design a powerhouse and its equipment, the use of the following spreadsheets is recommended, choosing the correct one for the turbine selected in the preliminary studies:

- 572p.xls – for powerhouse equipped with Pelton turbines;
- 572fv.xls – for powerhouse equipped with vertical-axis Francis turbines;
- 572fh.xls – for powerhouse equipped with horizontal-axis Francis turbines;
- 572ka.xls – for powerhouse equipped with Kaplan turbines with a steel spiral casing;
- 572kc.xls – for powerhouse equipped with Kaplan turbines with a semi-spiral casing made of concrete; and
- 572b.xls – for powerhouse equipped with Bulb turbines.

GENERAL

Basic Data

The main **information required for dimensioning** the turbine will come from items 4.6 and 5.3, that is:

- initial installed capacity, P' in MW;
- maximum net head, H_1 in m, from item 4.6;
- normal water level in the tailrace canal, NA_{fu}, from item 4.6;
- minimum water level in the tailrace canal, NA_{nfu}, from item 4.6;
- power factor, f_p, 0.90, in the absence of more accurate information;
- mean generator output, η_g, from item 4.6;
- mean turbine output, η_t, from item 4.6;
- mean water temperature in the summer, T, in °C; and
- frequency of the electricity system, f, in Hz (60 Hz in Brazil).

The main **information required for quantification purposes** is listed for each kind of turbine.

Type of Turbine

The **type of turbine** can be selected directly using Graph 5.7.2.01, as a function of the maximum net head and the unit capacity of the turbine, or from the following equivalent expressions (Eletrosul, 1996):

- for **Pelton** turbines: $150 \leq H_1 \leq 1500$ m
- for **vertical-axis Francis** turbines: $27 \leq H_1 \leq 600$ m
- for **horizontal-axis Francis** turbines: $27 \leq H_1 \leq 350$ m
- for **Kaplan** turbines: $8 \leq H_1 \leq 70$ m
- for **Bulb** turbines: $4 \leq H_1 \leq 23$ m

Whenever the head is such that more than one kind of turbine could be used, the decision should take into account the technical and operational characteristics of the generation equipment and also the costs and benefits of each option.

Number of units and capacities

The initial **total capacity of the set of turbines**, P'_t (kW), is given by:

$$P'_t = \frac{1000 \times P'}{\eta_g}$$

where:

P'	initial installed capacity, in MW; and
η_g	mean generator output.

The **number of generating units**, N_g, is given by:

$$N_g = \text{int}\left(\frac{P'_t}{1000 \times P_{1xt}} + 0.999\right) \geq 2$$

where:

- for **Pelton** turbines:

  $150 \leq H_1 \leq 200$ m: $P_{1xt} = 4.6 \times 10^{-14} \times H_1^{6.4526}$

  $200 \leq H_1 \leq 380$ m: $P_{1xt} = 2.0 \times 10^{-5} \times H_1^{2.691}$

  $380 \leq H_1 \leq 750$ m: $P_{1xt} = 0.5397 \times H_1^{0.978}$

  $750 \leq H_1 \leq 950$ m: $P_{1xt} = 350$

  $950 \leq H_1 \leq 1500$ m: $P_{1xt} = 3.331 \times 10^{9} \times H_1^{-2.3436}$

- for **vertical-axis Francis** turbines:

  $27 \leq H_1 \leq 46$ m: $P_{1xt} = 1.55 \times 10^{-10} \times H_1^{7.3423}$

  $46 \leq H_1 \leq 110$ m: $P_{1xt} = 2.0076 \times H_1^{1.2601}$

  $110 \leq H_1 \leq 200$ m: $P_{1xt} = 750$

  $200 \leq H_1 \leq 600$ m: $P_{1xt} = 440.010 \times H_1^{-1.2031}$

- for **horizontal-axis Francis** turbines:

  $27 \leq H_1 \leq 115$ m: $P_{1xt} = 0.1554 \times H_1^{1.0531}$

  $115 \leq H_1 \leq 350$ m: $P_{1xt} = 23.0$

- for **Kaplan** turbines:

  $8 \leq H_1 \leq 12$ m: $P_{1xt} = 0.25 \times H_1^{2.1072}$

  $12 \leq H_1 \leq 20$ m: $P_{1xt} = 0.2324 \times H_1^{2.1367}$

  $20 \leq H_1 \leq 30$ m: $P_{1xt} = 10.04 \times H_1^{0.8797}$

  $30 \leq H_1 \leq 50$ m: $P_{1xt} = 200$

  $50 \leq H_1 \leq 70$ m: $P_{1xt} = 632.384 \times H_1^{-2.06}$

- for **Bulb** turbines:

  $4.0 \leq H_1 \leq 5.5$ m: $P_{1xt} = 0.3516 \times H_1^{2.5465}$

  $5.5 \leq H_1 \leq 15.5$ m: $P_{1xt} = 4.52 \times H_1^{1.0484}$

  $15.5 \leq H_1 \leq 23.0$ m: $P_{1xt} = 80$

where:

H_1	maximum net head, in m;
P'_t	total initial capacity of the turbines, in kW;
P_{1xt}	maximum unit capacity of the turbine for the available head, in MW (Eletrosul, 1996); and
int(x)	function that returns the integer part of x.

The **initial capacity of a generating unit**, P'_1 (MW), is given by is given by:

$$P'_1 = \frac{P'}{N_g} \geq \eta_g \times P_{1nt}$$

where:

- for **Pelton** turbines: $P_{1nt} = 5$
- for **vertical-axis Francis** turbines:

$27 \le H_1 \le 200$ m: $P_{1nt} = 5$

$200 \le H_1 \le 350$ m: $P_{1nt} = 0.0071 \times H_1^{1.2386}$

$350 \le H_1 \le 600$ m: $P_{1nt} = 8.36 \times 10^{-6} \times H_1^{2.5312}$

- for **horizontal-axis Francis** turbines:

$27 \le H_1 \le 200$ m: $P_{1nt} = 5$

$200 \le H_1 \le 350$ m: $P_{1nt} = 0.0071 \times H_1^{1.2386}$

- for **Kaplan** turbines:

$8 \le H_1 \le 50$ m: $P_{1nt} = 5$

$50 \le H_1 \le 70$ m: $P_{1nt} = 0.0016 \times H_1^{2.06}$

- for **Bulb** turbines:

$4 \le H_1 \le 23$ m: $P_{1nt} = 5$

where:

P’	initial installed capacity, in MW;
N_g	number of generating units;
H_1	maximum net head, in m;
η_g	mean generator output; and
P_{1nt}	minimum unit capacity of the turbine for the available head, in MW (Eletrosul, 1996).

The **capacity of a generating unit**, P_1 (MW), is given by:

$$P_1 = k_p \times \text{int}\left(\frac{P'_1}{k_p} + 0.5\right)$$

where:

k_p	for
0.1	$P_1' \le 10$MW
0.5	$10 < P_1' \le 80$MW
1.0	$P_1' > 80$MW

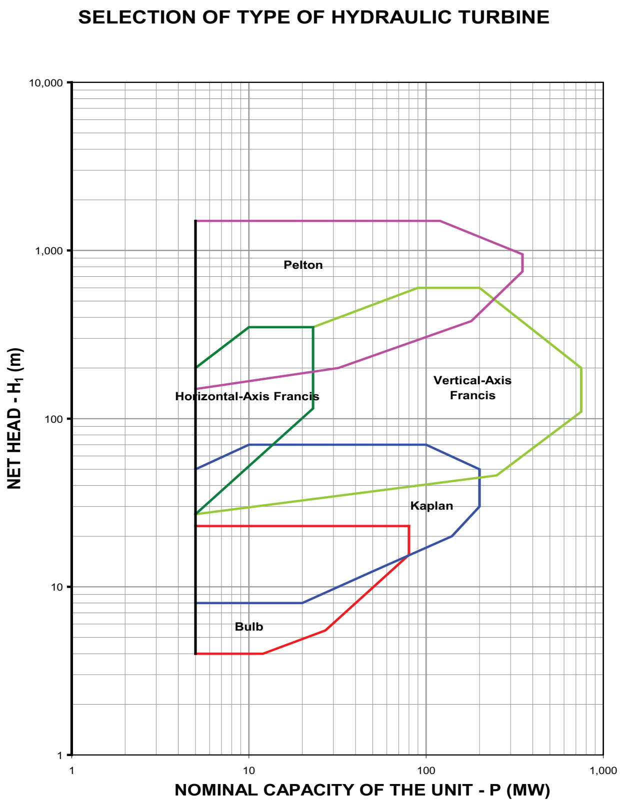
where:

P_1'	initial capacity of a generating unit, in MW; and
k_p	rounding coefficient.

The **installed capacity**, P (MW), is given by:

$$P = P_1 \times N_g$$

P_1	capacity of a generating unit, in MW; and
N_g	number of generating units.



Graph 5.7.2.01 – Selection of type of hydraulic turbine.

DIMENSIONS OF POWERHOUSES EQUIPPED WITH PELTON TURBINES

The other **information required for dimensioning purposes** is:

- space between the generating units, to be established by the design engineer, d_1 in m; and
- space both upstream and downstream from the generating unit, to be established by the design engineer, d_2, in m.

The main **information required for quantification purposes** is:

- mean elevation of the land where the powerhouse will stand, El_{te}, for surface powerhouses;
- mean thickness of the soil in the powerhouse area, e_{te} in m, for surface powerhouses;
- maximum water level in the tailrace canal, NA_{xfu}, for surface powerhouses;
- volume of surface rock excavation below the elevation of the assembly area, to be calculated from the design, V_{rcf} in m^3, for surface powerhouses;
- length of foundation to be treated, L_{tf} in m; and
- volume of concrete, V_{ccf} in m^3.

Velocities

The **specific initial velocity**, n'_s, is obtained from Graph 5.7.2.02 as a function of the maximum net head or from the equivalent expressions (Eletrosul, 1996):

For m: $150 \leq H_1 \leq 1500$ m: $n'_s = 0.01036 \times (2560 - H_1) \times j^{0.5}$

for:

j	maximum flow for each turbine (m³/s)
1	$Q_1 < 3.0$
2	$3.0 \leq Q_1 < 7.0$
3	$7.0 \leq Q_1 < 10.0$
4	$10.0 \leq Q_1 < 14.0$
5	$14.0 \leq Q_1 < 20.0$
6	$Q_1 \geq 20.0$

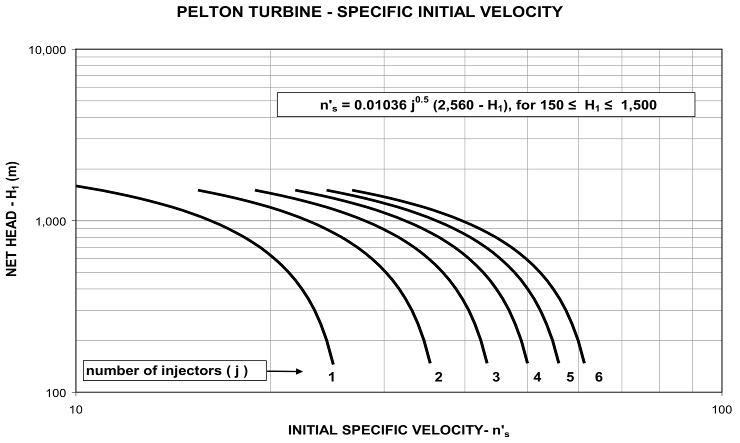
$$Q_t = \frac{10^6 \times P_1}{k \times H_1}$$

where:

$$k = \rho \times g \times \eta_{t1} \times \eta_{g1} \qquad \eta_{t1} = 0.89 \text{ and } \eta_{g1} = 0.95$$

where:

H_1	maximum net head, in m;
j	number of injectors;
Q_1	maximum flow for each turbine, in m³/s;
P_1	capacity of one generating unit, in MW;
k	coefficient;
ρ	1000 kg/m³ – specific mass of water;
η_{t1}	turbine output at maximum net head;
η_{g1}	generator output at maximum net head; and
g	9.81 m/s² – acceleration due to gravity.



Graph 5.7.2.02 – Initial Specific Velocity.

The position of the turbine axis is given by the table below:

position	maximum flow for each turbine (m³/s)
horizontal	$Q_1 < 7.0$
vertical	$Q_1 \geq 7.0$

where:

Q_1	maximum flow for each turbine, in m³/s.

The **initial velocity**, n' (rpm), is given by:

$$n' = \frac{n'_{sj} \times H_1^{1.25}}{P_{1tj}^{0.5}}$$

where:

$$n'_{sj} = \frac{n'_{sj}}{j^{0.5}} \text{ and } P_{1tj} = \frac{1000 \times P_1}{\eta_g \times j}$$

where:

n'_{sj}	specific initial velocity per injector;
H_1	maximum net head, in m;
P_{1tj}	capacity per injector in the turbine, in kW;
n'_s	specific initial velocity;
P_1	capacity of one generating unit, in MW;
η_g	generator output at maximum net head; and
j	number of injectors.

The **number of generator poles**, p, is obtained from Table 5.7.2.01, as a function of the initial velocity, or from the equivalent expressions:

For $n' \geq 1.2 \times f$: $p = 2 \times \text{int}\left(120 \times \frac{f}{n'} \times \frac{1}{2} + 0.5\right)$

without using 54, 74 and 94

For $n' < 1.2 \times f$: $p = 4 \times \text{int}\left(120 \times \frac{f}{n'} \times \frac{1}{4} + 0.5\right)$

where:

f	frequency of the electricity system, in Hz;
n'	initial velocity, in rpm; and
int(x)	function that returns the integer part of x.

Table 5.7.2.01 – Defining Synchronous Velocity

No. of generator poles	Synchronous Velocity		No. of generator poles	Synchronous Velocity	
	50 Hz	60 Hz		50 Hz	60 Hz
6	1000	1200	60	100.0	120.0
8	750.0	900.0	62	96.8	116.1
10	600.0	720.0	64	93.75	112.5
12	500.0	600.0	66	90.91	109.09
14	428.57	514.29	68	88.24	105.88
16	375.0	450.0	70	85.71	102.86
18	333.33	400.0	72	83.33	100.0
20	300.0	360.0	76	78.95	94.74
22	272.73	327.27	78	76.92	92.31
24	250.0	300.0	80	75.00	90
26	230.77	276.92	82	73.17	87.80
28	214.29	257.14	84	71.43	85.71
30	200.0	240.0	86	69.77	83.72
32	187.50	225.0	88	68.18	81.82
34	176.47	211.8	90	66.67	80.0
36	166.67	200.0	92	65.22	78.26
38	157.89	189.47	96	62.50	75.0
40	150.0	180.0	98	61.2	73.5
42	142.86	171.43	100	60.00	72.0
44	136.36	163.64	104	57.69	69.23
46	130.43	156.52	108	55.56	66.67
48	125.0	150.0	112	53.57	64.29
50	120.0	144.0	116	51.72	62.07
52	115.38	138.46	120	50.0	60.0
56	107.14	128.57	124	48.39	58.06
58	103.45	124.14	128	46.88	56.25
60	100.0	120.0	132	45.45	54.55

Notes:

- it is advisable to consult generator manufacturers before deciding on the number of poles highlighted in bold;
- for vertical-axis Francis turbines with a maximum unit turbine flow greater than 20 m^3/s and for all other turbine applications, if the initial velocity is less than 300 rpm for a system operating at 60 Hz, or less than 250 rpm for 50 Hz, **select** the number of poles corresponding to the **synchronous velocity that is immediately higher**;
- for vertical-axis Francis turbines with a maximum unit turbine flow greater than 20 m^3/s and for Pelton turbines, if the initial velocity is 300 rpm or higher for systems operating at 60 Hz or 250 rpm for 50 Hz, **select** the number of poles corresponding to the **synchronous velocity that is immediately lower** when the calculated velocity is between the synchronous velocity immediately below and the velocity corresponding to 75% of the difference between the synchronous velocity immediately above and the synchronous velocity immediately below plus the lowest synchronous velocity. From this point on, **select** the number of poles corresponding to the **synchronous velocity that is immediately higher**;
- for vertical-axis Francis turbines with a maximum unit turbine flow that is 20 m^3/s or lower, and for horizontal-axis Francis turbines, **select** the number of poles corresponding to the **synchronous velocity that is immediately lower** than the velocity calculated.

The **synchronous velocity**, n (rpm), is given by:

$$n = 120 \times \frac{f}{p}$$

where:

F	frequency of the electricity system, in Hz; and
P	number of generator poles.

The **specific velocity per injector**, n_{sj}, is given by:

$$n_{sj} = \frac{n_s}{j^{0.5}}$$

where: $n_s = n \times H_1^{-1.25} \times P_{1tj}^{0.5}$

where:

n_s	specific velocity;
J	number of injectors;
N	synchronous velocity, in rpm;
H_1	maximum net head, in m; and
P_{1tj}	capacity per turbine injector, in kW.

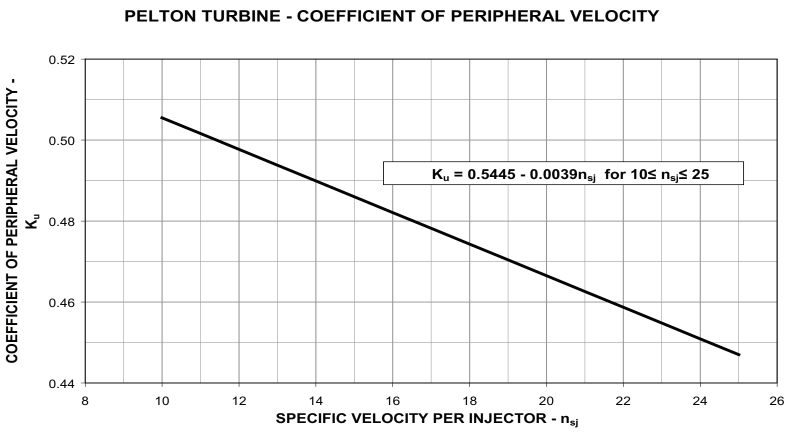
Diameter and position of the turbine rotor

The **coefficient of peripheral velocity**, K_u, is obtained from Graph 5.7.2.03 as a function of the specific velocity or by the equivalent expression (De Siervo & Lugaresi, 1978):

$$K_u = 0.5445 - 0.0039 \times n_{sj}$$

where:

n_{sj}	specific velocity per injector.



Graph 5.7.2.03 – Coefficient of peripheral velocity.

The **diameter of the center line of the jet**, D_2 (m), is given by:

$$D_2 = 0.01 \times \text{int}\left(84.5 \times K_u \times \frac{H_1^{0.5}}{n} \times \frac{1}{0.01} + 0.5\right)$$

where:

K_u	coefficient of peripheral velocity;
H_1	mean net head, in m; and
n	synchronous velocity.

The **installed elevation** of the turbine axis, $\mathbf{E}_{ld}$ (m) is given by:

$$E_{ld} = \frac{NA_{nfu} + NA_{xfu}}{2}$$

where:

NA_{nfu}	normal water level in the tailrace canal; and
NA_{xfu}	maximum water level in the tailrace canal.

Dimensions of the spiral casing and the draft tube

The **turbine dimensions** are given by the following expressions (Eletrosul, 1996). The dimensions are referred to in Fig. 5.7.2.01 and 5.7.2.02.

$D_3 = (1.028 + 0.0137 \times n_{sj}) \times D_2$

$L = 0.78 + 2.06 \times D_3$

$F = 1.09 + 0.71 \times L$

$G = 0.196 + 0.376 \times L$

$H = 0.62 + 0.513 \times L$

$I = 1.28 + 0.37 \times L$

$B = 0.595 + 0.694 \times L$

$C = 0.362 + 0.68 \times L$

$D = -0.219 + 0.70 \times L$

$E = 0.43 + 0.70 \times L$

where:

D_3, A, B, C, D, E	turbine dimensions, in m;
F, G, H, I, L	dimensions of steel-lined chamber, in m; and
D_2	diameter of center line of jet, in m.

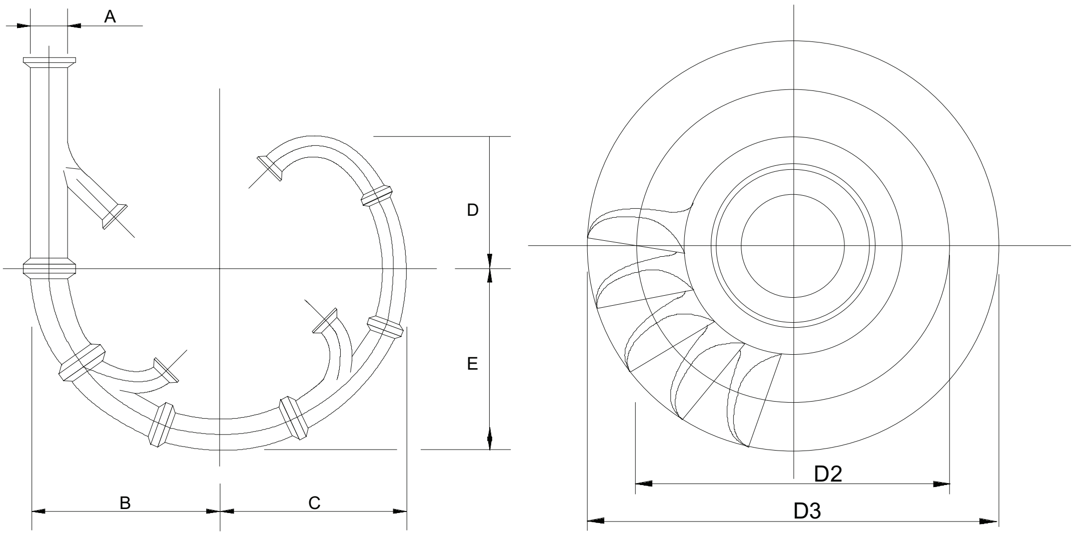


Fig. 5.7.2.01 – Plan of nozzles and rotor – Pelton Turbine.

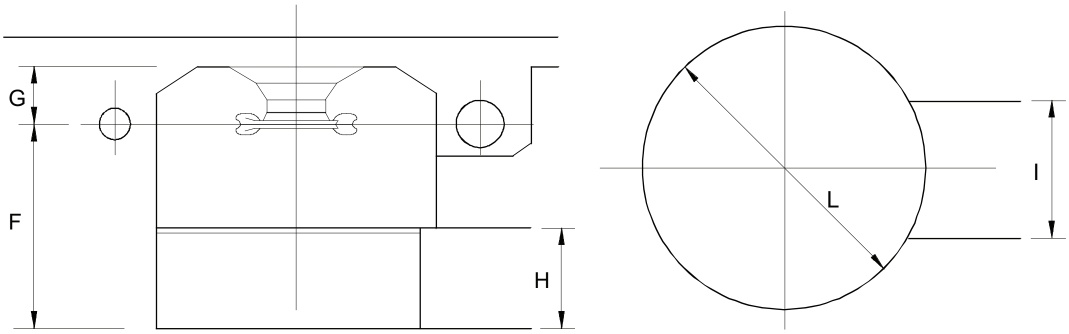


Fig. 5.7.2.02 – Plan and cross-section of the steel-lined chamber – Pelton Turbine.

Dimensions of the powerhouse

The **width of a block of the unit** of the powerhouse (perpendicular to flow), B_{1cf} (m), is given by:

$$B_{1cf} = B + C + d_1$$

where:

B, C	dimensions of the nozzle, in m; and
d_1	space between generating units as defined by the design engineer, in m.

The **total width of the powerhouse**, B_{cf} (m), excluding the assembly area, is given by:

$$B_{cf} = N_g \times B_{1cf} + 2.0$$

where:

N_g	number of generating units; and
B_{1cf}	width of a block of the unit of the powerhouse, in m.

The **width of the equipment assembly area**, B_{am} (m), is given by:

for: $Ng \leq 3$: $B_{am} = 1.5 \times B_{1cf}$

for: $Ng > 3$: $B_{am} = 2.25 \times B_{1cf}$

where:

B_{1cf}	width of a block of the unit of the powerhouse, in m; and
N_g	number of generating units.

The **length of the superstructure**, L_{cs} (m), is given by:

$L_{cs} = D + E + d_2$

where:

D, E	dimensions of the injector nozzle, in m; and
d_2	spacing in the direction of flow upstream and downstream from the generating unit defined by the design engineer, in m.

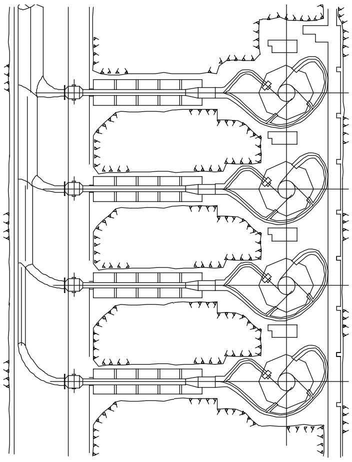


Fig. 5.7.2.03 – Plan of the valve house, powerhouse and assembly area – Pelton turbine.

Common excavation (account .11.13.00.12.10)

The **common excavation** volume, V_{tcf} (m³), for a **surface powerhouse** is given by:

$V_{tcf} = (B_{cf} + B_{am} + 2 \times B_{1cf} + 2 \times 0.6 \times h_r) \times L_{cs} \times e_{te}$

where: $h_r = El_{te} - e_{te} - (NA_{xfu} + 1.5)$

where:

B_{cf}	width of the powerhouse, in m;
B_{am}	width of the assembly area, in m;
B_{1cf}	width of a block of the unit of the powerhouse, in m;
h_r	mean depth of excavation in rock above the elevation of assembly area, in m;
L_{cs}	length of the superstructure, in m;
e_{te}	mean thickness of the layer of soil in the powerhouse area, in m;
El_{te}	mean elevation of the land in the powerhouse area; and
NA_{xfu}	maximum water level in the tailrace canal.

The **common excavation** volume, V_{tcf} (m³), for **underground powerhouses** is given by:

$V_{tcf} = 0$

The unit price of common excavation is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and

- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface rock excavation (account .11.13.00.12.11)

The volume of **excavation in rock**, V_{rcf} (m^3), for a **surface powerhouse** must be calculated from the design.

The unit price of excavation in rock is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

The price should be raised for projects in the Amazon region.

Underground excavation in rock (account .11.13.00.12.12)

In the absence of more accurate information, the volume of **underground excavation in rock**, V_{scf} (m^3), for **underground powerhouses** is given by:

$$V_{scf} = B_{cf} \times L_{cs} \times 2 \times L_{cs} + B_{am} \times L_{cs} \times L_{cs}$$

where:

B_{cf}	width of the powerhouse (perpendicular to flow), in m;
L_{cs}	length (direction of flow) of the powerhouse superstructure, in m; and
B_{am}	width of the assembly area, in m.

The unit price for underground excavation in rock, P_{us} (R$/$m^3$) (from December 2006 database) can be obtained from the expression below (or Graph B33, annex B, as a function of the area of the excavation section) and is applicable for projects in the south, southeast, central west and northeast regions of Brazil. This price per cubic meter measured using the project line includes excavation, loading, transportation up to 1.5 km and unloading:

valid for $4 \leq A_{se} \leq 300$: $P_{us} = 474.08 \times A_{se}^{-0.3987}$

for: $A_{se} = L_{cs}^2$

where:

A_{se}	area of the excavated section, in m^2; and
L_{cs}	length of the powerhouse superstructure, in m.

A detailed assessment should be made for any situation where underground excavation will make up a major portion of the overall budget, primarily to check the regional geographal conditions. Generally speaking, for those situations where the geological conditions are found to be poor, in the absence of more accurate information, the price could be up to 30% higher, depending on the judgement of the cost engineer.

The price should be raised for projects in the Amazon region.

Foundation Cleaning and Treatment (account .11.13.00.13)

The **area of the foundation to be cleaned**, A_{lf} (m^2), is given by:

$$A_{lf} = (B_{cf} + B_{am}) \times L_{cs}$$

where:

B_{cf}	width of the powerhouse, in m;
B_{am}	width of the assembly area, in m; and
L_{cs}	length of the powerhouse superstructure, in m.

The depth of the grout holes for the **foundation treatment** should be determined from the project design.

The unit prices for foundation cleaning and treatment services, expressed in Brazilian Reais (valid for the December 2006 database), can be used for projects in the south, southeast, central west and northeast regions of Brazil. They include the execution of the work, supply of inputs and equipment, and depend on the kind of surface and equipment to be used. The unit prices are:

- cleaning of rock surface: 39.70/m^2
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

The price should be raised for projects in the Amazon region.

Concrete (account .11.13.00.14)

The **volume of concrete** should be determined from the project design.

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the construction site, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the construction site, storage, preparation and installation.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of the powerhouse volume (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- concrete for infrastructure and end walls: 214.00/m^3
- dental concrete: 113.00/m^3
- concrete for the superstructure: 214.00/m^3

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

The price should be raised for projects in the Amazon region.

Installations and final works (account .11.13.00.15)

The **cost of installations and final works**, C_{ia} (R$), which covers all services for the final work on the powerhouse, such as dividing walls, coatings, installations, door and window frames, glass windows, etc., is obtained as a global cost using the expression below (or Graph B 20, as a function of installed capacity). It is valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996a):

Valid for 30 ≤ P ≤ 1,450 MW: $C_{ia} = 6{,}150 \times P^{1+\frac{15.34}{P}}$

where:

P	installed capacity, in MW.

Land developments in the plant area (account .11.12)

The **cost of land developments in the plant area**, C_{bau} (R$), which encompasses building the internal access roads to the different structures, guard houses and perimeter walls, landscaping, and others, is obtained as a global cost using the expression below (or Graph B 19, annex B, as a function of installed capacity). It is valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996a):

Valid for 30 ≤ P ≤ 1,450 MW: $C_{bau} = 1{,}565 + \left(\frac{772{,}973}{P}\right)$

where:

P	installed capacity, in MW.

Operators' Village (account .11.14)

This cost is included in the workers' camp account (account .17.21).

Turbines (account .13.13.00.23.28)

The **acquisition cost of each Pelton turbine** can be obtained from manufacturers.

Stoplogs for the draft tube (account .13.13.00.23.16)

There is no draft tube.

Generators (account .13.13.00.23.29)

The **acquisition cost of each horizontal-axis generator**, C_{gh} (R$), or **vertical-axis generator**, C_{gv} (R$), including the generator and associated equipment – FOB cost, cost of equipment purchase excluding transportation and insurance, assembly and testing and provisions for taxes payable, depending on the current tax regime – can be obtained from the expressions below (or from Graphs B 14 or B 16, annex B, as a function of the generator's capacity and its synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil.

- for horizontal-axis generators:

  valid for 0.0004 ≤ λ ≤ 0.0483: $C_{gh} = 29580(\lambda)^{0.6323}$

- for vertical-axis generators:

  valid for 0.0329 ≤ λ ≤ 1.9834: $C_{gv} = 42280(\lambda)^{0.6298}$

for: $\lambda = \frac{P_2}{n}$ and $P_2 = \frac{P_1}{f_p}$

where:

P_2	generator capacity, in MVA;
λ	magnetic torque, in MVA/rpm;
n	synchronous velocity, in rpm;
P_1	capacity of one generating unit, in MW; and
f_p	power factor.

The following percentages must be added to the FOB price:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for taxes and charges payable on the equipment.

Draft Tube Gantry Crane (account .13.13.00.23.20)

There is no draft tube.

Auxiliary Electrical Equipment (account .14.00.00.23)

The **acquisition cost of the auxiliary electrical equipment** should be taken as 18% of the overall cost of account .13 – Turbines and Generators.

Overhead Crane (account .15.13.00.23.20)

The cargo handling system is generally an indoor overhead crane. The **acquisition cost of the crane**, C_{prh} (R$), – FOB price – is obtained from the expression below (or from Graph B 17, annex B, as a function of the ratio between the generator capacity and the synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $68.9 \leq z \leq 4{,}582$: $C_{prv} = 25.12 \times z^{0.6961}$

for: $z = 1000 \times \dfrac{P_2}{n}$

where:

z	parameter, in kVA/rpm;
P_2	generator capacity, in MVA; and
n	synchronous velocity, in rpm.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB price.

Miscellaneous Equipment (account .15.00.00.23.31)

The **acquisition cost for miscellaneous equipment** should be taken as 6% of the overall cost of account .13 – Turbines and Generators.

POWERHOUSE EQUIPPED WITH VERTICAL-AXIS FRANCIS TURBINES

The main **information required for quantification purposes** is:

- mean elevation of the land in the powerhouse area, El_{te}, in m;
- mean thickness of the layer of soil in the powerhouse area, e_{te}, in m;
- volume of concrete for any additional excavation needed to make up for faults in the foundation, V_{cd}, in m^3;
- volume of concrete resulting from alterations to the project design so that the maximum water level in the tailrace canal is higher than the elevation of the generator floor, V_{cn}, in m^3;
- type of powerhouse; and
- maximum water level in the tailrace canal, NA_{xfu}, from item 5.1.2.

Velocities

The **specific initial velocity**, n'_s, is obtained from Graph 5.7.2.04 as a function of maximum net head or from the equivalent expressions (Eletrosul, 1996):

for $27 \leq H_1 \leq 358.06$ m: $n'_s = 95.2 \times \ln\left(\frac{1006}{H_1}\right)$ and for $358.06 < H_1 \leq 600$ m: $n'_s = 2772 \times H_1^{-0.568}$

for:

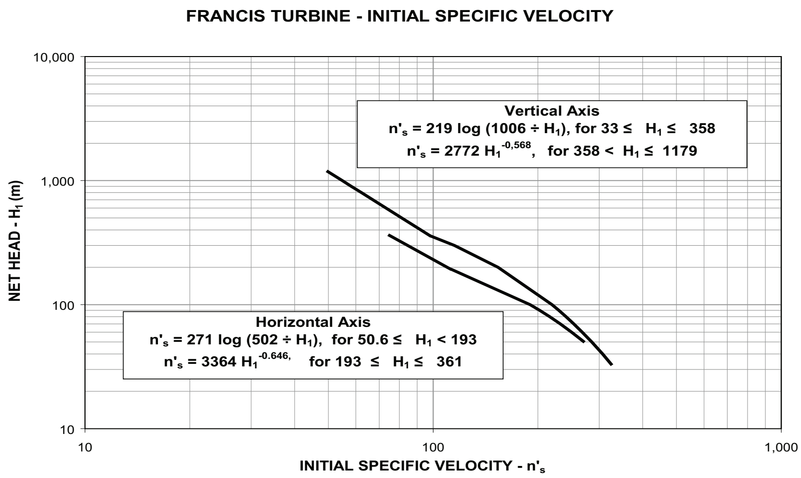
$$Q_1 = \frac{10^6 \times P_1}{k \times H_1} \qquad k = \rho \times g \times \eta_{t1} \times \eta_{g1}$$

$$\eta_{t1} = 0.856 \times Q_1^{0.013} \qquad \eta_{g1} = 0.92 \times P_2^{0.01}$$

$$P_2 = \frac{P_1}{f_p}$$

where:

Q_1	maximum turbine flow of each turbine, in m^3/s;
H_1	maximum net head, in m;
P_1	capacity of one generating unit, in MW;
k	coefficient;
ρ	1000 kg/m^3 – specific mass of water;
η_{t1}	turbine output at the maximum net head;
η_{g1}	generator output at the maximum net head;
g	9.81 m/s^2 – acceleration due to gravity;
P_2	generator capacity, in MVA; and
f_p	power factor.



Graph 5.7.2.04 – Initial Specific Velocity – Francis Turbines.

The **initial velocity**, n' (rpm), is given by:

$$n' = n'_s \times H_1^{1.25} \times P_{1t}^{-0.5}$$

for: $P_{1t} = \dfrac{10^3 \times P_1}{\eta_g}$

where:

n'_s	specific initial velocity;
H_1	maximum net head, in m;
P_{1t}	unit capacity of the turbine, in kW;
P_1	capacity of one generating unit, in MW; and
η_g	generator output at the maximum net head.

The **number of generator poles**, p, is obtained from Table 5.7.2.01, as a function of the initial velocity and the maximum unit turbine flow, or from the equivalent expressions:

for $n' \geq 5 \times f$: $p = 2 \times \text{int}\left(120 \times \frac{f}{n'} \times \frac{1}{2} + 0.778\right)$

for $1.2 \times f \leq n' < 5 \times f$: $p = 2 \times \text{int}\left(120 \times \frac{f}{n'} \times \frac{1}{2}\right)$

without using 54, 74 and 94

for $n' < 1.2 \times f$: $p = 4 \times \text{int}\left(120 \times \frac{f}{n'} \times \frac{1}{4}\right)$

where:

f	frequency of the electricity system, in Hz;
n'	initial velocity, in rpm; and
int(x)	function that returns the integer part of x.

Synchronous velocity, n (rpm), is given by:

$$n = 120 \times \frac{f}{p}$$

where:

f	frequency of the electricity system, in Hz; and
p	number of generator poles.

Specific velocity, n_s, is given by:

$$n_s = n \times H_1^{-1.25} \times P_{1t}^{0.5}$$

where:

n	synchronous velocity, in rpm;
H_1	maximum net head, in m; and
P_{1t}	unit capacity of the turbine, in kW.

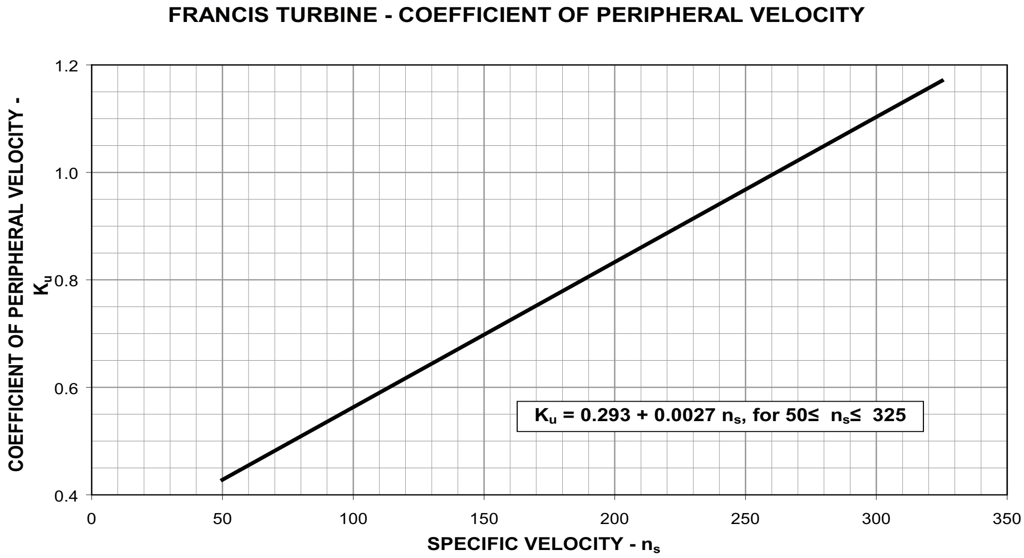
Diameter and position of the turbine rotor

The **coefficient of peripheral velocity coefficient of peripheral velocity**, K_u, is obtained from Graph 5.7.2.05 as a function of the specific velocity or from the equivalent expression (Lugaresi and Massa, 1987):

$$K_u = 0.293 + 0.0027 \times n_s$$

where:

n_s	specific velocity.



Graph 5.7.2.05 – Coefficient of peripheral velocity – Francis Turbines (PCE, 2007).

The diameter of the **turbine rotor**, D_3 (m), is given by:

$$D_3 = 0.01 \times \text{int}\left(84.5 \times K_u \times \frac{H_1^{0.5}}{n} \times \frac{1}{0.01} + 0.5\right)$$

where:

K_u	coefficient of peripheral velocity;
H_1	maximum net head, in m; and
n	synchronous velocity, in rpm.

The **suction head**, h_s (m), is given by: $h_s = K - \sigma \times H_1$

for: $K = 10.33 - 0.0012 \times NA_{fu} - 0.013 \times T$

$\sigma = 7.54 \times 10^{-5} \times n_s^{1.41}$

where:

K	variable, as a function of atmospheric pressure and steam pressure;
σ	Thoma coefficient (Siervo and Leva, 1976);
H_1	maximum net head, in m;
Na_{fu}	normal water level in the tailrace canal;
T	mean water temperature in the summer, in °C; and
n_s	specific velocity.

The **installation elevation**, El_d, is given by: $El_d = NA_{nfu} + h_s$

where:

NA_{nfu}	minimum water level downstream; and
h_s	suction head, in m.

Dimensions of the turbine, spiral casing, generator and draft tube

The **dimensions of horizontal-axis Francis turbines** and **generators** are given by the following expressions (De Siervo and De Leva, 1976). The dimensions refer to Figures 5.7.2.04 and 5.7.2.05.

$$A = D_3 \times \left(1.2 - \frac{19.56}{n_s}\right) \qquad B = D_3 \times \left(1.1 + \frac{54.80}{n_s}\right)$$

$$C = D_3 \times \left(1.32 + \frac{49.25}{n_s}\right) \qquad D = D_3 \times \left(1.50 + \frac{48.80}{n_s}\right)$$

$$R = 1.3 \times D_3 \qquad S = \frac{D_3 \times n_s}{-9.28 + 0.25 \times n_s}$$

$$Z = D_3 \times \left(2.63 + \frac{33.8}{n_s}\right)$$

for $Z \times R \geq 30\ m^2$: $U = 1.7$ m and $N_{vs} = 2$

for $Z \times R < 30\ m^2$: $U = 0$ m and $N_{vs} = 1$

$$Y = H'_2 + N$$

for $n_s \leq 110$: $H'_2 = D_3 \times \left(-0.05 + \frac{42}{n_s}\right)$

for $n_s > 110$: $H'_2 = \frac{D_3}{3.16 - 0.0013 \times n_s}$

for $n_s \leq 240$: $N = D_3 \times \left(1.54 + \frac{203.5}{n_s}\right)$

for $n_s > 240$: $N = 2.4 \times D_3$

where:

A, B, C, D, H'_2	turbine dimensions, in m;
N	height of the draft tube per se, in m;
R	height of the draft tube outlet, in m (Eletrosul, 1996);
S	length of the draft tube, in m;
U	thickness of the draft tube pillar, in m (Eletrosul, 1996);
Y	height from the draft tube to the center of the distributor, in m (Eletrosul, 1996);
Z	width of the draft tube, in m;
N_{vs}	number of openings for each draft tube;
D_3	diameter of the turbine rotor outlet, in m; and
n_s	specific velocity.

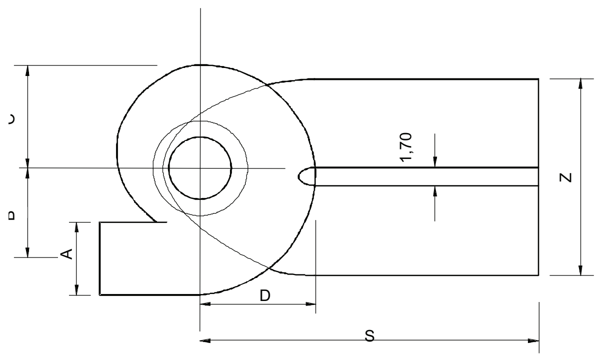


Fig. 5.7.2.04 – Plan of the spiral casing and draft tube – vertical-axis Francis turbine.

The **estimated diameter of the generator housing**, D_{pg} (m), is given by (COPEL, 1977):

$$D_{pg} = 9.0 \times \left(\frac{1000 \times P_1}{f_p \times n^2}\right)^{0.2}$$

where:

P_1	capacity of one generating unit, in MW;
f_p	power factor; and
n	synchronous velocity, in rpm.

Dimensions of the powerhouse

The **width of a block of the unit** of the powerhouse (perpendicular to flow), B_{1cf} (m), is given by:

$$B_{1cf} = \frac{A}{2} + B + C + 2 \times (1.3 + 0.1 \times D_3)$$

where:

A, B, C	dimensions of the spiral casing, in m; and
D_3	diameter of the turbine rotor, in m.

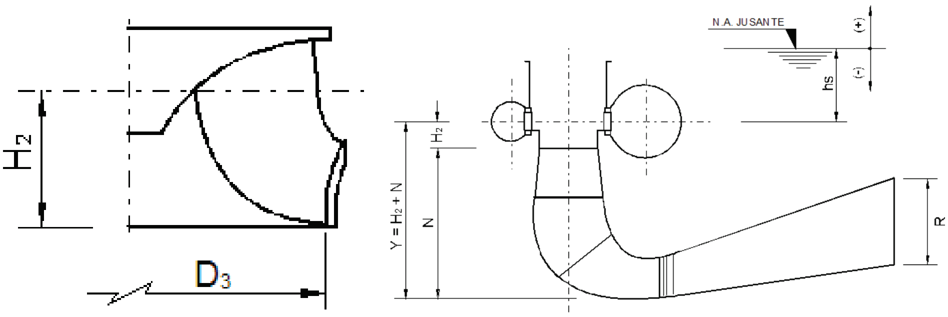


Fig. 5.7.2.05 – Cross-section of the spiral casing and draft tube – vertical-axis Francis turbine.

The **total width of the powerhouse**, B_{cf} (m), excluding the assembly area, is given by:

$$B_{cf} = N_g \times B_{1cf} + 2.0$$

where:

N_g	number of generating units; and
B_{1cf}	width of a block of the unit of the powerhouse, in m.

The **width of the equipment assembly area**, B_{am} (m), is given by:

for $N_g \leq 3$: $B_{am} = 1.5 \times B_{1cf}$

for $N_g > 3$: $B_{am} = 2.25 \times B_{1cf}$

where:

B_{1cf}	width of a block of the unit of the powerhouse, in m; and
N_g	number of generating units.

The **length of the superstructure**, L_{cs} (m), is given by:

$$L_{cs} = d_1 + d_2$$

for:

$$d_1 = \frac{D_{pg}}{2} + 2.1 + 0.2 \times D_3 \qquad d_2 = D + 2.1 + 0.2 \times D_3$$

where:

d_1	distance between the outside face of the upstream wall and the center line of the generating units, in m;
d_2	distance between the center line of the generating units and the outside face of the downstream wall, in m;
D_{pg}	diameter of the generator housing, in m;
D_3	diameter of the turbine rotor, in m; and
D	turbine dimensions, in m.

The **length of the powerhouse**, L_{cf} (m), is given by:

$$L_{cf} = d_1 + S$$

where:

d_1	distance between the outside face of the upstream wall and the center line of the generating units, in m; and
S	length of the draft tube, in m.

The **length of the equipment assembly area**, L_{am} (m), is given by:

$$L_{am} = L_{cs}$$

where:

L_{cs}	length of the superstructure, in m

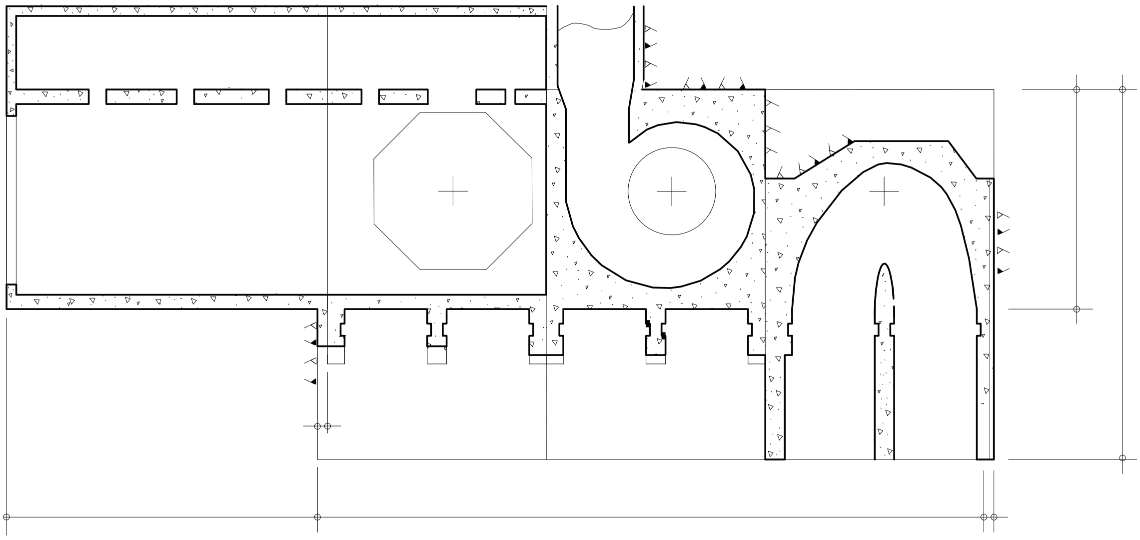


Fig. 5.7.2.06 – Plan of the powerhouse and assembly area for vertical-axis Francis turbines.

Common excavation (account .11.13.00.12.10)

The common excavation volume, V_{tcf} (m³), for **surface powerhouses** is given by:

$$V_{tcf} = (B_{cf} + B_{am} + 2 \times B_{1cf} + 2 \times 0.6 \times h_r) \times L_{cf} \times e_{te}$$

for: $h_r = El_{te} - e_{te} - (NA_{xfu} + 1.5)$

where:

B_{cf}	width of the powerhouse, in m;
B_{am}	width of the assembly area, in m;
B_{1cf}	width of a block of the unit of the powerhouse, in m;
h_r	mean depth of excavation in rock above the elevation of the assembly area, in m;
L_{cf}	length of the powerhouse, in m;
e_{te}	mean thickness of the layer of soil in the powerhouse area, in m;
El_{te}	mean elevation of the land in the powerhouse area; and
NA_{xfu}	maximum water level in the tailrace canal.

The common excavation volume, V_{tcf} (m³), for **underground powerhouses** is given by:

$$V_{tcf} = 0$$

The unit price of common excavation is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the

vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .11.13.00.12.11)

The volume of **excavation in rock**, V_{rcf} (m³), for a **surface powerhouse** is given by.

$$V_{rcf} = V_{re} + V_{rp} + V_{rd}$$

and is valid for $1.5 \leq D_3 \leq 8.0$ m:

$$V_{re} = (B_{cf} + B_{am} + 2 \times B_{1cf} + 0.6 \times h_r) \times L_{cf} \times h_r$$

$$V_{rp} = B_{cf} \times L_{cf} \times (NA_{xfu} + 1.5 - El_d)$$

$$V_{rd} = N_g \times 700 \times e^{0.54 \times D_3}$$

where:

V_{re}	volume of excavation in rock above the elevation of the assembly area, in m³;
V_{rp}	volume of excavation in rock between the elevation of the assembly area and the elevation of the center line of the turbine distributor, in m³;
V_{rd}	volume of excavation in rock below the center line of the turbine distributor, in m³ (COPEL, 1977);
B_{cf}	width of the powerhouse, in m;
B_{am}	width of the assembly area, in m;
B_{1cf}	width of a block of the unit of the powerhouse, in m;
h_r	mean depth of excavation in rock above the elevation of the assembly area, in m;
L_{cf}	length of the powerhouse, in m;
NA_{xfu}	maximum water level in the tailrace canal;
El_d	elevation of the center line of the turbine distributor;
N_g	number of generating units; and
D_3	diameter of the turbine rotor, in m.

The **volume of excavation in rock**, V_{rcf} (m³), for an **underground powerhouse** is given by:

$$V_{rcf} = 0$$

The unit price of excavation in rock is R$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Underground excavation in rock (account .11.13.00.12.12)

The volume of **underground excavation in rock**, V_{scf} (m^3), for an **underground powerhouse** is given by the following expression, in the absence of more accurate information:

$$V_{scf} = B_{cf} \times L_{cs} \times 2 \times L_{cs} + B_{am} \times L_{cs} \times L_{cs}$$

where:

B_{cf}	width of the powerhouse, in m;
L_{cs}	length of the powerhouse superstructure, in m; and
B_{am}	width of the assembly area, in m.

The unit price for underground excavation in rock, P_{us} (R\$/m^3) (from December 2006 database) can be obtained from the expression below (or Graph B33, annex B, as a function of the area of the excavation section) and is applicable for projects in the south, southeast, central west and northeast regions of Brazil. This price per cubic meter measured using the project line includes excavation, loading, transportation up to 1.5 km and unloading:

valid for $4 \leq A_{se} \leq 300$: $P_{us} = 474.08 \times A_{se}^{-0.3987}$

for: $A_{se} = L_{cs}^2$

where:

A_{se}	area of the excavation section, in m^2; and
L_{cs}	length of the powerhouse superstructure, in m.

A detailed assessment should be made for any situation where underground excavation will make up a major portion of the overall budget, primarily to check the regional geographal conditions. Generally speaking, for those situations where the geological conditions are found to be poor, in the absence of more accurate information, the price could rise by up to 30%, depending on the judgement of the cost engineer.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .11.13.00.13)

The **area of foundation to be cleaned**, A_{lf} (m^2), for the **powerhouse** is given by:

$$A_{lf} = B_{cf} \times L_{cf} + B_{am} \times L_{cs}$$

where:

B_{cf}	width of the powerhouse, in m;
L_{cf}	length of the powerhouse, in m;
B_{am}	width of the assembly area, in m; and
L_{cs}	length of the powerhouse superstructure, in m.

The **length of the grout holes**, L_{tf} (m), for treating the foundations for the **powerhouse,** is given by:

$$L_{tf} = \frac{B_{cf}}{3} \times L_{1tf} \qquad L_{1tf} = 1.5 \times (NA_{xfu} - El_d + Y) \leq 40 \text{ m}$$

where:

B_{cf}	width of the powerhouse, in m;
L_{1tf}	length of one grout hole, in m;

NA_{xfu}	maximum water level in the tailrace canal;
El_d	elevation of the center line of the turbine distributor;
Y	height from the draft tube to the center of the distributor, in m; and
3.0	spacing between the grout holes, in m.

For **underground powerhouses**, a grid of **rock anchors** of length, L_{pr} (m), should be used to fix the rock, given by the expression:

$$L_{pr} = 4.0 \times L_{cs} \times (B_{cf} + B_{am}) + 3.5 \times L_{cs} \times (2 \times B_{cf} + B_{am} + 2 \times L_{cs})$$

where:

L_{cs}	length of the powerhouse superstructure, in m;
B_{cf}	width of the powerhouse, in m; and
B_{am}	width of the assembly area, in m.

The unit prices for foundation cleaning and treatment services – expressed in Brazilian Reais (valid for the December 2006 database) can be used for projects in the south, southeast, central west and northeast regions of Brazil. They include the execution of the work, supply of inputs and equipment, depending on the kind of surface, and of the equipment to be used. The unit prices are:

- cleaning of the rock surface: 39.70/m^2
- rotary percussive drilling: 168/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .11.13.00.14)

The volume of **concrete**, V_{ccf} (m^3), for **indoor powerhouses** is given by:

$$V_{ccf} = N_g \text{ x } (V_{cf} + 1.5 \text{ x } V_{cs}) + V_{ce} + V_{cd} + V_{cn} + V_{ca}$$

for:

valid for $1.5 \leq D_3 \leq 8.0$ m: $V_{cf} = 485 \times e^{0.535 \times D_3}$

$V_{cs} = 215 \times e^{0.381 \times D_3}$ $\qquad$ $V_{ce} = 370 \times e^{0.314 \times D_3}$

for $N_g \leq 3$: $V_{ca} = V_{cs}$

for $N_g > 3$: $V_{ca} = 2 \times V_{cs}$

where:

N_g	number of generating units;
V_{cf}	volume of concrete for the infrastructure, in m^3 (COPEL, 1977);
V_{cs}	volume of concrete for the superstructure, in m^3 (COPEL, 1977);
V_{ce}	volume of concrete for a wall at each end, in m^3 (COPEL, 1977);
V_{cd}	volume of concrete for any additional excavation needed to make up for faults in the foundation, in m^3;
V_{cn}	volume of concrete resulting from alterations to the project design so that the maximum water level in the tailrace canal is higher than the elevation of the generator floor, in m^3;
V_{ca}	volume of concrete for the assembly area, in m^3; and
D_3	diameter of the turbine rotor, in m.

The volume of **concrete**, V_{ccf} (m^3), for a **semi-outdoor powerhouse** is given by:

$$V_{ccf} = N_g \times (V_{cf} + V_{cs}) + V_{ce} + V_{cd} + V_{cn} + V_{ca}$$

where:

N_g	number of generating units;
V_{cf}	volume of concrete for the infrastructure, in m^3;
V_{cs}	volume of concrete for the superstructure, in m^3;
V_{ce}	volume of concrete for a wall at each end, in m^3;
V_{cd}	volume of concrete for any additional excavation needed to make up for faults in the foundation, in m^3;
V_{cn}	volume of concrete resulting from alterations to the project design so that the maximum water level in the tailrace canal is higher than the elevation of the generator floor, in m^3;
V_{ca}	volume of concrete for the assembly area, in m^3; and
D_3	diameter of the turbine rotor, in m.

The volume of **concrete**, V_{ccf} (m^3), for an **outdoor powerhouse** is given by:

$$V_{ccf} = N_g \times (V_{cf} + 0.15 \times V_{cs}) + 0.6 \times V_{ce} + V_{cd} + V_{cn} + 0.25 \times V_{ca}$$

where:

N_g	number of generating units;
V_{cf}	volume of concrete for the infrastructure, in m^3;
V_{cs}	volume of concrete for the superstructure, in m^3;
V_{ce}	volume of concrete for a wall at each end, in m^3;
V_{cd}	volume of concrete for any additional excavation needed to make up for faults in the foundation, in m^3;
V_{cn}	volume of concrete resulting from alterations to the project design so that the maximum water level in the tailrace canal is higher than the elevation of the generator floor, in m^3;
V_{ca}	volume of concrete for the assembly area, in m^3; and
D_3	diameter of the turbine rotor, in m.

The volume of **concrete**, V_{ccf} (m^3), for an **underground powerhouse** is given by:

$$V_{ccf} = N_g \times (V_{cf} + 0.5 \times V_{cs}) + 0.6 \times V_{ce} + 0.25 \times V_{ca}$$

where:

N_g	number of generating units;
V_{cf}	volume of concrete for the infrastructure, in m^3;
V_{cs}	volume of concrete for the superstructure, in m^3;
V_{ce}	volume of concrete for a wall at each end, in m^3;
V_{cn}	volume of concrete resulting from alterations to the project design so that the maximum water level in the tailrace canal is higher than the elevation of the generator floor, in m^3;
V_{ca}	volume of concrete for the assembly area, in m^3; and
D_3	diameter of the turbine rotor, in m.

The volume of **shotcrete**, V_{cp} (m^3), for underground powerhouses is given by:

$$V_{cp} = 0.1 \times [(B_{cf} + B_{am}) \times 3 \times L_{cs} + 2 \times L_{cs} \times L_{cs}]$$

where:

B_{cf}	width of the powerhouse, in m;
B_{am}	width of the assembly area, in m; and
L_{cs}	length of the superstructure, in m.

The amounts of **cement and reinforcement steel** are:

	cement (kg/m^3)	reinforcement steel (kg/m^3)
infrastructure	275	50
superstructure	300	100
end wall	250	75
dental concrete	200	0
shotcrete	300	70

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the construction site, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the construction site, storage, preparation and installation.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of the powerhouse volume (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- concrete for the infrastructure and end walls: 214.00/m^3
- dental concrete: 113.00/m^3
- concrete for the superstructure: 214.00/m^3

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Installations and final work (account .11.13.00.15)

The **cost of installations and final works**, C_{ia} (R$), which covers all services for the final work on the powerhouse, such as dividing walls, coatings, installations, door and window frames, glass windows, etc., is obtained as a global cost using the expression below (or Graph B 20, as a function of installed capacity). It is valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996a):

Valid for $30 \leq P \leq 1450$ MW: $C_{ia} = 6{,}150 \times P^{1 + \frac{15.34}{P}}$

where:

P	installed capacity, in MW.

Land developments in the plant area (account .11.12)

The **cost of land developments in the plant area**, C_{bau} (R$), which encompasses building the internal access roads to the different structures, guard house and perimeter walls, landscaping, and others, is obtained as a global cost using the expression below (or Graph B 19, annex B, as a function of installed capacity), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996a):

Valid for $30 \leq P \leq 1450$ MW: $C_{bau} = 1{,}565 + \left(\frac{772{,}973}{P}\right)$

where:

P	installed capacity, in MW.

Operators' Village (account .11.14)

This cost is included in the workers' camp account (account .17.21).

Turbines (account .13.13.00.23.28)

The **acquisition cost of each vertical-axis** Francis turbine, C_{tf} (R$), which includes the electromechanical equipment, parts and materials normally supplied by the manufacturers – FOB cost excluding transportation, insurance, assembly and testing costs and provisions for charges and taxes payable according to the applicable tax legislation – can be obtained from the expression below (or from Graph. B 10, annex B, as a function of the unit capacity of the turbine and the synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil. (Eletrosul, 1996):

valid for $20 \leq z \leq 6000$: $C_{tf} = 0.0011 \times z^2 + 18.162 \times z + 3{,}279.8$

for:

$$z = \frac{P_{1t}}{n}$$

where:

z	parameter, in kW/rpm;
P_{1t}	unit capacity of the turbine, in kW; and
n	synchronous velocity, in rpm.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for the taxes and charges payable on the equipment.

Stoplogs for the draft tube (account .13.13.00.23.16)

The **number of stoplogs**, N_{sl}, is given by the following expressions:

for $N_g \leq 10$: $N_{sl} = 2 \times N_{vs}$

for $N_g > 10$: $N_{sl} = 3 \times N_{vs}$

where:

N_g	number of generating units; and
N_{vs}	number of openings for each draft tube.

The **acquisition cost of each stoplog** for the draft tube, C_{sl} (R$), – FOB cost – can be obtained from the expression below (or from Graph. B 25, annex B, as a function of its dimensions and hydrostatic load), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.16 \leq z \leq 54.5$: $C_{sl} = 72.9 \times z^{0.716}$

for:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000} \qquad H_{cp} = R$$

$$H_x = NA_{xfu} - El_d + Y \qquad B_{cp} = \frac{Z - U}{N_{vs}}$$

where:

z	parameter, in m^4;
B_{cp}	width of the stoplog, in m;
H_{cp}	height of the stoplog, in m;
H_x	maximum hydrostatic load on the sill of the stoplog, in m;
R	height of the opening for the draft tube at the outlet, in m;
NA_{xfu}	maximum water level in the tailrace canal;

El_d	elevation of the center line of the turbine distributor;
Y	height from the draft tube to the center of the distributor, in m;
Z	width of the draft tube, in m;
U	thickness of the draft tube pillar, in m; and
N_{vs}	number of openings for each draft tube.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

The **overall acquisition cost for the fixed parts and parts embedded in the concrete** of the stoplogs for the draft tube, C_{gpf} (R$), – FOB cost – is given by the expression below, valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

$$C_{gpf} = 2 \times N_{vs} \times N_g \times (H_x + 2.0) \times 2{,}084.80$$

where:

N_g	number of generating units.
H_x	maximum hydrostatic load on the sill of the stoplog, in m; and
N_{vs}	number of openings for each draft tube.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

Draft Tube Gantry Crane (account .13.13.00.23.20)

As the **acquisition cost of draft tube gantry crane** is low, it can be ignored at this stage.

Generators (account .13.13.00.23.29)

The **acquisition cost of each vertical-axis generator**, C_{gv} (R$), including the voltage regulator and auxiliary electromechanical equipment – FOB cost – can be obtained from the expressions below (or from Graph. B 16, annex B, as a function of the ratio between the generator capacity and its synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.0329 \le \lambda \le 1.9834$: $C_{gv} = 42280(\lambda)^{0.6298}$

for: $\lambda = \dfrac{P_2}{n}$ and $P_2 = \dfrac{P_1}{f_p}$

where:

P_2	generator capacity, in MVA;
λ	magnetic torque, in MVA/rpm;
n	synchronous velocity, in rpm;
P_1	capacity of one generating unit, in MW; and
f_p	power factor.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for the taxes and charges payable on the equipment.

Auxiliary Electrical Equipment (account .14.00.00.23)

The **acquisition cost of the auxiliary electrical equipment** should be taken as 18% of the overall cost of account .13 – Turbines and Generators.

Bridge and gantry cranes (account .15.13.00.23.20)

The cargo handling system can make use of either one outdoor gantry crane or one or two indoor gantry cranes. The **acquisition cost of the crane or cranes**, C_{prv} (R$), – FOB cost – can be obtained from the expression below (or from Graph. B 17, annex B, as a function of the ratio between the generator capacity and its synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $68.9 \leq z \leq 4582$: $C_{prv} = 25.12 \times z^{0.6961}$

for:

$$z = 1000 \times \frac{P_2}{n}$$

where:

z	parameter, in kVA/rpm;
P_2	generator capacity, in MVA; and
n	synchronous velocity, in rpm.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

The **acquisition cost of the gantry crane**, C_{pcr} (R$), – FOB cost – can be obtained from the expression below (or from Graph. B 18, annex B, as a function of the ratio between the generator capacity and its synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $68.9 \leq z \leq 4582$: $C_{prv} = 59.506 \times z^{0.6621}$

for:

$$z = 1000 \times \frac{P_2}{n}$$

where:

z	parameter, in kVA/rpm;
P_2	generator capacity, in MVA; and
n	synchronous velocity, in rpm.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

Miscellaneous Equipment (account .15.00.00.23.31)

The **acquisition cost of miscellaneous equipment** should be taken as 6% of the overall cost of account .13 – Turbines and Generators.

POWERHOUSE EQUIPPED WITH HORIZONTAL-AXIS FRANCIS TURBINES

The main **information required for quantification purposes** is:

- width of a block of the unit of the powerhouse (perpendicular to flow), B_{1cf} in m;
- length of the powerhouse (direction of flow), L_{cf} in m;
- mean elevation of the land in the powerhouse area, El_{te};
- mean thickness of the layer of soil in the powerhouse area, e_{te} in m;
- maximum water level in the tailrace canal, NA_{xfu}, of item 5.1.2;

- volume of surface excavation in rock below the elevation of the assembly area, V_{rp} in m^3; and
- volume of concrete, V_{ccf} in m^3.

Velocities

Specific initial velocity, n'_s, can be obtained from Graph. 5.7.2.04 as a function of the maximum net head or from the equivalent expressions (Eletrosul, 1996):

for $27 \leq H_1 \leq 193.42$ m: $n'_s = 117.6 \times \ln\left(\frac{502}{H_1}\right)$

for $193.42 < H_1 \leq 350$ m: $n'_s = 3364 \times H_1^{-0.646}$

for:

$$Q_1 = \frac{10^6 \times P_1}{k \times H_1} \qquad k = \rho \times g \times \eta_{t1} \times \eta_{g1}$$

$$\eta_{t1} = 0.856 \times Q_1^{0.013} \qquad \eta_{g1} = 0.92 \times P_2^{0.01}$$

$$P_2 = \frac{P_1}{f_p}$$

where:

H_1	maximum net head, in m;
P_1	capacity of one generating unit, in MW;
k	coefficient;
ρ	1000 kg/m³ – specific mass of water;
η_{t1}	turbine output at the maximum net head;
η_{g1}	generator output at the maximum net head;
g	9.81 m/s² – acceleration due to gravity;
P_2	generator capacity, in MVA; and
f_p	power factor.

Initial velocity, n' (rpm), is given by:

$$n' = n'_s \times H_1^{1.25} \times P_{1t}^{-0.5}$$

for:

$$P_{1t} = \frac{10^3 \times P_1}{\eta_g}$$

where:

H_1	maximum net head, in m;
P_{1t}	unit capacity of the turbine, in kW;
n'_s	specific initial velocity;
P_1	capacity of one generating unit, in MW; and
η_g	generator output at the maximum net head.

The **number of generator poles**, p, can be obtained from Table 5.7.2.01, as a function of the initial velocity and maximum unit turbine flow, or from the equivalent expressions:

for $n' \geq 1.2 \times f$: $p = 2 \times \text{int}\left(120 \times \frac{f}{n'} \times \frac{1}{2} + 0.999\right)$

without using 54, 74 and 94

for n' < 1.2 × f: $p = 4 \times \text{int}\left(120 \times \frac{f}{n'} \times \frac{1}{4} + 0.999\right)$

where:

f	frequency of the electricity system, in Hz;
n'	initial velocity, in rpm; and
int(x)	function that returns the integer part of x.

Synchronous velocity, n (rpm), is given by:

$$n = 120 \times \frac{f}{p}$$

where:

F	frequency of the electricity system, in Hz; and
p	number of generator poles.

Specific velocity, n_s, is given by:

$$n_s = n \times H_1^{-1.25} \times P_{1t}^{0.5}$$

where:

n	synchronous velocity, in rpm;
H_1	maximum net head, in m; and
P_{1t}	unit capacity of the turbine, in kW.

Diameter and position of the turbine rotor

The **coefficient of peripheral velocity**, K_u, can be obtained from Graph. 5.7.2.05 as a function of the specific velocity or from the equivalent expression (Lugaresi and Massa, 1987):

$$K_u = 0.293 + 0.0027 \times n_s$$

where:

n_s	specific velocity.

The diameter of the **turbine rotor**, D_3 (m), is given by:

$$D_3 = 0.01 \times \text{int}\left(84.5 \times K_u \times \frac{H_1^{0.5}}{n} \times \frac{1}{0.01} + 0.5\right)$$

where:

K_u	coefficient of peripheral velocity;
H_1	maximum net head, in m; and
n	synchronous velocity, in rpm.

The **suction head**, h_s (m), is given by:

$$h_s = K - \sigma \times H_1 - D_3$$

for:

$$K = 10.33 - 0.0012 \times NA_{fu} - 0.013 \times T$$

$$\sigma = 7.54 \times 10^{-5} \times n_s^{1.41}$$

where:

K	variable, as a function of atmospheric pressure and steam pressure;
σ	Thoma coefficient (Siervo and Leva, 1976);
H_1	maximum net head, in m;

D_3	outlet diameter of the turbine rotor, in m;
NA_{fu}	normal water level in the tailrace canal;
T	mean water temperature in the summer, and °C; and
n_s	specific velocity.

The elevation of installation, El_d, is given by:

$$El_d = NA_{nfu} + h_s$$

where:

NA_{nfu}	minimum water level downstream; and
h_s	suction head, in m.

Dimensions of the turbine, the spiral casing, the generator and the draft tube

The **dimensions of a horizontal-axis Francis turbine** are given by the following expressions (Eletrosul, 1986). The dimensions in question are in Figures 5.7.2.07 and 5.7.2.08.

$A = 1.15 \times D_3$ $\quad$ $B = 1.50 \times D_3$

$C = 3.80 \times D_3$ $\quad$ $D = 1.90 \times D_3$

$E = 2.0 \times D_3$ $\quad$ $R = 2.0 \times D_3$

$S = 5.2 \times D_3$ $\quad$ $Y = 2.60 \times D_3$

where:

A, B, C, D, E	turbine dimensions, in m;
R	height of the opening for the draft tube at the outlet, in m (Eletrosul, 1996);
S	length of the draft tube, in m;
Y	height from the draft tube to the center of the distributor, in m (Eletrosul, 1996); and
D_3	outlet diameter of the turbine rotor, in m.

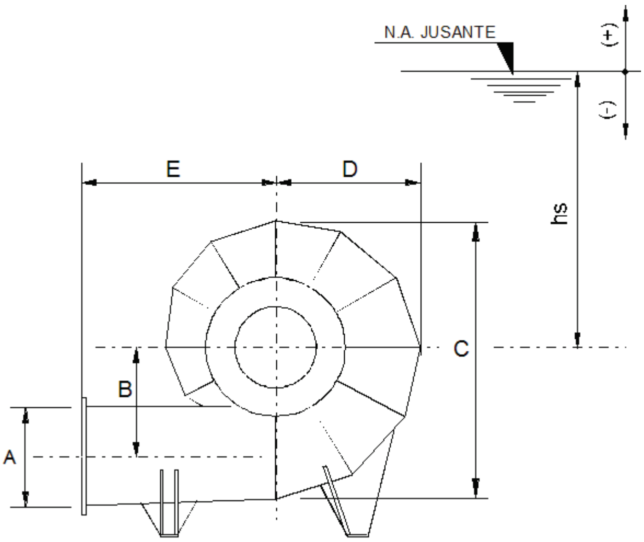


Fig. 5.7.2.07 – View of spiral casing – horizontal-axis Francis turbine.

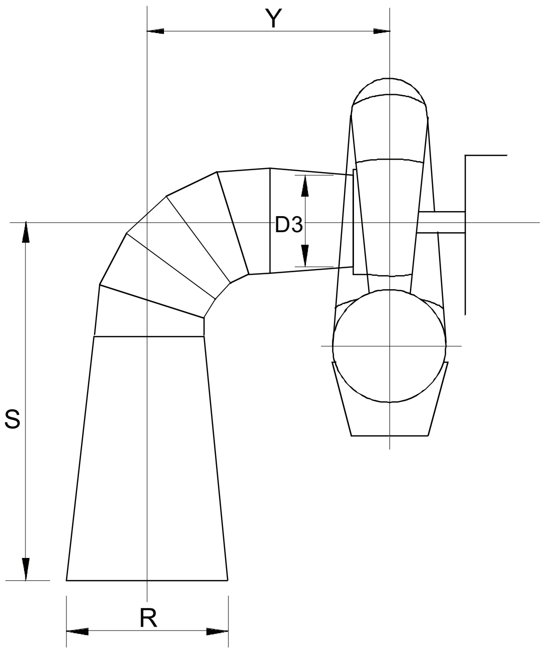


Fig. 5.7.2.08 – View of draft tube – horizontal-axis Francis turbine.

The **estimated diameter of the generator housing**, D_{pg} (m), is given by (COPEL, 1977):

$$D_{pg} = 9.0 \times \left(\frac{1000 \times P_1}{f_p \times n^2} \right)^{0.2}$$

where:

P_1	capacity of one generating unit, in MW;
f_p	power factor; and
n	synchronous velocity, in rpm.

Dimensions of the powerhouse

The **width of a block of one unit** for the powerhouse (perpendicular to flow), B_{1cf} (m), is determined by the design engineer based on the layout inside the powerhouse.

The **total width of the powerhouse**, B_{cf} (m), excluding the assembly area, is given by:

$$B_{cf} = N_g \times B_{1cf} + 2.0$$

where:

N_g	number of generating units; and
B_{1cf}	width of a block of the unit of the powerhouse, in m.

The **width of the equipment assembly area**, B_{am} (m), is given by:

for $N_g \leq 3$: $B_{am} = 1.5 \times B_{1cf}$

for $N_g > 3$: $B_{am} = 2.25 \times B_{1cf}$

where:

B_{1cf}	width of a block of the unit of the powerhouse, in m; and
N_g	number of generating units.

The **length of the powerhouse** (direction of flow), L_{cf} (m), is defined by the design engineer.

the **length of the equipment assembly area**, L_{am} (m), is given by:

$$L_{am} = L_{cf}$$

where:

L_{cf}	length of the powerhouse, in m.

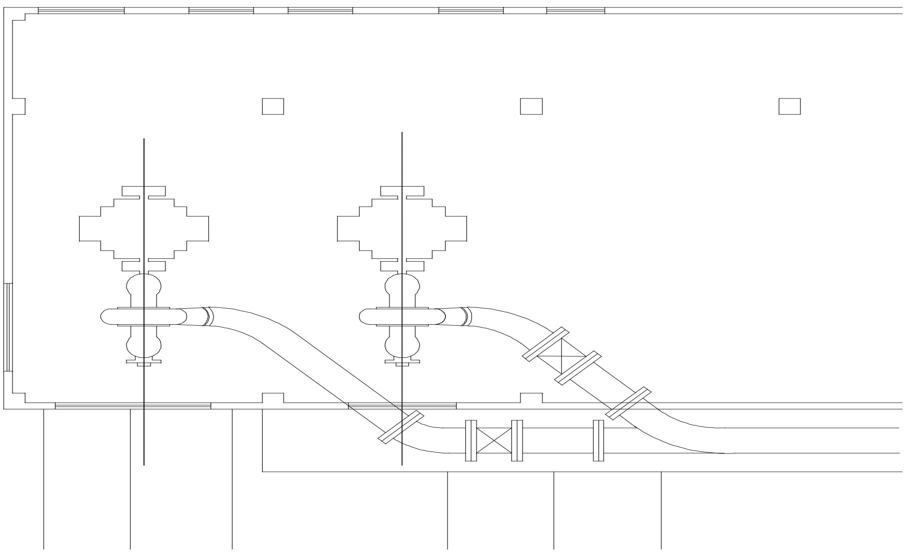


Figure 5.7.2.09 – Powerhouse with horizontal-axis Francis turbines.

Common excavation (account .11.13.00.12.10)

The common excavation volume, V_{tcf} (m^3), is given by:

$$V_{tcf} = (B_{cf} + B_{am} + 2 \times B_{1cf} + 2 \times 0.6 \times h_r) \times L_{cf} \times e_{te}$$

for: $h_r = El_{te} - e_{te} - (NA_{xfu} + 1.5)$

where:

B_{cf}	width of the powerhouse, in m;
B_{am}	width of the assembly area, in m;
B_{1cf}	width of one block of a unit of the powerhouse, in m;
h_r	mean depth of excavation in rock above the elevation of the assembly area, in m;
L_{cf}	length of the powerhouse, in m;
e_{te}	mean thickness of the layer of soil in the powerhouse area, in m;
El_{te}	mean elevation of the land in the powerhouse area; and
NA_{xfu}	maximum water level in the tailrace canal.

The unit price of common excavation is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .11.13.00.12.11)

The **volume of excavation in rock**, V_{rcf} (m^3), is given by:

$$V_{rcf} = V_{re} + V_{rp}$$

for: $V_{re} = (B_{cf} + B_{am} + 2 \times B_{1cf} + 0.6 \times h_r) \times L_{cf} \times h_r$

where:

V_{re}	volume of excavation in rock above the elevation of the assembly area, in m^3;
V_{rp}	volume of excavation in rock below the elevation of the assembly area, determined from the project design, in m^3;
B_{cf}	width of the powerhouse, in m;
B_{am}	width of the assembly area, in m;
B_{1cf}	width of a block of the unit of the powerhouse, in m;
h_r	mean depth of excavation in rock above the elevation of the assembly area, in m; and
L_{cf}	length of the powerhouse, in m.

The unit price of excavation in rock is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .11.13.00.13)

The area of foundation to be cleaned, A_{lf} (m²), is given by:

$$A_{lf} = (B_{cf} + B_{am}) \times L_{cf}$$

where:

B_{cf}	width of the powerhouse, in m;
B_{am}	width of the assembly area, in m; and
L_{cf}	length of the powerhouse, in m.

The **length of the grout holes**, L_{tf} (m), for treating the powerhouse foundations**,** is given by:

$$L_{tf} = \frac{B_{cf}}{3} \times L_{1tf}$$

for: $L_{1tf} = 1.5 \times (NA_{xfu} - El_d + Y) \leq 40 \text{ m}$

where:

B_{cf}	width of the powerhouse, in m;
L_{1tf}	length of one grout hole, in m;
NA_{xfu}	maximum water level in the tailrace canal;
El_d	elevation of the center line of the turbine distributor;
Y	height from the draft tube to the center of the distributor, in m; and
3.0	spacing between the grout holes, in m.

The unit prices for foundation cleaning and treatment services, expressed in Brazilian Reais (valid for the December 2006 database), can be used for projects in the south, southeast, central west and northeast regions of Brazil. They include the execution of the work, supply of inputs and equipment, and depend on the kind of surface and the equipment to be used. The unit prices are:

- cleaning of the rock surface: 39.70/m²
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .11.13.00.14)

The volume of concrete should be determined from the project design.

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the construction site, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the construction site, storage, preparation and installation.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of the powerhouse volume (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- concrete for infrastructure and end walls: 214.00/m^3
- dental concrete: 113.00/m^3
- concrete for superstructure: 214.00/m^3

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Installations and final work (account .11.13.00.15)

The **cost of installations and final works**, C_{ia} (R$), which covers all services for the final work on the powerhouse, such as dividing walls, coatings, installations, door and window frames, glass windows, etc., is obtained as a global cost using the expression below (or Graph B 20, as a function of installed capacity). It is valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996a):

valid for $30 \leq P \leq 1450$ MW: $C_{ia} = 6{,}150 \times P^{1 + \frac{15.34}{P}}$

where:

P	installed capacity, in MW.

Land developments in the plant area (account .11.12)

The **cost of land developments in the plant area**, C_{bau} (R$), which encompasses building the internal access roads to the different structures, guard houses and perimeter walls, landscaping, and others, is obtained as a global cost using the expression below (or Graph B 19, annex B, as a function of installed capacity). It is valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996a):

valid for $30 \leq P \leq 1450$ MW: $C_{bau} = 1{,}565 + \left(\frac{772{,}973}{P}\right)$

where:

P	installed capacity, in MW.

Operators' Village (account .11.14)

This cost is included in the workers' camp account (account .17.21).

Turbines (account .13.13.00.23.28)

The **acquisition cost of each horizontal-axis Francis turbine**, C_{tf} (R$), which includes the electromechanical equipment, parts and materials normally supplied by the manufacturers – FOB cost excluding transportation, insurance, assembly and testing costs and provisions for charges and taxes payable according to the applicable tax legislation – can be obtained from the expression below (or from

Graph. B 10, annex B, as a function of the unit capacity of the turbine and the synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $20 \leq z \leq 6000$: $C_{tf} = 0.0011 \times z^2 + 18.162 \times z + 3{,}279.8$

for: $z = \frac{P_{1t}}{n}$

where:

z	parameter, in kW/rpm;
P_{1t}	unit capacity of the turbine, in kW; and
n	synchronous velocity, in rpm.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for the taxes and charges payable on the equipment.

Stoplogs for the draft tube (account .13.13.00.23.16)

Normally, there is no stoplog used for the draft tube for this kind of turbine. If necessary, make a specific design for their usage.

Generators (account .13.13.00.23.29)

The **acquisition cost of each horizontal-axis generator**, C_{gh} (R$), which includes the generator and associated equipment – FOB cost – can be obtained from the expression below (or from Graph. B 14, annex B, as a function of the ratio between the generator capacity and its synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.0004 \leq \lambda \leq 0.0483$: $Cgh = 29580(\lambda)^{0.6323}$

for: $\lambda = \frac{P_2}{n}$ and $P_2 = \frac{P_1}{f_p}$

where:

P_2	generator capacity, in MVA;
λ	magnetic torque, in MVA/rpm;
n	synchronous velocity, in rpm;
P_1	capacity of one generating unit, in MW; and
f_p	power factor.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for the taxes and charges payable on the equipment.

Draft Tube Gantry Crane (account .13.13.00.23.20)

As the **acquisition cost of draft tube gantry crane** is low, it can be ignored at this stage.

Auxiliary electrical equipment (account .14.00.00.23)

The **acquisition cost of the auxiliary electrical equipment** should be taken as 18% of the overall cost of account .13 – Turbines and Generators.

Bridge and gantry cranes (account .15.13.00.23.20)

The cargo handling system is usually made up of one indoor bridge crane. The **acquisition cost of the crane**, C_{prv} (R$), – FOB cost – can be obtained from the expression below (or from Graph. B 17, annex B, as a function of the ratio between the generator capacity and its synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $8 \leq \lambda \leq 8$: $C_{prh} = (9.4666 \times \lambda) + 9.1722$

for: $\lambda = \frac{P_2}{n}$ and $P_2 = \frac{P_1}{f_p}$

where:

P_2	generator capacity, in MVA;
λ	magnetic torque, in MVA/rpm;
n	synchronous velocity, in rpm;
P_1	capacity of one generating unit, in MW; and
f_p	power factor.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

Miscellaneous Equipment (account .15.00.00.23.31)

The **acquisition cost of the miscellaneous equipment** should be taken as 6% of the overall cost of account .13 – Turbines and Generators.

POWERHOUSE EQUIPPED WITH KAPLAN TURBINES WITH A STEEL SPIRAL CASING

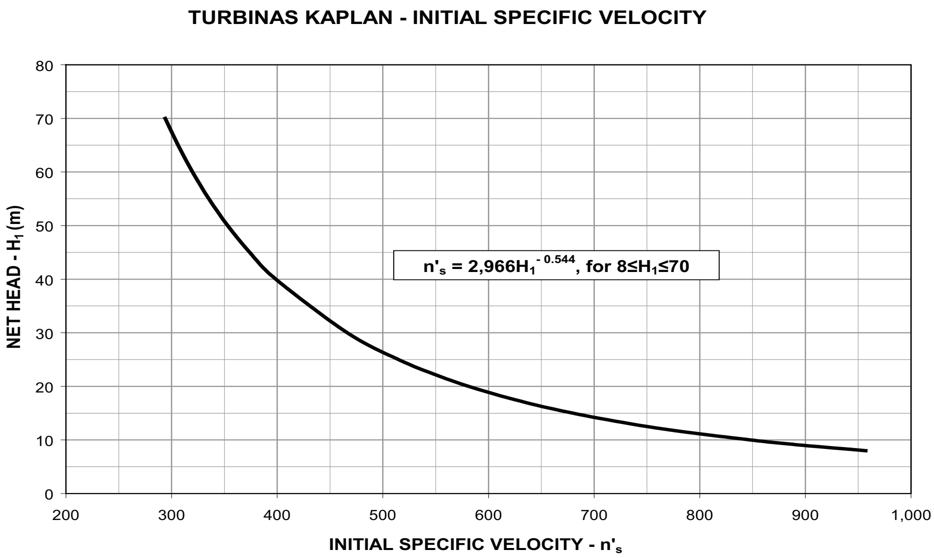
Velocities

The **specific initial velocity**, n'_s, can be obtained from Graph. 5.7.2.06 as a function of the maximum net head or from the equivalent expressions (Eletrosul, 1996):

for $8 \leq H_1 \leq 70$ m: $n'_s = 2966 \times H_1^{-0.544}$

where:

H_1	maximum net head, in m.



Graph 5.7.2.06 – Initial Specific Velocity – Kaplan turbines.

The **initial velocity**, n' (rpm), is given by:

$$n' = n'_s \times H_1^{1.25} \times P_{1t}^{-0.5}$$

for: $P_{1t} = \dfrac{10^3 \times P_1}{\eta_g}$

where:

n'_s	specific initial velocity;
H_1	maximum net head, in m;
P_{1t}	unit capacity of the turbine, in kW;
P_1	capacity of one generating unit, in MW; and
η_g	generator output at the maximum net head.

The **number of generator poles**, p, can be obtained from Table 5.7.2.01, as a function of the initial synchronous velocity, or from the equivalent expressions:

for $n' \geq 1.2 \times f$: $p = 2 \times \text{int}\left(120 \times \dfrac{f}{n'} \times \dfrac{1}{2} + 0.999\right)$

without using 54, 74 and 94

for $n' < 1.2 \times f$: $p = 4 \times \text{int}\left(120 \times \dfrac{f}{n'} \times \dfrac{1}{4} + 0.999\right)$

where:

f	frequency of the electricity system, in Hz;
n'	initial velocity, in rpm; and
int(x)	function that returns the integer part of x.

The **synchronous velocity**, n (rpm), is given by:

$$n = 120 \times \frac{f}{p}$$

where:

f	frequency of the electricity system, in Hz; and
p	number of generator poles.

The **specific velocity**, n_s, is given by:

$$n_s = n \times H_1^{-1.25} \times P_{1t}^{0.5}$$

where:

n	synchronous velocity, in rpm;
H_1	maximum net head, in m; and
P_{1t}	capacity of one turbine, in kW.

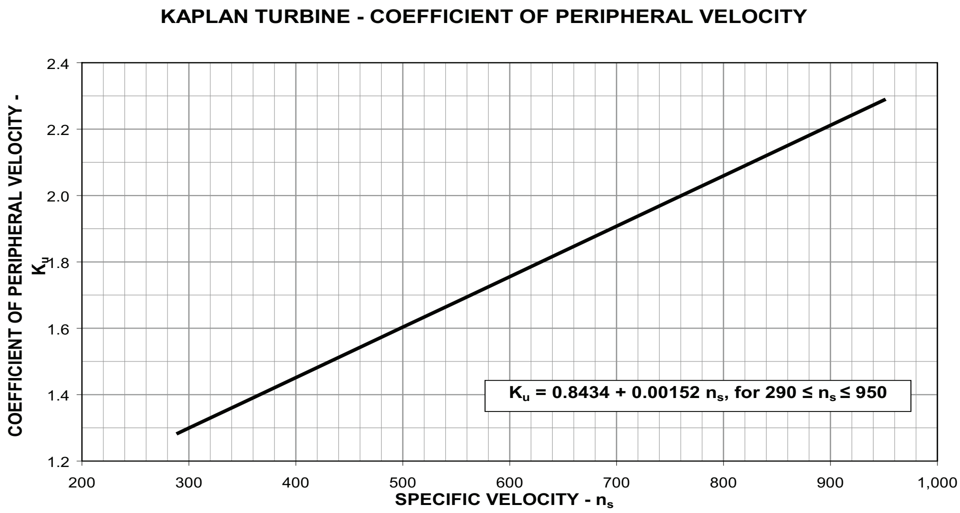
Diameter and position of the turbine rotor

The **coefficient of peripheral velocity**, K_u, can be obtained from Graph. 5.7.2.07 as a function of the specific velocity or from the equivalent expression (Schweiger and Gregori, 1987):

$$K_u = 0.8434 + 0.00152 \times n_s$$

where:

n_s	specific velocity.



Graph 5.7.2.07 – Coefficient of peripheral velocity – Kaplan turbines.

The diameter of the **turbine rotor**, D_K (m), is given by:

$$D_K = 0.01 \times \text{int}\left(84.5 \times K_u \times \frac{H_1^{0.5}}{n} \times \frac{1}{0.01} + 0.5\right)$$

where:

K_u	coefficient of peripheral velocity;
H_1	mean net head, in m; and
n	synchronous velocity.

The **suction head**, h_s (m), is given by:

$$h_s = K - \sigma \times H_1$$

for:

$$K = 10.33 - 0.0012 \times NA_{fu} - 0.013 \times T$$

$$\sigma = 6.40 \times 10^{-5} \times n_s^{1.46}$$

where:

K	variable, as a function of atmospheric pressure and steam pressure, in m;
σ	Thoma coefficient (De Siervo and De Leva, 1977);
H_1	maximum net head, in m;
NA_{fu}	normal water level in the tailrace canal;
T	mean water temperature in the summer, in °C; and
n_s	specific velocity.

The **elevation of the center line of the turbine distributor**, El_d, is given by:

$$E_{ld} = NA_{nfu} + h_s + H'_1$$

where:

NA_{nfu}	minimum water level downstream;
h_s	suction head, in m; and
D_K	diameter of the turbine rotor.

Dimensions of the turbine, the spiral casing, the generator and the draft tube

The **turbine and generator dimensions** are given by the following expressions (De Siervo and De Leva, 1978). The dimensions in question are in Figures 5.7.2.10 and 5.7.2.11.

$A = D_k \times 0.40 \times n_s^{020}$

$B = D_k \times (1.26 + 3.79 \times 10^{-4} \times n_s)$

$C = D_k \times (1.46 + 3.24 \times 10^{-4} \times n_s)$

$D = D_k \times (1.59 + 5.74 \times 10^{-4} \times n_s)$

$M = 2.25 \times D_k$ $\qquad$ $R = 1.3 \times D_k$

$$S = D_K \times \left(4.26 + \frac{201.51}{n_s}\right) \qquad Z = D_k \times \left(2.58 + \frac{102.66}{n_s}\right)$$

$Y = H_1' + M$ $\qquad$ $H_1' = 0.42 \times D_k$

for $Z \times R \geq 30\ m^2$: $U = 1.7\ m$ $N_{vs} = 2$

for $Z \times R < 30\ m^2$: $U = 0\ m$ $N_{vs} = 1$

where:

A, B, C, D, H'_1	turbine dimensions, in m;
M	height of the draft tube per se, in m;
R	height of the opening for the draft tube at the outlet, in m (Eletrosul, 1996);
S	length of the draft tube, in m;
U	thickness of the draft tube pillar, in m (Eletrosul, 1996);
Y	height from the draft tube to the center of the distributor, in m (Eletrosul, 1996);
Z	width of the draft tube, in m;
N_{vs}	number of openings for each draft tube;
D_K	outlet diameter of the turbine rotor, in m; and
n_s	specific velocity.

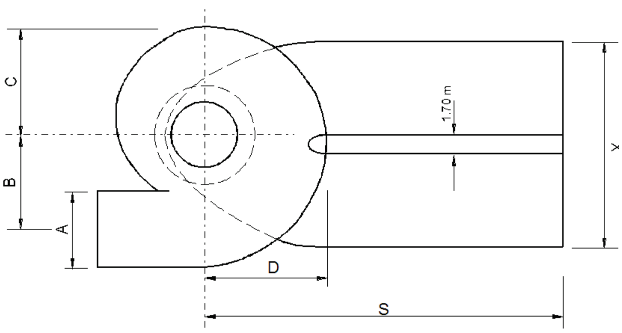


Fig. 5.7.2.10 – Plan of the spiral casing and draft tube – Kaplan turbine with steel spiral casing.

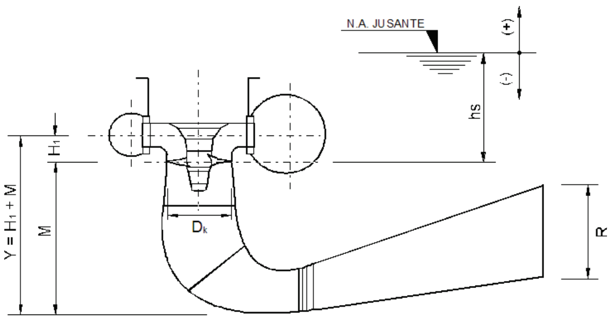


Fig. 5.7.2.11 – Cross-section of spiral casing and draft tube – Kaplan turbine with steel spiral casing.

The **estimated diameter of the generator housing**, D_{pg} (m), is given by (COPEL, 1977):

$$D_{pg} = 9.0 \times \left(\frac{1000 \times P_1}{f_p \times n^2}\right)^{0.2}$$

where:

P_1	capacity of one generating unit, in MW;
f_p	power factor; and
n	synchronous velocity, in rpm.

Dimensions of the powerhouse

The **width of a block of the unit of the powerhouse** (perpendicular to flow), B_{1cf} (m), is given by:

$$B_{1cf} = \frac{A}{2} + B + C + 2 \times (1.3 + 0.09 \times D_K)$$

where:

A, B, C	dimensions of the spiral casing, in m; and
D_K	diameter of the turbine rotor, in m.

The total width of the powerhouse, B_{cf} (m), excluding the assembly area, is given by:

$$B_{cf} = N_g \times B_{1cf} + 2.0$$

where:

N_g	number of generating units; and
B_{1cf}	width of a block of the unit of the powerhouse, in m.

The **width of the equipment assembly area**, B_{am} (m), is given by:

for $N_g \leq 3$: $B_{am} = 1.5 \times B_{1cf}$

for $N_g > 3$: $B_{am} = 2.25 \times B_{1cf}$

where:

B_{1cf}	width of a block of the unit of the powerhouse, in m; and
N_g	number of generating units.

The **length of the superstructure**, L_{cs} (m), is given by:

$$L_{cs} = d_1 + d_2$$

for:

$$d_1 = \frac{D_{pg}}{2} + 2.1 + 0.2 \times D_K \qquad d_2 = D + 2.1 + 0.2 \times D_k$$

where:

d_1	distance between the outside face of the upstream wall and the center line of the generating units, in m;
d_2	distance between the center line of the generating units and the outside face of the downstream wall, in m;
D_{pg}	diameter of the generator housing, in m;
D_K	diameter of the turbine rotor, in m; and
D	turbine dimensions, in m.

The **length of the powerhouse**, L_{cf} (m), is given by:

$$L_{cf} = d_1 + S$$

where:

d_1	distance between the outside face of the upstream wall and the center line of the generating units, in m; and
S	length of the draft tube, in m.

The **length of the equipment assembly area**, L_{am} (m), is given by:

$$L_{am} = L_{cs}$$

where:

L_{cs}	length of the superstructure, in m.

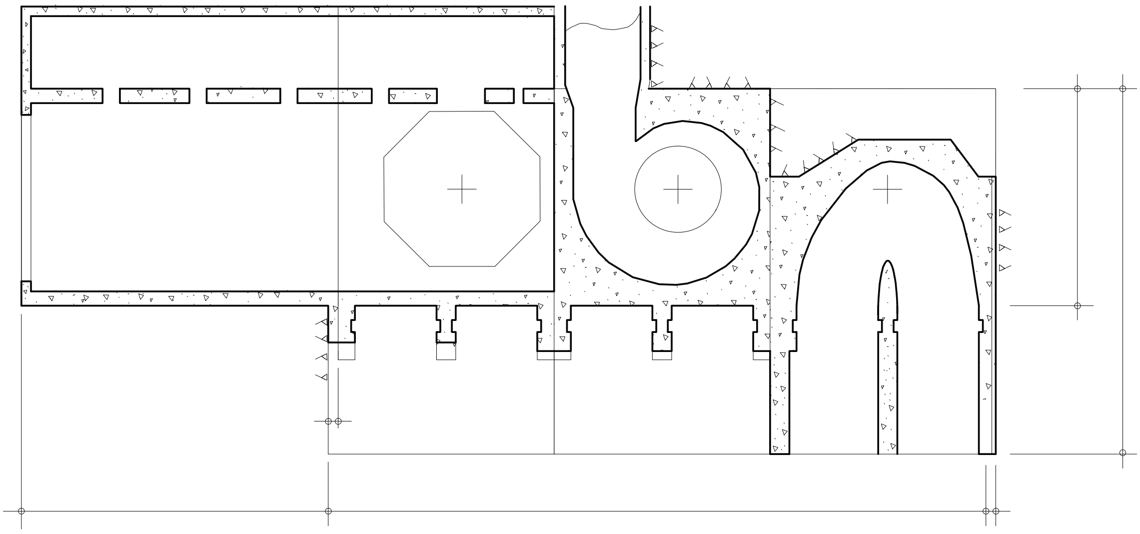


Fig. 5.7.2.12 – Plan of the powerhouse and assembly area for Kaplan turbines with a steel spiral casing.

Common excavation (account .11.13.00.12.10)

The common excavation volume, V_{tcf} (m^3), for the **powerhouse** is given by:

$$V_{tcf} = (B_{cf} + B_{am} + 2 \times B_{1cf} + 2 \times 0.6 \times h_r) \times L_{cf} \times e_{te}$$

for: $h_r = El_{te} - e_{te} - (NA_{xfu} + 1.5)$

where:

B_{cf}	width of the powerhouse, in m;
B_{am}	width of the assembly area, in m;
B_{1cf}	width of a block of a unit of the powerhouse, in m;
h_r	mean depth of excavation in rock above the elevation of the assembly area, in m;
L_{cf}	length of the powerhouse, in m;
e_{te}	mean thickness of the layer of soil in the powerhouse area, in m;
El_{te}	mean elevation of the land in the powerhouse area; and
NA_{xfu}	maximum water level in the tailrace canal.

The unit price of common excavation is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .11.13.00.13)

The whole of the foundation area should be cleaned. The area of foundation to be cleaned, A_{lf} (m^2), for the **powerhouse** is given by:

$$A_{lf} = B_{cf} \times L_{cf} + B_{am} \times L_{cs}$$

where:

B_{cf}	width of the powerhouse, in m;
L_{cf}	length of the powerhouse, in m;
B_{am}	width of the assembly area, in m; and
L_{cs}	length of the powerhouse superstructure, in m.

The **length of the grout holes**, L_{tf} (m), for treating the powerhouse foundations is given by:

$$L_{tf} = \frac{B_{cf}}{3} \times L_{1tf}$$

for: $L_{1tf} = 1.5 \times (NA_{xfu} - El_d + Y) \leq 40 \text{ m}$

where:

B_{cf}	width of the powerhouse, in m;
L_{1tf}	length of one grout hole, in m;
NA_{xfu}	maximum water level in the tailrace canal;
El_d	elevation of the center line of the turbine distributor; and
Y	height from the draft tube to the center of the distributor, in m.

The unit price of common excavation is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .11.13.00.12.11)

The **volume of excavation in rock**, V_{rcf} (m^3), for the **powerhouse** is given by:

$$V_{rcf} = V_{re} + V_{rp} + V_{rd}$$

and is valid for $1.5 \leq D_k \leq 8.0$ m:

$$V_{re} = (B_{cf} + B_{am} + 2 \times B_{1cf} + 0.6 \times h_r) \times L_{cf} \times h_r$$

$$V_{rp} = B_{cf} \times L_{cf} \times (NA_{xfu} + 1.5 - El_d)$$

$$V_{rd} = N_g \times 700 \times e^{0.54 \times D_K}$$

where:

V_{re}	volume of excavation in rock above the elevation of the assembly area, in m^3;
V_{rp}	volume of excavation in rock between the elevation of the assembly area and the elevation of the center line of the turbine distributor, in m^3;
V_{rd}	volume of excavation in rock below the center line of the turbine distributor, in m^3;
B_{cf}	width of the powerhouse, in m;
B_{am}	width of the assembly area, in m;
B_{1cf}	width of a block of the unit of the powerhouse, in m;
h_r	mean depth of excavation in rock above the elevation of the assembly area, in m;

L_{cf}	length of the powerhouse, in m;
NA_{xfu}	maximum water level in the tailrace canal;
El_d	elevation of the center line of the turbine distributor;
N_g	number of generating units; and
D_K	diameter of the turbine rotor, in m.

The unit price of excavation in rock is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .11.13.00.13)

The whole of the foundation area should be cleaned. The area of foundation to be cleaned, A_{lf} (m^2), for the **powerhouse** is given by:

$$A_{lf} = B_{cf} \times L_{cf} + B_{am} \times L_{cs}$$

where:

B_{cf}	width of the powerhouse, in m;
L_{cf}	length of the powerhouse, in m;
B_{am}	width of the assembly area, in m; and
L_{cs}	length of the powerhouse superstructure, in m.

The **length of the grout holes**, L_{tf} (m), for treating the powerhouse foundations is given by:

$$L_{tf} = \frac{B_{cf}}{3} \times L_{1tf}$$

for: $L_{1tf} = 1.5 \times (NA_{xfu} - El_d + Y) \leq 40 \text{ m}$

where:

B_{cf}	width of the powerhouse, in m;
L_{1tf}	length of one grout hole, in m;
NA_{xfu}	maximum water level in the tailrace canal;
El_d	elevation of the center line of the turbine distributor; and
Y	height from the draft tube to the center of the distributor, in m.

The unit prices for foundation cleaning and treatment services, expressed in Brazilian Reais (valid for the December 2006 database), can be used for projects in the south, southeast, central west and northeast regions of Brazil. They include the execution of the work, supply of inputs and equipment, and depend on the kind of surface and the equipment to be used. The unit prices are:

- cleaning of the rock surface: 39.70/m^2
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .11.13.00.14)

The volume of **concrete**, V_{ccf} (m^3), for an **indoor powerhouse** is given by:

$$V_{ccf} = N_g \times (V_{cf} + 1.5 \times V_{cs}) + V_{ce} + V_{cd} + V_{cn} + V_{ca}$$

for:

and valid for $1.5 \leq D_k \leq 8.0$ m

$$V_{cf} = 530 \times e^{0.535 \times D_K} \qquad V_{cs} = 235 \times e^{0.381 \times D_K}$$

$$V_{ce} = 410 \times e^{0.314 \times D_K}$$

for $N_g \leq 3$: $V_{ca} = V_{cs}$

for $N_g > 3$: $V_{ca} = 2 \times V_{cs}$

where:

N_g	number of generating units;
V_{cf}	volume of concrete for the infrastructure, in m^3;
V_{cs}	volume of concrete for the superstructure, in m^3;
V_{ce}	volume of concrete for one wall at each end, in m^3;
V_{cd}	volume of concrete for when additional excavation is required because of poor foundations, in m^3;
V_{cn}	volume of concrete resulting from alterations to the project design so that the maximum water level in the tailrace canal is higher than the elevation of the generator floor, in m^3;
V_{ca}	volume of concrete for the assembly area, in m^3; and
D_K	diameter of the turbine rotor, in m.

The volume of **concrete**, V_{ccf} (m^3), for a **semi-outdoor powerhouse** is given by:

$$V_{ccf} = N_g \times (V_{cf} + V_{cs}) + V_{ce} + V_{cd} + V_{cn} + V_{ca}$$

where:

N_g	number of generating units;
V_{cf}	volume of concrete for the infrastructure, in m^3;
V_{cs}	volume of concrete for the superstructure, in m^3;
V_{ce}	volume of concrete for one wall at each end, in m^3;
V_{cd}	volume of concrete for when additional excavation is required because of poor foundations, in m^3;
V_{cn}	volume of concrete resulting from alterations to the project design so that the maximum water level in the tailrace canal is higher than the elevation of the generator floor, in m^3;
V_{ca}	volume of concrete for the assembly area, in m^3; and
D_K	diameter of the turbine rotor, in m.

The volume of **concrete**, V_{ccf} (m^3), for a **surface powerhouse** is given by:

$$V_{ccf} = N_g \times (V_{cf} + 0.15 \times V_{cs}) + 0.6 \times V_{ce} + V_{cd} + V_{cn} + 0.25 \times V_{ca}$$

where:

N_g	number of generating units;
V_{cf}	volume of concrete for the infrastructure, in m^3;
V_{cs}	volume of concrete for the superstructure, in m^3;
V_{ce}	volume of concrete for one wall at each end, in m^3;
V_{cd}	volume of concrete for when additional excavation is required because of poor foundations, in m^3;

V_{cn}	volume of concrete resulting from alterations to the project design so that the maximum water level in the tailrace canal is higher than the elevation of the generator floor, in m^3;
V_{ca}	volume of concrete for the assembly area, in m^3; and
D_K	diameter of the turbine rotor, in m.

The amounts of cement and reinforcement steel are as follows:

	cement (kg/m^3)	reinforcement steel (kg/m^3)
infrastructure	275	50
superstructure	300	100
end wall	250	75
dental	200	0
shotcrete	300	70

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the construction site, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the construction site, storage, preparation and installation.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of the powerhouse volume (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- concrete for the infrastructure and end walls: 214.00/m^3
- concrete dental: 113.00/m^3
- concrete for the superstructure: 214.00/m^3

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Installations and final work (account .11.13.00.15)

The **cost of installations and final works**, C_{ia} (R$), which covers all services for the final work on the powerhouse, such as dividing walls, coatings, installations, door and window frames, glass windows, etc., is obtained as a global cost using the expression below (or Graph B 20, as a function of installed capacity). It is valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996a):

valid for $30 \le P \le 1450$ MW: $C_{ia} = 6{,}150 \times P^{1 + \frac{15.34}{P}}$

where:

P	installed capacity, in MW.

Land developments in the plant area (account .11.12)

The **cost of land developments in the plant area**, C_{bau} (R$), which encompasses building the internal access roads to the different structures, guard houses and perimeter walls, landscaping, and others, is obtained as a global cost using the expression below (or Graph B 19, annex B, as a function of installed capacity). It is valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996a):

valid for 30 ≤ P ≤ 1450 MW: $C_{bau} = 1{,}565 + \left(\frac{772{,}973}{P}\right)$

where:

P	installed capacity, in MW.

Operators' Village (account .11.14)

This cost is included in the workers' camp account (account .17.21).

Turbines (account .13.13.00.23.28)

The **acquisition cost of each** Kaplan **turbine** with a steel spiral casing, C_{tka} (R$), which includes the electromechanical equipment, parts and materials normally supplied by the manufacturers – FOB cost excluding transportation, insurance, assembly and testing costs and provisions for charges and taxes payable according to the applicable tax legislation – can be obtained from the expression below (or from Graph. B 11, annex B, as a function of the unit capacity of the turbine and the synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for 20 ≤ z ≤ 2500: $C_{tka} = 0.0058 \times z^2 + 40.609 \times z + 3{,}122.5$

for: $z = \frac{P_{1t}}{n}$

where:

z	parameter, in kW/rpm;
P_{1t}	capacity of one turbine, in kW; and
n	synchronous velocity, in rpm.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for the taxes and charges payable on the equipment.

Stoplogs for the draft tube (account .13.13.00.23.16)

The number of stoplogs, N_{sl}, is given by the following expressions:

for $N_g \leq 10$: $N_{sl} = 2 \times N_{vs}$

for $N_g > 10$: $N_{sl} = 3 \times N_{vs}$

where:

N_g	number of generating units; and
N_{vs}	number of openings for each draft tube.

The **acquisition cost of each stoplog** for the draft tube, C_{sl} (R$), – FOB cost – can be obtained from the expression below (or from Graph. B 25, annex B, as a function of its dimensions and hydrostatic load), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for 0.16 ≤ z ≤ 54.5: $C_{sl} = 72.9 \times z^{0.716}$

for:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000} \qquad H_{cp} = R$$

$$H_x = NA_{xfu} - El_d + Y \qquad B_{cp} = \frac{Z - U}{N_{vs}}$$

where:

z	parameter, in m^4;
B_{cp}	width of the stoplog, in m;
H_{cp}	height of the stoplog, in m;
H_x	maximum hydrostatic load on the sill of the stoplog, in m;
R	height of the opening for the draft tube at the outlet, in m;
NA_{xfu}	maximum water level in the tailrace canal;
El_d	elevation of the center line of the turbine distributor;
Y	height from the draft tube to the center of the distributor, in m;
Z	width of the draft tube, in m;
U	width of the draft tube pillar, in m; and
N_{vs}	number of openings for each draft tube.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

The **overall acquisition cost for the fixed parts and parts embedded in the concrete** of the stoplogs for the draft tube, C_{gpf} (R$), – FOB cost – is given by below, valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

$$C_{gpf} = 2 \times N_{vs} \times N_g \times (H_x + 2.0) \times 2{,}084.80$$

where:

N_g	number of generating units;
H_x	maximum hydrostatic load on the sill of the stoplog, in m; and
N_{vs}	number of openings for each draft tube.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

Draft Tube Gantry Crane (account .13.13.00.23.20)

As the **acquisition cost of draft tube gantry crane** is low, it can be ignored at this stage.

Generators (account .13.13.00.23.29)

The **acquisition cost of each vertical-axis generator**, C_{gv} (R$), which includes the generator and related equipment – FOB cost – can be obtained from the expression below (or from Graph. B 16, annex B, as a function of the ratio between the generator capacity and its number of poles and taking the synchronous velocity into account), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.0004 \leq \lambda \leq 0.0483$: $C_{gv} = 42280 \times \lambda^{0.6298}$

for: $\lambda = \dfrac{P_2}{n}$ and $P_2 = \dfrac{P_1}{f_p}$

where:

P_2	generator capacity, in MVA;
λ	magnetic torque, in MVA/rpm;
n	synchronous velocity, in rpm;
P_1	capacity of one generating unit, in MW; and
f_p	power factor.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 15.0%: for the taxes and charges payable on the equipment.

Auxiliary Electrical Equipment (account .14.00.00.23)

The **acquisition cost of the auxiliary electrical equipment** should be taken as 18% of the overall cost of account .13 – Turbines and Generators.

Bridge and gantry cranes (account .15.13.00.23.20)

The cargo handling system can make use of either one outdoor gantry crane or one or two indoor gantry cranes. The **acquisition cost of the crane or cranes**, C_{prv} (R$), – FOB cost – can be obtained from the expression below (or from Graph. B 17, annex B, as a function of the ratio between the generator capacity and its synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $68.9 \leq z \leq 4582$: $C_{prv} = 25.12 \times z^{0.6961}$

for: $z = 1000 \times \frac{P_2}{n}$

where:

z	parameter, in kVA/rpm;
P_2	generator capacity, in MVA; and
n	synchronous velocity, in rpm.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

The **acquisition cost of the bridge crane**, C_{pcr} (R$), – FOB cost – can be obtained from the expression below (or from Graph. B 18, annex B, as a function of the ratio between the generator capacity and its number of poles), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $68.9 \leq z \leq 4582$: $C_{pcr} = 59.506 \times z^{0.6621}$

for: $z = 1000 \times \frac{P_2}{n}$

where:

z	parameter, in kVA/rpm;
P_2	generator capacity, in MVA; and
n	synchronous velocity, in rpm.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

Miscellaneous Equipment (account .15.00.00.23.31)

The **acquisition cost of miscellaneous equipment** should be taken as 6% of the overall cost of account .13 – Turbines and Generators.

POWERHOUSE EQUIPPED WITH KAPLAN TURBINES WITH A SEMI-SPIRAL CONCRETE CASING

The main **information required for dimensioning** purposes is:

- mean elevation of the land in the powerhouse area, El_{te}, in m;
- mean thickness of the layer of soil in the powerhouse area, e_{te}, in m;
- volume of concrete for when additional excavation is required because of poor foundations, V_{cd}, in m³;

- volume of concrete resulting from alterations to the project design so that the maximum water level in the tailrace canal is higher than the elevation of the generator floor, V_{cn}, in m^3;
- type of powerhouse; and
- maximum water level in the tailrace canal, NA_{xfu}, from item 5.1.2.

Velocities

The **specific initial velocity**, n'_s, can be obtained from Graph. 5.7.2.06 as a function of the maximum net head or from the equivalent expressions (Eletrosul, 1996):

for $8 \leq H1 \leq 70$ m: $n'_s = 2966 \times H_1^{-0.544}$

where:

H_1	maximum net head, in m.

The **initial velocity**, n' (rpm), is given by:

$$n' = n'_s \times H_1^{1.25} \times P_{1t}^{-0.5}$$

for: $P_{1t} = \dfrac{10^3 \times P_1}{\eta_g}$

where:

n'_s	specific initial velocity;
H_1	maximum net head, in m;
P_{1t}	unit capacity of the turbine, in kW;
P_1	capacity of one generating unit, in MW; and
η_g	0.98 – generator output at maximum net head.

The **number of generator poles**, p, can be obtained from Table 5.7.2.01, as a function of the initial synchronous velocity, or from the equivalent expressions:

for $n' \geq 1.2 \times f$: $p = 2 \times \text{int}\left(120 \times \dfrac{f}{n'} \times \dfrac{1}{2} + 0.999\right)$

without using 54, 74 and 94

for $n' < 1.2 \times f$: $p = 4 \times \text{int}\left(120 \times \dfrac{f}{n'} \times \dfrac{1}{4} + 0.999\right)$

where:

f	frequency of the electricity system, in Hz;
n'	initial velocity, in rpm; and
int(x)	function that returns the integer part of x.

The **synchronous velocity**, n (rpm), is given by:

$$n = 120 \times \frac{f}{p}$$

where:

f	frequency of the electricity system, in Hz; and
p	number of generator poles.

The **specific velocity**, n_s, is given by:

$$n_s = n \times H_1^{-1.25} \times P_{1t}^{0.5}$$

where:

n	synchronous velocity, in rpm;
H_1	maximum net head, in m; and
P_{1t}	capacity of one turbine, in kW.

Diameter and position of the turbine rotor

The **coefficient of peripheral velocity**, K_u, can be obtained from Graph. 5.7.2.07 as a function of the specific velocity or from the equivalent expression (Schweiger and Gregori, 1987):

$$K_u = 0.8434 + 0.00152 \times n_s$$

where:

n_s	specific velocity.

The diameter of the **turbine rotor**, D_K (m), is given by:

$$D_K = 0.01 \times \text{int}\left(84.5 \times K_u \times \frac{H_1^{0.5}}{n} \times \frac{1}{0.01} + 0.5\right)$$

where:

K_u	coefficient of peripheral velocity;
H_1	mean net head, in m; and
n	synchronous velocity.

The **suction head**, h_s (m), is given by:

$$h_s = K - \sigma \times H_1$$

for: $K = 10.33 - 0.0012 \times NA_{fu} - 0.013 \times T$

$$\sigma = 6.40 \times 10^{-5} \times n_s^{1.46}$$

where:

K	height at pressure, in m;
σ	Thoma coefficient (Siervo and Leva, 1977);
H_1	maximum net head, in m;
NA_{fu}	normal water level in the tailrace canal;
T	mean water temperature in the summer, in °C; and
n_s	specific velocity.

The **elevation of the center line of the turbine distributor**, El_d, is given by:

$$El_d = NA_{nfu} + h_s + H'_1$$

where:

NA_{nfu}	minimum water level downstream;
h_s	suction head, in m;
H'_1	turbine dimensions defined below; and
D_K	diameter of the turbine rotor.

Dimensions of the turbine, the semi-spiral casing, the generator and the draft tube

The **dimensions of the turbine** and **generator** are given by the following expressions (Eletrosul, 1996). The dimensions in question are in Fig. 5.7.2.13 and 5.7.2.14.

$B = 1.8 \times D_k$ $\quad$ $C = 1.2 \times D_k$

$D = 1.5 \times D_k$ $\quad$ $F = 1.65 \times D_k$

$G = 1.3 \times D_k$

$R = 1.2 \times D_k$

$S = 4.6 \times D_k$

$X = 3.0 \times D_k$

$Y = 2.65 \times D_k$

$H'_1 = 0.4 \times D_k$

$U = 1.7m$

where:

H'_1	turbine dimensions, in m;
B, C, D	dimensions of the semi-spiral casing, in m;
R	height of the opening for the draft tube at the outlet, in m;
S	length of the draft tube, in m;
U	width of the draft tube pillar, in m;
Y	height from the draft tube to the center of the distributor, in m;
X	width of the draft tube, in m;
N_{vs}	2 – number of openings for each draft tube;
D_K	diameter of the turbine rotor outlet, in m; and
n_s	specific velocity.

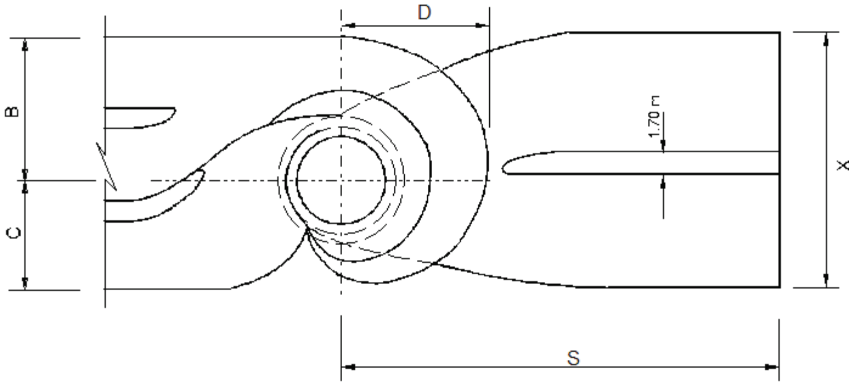


Fig. 5.7.2.13 – Plan of semi-spiral casing and draft tube – Kaplan turbine with semi-spiral casing made of concrete.

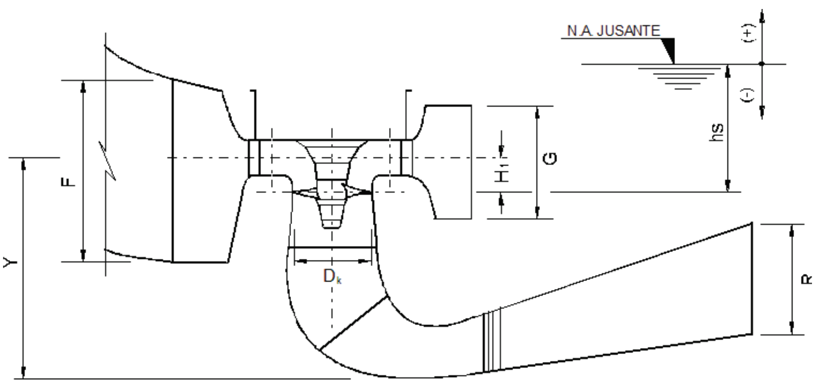


Fig. 5.7.2.14 – Cross-section of semi-spiral casing and draft tube – Kaplan turbine with semi-spiral casing made of concrete.

The **estimated diameter of the generator housing**, D_{pg} (m), is given by (COPEL, 1977):

$$D_{pg} = 9.0 \times \left(\frac{1000 \times P_1}{f_p \times n^2} \right)^{0.2}$$

where:

P_1	capacity of one generating unit, in MW;
f_p	power factor; and
n	synchronous velocity, in rpm.

Dimensions of the powerhouse

The **width of a block of the unit** for the powerhouse (perpendicular to flow), B_{1cf} (m), is given by:

$$B_{1cf} = B + C + 2 \times (0.6 + 0.2 \times D_K)$$

where:

A, B, C	dimensions of the semi-spiral casing, in m; and
D_K	diameter of the turbine rotor, in m.

The **total width** of the powerhouse, B_{cf} (m), excluding the assembly area, is given by:

$$B_{cf} = N_g \times B_{1cf} + 2.0$$

where:

N_g	number of generating units; and
B_{1cf}	width of a block of the unit of the powerhouse, in m.

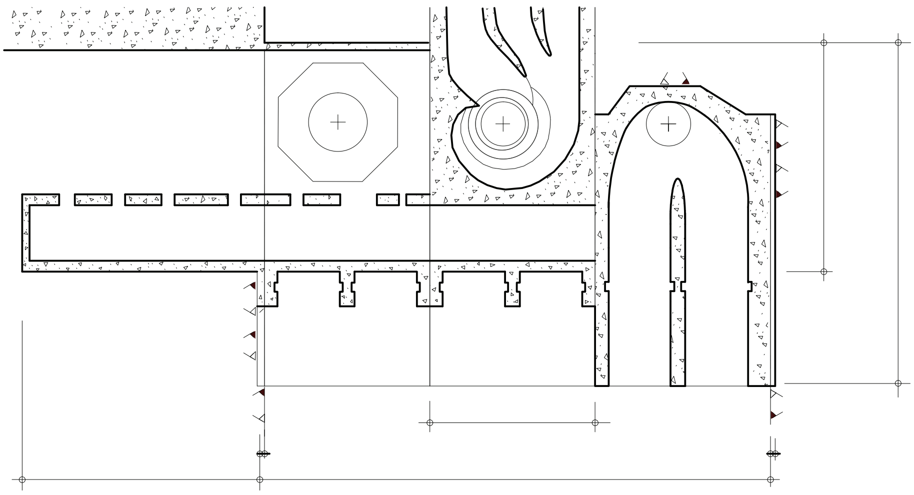


Fig. 5.7.2.15 – Plan of the powerhouse and assembly area – Kaplan turbine with semi-spiral casing made of concrete.

The **width of the equipment assembly area**, B_{am} (m), is given by:

for $N_g \leq 3: B_{am} = 1.5 \times B_{1cf}$

for $N_g > 3: B_{am} = 2.25 \times B_{1cf}$

where:

B_{1cf}	width of a block of the unit of the powerhouse, in m; and
N_g	number of generating units.

The **length of the superstructure**, L_{cs} (m), is given by:

$$L_{cs} = d_1 + d_2$$

for: $d_1 = \frac{D_{pg}}{2} + 2.1 + 0.2 \times D_K$ and $d_2 = D + 2.1 + 0.2 \times D_K$

where:

d_1	distance between the outside face of the upstream wall and the central line of the generating units, in m;
d_2	distance between the central line of the generating units and the outside face of the downstream wall, in m;
D_{pg}	diameter of the generator housing, in m;
D_K	diameter of the turbine rotor, in m; and
D	turbine dimensions, in m.

The **length of the powerhouse**, L_{cf} (m), is given by:

$$L_{cf} = d_1 + S$$

where:

d_1	distance between the outside face of the upstream wall and the center line of the generating units, in m; and
S	length of the draft tube, in m.

The **length of the equipment assembly area**, L_{am} (m), is given by:

$$L_{am} = L_{cs}$$

where:

L_{cs}	length of the superstructure, in m.

Common excavation (account .11.13.00.12.10)

The common excavation volume, V_{tcf} (m^3), in the **powerhouse** is given by:

$$V_{tcf} = (B_{cf} + B_{am} + 2 \times B_{1cf} + 2 \times 0.6 \times h_r) \times L_{cf} \times e_{te}$$

for: $h_r = El_{te} - e_{te} - (NA_{xfu} + 1.5)$

where:

B_{cf}	width of the powerhouse, in m;
B_{am}	width of the assembly area, in m;
B_{1cf}	width of a block of the unit of the powerhouse, in m;
h_r	mean depth of excavation in rock above the elevation of the assembly area, in m;
L_{cf}	length of the powerhouse, in m;
e_{te}	mean thickness of the layer of soil in the powerhouse area, in m;
El_{te}	mean elevation of the land in the powerhouse area; and
NA_{xfu}	maximum water level in the tailrace canal.

The unit price of common excavation is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .11.13.00.12.11)

The **volume of excavation in rock**, V_{rcf} (m^3), for the **powerhouse** is given by:

$$V_{rcf} = V_{re} + V_{rp} + V_{rd}$$

valid for $1.5 \leq D_K \leq 8.0\,m$

$$V_{re} = (B_{cf} + B_{am} + 2 \times B_{1cf} + 0.6 \times h_r) \times L_{cf} \times h_r$$

$$V_{rp} = B_{cf} \times L_{cf} \times (NA_{xfu} + 1.5 - El_d)$$

$$V_{rd} = N_g \times 700 \times e^{0.54 \times D_K}$$

where:

V_{re}	volume of excavation in rock above the elevation of the assembly area, in m³;
V_{rp}	volume of excavation in rock between the elevation of the assembly area and the elevation of the center line of the turbine distributor, in m³;
V_{rd}	volume of excavation in rock below the center line of the turbine distributor, in m³;
B_{cf}	width of the powerhouse, in m;
B_{am}	width of the assembly area, in m;
B_{1cf}	width of a block of the unit of the powerhouse, in m;
h_r	mean depth of excavation in rock above the elevation of the assembly area, in m;
L_{cf}	length of the powerhouse, in m;
NA_{xfu}	maximum water level in the tailrace canal;
El_d	elevation of the center line of the turbine distributor;
N_g	number of generating units; and
D_K	diameter of the turbine rotor, in m.

The unit price of excavation in rock is R$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .11.13.00.13)

The whole of the foundation area should be cleaned. The area of foundation to be cleaned, A_{lf} (m²), for the **powerhouse** is given by:

$$A_{lf} = B_{cf} \times L_{cf} + B_{am} \times L_{cs}$$

B_{cf}	width of the powerhouse, in m;
L_{cf}	length of the powerhouse, in m;
B_{am}	width of the assembly area, in m; and
L_{cs}	length of the powerhouse superstructure, in m.

The **length of the grout holes**, L_{tf} (m), for treating the powerhouse foundations is given by:

$$L_{tf} = \frac{B_{cf}}{3} \times L_{1tf}$$

for: $L_{1tf} = 1.5 \times \left(NA_{xfu} - El_d + Y\right) \leq 40\ \text{m}$

where:

B_{cf}	width of the powerhouse, in m;
L_{1tf}	length of one grout hole, in m;
NA_{xfu}	maximum water level in the tailrace canal;
El_d	elevation of the center line of the turbine distributor; and
Y	height from the draft tube to the center of the distributor, in m.

The unit prices for foundation cleaning and treatment services, expressed in Brazilian Reais (valid for the December 2006 database), can be used for projects in the south, southeast, central west and northeast regions of Brazil. They include the execution of the work, supply of inputs and equipment, and depend on the kind of surface and the equipment to be used. The unit prices are:

- cleaning of the rock surface: 39.70/m²
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .11.13.00.14)

The volume of **concrete**, V_{ccf} (m³), for a **indoor powerhouse** is given by:

$$V_{ccf} = N_g \times (V_{cf} + 1.5 \times V_{cs}) + V_{ce} + V_{cd} + V_{cn} + V_{ca}$$

valid for $1.5 \leq D_K \leq 8.0\,\text{m}$:

$$V_{cf} = 485 \times e^{0.535 \times D_K}$$

$$V_{cs} = 215 \times e^{0.381 \times D_K}$$

$$V_{ce} = 370 \times e^{0.314 \times D_K}$$

for $N_g \leq 3$: $V_{ca} = V_{cs}$

for $N_g > 3$: $V_{ca} = 2 \times V_{cs}$

where:

N_g	number of generating units;
V_{cf}	volume of concrete for the infrastructure, in m³;
V_{cs}	volume of concrete for the superstructure, in m³;
V_{ce}	volume of concrete for one wall at each end, in m³;
V_{cd}	volume of concrete for when additional excavation is required because of poor foundations, in m³;
V_{cn}	volume of concrete resulting from alterations to the project design so that the maximum water level in the tailrace canal is higher than the elevation of the generator floor, in m³;
V_{ca}	volume of concrete for the assembly area, in m³; and
D_K	diameter of the turbine rotor, in m.

The volume of **concrete**, V_{ccf} (m³), for a **semi-outdoor powerhouse** is given by:

$$V_{ccf} = N_g \times (V_{cf} + V_{cs}) + V_{ce} + V_{cd} + V_{cn} + V_{ca}$$

where:

N_g	number of generating units;
V_{cf}	volume of concrete for the infrastructure, in m³;
V_{cs}	volume of concrete for the superstructure, in m³;
V_{ce}	volume of concrete for one wall at each end, in m³;
V_{cd}	volume of concrete for when additional excavation is required because of poor foundations, in m³;
V_{cn}	volume of concrete resulting from alterations to the project design so that the maximum water level in the tailrace canal is higher than the elevation of the generator floor, in m³;
V_{ca}	volume of concrete for the assembly area, in m³; and
D_K	diameter of the turbine rotor, in m.

The amounts of cement and reinforcement steel are as follows:

	cement (kg/m³)	reinforcement steel (kg/m³)
infrastructure	275	50
superstructure	300	100
end wall	250	75
dental concrete	200	0
shotcrete	300	70

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the construction site, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the construction site, storage, preparation and installation.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of the powerhouse volume (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- concrete for the infrastructure and end walls: 214.00/m³
- dental concrete: 113.00/m³
- concrete for the superstructure: 214.00/m³

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Installations and final work (account .11.13.00.15)

The **cost of installations and final works**, C_{ia} (R$), which covers all services for the final work on the powerhouse, such as dividing walls, coatings, installations, door and window frames, glass windows, etc., is obtained as a global cost using the expression below (or Graph B 20, as a function of installed capacity). It is valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996a):

valid for $30 \le P \le 1450$ MW: $C_{ia} = 6{,}150 \times P^{1 + \frac{15.34}{P}}$

where:

P	installed capacity, in MW.

Land developments in the plant area (account .11.12)

The **cost of land development in the plant area**, C_{bau} (R$), which encompasses building the internal access roads to the different structures, guard houses and perimeter walls, landscaping, and others, is obtained as a global cost using the expression below (or Graph B 19, annex B, as a function of installed capacity). It is valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996a):

valid for $30 \le P \le 1450$ MW: $C_{bau} = 1{,}565 + \left(\frac{772{,}973}{P}\right)$

where:

P	installed capacity, in MW.

Operators' Village (account .11.14)

This cost is included in the workers' camp account (account .17.21).

Turbines (account .13.13.00.23.28)

The **acquisition cost of each Kaplan turbine** with a semi-spiral steel casing, C_{tkc} (R$), which includes the electromechanical equipment, parts and materials normally supplied by the manufacturers – FOB cost excluding transportation, insurance, assembly and testing costs and provisions for charges and taxes payable according to the applicable tax legislation – can be obtained from the expression below (or from Graph. B 11, annex B, as a function of the unit capacity of the turbine and the synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $20 \le z \le 2500: C_{tkc} = -0.0058 \times z^2 + 40.609 \times z + 3{,}122.5$

for: $z = \frac{P_{1t}}{n}$

where:

Z	parameter, in kW/rpm;
P_{1t}	capacity of one turbine, in kW; and
n	synchronous velocity, in rpm.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for the taxes and charges payable on the equipment.

Stoplogs for the draft tube (account .13.13.00.23.16)

The **number of stoplogs**, N_{sl}, is given by the following expressions:

for $N_g \le 10: N_{sl} = 2 \times N_{vs}$

for $N_g > 10: N_{sl} = 3 \times N_{vs}$

where:

N_g	number of generating units; and
N_{vs}	number of openings for each draft tube.

The **acquisition cost of each stoplog** for the draft tube, C_{sl} (R$), – FOB cost – can be obtained from the expression below (or from Graph. B 25, annex B, as a function of its dimensions and hydrostatic load), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.16 \le z \le 54.5: C_{sl} = 72.9 \times z^{0.716}$

for: $z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$

$H_{cp} = R$

$H_x = NA_{xfu} - El_d + Y$

$B_{cp} = \frac{Z - U}{N_{vs}}$

where:

z	parameter, in m^4;
B_{cp}	width of the stoplog, in m;
H_{cp}	height of the stoplog, in m;
H_x	maximum hydrostatic load on the sill of the stoplog, in m;
R	height of the draft tube aperture at the outlet, in m;
NA_{xfu}	maximum water level in the tailrace canal;
El_d	elevation of the center line of the turbine distributor;
Y	height from the draft tube to the center of the distributor, in m;
Z	width of the draft tube, in m;
U	width of the pillar of the draft tube, in m; and
N_{vs}	number of openings for each draft tube.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

The **overall acquisition cost for the fixed parts and parts embedded in the concrete** of the stoplogs for the draft tube, C_{gpf} (R$), – FOB cost – is given by the expression below, valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

$$C_{gpf} = 2 \times N_{vs} \times N_g \times (H_x + 2.0) \times 2{,}084.80$$

where:

N_g	number of generating units;
H_x	maximum hydrostatic load on the sill of the stoplog, in m; and
N_{vs}	number of openings for each draft tube.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

Draft Tube Gantry Crane (account .13.13.00.23.20)

As the **acquisition cost of draft tube gantry crane** is low, it can be ignored at this stage.

Generators (account .13.13.00.23.29)

The **acquisition cost of each vertical-axis generators**, C_{gv} (R$), including the voltage regulator and auxiliary electromechanical equipment – FOB cost – can be obtained from the expression below (or from Graph. B 16, annex B, as a function of the ratio between the generator capacity and its number of poles and taking the synchronous velocity into account), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.0004 \le \lambda \le 0.0483$: $C_{gv} = 42280 \times \lambda^{0.6298}$

for: $\lambda = \dfrac{P_2}{n}$ and $P_2 = \dfrac{P_1}{f_p}$

where:

P_2	generator capacity, in MVA;
λ	magnetic torque, in MVA/rpm;
n	synchronous velocity, in rpm;
P_1	capacity of one generating unit, in MW; and
f_p	power factor.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for the taxes and charges payable on the equipment.

Auxiliary electrical equipment (account .14.00.00.23)

The **acquisition cost of the auxiliary electrical equipment** should be taken as 18% of the overall cost of account .13 – Turbines and Generators.

Bridge and gantry cranes (account .15.13.00.23.20)

The cargo handling system can make use of either one outdoor gantry crane or one or two indoor gantry cranes. The **acquisition cost of the crane or cranes**, C_{prv} (R$), – FOB cost – can be obtained from the expression below (or from Graph. B 17, annex B, as a function of the ratio between the generator capacity and its synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $68.9 \leq z \leq 4582$: $C_{prv} = 25.12 \times z^{0.6961}$

for: $z = 1000 \times \dfrac{P_2}{n}$

where:

z	parameter, in kVA/rpm;
P_2	generator capacity, in MVA; and
n	synchronous velocity, in rpm.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

The **acquisition cost of the gantry crane**, C_{pcr} (R$), – FOB cost – can be obtained from the expression below (or from Graph. B 18, annex B, as a function of the ratio between the generator capacity and its number of poles), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $68.9 \leq z \leq 4582$: $C_{pcr} = 59.506 \times z^{0.6621}$

for: $z = 1000 \times \dfrac{P_2}{n}$

where:

z	parameter, in kVA/rpm;
P_2	generator capacity, in MVA; and
n	synchronous velocity, in rpm.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

Miscellaneous Equipment (account .15.00.00.23.31)

The **acquisition cost of the miscellaneous equipment** should be taken as 6% of the overall cost of account .13 – Turbines and Generators.

POWERHOUSE EQUIPPED WITH BULB TURBINES

The main information required for dimensioning purposes is:

- mean elevation of the land in the powerhouse area, El_{te};, in m;
- mean thickness of the layer of soil in the powerhouse area, e_{te}, in m;
- maximum water level in the tailrace canal, NA_{xfu}, from item 5.1.2; and
- volume of concrete, V_{ccf} in m^3.

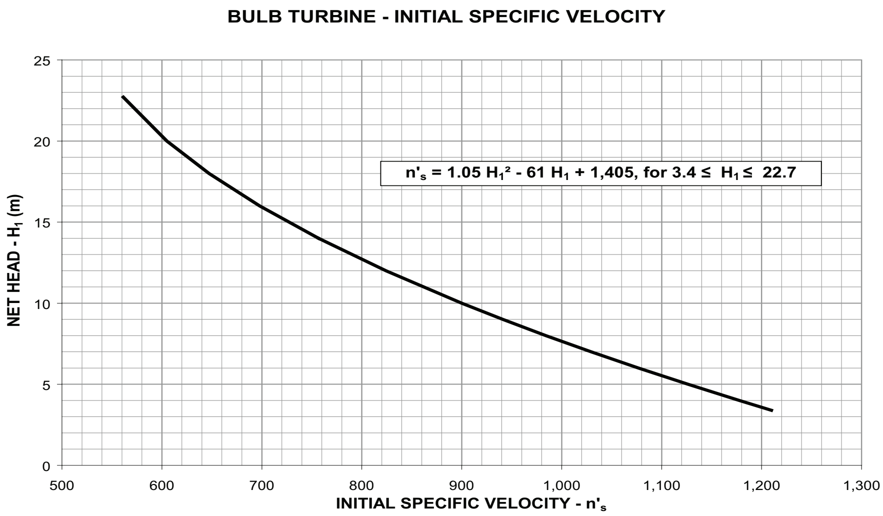
Velocities

The **specific initial velocity**, n'_s, can be obtained from Graph. 5.7.2.08 as a function of the maximum net head or from the equivalent expressions (Eletrosul, 1996):

for $3.4 \leq H_1 \leq 22.7$ m: $n'_s = 1.05 \times H_1^2 - 61 \times H_1 + 1405$

where:

H_1	maximum net head, in m



Graph 5.7.2.08 – Initial Specific Velocity – Bulb turbines.

The **initial velocity**, n' (rpm), is given by:

$$n' = n'_s \times H_1^{1.25} \times P_{1t}^{-0.5}$$

for: $P_{1t} = \dfrac{10^3 \times P_1}{\eta_g}$

where:

n'_s	specific initial velocity;
H_1	maximum net head, in m;
P_{1t}	unit capacity of the turbine, in kW;
P_1	capacity of one generating unit, in MW; and
η_g	generator output at maximum net head.

The **number of generator poles**, p, can be obtained from Table 5.7.2.01, as a function of the initial velocity, or from the equivalent expressions:

for $n' \geq 1.2 \times f$: $p = 2 \times \text{int}\left(120 \times \dfrac{f}{n'} \times \dfrac{1}{2} + 0.999\right)$

without using 54, 74 and 94

for $n' < 1.2 \times f$: $p = 4 \times \text{int}\left(120 \times \dfrac{f}{n'} \times \dfrac{1}{4} + 0.999\right)$

where:

f	frequency of the electricity system, in Hz;
n'	initial velocity, in rpm; and
int(x)	function that returns the integer part of x.

The **synchronous velocity**, n (rpm), is given by:

$$n = 120 \times \frac{f}{p}$$

where:

f	frequency of the electricity system, in Hz; and
p	number of generator poles.

The **specific velocity**, n_s, is given by:

$$n_s = n \times H_1^{-1.25} \times P_{1t}^{0.5}$$

where:

n	synchronous velocity, in rpm;
H_1	maximum net head, in m; and
P_{1t}	capacity of one turbine, in kW.

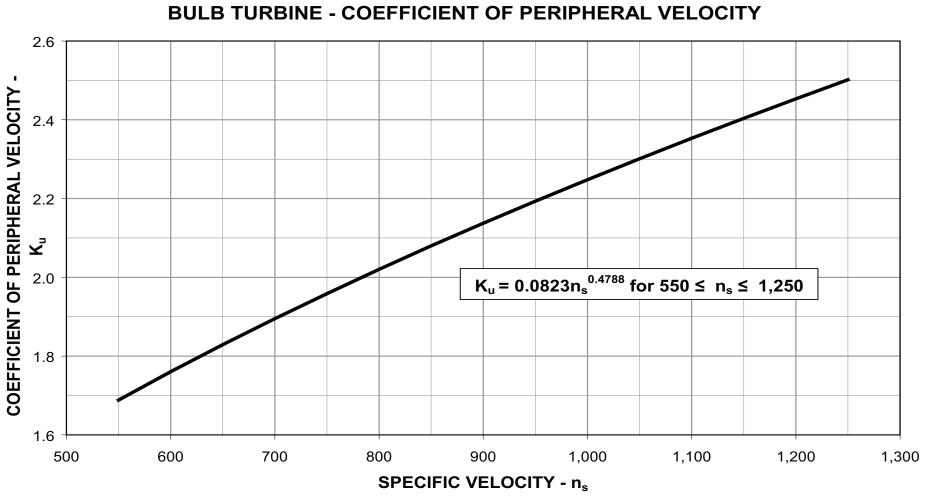
Diameter and position of the turbine rotor

The **coefficient of peripheral velocity**, K_u, can be obtained from Graph. 5.7.2.09 as a function of the specific velocity or from the equivalent expression (Cruz, 1995):

$$K_u = 0.0823 \times n_s^{0.4788}$$

where:

n_s	specific velocity



Graph 5.7.2.09 – Coefficient of peripheral velocity.

The diameter of the **turbine rotor**, D_K (m), is given by:

$$D_K = 0.01 \times \text{int}\left(84.5 \times K_u \times \frac{H_1^{0.5}}{n} \times \frac{1}{0.01} + 0.5\right)$$

where:

K_u	coefficient of peripheral velocity;
H_1	mean net head, in m; and
n	synchronous velocity.

The **suction head**, h_s (m), is given by:

$$h_s = K - \sigma \times H_1$$

for:

$$K = 10.33 - 0.0012 \times NA_{fu} - 0.013 \times T$$

$$\sigma = 0.0035 \times n_s - 1.12$$

where:

K	Variable, as a function of atmospheric pressure and the water vapor pressure, in m;
σ	Thoma coefficient;
H_1	maximum net head, in m;
NA_{fu}	normal water level in the tailrace canal;
T	mean water temperature in the summer, in °C; and
n_s	specific velocity.

The **installation elevation**, El_d, is given by:

$$El_d = NA_{nfu} + h_s$$

where:

NA_{nfu}	minimum water level downstream;
h_s	suction head, in m; and
D_K	diameter of the turbine rotor.

Main dimensions of the turbine

The **turbine dimensions** are given by the following expressions (Eletrosul, 1996). The dimensions in question are in Fig. 5.7.2.16 and 5.7.2.17.

$A = 2.25 \times D_k$ $B = 2.00 \times D_k$

$D_B = 1.25 \times D_k$ $L = 2.40 \times D_k$

$T = 2.87 \times D_k$ $Q = 1.58 \times D_k$

$R = 1.58 \times D_k$ $S = 5.12 \times D_k$

$X = 2.10 \times D_k$

where:

A, B, D_B, L, T, Q, R, S, X	turbine dimensions, in m; and
D_K	diameter of the turbine rotor, in m.

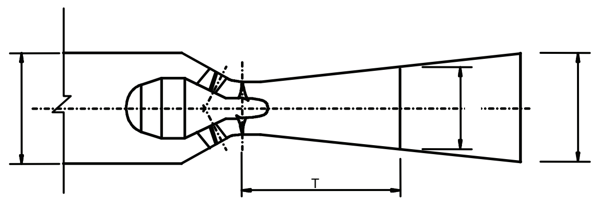


Fig. 5.7.2.16 – Plant of a unit with a Bulb turbine.

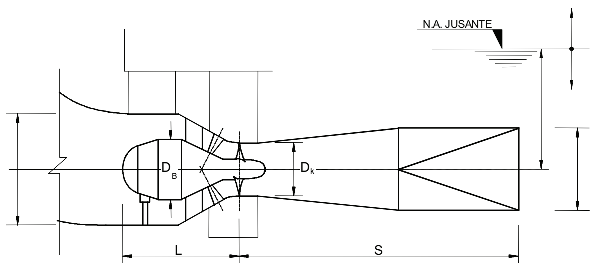


Fig. 5.7.2.17 – Cross-section of a unit with a Bulb turbine.

Dimensions of the powerhouse

The **width of a block of the unit of the powerhouse** (perpendicular to flow), B_{1cf} (m), is given by:

$$B_{1cf} = 2.55 \times D_k$$

where:

D_K	diameter of the turbine rotor, in m.

The total width of the powerhouse, B_{cf} (m), excluding the assembly area, is given by:

$$B_{cf} = N_g \times B_{1cf} + 2.0$$

where:

N_g	number of generating units; and
B_{1cf}	width of a block of the unit of the powerhouse, in m.

The **width of the equipment assembly area**, B_{am} (m), is given by:

for $N_g \leq 3: B_{am} = 1.5 \times B_{1cf}$

for $N_g > 3: B_{am} = 2.25 \times B_{1cf}$

where:

B_{1cf}	width of a block of the unit of the powerhouse, in m; and
N_g	number of generating units.

The **length of the assembly area**, L_{cs} (m), is given by:

$$L_{cs} = 2 \times L$$

where:

L	turbine dimensions, in m.

The **length of the powerhouse**, L_{cf} (m), is given by:

$$L_{cf} = L + S$$

where:

L	turbine dimensions, in m; and
S	length of the draft tube, in m.

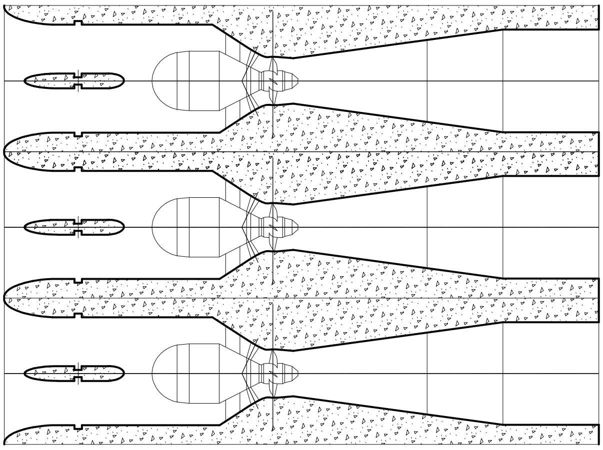


Fig. 5.7.2.18 – Plan of the powerhouse – Bulb turbine.

Common excavation (account .11.13.00.12.10)

In the absence of more accurate information, the common excavation volume, V_{tcf} (m^3), for a **surface powerhouse**, can be determined by:

$$V_{tcf} = (B_{cf} + B_{am} + 2 \times B_{1cf} + 2 \times 0.6 \times h_r) \times L_{cf} \times e_{te}$$

for: $h_r = El_{te} - e_{te} - (NA_{xfu} + 1.5)$

where:

B_{cf}	width of the powerhouse, in m;
B_{am}	width of the assembly area, in m;
B_{1cf}	width of a block of the unit of the powerhouse, in m;
h_r	mean depth of excavation in rock above the elevation of the assembly area, in m;
L_{cf}	length of the powerhouse, in m;
e_{te}	mean thickness of the layer of soil in the powerhouse area, in m;
El_{te}	mean elevation of the land in the powerhouse area; and
NA_{xfu}	maximum water level in the tailrace canal.

The unit price of common excavation is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .11.13.00.12.11)

In the absence of more accurate information, the excavation in rock, V_{tcf} (m^3), for the **powerhouse**, can be determined by:

$$V_{rcf} = V_{re} + V_{rp} + V_{rd}$$

for:

$$V_{re} = \left(B_{cf} + B_{am} + 2 \times B_{1cf} + 0.6 \times h_r\right) \times L_{cf} \times h_r$$

$$V_{rp} = B_{cf} \times L_{cf} \times \left(NA_{xfu} + 1.5 - El_d + \frac{A}{2}\right)$$

where:

V_{re}	volume of excavation in rock above the elevation of the assembly area, in m^3;
V_{rp}	volume of excavation in rock below the elevation of the assembly area, in m^3;
B_{cf}	width of the powerhouse, in m;
B_{am}	width of the assembly area, in m;
B_{1cf}	width of a block of the unit of the powerhouse, in m;
h_r	mean depth of excavation in rock above the elevation of the assembly area, in m;
L_{cf}	length of the powerhouse, in m;
NA_{xfu}	maximum water level in the tailrace canal;
El_d	elevation of the center line of the turbine distributor; and
A	height of the opening at the inlet, in m.

The unit price of excavation in rock is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the

vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .11.13.00.13)

The area of foundation to be cleaned, A_{lf} (m^2), of the **powerhouse** is given by:

$$A_{lf} = B_{cf} \times L_{cf} + B_{am} \times L_{cs}$$

where:

B_{cf}	width of the powerhouse, in m;
L_{cf}	length of the powerhouse, in m;
B_{am}	width of the assembly area, in m; and
L_{cs}	length of the assembly area, in m.

The **length of the grout holes**, L_{tf} (m), for treating the powerhouse foundations**,** is given by:

$$L_{tf} = \frac{B_{cf}}{3} \times L_{1tf}$$

for:

$$L_{1tf} = 1.5 \times \left(NA_{xfu} - El_d + \frac{R}{2} \right)$$

where:

B_{cf}	width of the powerhouse, in m;
L_{1tf}	length of one grout hole, in m;
NA_{xfu}	maximum water level in the tailrace canal;
El_d	elevation of the center line of the turbine distributor;
R	height of the opening for the draft tube, in m; and
3.0	spacing between the grout holes, in m.

The unit prices for foundation cleaning and treatment services, expressed in Brazilian Reais (valid for the December 2006 database), can be used for projects in the south, southeast, central west and northeast regions of Brazil. They include the execution of the work, supply of inputs and equipment, and depend on the kind of surface and the equipment to be used. The unit prices are:

- cleaning of the rock surface: 39.70/m^2
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .11.13.00.14)

The volume of **concrete** should be determined from the project design.

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the construction site, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the construction site, storage, preparation and installation.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of the powerhouse volume (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- concrete for the infrastructure and end walls: 214.00/m^3
- dental concrete: 113.00/m^3
- concrete for the superstructure: 214.00/m^3.

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Installations and final work (account .11.13.00.15)

The **cost of installations and final works**, C_{ia} (R$), which covers all services for the final work on the powerhouse, such as dividing walls, coatings, installations, door and window frames, glass windows, etc., is obtained as a global cost using the expression below (or Graph B 20, as a function of installed capacity). It is valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996a):

valid for $30 \leq P \leq 1450$ MW: $C_{ia} = 6{,}150 \times P^{1 + \frac{15.34}{P}}$

where:

P	installed capacity, in MW.

Land developments in the plant area (account .11.12)

The **cost of land developments in the plant area**, C_{bau} (R$), which encompasses building the internal access roads to the different structures, guard houses and perimeter walls, landscaping, and others, is obtained as a global cost using the expression below (or Graph B 19, annex B, as a function of installed capacity). It is valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996a):

valid for $30 \leq P \leq 1450$ MW: $C_{bau} = 1{,}565 + \left(\frac{772{,}973}{P}\right)$

where:

P	installed capacity, in MW.

Operators' Village (account .11.14)

This cost is included in the workers' camp account (account .17.21).

Turbines (account .13.13.00.23.28)

The **acquisition cost of each Bulb turbine**, C_{tb} (R$), which includes the electromechanical equipment, parts and materials normally supplied by the manufacturers – FOB cost excluding transportation, insurance, assembly and testing costs and provisions for charges and taxes payable according to the applicable tax legislation – can be obtained from the expression below (or from Graph. B 13, annex B, as a function of the unit capacity of the turbine and the synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $30 \le P \le 700$: $C_{tb} = 39.434 \times z + 3{,}791.7$

for: $z = \dfrac{P_{1t}}{n}$

where:

z	parameter, in kW/rpm;
P_{1t}	capacity of one turbine, in kW; and
n	synchronous velocity, in rpm.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for the taxes and charges payable on the equipment.

Stoplogs for the draft tube (account .13.13.00.23.16)

The number of stoplogs, N_{sl}, is given by the following expressions:

for $N_g \le 10$: $N_{sl} = 2$

for $N_g > 10$: $N_{sl} = 3$

where:

N_g	number of generating units.

The **acquisition cost of each stoplog** for the draft tube, C_{sl} (R$), – FOB cost – can be obtained from the expression below (or from Graph. B 25, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.16 \le z \le 54.5$: $C_{sl} = 72.9 \times z^{0.716}$

for:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000} \qquad H_{cp} = R$$

$$H_x = NA_{xfu} - El_d + \frac{R}{2} \qquad B_{cp} = Q$$

where:

z	parameter, in m^4;
B_{cp}	width of the stoplog, in m;
H_{cp}	height of the stoplog, in m;
H_x	maximum hydrostatic load on the sill of the stoplog, in m;
R	height of the opening for the draft tube at the outlet, in m;

NA_{xfu}	maximum water level in the tailrace canal;
El_d	elevation of the center line of the turbine distributor; and
Q	width of the draft tube, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

The **overall acquisition cost for the fixed parts and parts embedded in the concrete** of the stoplogs for the draft tube, C_{gpf} (R$), – FOB cost – is given below, valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

$$C_{gpf} = 2 \times N_g \times (H_x + 2.0) \times 2{,}084.80$$

where:

N_g	number of generating units; and
H_x	maximum hydrostatic load on the sill of the stoplog, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

Draft Tube Gantry Crane (account .13.13.00.23.20)

As the **acquisition cost of draft tube gantry crane** is low, it can be ignored at this stage.

Generators (account .13.13.00.23.29)

The **acquisition cost of each horizontal-axis Bulb generator**, C_{gb} (R$), including the voltage regulator and auxiliary electromechanical equipment – FOB cost – can be obtained from the expression below (or from Graph. B 15, annex B, as a function of the ratio between the generator capacity and its number of poles), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

For $0.0396 \le \lambda \le 0.9289$: $C_{gb} = 34120 \times \lambda^{0.7091}$

for: $\lambda = \dfrac{P_2}{n}$ and $P_2 = \dfrac{P_1}{f_p}$

where:

λ	magnetic torque, in MVA/rpm;
P_2	generator capacity, in MVA;
N	synchronous velocity, in rpm;
P_1	capacity of one generating unit, in MW; and
f_p	power factor.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for the taxes and charges payable on the equipment.

Auxiliary electrical equipment (account .14.00.00.23)

The **acquisition cost of the auxiliary electrical equipment** should be taken as 18% of the overall cost of account .13 – Turbines and Generators.

Bridge and gantry cranes (account .15.13.00.23.20)

The cargo handling system usually makes use one indoor bridge crane. The **acquisition cost of the crane**, C_{prv} (R$), – FOB cost – can be obtained from the expression below (or from Graph. B 17, annex B, as a function of the ratio between the generator capacity and its synchronous velocity), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $68.9 \leq z \leq 4582$: $C_{prv} = 25.12 \times z^{0.6961}$

for: $z = 1000 \times \frac{P_2}{n}$

where:

z	parameter, in kVA/rpm;
P_2	generator capacity, in MVA; and
n	synchronous velocity, in rpm.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

The **acquisition cost of the gantry crane**, C_{pcr}, – FOB cost – can be obtained from the expression below (or from Graph. B 18, annex B, as a function of the ratio between the generator capacity and its number of poles), in Brazilian Reals, valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $68.9 \leq z \leq 4582$: $C_{pcr} = 59.506 \times z^{0.6621}$

for: $z = 1000 \times \frac{P_2}{n}$

where:

z	parameter, in kVA/rpm;
P_2	generator capacity, in MVA; and
n	synchronous velocity, in rpm.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

Miscellaneous Equipment (account .15.00.00.23.31)

The **acquisition cost of miscellaneous equipment** should be taken as 6% of the overall cost of account .13 – Turbines and Generators.

5.7.3 River Diversion (account .12.16)

GENERAL

The main information used for calculating this item can be obtained from the overall layout of the project and the hydrometeorological studies, namely:

- topography;
- geological features;
- type of dam;
- type of diversion;
- flood flow / recurrence time curve;

The project flow through the river diversion can be obtained from the hydrology studies as a function of the **recurrence time**, T_r (years), which is obtained from:

$$T_r = 5 \times \text{int}\left(\frac{1}{1 - \sqrt[t]{1-R}} \times \frac{1}{5} + 0.999 \right)$$

where:

T_r	recurrence time of the diversion flow, in years;
t	during the stage of diversion under study, in whole years; and
R	risk, defined as the likelihood of flooding during the period of exposure.

for:

risk	Diversion Scheme
3%	through tunnels or galleries in layouts with an earthfill dams
5%	through tunnels or galleries in layouts with a rockfill dams
5%	by narrowing the river bed, in layouts with an earthfill dam
2%	through sluiceways in layouts with an earthfill dam
5%	by narrowing the river bed, in layouts with a rockfill dam
3%	through sluiceways, in layouts with a rockfill dam and
10%	when the structures at risk are made of concrete

COFFERDAMS (ACCOUNT .12.16.22)

Cofferdams for diverting the river through tunnels or galleries

The basic data needed for this are:

- water level upstream from the upstream cofferdam, NA_{dm}, from this item;
- water level downstream from the downstream cofferdam, NA_{dj}, from item 5.1.2.;
- freeboard, H_{bl};
- thickness of the layer of topsoil (material removed from the foundations), e_{te};
- length of the cofferdam k, L_{dk}, in m;
- number of sections in the cofferdam k, n_k; and
- elevation of the bottom of the river or the land in sections i along the axis of cofferdam k, El_{teki}.

Earth-rock cofferdam (account .12.16.22.19)

The **volume of the cofferdam**, V_d (m^3), is given by:

$$V_d = \sum_k V_{dk}$$

for:

$$V_{dk} = V_{dek} + V_{dak} + V_{dtk}$$

$$V_{dtk} = 0.15 \text{ x } V_{dek}$$

$$V_{dek} = \left(1.5 \times H_{dk}^2 + 7 \times H_{dk}\right) \times L_{dk}$$

$$V_{dak} = \left(0.75 \times H_{dk}^2 + 3 \times H_{dk}\right) \times L_{dk}$$

$$H_{dk} = \sqrt{\frac{1}{n_k} \sum_i H_{dki}^2}$$

$$H_{dki} = NA_{dk} + H_{bl} - (El_{teki} - e_{te})$$

k	For
m	upstream cofferdam
j	downstream cofferdam

where:

V_{dk}	volume of the cofferdam k, in m^3;
V_{dek}	volume of rockfill for the cofferdam k, in m^3;
V_{dak}	volume of earthfill for the cofferdam k, in m^3;
V_{dtk}	volume of transitions for the cofferdam k, in m^3;
H_{dk}	mean height of the cofferdam k, in m;
L_{dk}	length of the cofferdam k, in m;
H_{dki}	height of the cofferdam k at section i, in m;
n_k	number of sections in the cofferdam k;
NA_{dk}	water level on the outside of the cofferdam k;
H_{bl}	freeboard, in m;
El_{teki}	elevation of the river bed or land in section i, along the axis of cofferdam k; and
e_{te}	thickness of the layer of topsoil (material removed from the foundations), in m.

The cost of building cofferdams will depend on the kind of section and above all on the provenance of the building materials.

The unit prices for miscellaneous **earthfill services**, expressed in Brazilian Reais, valid for December 2006 and for projects in the south, southeast, central west and northeast regions of Brazil, are:

- necessary excavation of compacted rockfill: 1.97/m^3;
- necessary excavation of compacted earthfill: 2.69/m^3;
- transitions, filters and drains: 19.49/m^3;
- dumped rockfill from quarry: 13.76/m^3;
- compacted rockfill from quarry: 15.18/m^3;
- dumped earthfill from quarry: 7.12/m^3;
- compacted earthfill from quarry: 7.93/m^3.

This value corresponds to the price per cubic meter measured using the cross-section of the earthfill or rockfill as defined by the design lines of the cofferdam, and includes only spreading and compactation services. The unit price of the material from the borrow area should be added to the transportation cost, according to the type of material and mean distance it is to be transported:

- rockfill: 2.21/m^3.km
- earthfill: 2.55/m^3.km

When the cofferdams are made of dumped earthfill and rockfill from necessary excavations, the cost of the earthfill can be taken as zero. However, when the area for dumping at the extreme edge of the earthfill or rockfill is narrow and hard to access, the unit prices of dumped earthfill and rockfill can be estimated as being 50% of the cost of the compaction service.

In order to make up for differences in volumes, in each situation an estimate should be made of the proportion of volumes per kind of service, from which the mean weighted construction cost of the cofferdam can be determined.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Removal of cofferdams (account .12.16.22.21)

The **volume of cofferdam to be removed**, V_{dr} (m^3), is given by:

$$V_{dr} = \sum_k V_{drk}$$

for: $$V_{drk} = \left(2.25 \times H_{drk}^2 + 10 \times H_{drk}\right) \times L_{drk}$$

where:

V_{drk}	volume of cofferdam to be removed k, lengthwise to the river, in m^3;
H_{drk}	mean height of the part of cofferdam k to be removed, in m; and
L_{drk}	length of the section of cofferdam k to be removed, in m.

The mean unit price for **removing a cofferdam** above and below the water level is R$ 6.60/m^3 (December 2006 price), for projects in the south, southeast, central west and northeast regions of Brazil. This corresponds to the price per cubic meter and includes excavating and loading using earthfill equipment and transportation up to 1.5 km.

The cost of removing special cofferdams should be allocated, exceptionally, to their building cost.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Dewatering and other costs (account .12.16.22.22)

The cost of dewatering the dried area and keeping it in a suitable state during the construction period, as well as other miscellaneous costs, can be calculated as being 15% of the total cost of the cofferdam.

Cofferdams to divert the river in different phases

The **basic data used for this calculation** are:

- water level upstream from the upstream section of the first-phase cofferdam, NA_{dm1};
- water level downstream from the downstream section of the first-phase cofferdam, NA_{dj1};
- water level upstream from the upstream section of the second-phase cofferdam, NA_{dm2};
- water level downstream from the downstream section of the second-phase cofferdam, NA_{dj2};
- water level on the outside of the longitudinal cofferdam in section i, NA_{dli} ;
- elevation of the river bottom or land in sections i along the axis of cofferdam k, El_{teki};
- number of sections in cofferdam k, n_k;
- length of the upstream section of the first-phase cofferdam, L_{dm1}, in m;
- length of the downstream section of the first-phase cofferdam, L_{dj1}, in m;
- length of the upstream section of the second-phase cofferdam, L_{dm2}, in m;
- length of the downstream section of the second-phase cofferdam, L_{dj2}, in m;
- elevation of the river bottom or land in sections i along the axis of cofferdam l, El_{teli};
- number of sections in cofferdam l, n_l;
- length of the cofferdam longitudinal to the river, L_{dl}, in m; and
- volume of concrete for the deflector baffle, V_{cd} in m^3, when required.

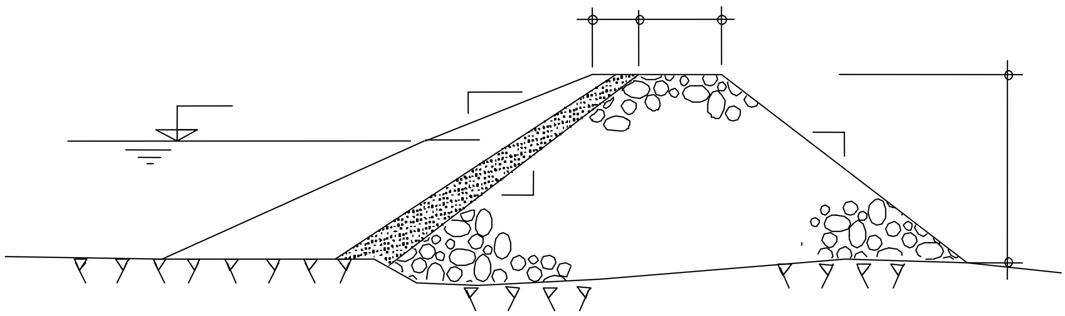


Fig 5.7.3.01 – Cross-section of a cofferdam across the river.

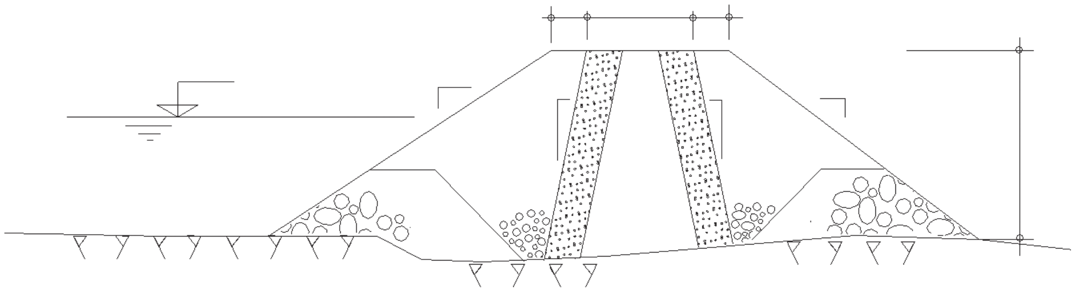


Fig 5.7.3.02 – Cross-section of a longitudinal cofferdam.

Earth-rock cofferdam (account .12.16.22.19)

The **volume of the cofferdam**, V_d (m³), is given by:

$$V_d = \sum_k V_{dk} + V_{dl}$$

for:

$$V_{dk} = V_{dek} + V_{dak} + V_{dtk}$$

$$V_{dek} = \left(1.5 \times H_{dk}^2 + 7 \times H_{dk}\right) \times L_{dk}$$

$$V_{dak} = \left(0.75 \times H_{dk}^2 + 3 \times H_{dk}\right) \times L_{dk}$$

$$V_{dtk} = 0.15 \text{ x } V_{dek}$$

$$H_{dk} = \sqrt{\frac{1}{n_k} \sum_i H_{dki}^2}$$

$$H_{dki} = NA_{dk} + 2 - El_{teki}$$

$$V_{dl} = V_{del} + V_{dal}$$

$$V_{del} = \left(1.3 \times H_{dl}^2 + 4 \times H_{dl}\right) \times L_{dl}$$

$$V_{dal} = \left(0.2 \times H_{dl}^2 + 6 \times H_{dl}\right) \times L_{dl}$$

$$H_{dl} = \sqrt{\frac{1}{n_l} \sum_i H_{dli}^2}$$

$$H_{dli} = NA_{dli} + 2 - El_{teli}$$

k	for
m1	upstream first-phase cofferdam
j1	downstream first-phase cofferdam
m2	upstream second-phase cofferdam
j2	downstream second-phase cofferdam

where:

V_{dk}	volume of cofferdam k across the river rio, in m³;
V_{dek}	volume of rockfill for cofferdam k, in m³;
V_{dak}	volume of earthfill for cofferdam k, in m³;
V_{dtk}	volume of transition for cofferdam k, in m³;
H_{dk}	mean height of cofferdam k, in m;
L_{dk}	length of cofferdam k, in m;
H_{dki}	height of cofferdam k in section i, in m;
n_k	number of sections in cofferdam k;
NA_{dk}	water level on the outside of cofferdam k;
El_{teki}	elevation of the river bottom or land in section i, along the axis of cofferdam k;
V_{dl}	volume of longitudinal cofferdam, in m³;
V_{del}	volume of rockfill for the longitudinal cofferdam, in m³;
V_{dal}	volume of earthfill for longitudinal cofferdam, in m³;
H_{dl}	mean height of longitudinal cofferdam, in m;
L_{dl}	length of longitudinal cofferdam, in m;
1.50	coefficient for offsetting the unit price of the inclined inclined filter for the earthfill;
H_{dli}	height of the longitudinal cofferdam at section i, in m;

n_l	number of sections in cofferdam l;
NA_{dli}	water level on the outside of the longitudinal cofferdam at section i; and
El_{teli}	elevation of the river bottom or land at section i, along the axis of the longitudinal cofferdam.

The cost of building cofferdams will depend on the kind of section and above all on the provenance of the building materials.

The unit prices for miscellaneous **earthfill services**, expressed in Brazilian Reais, valid for December 2006 and for projects in the south, southeast, central west and northeast regions of Brazil, are:

- necessary excavation of compacted rockfill: 1.97/m^3;
- necessary excavation of compacted earthfill: 2.69/m^3;
- transitions and filtros: 19.49/m^3;
- dumped rockfill from quarry: 13.76/m^3;
- compacted rockfill from quarry: 15.18/m^3;
- dumped earthfill from quarry: 7.12/m^3;
- compacted earthfill from quarry: 7.93/m^3.

This value corresponds to the price per cubic meter measured using the cross-section of the earthfill or rockfill as defined by the design lines of the cofferdam, and includes only spreading and compactation services. The unit price of the material from the borrow area should be added to the transportation cost, according to the type of material and mean distance it is to be transported:

- rockfill: 2.21/m^3.km
- earthfill: 2.55/m^3.km

When the cofferdams are made of dumped earthfill and rockfill from necessary excavations, the cost of the earthfill can be taken as zero. However, when the area for dumping at the extreme edge of the earthfill or rockfill is narrow and hard to access, the unit prices of dumped earthfill and rockfill can be estimated as being 50% of the cost of the compaction service.

In order to make up for differences in volumes, in each situation an estimate should be made of the proportion of volumes per kind of service, from which the mean weighted construction cost of the cofferdam can be determined.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.16.22.14)

The **volume of concrete in the cofferdam**, V_{cd} (m^3), corresponds to the volume for the baffle deflector and should be defined from the project design, in the same way as the volumes of cement and reinforcement steel.

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the construction site, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the construction site, storage, preparation and installation.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of the powerhouse volume (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Removal of cofferdams (account .12.16.22.21)

The **volume of cofferdam to be removed**, V_{dr} (m^3), is given by:

$$V_{dr} = \sum_k V_{drk} + V_{drl}$$

for:

$$V_{drk} = \left(2.25 \times H_{drk}^2 + 10 \times H_{drk}\right) \times L_{drk}$$

$$V_{drl} = \left(1.5 \times H_{drl}^2 + 10 \times H_{drl}\right) \times L_{drl}$$

where:

V_{drk}	volume of transverse cofferdam, k, to be removed, in m^3;
V_{drl}	volume of longitudinal cofferdam to be removed, in m^3;
H_{drk}	mean height of the part of cofferdam k to be removed, in m;
L_{drk}	length of the section of cofferdam k to be removed, in m;
H_{drl}	mean height of the part of the longitudinal cofferdam to be removed, in m; and
L_{drl}	length of the section of the longitudinal cofferdam to be removed, in m.

The mean unit price for **removing a cofferdam** above and below the water level is R$ 6.60/m^3 (December 2006 price), for projects in the south, southeast, central west and northeast regions of Brazil. This corresponds to the price per cubic meter and includes excavating and loading using earthfill equipment and transportation up to 1.5 km.

The cost of removing special cofferdams should be allocated, exceptionally, to their building cost.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Dewatering and other costs (account .12.16.22.22)

The cost of dewatering the dried area and keeping it in a suitable state during the construction period, as well as other miscellaneous costs, can be calculated as being 15% of the total cost of the cofferdam.

Special Cofferdams (account .12.16.22.20)

The cost of building and removing **special cofferdams** will depend on the kind of structure used and can be calculated after undertaking specific market research.

The cost of dewatering the dried area and keeping it in a suitable state during the construction period, as well as other miscellaneous costs, can be calculated as being 15% of the total cost of an equivalent earth-rock cofferdam.

DIVERSION TUNNELS (ACCOUNT .12.16.23)

Basic data

The main **information required for dimensioning purposes** is:

- design flow in the diversion for a recurrence time of k years, Q_k in m^3/s, from item 5.1.2.;
- length of the tunnels, L_{td}, in m;

- deflections from the tunnels' axis, δ, in degrees;
- length of the section lined with structural concrete, L_c, in m;
- length of the section lined with shotcrete, L_{cp}, in m;
- natural water level at the outlet of the downstream canal from the tunnels for the design flow in the diversion, NA_{dcr}, from item 5.1.2, in m;
- minimum physical elevation of the sill at the tunnel inlet, El_{den}, when there is some restriction, in m; and
- type of inlet to the tunnel.

The **information required for quantification purposes** is:

- maximum normal water level in the reservoir, NA_{max}, from item 4.6, in m;
- geological conditions of the region crossed by the tunnels;
- mean thickness of the layer of soil in the area of the diversion tunnels, e_{te}, in m;
- mean elevation of the land in section i – 0, 1 and 2 as indicated in Fig. 5.7.5.05 – perpendicular to the longitudinal axis of the approach channel, El_{tai}, in m;
- mean elevation of the land in section i – 0, 1 and 2 as indicated in Fig. 5.7.5.05 – perpendicular to the longitudinal axis of the downstream channel, El_{tri}, in m;
- length of the approach channel, L_{ca}, in m; and
- length of the downstream channel, L_{cr}, in m.

Considerations and recommendations

This text applies to tunnels with a typical cross-section, such as shown in Figures. 5.7.3.03 and 5.7.3.04.

In order to plug the tunnels, there may be no need to exclude water from the stretch downstream from the structure. Otherwise, a cofferdam can be built in the downstream channel, although the respective costs of such must be taken into account.

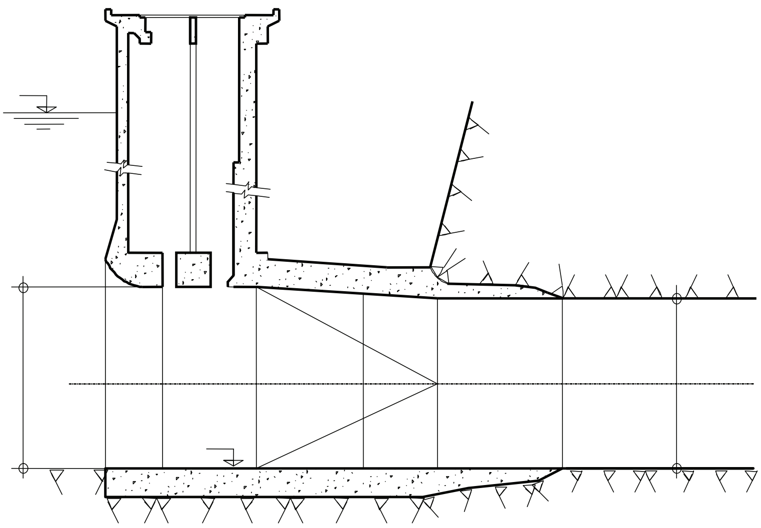


Fig. 5.7.3.03 – Typical longitudinal section of a diversion channel.

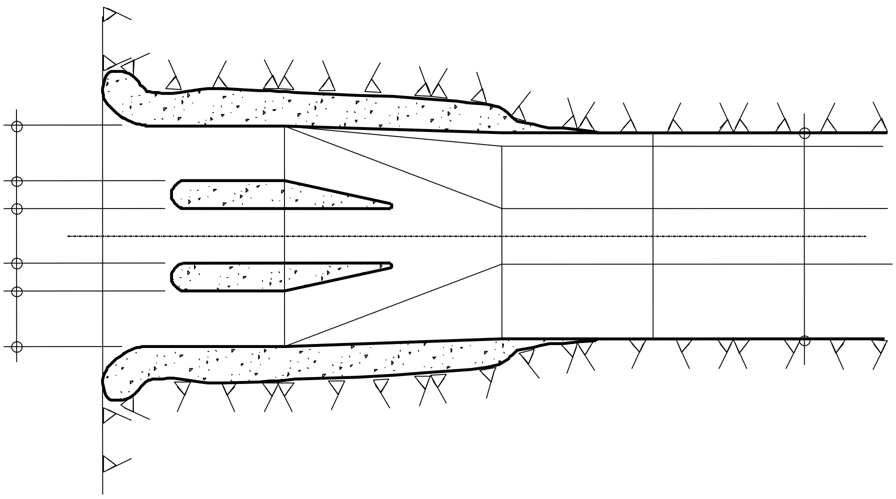


Fig. 5.7.3.04 – Plan of a diversion channel.

When defining the profile and diameter, there are some restrictions that must be respected, but also some suggestions for overcoming them:

- one physical restriction on setting the minimum elevation of the sill at the tunnel inlet is when the elevation determined by the spreadsheet is too low, the result of rapids in the section in question, for instance.
- In order to comply with the minimum **diameter restrictions**, the mean velocity of discharge through the tunnels can be reduced.

One example of a minimum physical elevation of the sill at the tunnel inlet, El_{den}, due to some restriction, is when there is a rapid between the inlet to the approach channel and the outlet of the downstream channel, and the approach channel is not long enough to include a ramp that would raise the elevation to the level required.

Number and diameter of diversion tunnels

Iniially, the **mean velocity of discharge in the tunnels**, v_{td} (m/s), is obtained by:

$$V_{td} = 0.8 \times V_{max}$$

for:

v_{max}	Type of Lining
10.0	unlined tunnels or tunnels lined with shotcrete
15.0	tunnels lined with structural concrete

where:

v_{max}	maximum discharge velocity allowed in the tunnels, in m/s

The **number of diversion tunnels**, N_{td}, in the absence of more accurate information, is given by:

$$N_{td} = \text{int}\left[\frac{Q_k}{201 \times v_{td}} + 0.99\right]$$

where:

int(x)	function that returns the integer part of the argument;
Q_k	design flow through the diversion, in m^3/s; and
v_{td}	mean velocity of discharge in the tunnels, in m/s.

The **internal diameter of the diversion tunnels**, D_{td} (m), is given by:

$$D_{td} = \sqrt{\frac{Q_k}{0.8927 \times N_{td} \times v_{td}}} \geq 2.0\text{ m}$$

where:

Q_k	design flow through the diversion, in m³/s; and
v_{td}	mean velocity of discharge in the tunnels, in m/s.

Course of the diversion tunnels

The **elevation of the sill at the tunnels' outlets**, El_{ds}, is given by:

$$El_{ds} = NA_{dcr} - 0.9 \times D_{td}$$

where:

NA_{dcr}	water level in the downstream channel of the tunnels, for flow Q_k; and
D_{td}	internal diameter of the tunnels, in m.

The **elevation of the sill at the inlet** of the diversion tunnels, El_{de}, is given by:

$$El_{de} = El_{ds} + 0.005 \times L_{td} \geq El_{den}$$

for $El_{de} = El_{den}$:

$$i_{td} = \frac{El_{de} - El_{ds}}{L_{td}} \leq 0.025$$

and for: $i_{td} = 0.025$

$$El_{ds} = El_{de} - i_{td} \times L_{td}$$

where:

El_{ds}	elevation of the sill at the tunnel outlet, in m;
i_{td}	slope of the tunnels, in m/m;
L_{td}	length of the tunnels, in m; and
El_{den}	minimum physical elevation of the sill at the tunnel inlet, in m.

The **elevation of the bottom of the approach channel** and **downstream channel**, El_{ca} and El_{cr}, is given by:

$$El_{ca} = El_{de} \qquad\qquad El_{cr} = El_{ds}$$

where:

El_{de}	elevation of the sill at the tunnel inlet, in m; and
El_{ds}	elevation of the sill at the tunnel outlet, in m.

Water level at the upstream cofferdam for tunnels with subcritical or critical flow

Subcritical or critical flow is when:

$$i_{td} \leq i_c$$

for:

$$i_c = 6.23 \times n^2 \times \frac{v_{td}^2}{D_{td}^{4/3}}$$

where:

i_{td}	slope of the tunnel, in m/m;
i_c	slope of the energy head line under critical streamflow, in m/m;
n	Manning's coefficient for the predominant lining;
v_{td}	mean velocity of discharge in the tunnels, in m/s; and
D_{td}	internal diameter of the tunnels, in m.

The water level **at the upstream cofferdam** for tunnels with subcritical or critical flow, NA_{dm}, is given by:

$$NA_{dm} = El_{ds} + D_{td} + \frac{v_{td}^2}{2 \times g} + h_p$$

for:

$$h_p = h_e + h_o + h_f \qquad h_e = k_e \times \frac{v_{cp}^2}{2 \times g}$$

$$h_o = \sum k_{oi} \times \frac{v_{td}^2}{2 \times g}$$

$$h_f = 6.23 \times [(L_{td} - L_c - L_{cp}) \times n^2 + L_c \times n_{cr}^2 + L_{cp} \times n_{cp}^2] \times \frac{v_{td}^2}{D_{td}^{4/3}}$$

$$V_{cp} = 0.8977 \text{ x } V_{td}$$

$$k_{oi} = 0.132 \times \frac{\delta}{90^\circ}$$

k_e	Type of inlet	n	Type of Lining
0.50	sharp angle	0.035	unlined
0.13	rounded	0.022	shotcrete
0.75	protruding into the reservoir	0.013	structural concrete

where:

El_{ds}	elevation of the sill at the tunnel intake, in m;
D_{td}	internal diameter of the tunnels, in m;
v_{td}	mean velocity of discharge in the tunnels, in m/s;
g	9.81 m/s^2 – acceleration due to gravity;
h_p	total head loss in the tunnels, in m;
h_e	head loss at the inlet, in m;
h_o	head loss at bends, in m;
h_f	continuous head loss throughout the tunnels, in m;
k_e	head loss coefficient at the inlet;
v_{cp}	mean flow rate at the section with gates, in m/s;
k_{oi}	head loss coefficient at bends in the tunnels;
L_{td}	length of the tunnels, in m;
L_c	length of the section lined with structural concrete, in m;
L_{cp}	length of the section lined with shotcrete, in m;
n	Manning's coefficient for the unlined section;
n_{cr}	Manning's coefficient for the section lined with structural concrete;
n_{cp}	Manning's coefficient for the section lined with shotcrete; and
δ	deflection of the tunnel axis, in degrees.

Water level at the upstream cofferdam for tunnels with supercritical flow

Flow is supercritical if: $i_{td} > i_c$

where:

i_{td}	slope of the tunnel, in m/m; and
i_c	slope of the energy head line under critical streamflow, in m/m.

The water level **at the upstream cofferdam** for tunnels with supercritical flow, NA_{dm}, can be obtained from Graph 5.7.3.01 (COPEL, 1977) or by:

$$NA_{dm} = El_{de} + H$$

for:

$$H = k_H \times H_{cp}$$

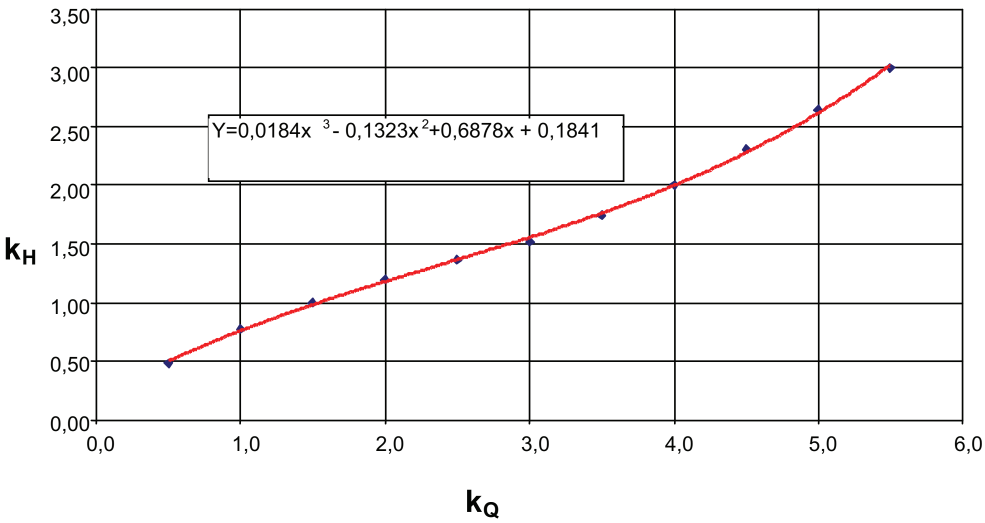
$$k_H = 0.0184 \times k_Q^3 - 0.1323 \times k_Q^2 + 0.688 \times k_Q + 0.18$$

$$k_Q = \frac{Q_k}{N_{td} \times N_v \times B_{cp} \times H_{cp}^{3/2}} \qquad N_v = \mathrm{int}\left(\frac{D_{td}}{4.5} + 0.9\right)$$

$$B_{cp} = \frac{0.88 \times D_{td}}{N_v} \qquad H_{cp} = 1.13 \times D_{td}$$

where:

El_{de}	elevation of the sill at the diversion tunnel intake , in m;
H	hydrostatic load upstream from the tunnels, in m;
k_Q, k_H	coefficients;
H_{cp}	height of the gates for the diversion tunnel, in m;
N_{td}	number of diversion tunnels;
N_v	number of openings for the inlet structure of each diversion tunnel;
B_{cp}	width of the gates for the diversion tunnel, in m;
D_{td}	internal diameter of the tunnels, in m; and
int(x)	function that returns the integer part of x.



Graph 5.7.3.01 – Hydrostatic load on the upstream side of the inlet structure.

Common excavation (account .12.16.23.12.10)

The **common excavation volume** for diversion tunnels, V_{ttd} (m³), in the absence of more accurate information, can be determined by:

$$V_{ttd} = V_{tca} + V_{tcr}$$

for:

$$V_{tca} = \left(\frac{V_{ta0}}{2} + V_{ta1} + V_{ta2}\right) \times \frac{L_{ca}}{3}$$

$$V_{tai} = \left[B_{ca} - 6 + 2 \times \left(0.6 \times h_{rai} + e_{te}\right)\right] \times e_{te}$$

$$h_{rai} = El_{tai} - El_{ca} - e_{te}, \ i = 0,\ 1,\ 2$$

$$B_{ca} = (N_{td} - 1) \times 2 \times D_{td} + 2 \times D_{td}$$

$$V_{tcr} = \left(\frac{V_{tr0}}{2} + V_{tr1} + V_{tr2}\right) \times \frac{L_{cr}}{3}$$

$$V_{tri} = \left[B_{cr} - 6 + 2 \times \left(0.6 \times h_{rri} + e_{te}\right)\right] \times e_{te}$$

$$h_{rri} = El_{tri} - El_{cr} - e_{te}, \ i = 0,\ 1,\ 2$$

$$B_{cr} = (N_{td} - 1) \times 2 \times D_{td} + 1.5 \times D_{td}$$

where:

V_{tca}	common excavation volume for the approach channel, in m³;
V_{tcr}	common excavation volume for the downstream channel, in m³;
V_{tai}	common excavation volume per meter in section i of the approach channel, in m³/m;
L_{ca}	length of the approach channel, in m;
B_{ca}	width of the bottom of the approach channel, in m;
h_{rai}	depth of excavation in rock in section i of the approach channel, in m;
e_{te}	mean thickness of the layer of soil in the area of the spillway per se, in m;
El_{tai}	mean elevation of the land in section i of the approach channel, in m;
El_{ca}	elevation of the bottom of the approach channel;
N_{td}	number of diversion tunnels, in m;
D_{td}	diameter of the diversion tunnels, in m;
V_{tri}	common excavation volume per meter in section i of the downstream channel, in m³/m;
L_{cr}	length of the downstream channel, in m;
B_{cr}	width of the bottom of the downstream channel, in m;
h_{rri}	depth of excavation in rock in section i of the downstream channel, in m;
El_{tri}	mean elevation of the land in section i perpendicular to the longitudinal axis of the downstream channel, in m;
El_{cr}	elevation of the bottom of the downstream channel, in m.

The unit price of common excavation is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and

- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.16.23.12.11)

The **volume of excavation in rock** for the diversion tunnels, V_{rtd} (m^3), in the absence of more accurate information, can be determined by:

$$V_{rtd} = V_{rca} + V_{rcr}$$

for:

$$V_{rca} = \left(\frac{V_{ra0}}{2} + V_{ra1} + V_{ra2} \right) \times \frac{L_{ca}}{3}$$

$$V_{rai} = [B_{ca} - 6 + 0.6 \times h_{rai}] \times h_{rai}$$

$$V_{rcr} = \left(\frac{V_{rr0}}{2} + V_{rr1} + V_{rr2} \right) \times \frac{L_{cr}}{3}$$

$$V_{rri} = [B_{cr} - 6 + 0.6 \times h_{rri}] \times h_{rri}$$

where:

V_{rca}	volume of excavation in rock for the approach channel, in m^3 (COPEL, 1996);
V_{rcr}	volume of excavation in rock for the downstream channel, in m^3 (COPEL, 1996);
V_{rai}	volume of excavation in rock per meter in section i of the approach channel, in m^3/m;
L_{ca}	length of the approach channel, in m;
B_{ca}	width of the bottom of the approach channel, in m;
h_{rai}	depth of excavation in rock in section i of the approach channel, in m;
V_{rri}	volume of excavation in rock per meter in section i of the downstream channel, in m^3/m;
L_{cr}	length of the downstream channel, in m;
B_{cr}	width of the bottom of the downstream channel, in m; and
h_{rri}	depth of excavation in rock in section i of the downstream channel, in m.

The unit price of excavation in rock is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Underground excavation in rock (account .12.16.23.12.12)

The **volume of underground excavation in rock**, V_{std} (m^3), is given by:

$$V_{std} = 0.8927 \times [D_{td}^2 \times (L_{td} - L_c) + (D_{td} + 2 \times e_c)^2 \times L_c] \times N_{td}$$

for:

$$e_c = k_g \times \left[0.091 \times D_{td}^{0.62} + 0.0034 \times (H - 30)\right]$$

$$H = NA_{max} - El_{ca}$$

k_g	geological conditions
1.0	good
1.4	mean
2.0	variable

where:

D_{td}	internal diameter of the diversion tunnel, in m;
e_c	thickness of the structural concrete lining in the tunnels, in m;
e_{cp}	0.05 m, mean thickness of the shotcrete lining;
k_g	coefficient to represent the geological conditions;
H	mean hydrostatic load in the tunnel, in m;
NA_{max}	maximum normal water level in the reservoir;
El_{ca}	elevation of the sill at the tunnel inlet;
L_{td}	length of the tunnel, in m;
L_c	length of the section lined with structural concrete, in m;
L_{cp}	length of the section lined with shotcrete, in m; and
N_{td}	number of diversion tunnels.

The unit price of **underground excavation in rock**, $P_{R\$}$ (R\$/m³), valid for December 2006, which can be used for projects in the south, southeast, central west and northeast regions of Brazil, can be obtained from the expression below (or from Graph. B 33, annex B, as a function of the area of the excavated section). This price corresponds to the price per cubic meter measured using the project line and includes excavating, loading, transportation up to 1.5 km and unloading:

valid for $4 \le A_{se} \le 300$: $P_{st} = 474.08 \times A_{se}^{-0.3987}$

for: $A_{se} = 0.8927 \times D_{td}^2$

where:

A_{se}	area of the excavated section, in m²; and
D_{td}	internal diameter of the diversion tunnel, in m.

A careful assessment should be made for those circumstances in whcih the tunnels represent a significant portion of the cost estimate, checking first and foremost the geological conditions of the region and long routes. Generally speaking, when the geological conditions are deemed poor and when there is no more reliable information available, depending on the judgement of the cost engineer, the price may rise by up to 30%.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.16.23.13)

The area of foundation to be cleaned, A_{lf} (m²), is given by:

$$A_{lf} = A_{lfr} + A_{lfe} + A_{lft} + A_{lfd}$$

for:

$$A_{lfr} = N_{td} \times \frac{\pi}{2} \times D_{td} \times (L_c + L_{cp})$$

$$A_{lfe} = 2.6 \times D_{td}^2 + 1.65 \times D_{td} + 70$$

$$A_{lft} = N_{td} \times \pi \times D_{td}^2$$

$$A_{lfd} = 12 \times D_{td} + (2 \times N_{td} - 1) \times D_{td}^2$$

where:

A_{lfr}	area of foundation to be cleaned for the lining, in m²;
A_{lfe}	area of foundation to be cleaned for the inlet structure, in m²;
A_{lft}	area of foundation to be cleaned for the plug, in m²;
A_{lfd}	area of foundation to be cleaned for the outlet structure, in m²;
N_{td}	number of diversion tunnels;
L_{td}	length of the tunnels, in m;
L_c	length of the section lined with structural concrete, in m;
L_{cp}	length of the section lined with shotcrete, in m; and
D_{td}	internal diameter of the diversion tunnels, in m.

The **total length of the rock anchors**, L_{tfp} (m), is given by:

$$L_{tfp} = 11.0 \times D_{td} \times (L_{td} - L_c - L_{cp})$$

where:

D_{td}	internal diameter of the diversion tunnel , in m;
L_{td}	length of the tunnel, in m;
L_c	length of the section lined with structural concrete, in m; and
L_{cp}	length of the section lined with shotcrete, in m.

The unit prices for foundation cleaning and treatment services, expressed in Brazilian Reais (valid for the December 2006 database), can be used for projects in the south, southeast, central west and northeast regions of Brazil. They include the execution of the work, supply of inputs and equipment, and depend on the kind of surface and the equipment to be used. The unit prices are:

- cleaning of the rock surface: 39.70/m²
- rotary percussive drilling: 168.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.16.23.14)

The **volume of concrete for the tunnel**, V_{ctd} (m³), is given by:

$$V_{ctd} = V_{cte} + V_{ctr} + V_{ctt} + V_{cts} + V_{ctp}$$

for:

$$V_{cte} = 2.76 \times D_{td}^3 + 2 \times D_{td}^2 + 250 \times D_{td} + 325$$

$$V_{ctr} = 0.8927 \times N_{td} \times L_c \times \left[(D_{td} + 2 \times e_c)^2 - D_{td}^2 \right]$$

$$V_{ctt} = 1.5 \times N_{td} \times D_{td}^3$$

$$V_{cts} = 2.5 \times \left[(2 \times N_{td} - 1) \times D_{td} + 12 \right]$$

$$V_{ctp} = 2.57 \times D_{td} \times L_{cp} \times N_{td} \times e_{cp}$$

where:

V_{cte}	volume of concrete for the inlet structure, in m^3;
V_{ctr}	volume of concrete for lining the tunnels, in m^3;
V_{ctt}	volume of concrete for the plug, in m^3;
V_{cts}	volume of concrete for the outlet structure, in m^3;
V_{ctp}	volume of shotcrete, in m^3;
N_{td}	number of diversion tunnels;
L_c	length of the section lined with structural concrete, in m;
L_{cp}	length of the section lined with shotcrete, in m;
k	coefficient to represent the geological conditions;
H	mean hydrostatic load in the tunnel, in m;
D_{td}	internal diameter of the tunnels, in m;
e_{cp}	0.05 m, mean thickness of the shotcrete; and
e_c	thickness of the structural concrete lining on the tunnels, in m.

The amounts of cement and reinforcement steel are as follows:

	cement (kg/m³)	reinforcement steel (kg/m³)
inlet structure	280	80
lining and outlet structure	250	50
plug	220	20
shotcrete	300	70

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the construction site, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast. This price per ton is for the steel used, and includes its supply, transportation to the construction site, storage, preparation and installation.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of the powerhouse volume (December 2006 database) and are valid for projects in the south, southeast, central west and northeast. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- inlet structure: 214.00/m^3
- plugs, lining and outlet structure: 128.00/m^3
- shotcrete: 378.00/m^3

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Emergency gates for the diversion (account .12.16.23.23.16)

The **acquisition cost of an emergency gate** for the diversion tunnel, C_{cp} (R$) – FOB cost excluding transportation, insurance, assembly and testing costs and provisions for charges and taxes payable according to the applicable tax legislation – is given below (or obtained from Graph B 23, annex B, as a function of its dimensions and maximum hydrostatic load), valid for the December 2006 database and for projects anywhere in Brazil:

valid for $0.13 \le z \le 9.17$: $C_{cp} = -4.3986 \times z^2 + 124.79 \times z + 110.2$

and for $9.17 \le z \le 125.39$: $C_{cp} = -0.128 \times z^2 + 57.311 \times z + 369.83$

for:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_{xe}}{1000} \qquad H_{xe} = \frac{NA_{max} - El_{td}}{3}$$

$$B_{cp} = \frac{0.88 \times D_{td}}{N_v} \qquad H_{cp} = 1.13 \times D_{td}$$

$$n_v = \mathrm{int}\left(\frac{D_{td}}{4.5} + 0.9\right)$$

where:

z	parameter, in m^4;
B_{cp}	width of the gates for the diversion tunnel, in m;
H_{cp}	height of the gates for the diversion tunnel, in m;
H_x	maximum hydrostatic load on the sill of the gate in the diversion tunnel, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
D_{td}	internal diameter of the tunnels, in m;
n_v	number of openings for the inlet structure of each diversion tunnel; and
int(x)	function that returns the integer part of x.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0 %: for the taxes and charges payable on the equipment.

Gates to close the diversion tunnel (account .12.16.23.23.17)

The **acquisition cost of each gate to close** the diversion tunnel, C_{sl} (R$) – FOB cost – is given below (or obtained from Graph B 25, annex B, as a function of its dimensions and maximum hydrostatic load), valid for the December 2006 database and for projects anywhere in Brazil:

valid for $0.16 \leq z \leq 54.43$: $C_{sl} = 72.896 \times z^{0.716}$

for:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000} \qquad H_x = NA_{max} - El_{td}$$

where:

z	parameter, in m^4;
B_{cp}	width of the gates for the diversion tunnel, in m;
H_{cp}	height of the gates for the diversion tunnel, in m; and
H_x	maximum hydrostatic load on the sill of the gate in the diversion tunnel, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

The **overall acquisition cost for fixed parts and parts embedded in the concrete** of the gates to close the diversion tunnel, C_{gpf} (R$) – FOB cost – is given below, valid for the December 2006 database and for projects anywhere in Brazil:

$$C_{gpf} = 2 \times N_v \times N_{td} \times H_{td} \times 2084.80$$

for: $H_{td} = 2.5 \times H_{cp}$

where:

N_v	number of openings for the inlet structure of each diversion tunnel;
N_{td}	number of diversion tunnels;
H_{td}	height of the inlet structure from the sill, in m; and
H_{cp}	height of the gates for the diversion tunnel, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

Crane (account .12.16.23.23.20)

A construction hoist should be used.

DIVERSION CHANNELS (ACCOUNT .12.16.24)

The main **information required for dimensioning purposes** is:

- mean elevation of the river bottom in the section immediately downstream from the canal, El_s, in m;
- mean elevation of the canal bottom at the inlet section, El_{ec}, in m;
- mean elevation of the canal bottom at the outlet section, El_{sc}, in m;
- width of the river in the section immediately downstream from the canal, B_s, in m;
- width of the canal in the inlet section, B_{ec}, in m;
- width of the canal at the outlet section, B_{sc}, in m;
- length of the diversion channel, L_{cd} in m;
- design flow in the diversion for a recurrence time of k years, Q_k in m^3/s, from item 5.1.2.;
- natural water level of the river in the section immediately downstream from the canal for flow Q_k, NA_{dcn}, from item 5.1.2.; and
- type of channel bottom.

The main **information required for quantification purposes**, when the canal is excavated in one of the banks, is:

- mean thickness of the layer of soil in the diversion channel area, e_{te}, in m;
- mean elevation of the land in section i – 0, 1 and 2 as indicated in Fig. 5.7.3.05 – perpendicular to the longitudinal axis of the upstream half of the diversion channel, El_{tai}, in m;
- mean elevation of the land in section i – 0 and 1 as indicated in Fig. 5.7.3.05 – perpendicular to the longitudinal axis of the downstream half of the diversion channel, El_{tri}, in m; and
- length of the upstream half of the diversion channel, L_{ca}, in m.

Considerations and recommendations

This **text** applies to canals which either narrow the river bed, as shown in Fig. 5.7.3.05, or which are excavated in an abutment.

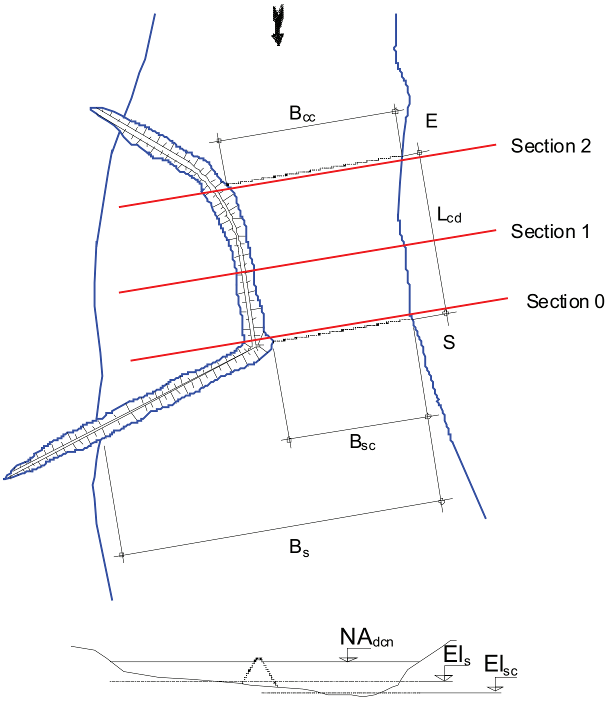


Fig 5.7.3.05 – Typical plan and cross-section of a diversion channel.

When the **canal excavated in one of the abutments** is short, the same simplified methodology as used for the approach and downstream channels can be used to calculate the construction quantities. It is best for the division to be made at the section where the land is highest.

In the absence of more accurate information about the **river bottom** or when the bottom is very uneven, the following is acceptable:

$$El_{ec} = El_{sc}$$

where:

El_{ec}	mean elevation of the canal bottom at the inlet section, in m; and
El_{sc}	mean elevation of the canal bottom at the outlet section, in m.

Characteristics of critical flow at the canal outlet

The characteristics of critical flow at the canal outlet – depth of the water column, y_{cs} (m), slope of the energy head line, i_{cs} (%), and energy head, E_{cs} – for flow Q_k, are given by:

$$y_{cs} = \sqrt[3]{\frac{1}{g} \times \left(\frac{Q_k}{B_{sc}}\right)^2} \qquad i_{cs} = 100 \times \frac{n^2 \times v_{cs}^2}{y_{cs}^{4/3}}$$

$$E_{cs} = NA_{cs} + \frac{v_{cs}^2}{2 \times g}$$

for:

$$NA_{cs} = El_{sc} + y_{cs} \qquad v_{cs} = \frac{Q_k}{B_{sc} \times y_{cs}}$$

n	Type of Bottom
0.025	canal excavated in earth
0.035	canal excavated in rock
0.040	narrowing with an uneven bottom

where:

g	9.81 m/s^2 – acceleration due to gravity;
Q_k	diversion flow for a recurrence time of k years, in m^3/s;
B_{sc}	width of the canal at the outlet section, in m;
n	Manning's roughness coefficient;
v_{cs}	critical velocity at the outlet section of the canal, in m/s;
NA_{cs}	critical water level at the outlet section of the canal, in m; and
El_{sc}	mean elevation of the canal bottom at the outlet section, in m.

Natural channel flow of the river in the section immediately downstream from the canal

The natural channel flow of the river in the section immediately downstream from the canal – depth of the water column, y_s (m), slope of the energy head line, i_s (%), and energy head, E_{dcn} – for flow Q_k, are given by:

$$y_s = NA_{dcn} - El_s \qquad i_s = 100 \times \frac{n^2 \times v_s^2}{y_s^{4/3}}$$

$$E_{dcn} = NA_{dcn} + \frac{v_s^2}{2 \times g}$$

for:

$$v_s = \frac{Q_k}{B_s \times y_s}$$

where:

NA_{dcn}	natural water level in the section immediately downstream from the canal for flow Q_k, in m;
El_s	mean elevation of the river bottom in the section immediately downstream from the canal, in m;
n	Manning's roughness coefficient;
v_s	mean velocity of the river in the section immediately downstream from the canal, in m/s;
g	9.81 m/s^2 – acceleration due to gravity;
Q_k	diversion flow for a recurrence time of k years, in m^3/s; and
B_s	width of the river in the section immediately downstream from the canal, in m.

Water level for subcritical flow with unsubmerged flow control

For **subcritical flow with unsubmerged flow control,** the following applies:

$i_S < i_{CS}$ and $E_{dcn} < E_{CS}$

where:

i_s	slope of the energy head line in the section immediately downstream from the canal for natural channel flow, in %;
i_{cs}	critical slope of the energy head line at the canal outlet, in %;
E_{dcn}	energy head, for flow Q_k, in the section immediately downstream from the canal for natural channel flow, in m; and
E_{cs}	critical energy head, for flow Q_k, at the canal outlet, in m.

The water levels along the canal, NA_{dl}, and at the upstream cofferdam, NA_{dm}, are given by:

$$NA_{dl} = El_{sc} + y_m \qquad NA_{dm} = E_{cs} + h_p$$

for:

$$y_m = 1.25 \times y_{cs} \qquad h_p = L_{cd} \times \frac{n^2 \times v_m^2}{y_m^{4/3}}$$

$$v_m = \frac{Q_k}{B_{mc} \times y_m} \qquad B_{mc} = \frac{B_{ec} + B_{sc}}{2}$$

where:

El_{sc}	mean elevation of the canal bottom at the outlet section, in m;
y_m	mean depth of discharge along the canal, in m;
E_{cs}	critical energy head, for flow Q_k, at the canal outlet, in m;
h_p	head loss along the canal, in m;
y_{cs}	critical depth of discharge at the canal outlet, in m;
L_{cd}	length of the canal, in m;
n	Manning's roughness coefficient;
v_m	mean velocity of discharge along the canal, in m/s;
Q_k	diversion flow for a recurrence time of k years, in m^3/s;
B_{mc}	mean width of the canal, in m;
B_{ec}	width of the canal in the inlet section, in m; and
B_{sc}	width of the canal at the outlet section, in m.

Water level for subcritical flow with submerged flow control

For **subcritical flow with submerged flow control**, the following applies:

$i_S < i_{CS}$ and $E_{dcn} \geq E_{CS}$

where:

i_s	slope of the energy head line in the section immediately downstream from the canal for natural channel flow, in %;
i_{cs}	critical slope of the energy head line at the canal outlet, in %;
E_{dcn}	energy head, for flow Q_k, in the section immediately downstream from the canal for natural channel flow, in m; and
E_{cs}	critical energy head, for flow Q_k, at the canal outlet, in m.

The water levels along the canal, NA_{dl}, and at the upstream cofferdam, NA_{dm}, are given by:

$$NA_{dl} = NA_{dcn} \qquad NA_{dm} = NA_{dcn} + h_p + h_{vn}$$

for:

$$h_p = L_{cd} \times \frac{n^2 \times v_n^2}{y_n^{4/3}} \qquad h_{vn} = \frac{v_n^2}{2 \times g}$$

$$v_n = \frac{Q_k}{B_{mc} \times y_n} \qquad y_n = E_{dcn} - \frac{v_n'^2}{2 \times g} - El_{sc}$$

$$v_n' = \frac{B_s}{B_{mc}} \times v_s$$

where:

NA_{dcn}	natural water level in the section immediately downstream from the canal for flow Q_k, in m;
h_p	head loss along the canal, in m;
h_{vn}	mean velocity head along the canal, in m;
L_{cd}	length of the canal, in m;
n	Manning's roughness coefficient;
v_n	mean velocity of discharge along the canal, in m/s;
y_n	mean depth of discharge along the canal, in m;
g	9.81 m/s^2 – acceleration due to gravity;
Q_k	diversion flow for a recurrence time of k years, in m^3/s;

B_{mc}	mean width of the canal, in m;
E_{dcn}	energy head, for flow Q_k, in the section immediately downstream from the canal for natural channel flow, in m;
v'_n	first approximation of the mean velocity in the canal, in m/s;
El_{sc}	mean elevation of the canal bottom at the outlet section, in m;
B_s	width of the river in the section immediately downstream from the canal, in m; and
v_s	mean velocity of the river in the section immediately downstream from the canal, in m/s.

Water levels for critical and supercritical flows

For **critical and supercriticial flows**, the following applies:

$$i_s \geq i_{cs}$$

where:

i_s	slope of the energy head line in the section immediately downstream from the canal for natural channel flow, in %; and
i_{cs}	critical slope of the energy head line at the canal outlet, in %.

The water levels along the canal, NA_{dl}, and near the upstream cofferdam, NA_{dm}, are given by:

$NA_{dl} = NA_{cs}$ to NA_{ce} (variable)

$$NA_{dm} = E_{ce}$$

for:

$$NA_{ce} = El_{ec} + y_{ce} \qquad E_{ce} = NA_{ce} + h_{vce}$$

$$y_{ce} = \sqrt[3]{\frac{1}{g} \times \left(\frac{Q_k}{B_{ec}}\right)^2} \qquad h_{vce} = \frac{v_{ce}^2}{2 \times g}$$

$$v_{ce} = \frac{Q_k}{B_{ec} \times y_{ce}}$$

where:

NA_{cs}	critical water level at the canal outlet, in m;
NA_{ce}	critical water level at the canal inlet, in m;
E_{ce}	critical energy head at the canal inlet, in m;
g	9.81 m/s^2 – acceleration due to gravity;
Q_k	diversion flow for a recurrence time of k years, in m^3/s;
B_{ec}	width of the canal in the inlet section, in m;
El_{ec}	mean elevation of the canal bottom at the inlet section, in m;
h_{vce}	velocity head in the canal at the inlet section, in m; and
v_{ce}	critical velocity at the inlet section of the canal, in m/s.

Common excavation (account .12.16.24.12.10)

In the absence of more accurate information, the **common excavation volume** for the diversion channel, V_{tcd} (m^3), where there is one, can be calculated as:

$$V_{tcd} = V_{tca} + V_{tcr}$$

for:

$$V_{tca} = \left(\frac{V_{ta0}}{2} + V_{ta1} + V_{ta2}\right) \times \frac{L_{ca}}{3}$$

$$V_{tai} = \left[B_{mc} - 6 + 2 \times (0.6 \times h_{rai} + e_{te})\right] \times e_{te}$$

$$h_{rai} = El_{tai} - El_{ec} - e_{te}\text{ , i = 0, 1, 2}$$

$$V_{tcr} = \left(\frac{V_{tr0}}{2} + V_{tr1} + V_{tr2}\right) \times \frac{L_{cr}}{3}$$

$$V_{tri} = \left[B_{mc} - 6 + 2 \times \left(0.6 \times h_{rri} + e_{te}\right)\right] \times e_{te}$$

$$L_{cr} = L_{cd} - L_{ca}$$

$$h_{rri} = El_{tri} - El_{sc} - e_{te}\text{ , i = 0, 1, 2}$$

where:

V_{tca}	common excavation volume in the upstream half of the canal, in m^3;
V_{tcr}	common excavation volume in the downstream half of the canal, in m^3;
V_{tai}	common excavation volume per meter in section i of the upstream half of the canal, in m^3/m;
L_{ca}	length of the upstream half of the canal, in m;
B_{mc}	mean width of the canal, in m;
h_{rai}	depth of excavation in rock in section i of the upstream half of the canal, in m;
e_{te}	mean thickness of the layer of soil in the canal area, in m;
El_{tai}	mean elevation of the land in section i of the upstream half of the canal, in m;
El_{ec}	elevation of the bottom of the canal in the inlet section, in m;
V_{tri}	common excavation volume per meter in section i of the downstream half of the canal, in m^3/m;
L_{cr}	length of the downstream half of the canal, in m;
h_{rri}	depth of excavation in rock in section i of the downstream half of the canal, in m;
L_{cd}	length of the canal, in m;
El_{tri}	mean elevation of the land in section i, perpendicular to the longitudinal axis of the downstream half of the canal, in m; and
El_{sc}	elevation of the bottom of the canal in the outlet section, in m.

The unit price of common excavation is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.16.24.12.11)

In the absence of more accurate information, the **volume of excavation in rock** for the diversion channel, V_{tcd} (m^3), where there is one, can be calculated as:

$$V_{rcd} = V_{rca} + V_{rcr}$$

for:

$$V_{rca} = \left(\frac{V_{ra0}}{2} + V_{ra1} + V_{ra2}\right) \times \frac{L_{ca}}{3}$$

$$V_{rai} = \left[B_{mc} - 6 + 0.6 \times h_{rai}\right] \times h_{rai}$$

$$V_{rcr} = \left(\frac{V_{rr0}}{2} + V_{rr1} + V_{rr2}\right) \times \frac{L_{cr}}{3}$$

$$V_{rri} = \left[B_{mc} - 6 + 0.6 \times h_{rri}\right] \times h_{rri}$$

where:

V_{rca}	volume of excavation in rock in the upstream half of the canal, in m³ (COPEL, 1996);
V_{rcr}	volume of excavation in rock in the downstream half of the canal, in m³ (COPEL, 1996);
V_{rai}	volume of excavation in rock per meter in section i of the upstream half of the canal, in m³/m;
L_{ca}	length of the upstream half of the canal, in m;
B_{mc}	mean width of the canal, in m;
h_{rai}	depth of excavation in rock in section i of the upstream half of the canal, in m;
V_{rri}	volume of excavation in rock per meter in section i of the downstream half of the canal, in m³/m;
L_{cr}	length of the downstream half of the canal, in m; and
h_{rri}	depth of excavation in rock in section i of the downstream half of the canal, in m.

The unit price of excavation in rock is R$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.16.24.13)

There is no need to include foundation cleaning or treatment for the purposes of this stage of the studies.

Concrete (account .12.16.24.14)

At this stage of the studies, there is no provision for concrete lining or a concrete spur wall.

Water column profile along the diversion canal

When the profile of the water column along the canal has to be determined more carefully for subcritical channel flows with controlled flow at the outlet, the additional data required are:

- mean longitudinal slope of the bottom of the canal, i_{cn} in %;
- mean width of the channel bottom, B_d in m;
- mean slope of the side slopes, horizontal distance for a difference in level of 1.0 m, in m; and
- width of the canal bottom at section 0, narrowed, B_c, in m.

First, the characteristics of the critical flow are determined for section 0 at the canal outlet. The depth of the water column, y_c (m), is obtained iteratively by:

$$y_c = \sqrt[3]{\frac{1}{g} \times \left(\frac{Q_k}{B_m}\right)^2}$$

The mean width of the canal at section 0 can be estimated as:

$$B_m = B_c + m \times y_c$$

where:

g	9.81 m/s^2 – acceleration due to gravity;
Q_k	diversion flow for a recurrence time of k years, in m^3/s;
B_c	width of the canal bottom at section 0, in m; and
m	mean slope of the side slopes, horizontal distance for a difference in level of 1.0 m, in m.

The specific energy, H_c (m), and water level, NA_c, at section 0 are given by:

$$H_c = y_c + \frac{v_c^2}{2 \times g} \qquad NA_c = El_0 + y_c$$

for:

$$v_c = \frac{Q_k}{(B_c + m \times y_c) \times y_c}$$

where:

y_c	critical depth, in m;
v_c	critical velocity, in m/s;
g	9.81 m/s^2 – acceleration due to gravity;
El_0	elevation of the bottom of the canal in section i, in m;
Q_k	diversion flow for a recurrence time of k years, in m^3/s;
B_c	width of the canal bottom at section 0, in m; and
m	mean slope of the side slopes, horizontal distance for a difference in level of 1.0 m, in m.

The water levels are determined successively from one section to the next. The distance between two successive sections i-1 and i, Δx (m), for which the mean depth of the water column of the second section is fixed, y_i (m), is given by:

$$\Delta x = \frac{\Delta H}{i_m - i_{cn}}$$

for:

$$\Delta H = H_i - H_{i-1}$$

$$H_i = y_i + \frac{v_i^2}{2 \times g}$$

$$y_i = y_{i-1} + 0.15 \times \frac{v_{i-1}^2}{2 \times g}$$

$$v_i = \frac{Q_k}{(B_d + m \times y_i) \times y_i}$$

$$i_m = \frac{i_i + i_{i-1}}{2}$$

$$i_i = \frac{n^2 \times v_i^2}{R_{hi}^{4/3}}$$

$$R_{hi} = \frac{A_i}{B_d + 2 \times \sqrt{1+m^2} \times y_i}$$

n	Type of Bottom
0.025	canal excavated in earth
0.035	canal excavated in rock
0.040	narrowing with an uneven bottom

where:

ΔH	difference in specific energy between section i and i-1, in m;
i_m	mean slope of the energy head line, in m/m;
i_{cn}	mean slope of the channel bottom, in m/m;
H_i	specific energy in section i, in m;
y_i	mean depth of the water column in section i, in m;
v_i	mean velocity of discharge in section i, in m/s;
g	9.81 m/s^2 – acceleration due to gravity;
Q_k	diversion flow for a recurrence time of k years, in m^3/s;
B_d	mean width of the channel bottom, in m;
m	mean slope of the side slopes, in m/m;
i_i	slope of the energy head line in section i, in m/m;
n	Manning's coefficient; and
R_{hi}	hydraulic radius of section i, in m.

The water level in section i, NA_i, is given by:

$$NA_i = El_i + y_i$$

for: $El_i = El_{i-1} + i_{cn} \times \Delta x$

where:

El_i	elevation of the bottom of the canal in section i, in m;
y_i	depth of the water column in section i, in m;
i_{cn}	mean slope of the bottom of the canal, in m/m; and
Δx	distance between sections i and i-1, in m.

The calculation is repeated until the sum of the distances between the sections is greater than the length of the canal.

The mean velocity limits must be observed. If the limit is exceeded, reduce the mean velocity in the canal by decreasing the controlled cross-section of the canal. The velocity restriction in the controlled cross-section can be overcome by protecting the surface with larger rockfill or with concrete lining.

The mean velocity limits in these sections are:

Velocity	Type of Lining
1.5 m/s	unlined earth
4.0 m/s	unlined rockfill
10.0 m/s	unlined rock
15.0 m/s	rock lined with concrete

DIVERSION GALLERIES (ACCOUNT .12.16.24)

The main **information required for dimensioning purposes** comes from item 5.1.2. Hydrometeorological Data, as follows:

- design flow in the diversion for a recurrence time of k years, Q_k in m^3/s, from item 5.1.2.;

- water level in the downstream channel for the galleries, for the design flow calculated for the diversion, NA_{dcr}, from item 5.1.2., in m;
- elevation of the bottom of the approach channel, El_{ca}, in m; and
- length of the galleries, L_{ga} in m.

The main **information required for quantification purposes** is as follows:

- maximum normal water level in the reservoir, NA_{max}, from item 4.6., in m;
- mean elevation of the land in the gallery area, El_{te}, in m;
- mean thickness of the layer of soil in the gallery area, e_{te}, in m;
- mean elevation of the land in section i – 0, 1 and 2 as indicated in Fig. 5.7.5.05 – perpendicular to the longitudinal axis of the approach channel, El_{tai}, in m;
- mean elevation of the land in section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the downstream channel, El_{tri}, in m;
- length of the approach channel, L_{ca}, in m; and
- length of the downstream channel, L_{cr}, in m.

Considerations and recommendations

This text applies to galleries with a typical cross-section, as shown in Fig. 5.7.3.06.

For plugging the gallery, there may be no need to exclude water from the stretch downstream from the structure. Otherwise, a cofferdam can be built in the downstream channel, always taking into account the respective costs.

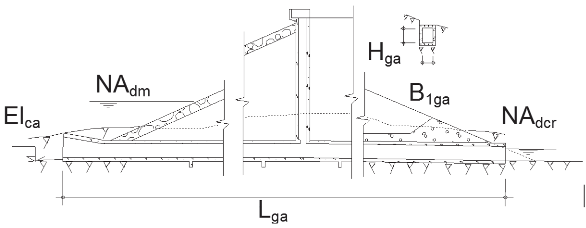


Fig. 5.7.3.06 – Cross-section and longitudinal section of a diversion gallery in a dam.

Coefficient k_Q can be reduced to meet the minimum **wigth or height** restrictions.

In order to stay within the **velocity limits,** the width or the number of openings can be increased or coefficient k_Q can be reduced.

Whatever the case, when any of the dimensions is changed, the following ratio must be maintained:

$$k_Q \times N_{ga} \times B_{1ga} \times H_{ga}^{3/2} = Q_k$$

where:

k_Q	coefficient;
N_{ga}	number of galleries;
B_{1ga}	width of one opening in the galleries, in m;
H_{ga}	height of one opening in the galleries, in m; and
Q_k	design flow in the diversion for a recurrence time of k years, in m^3/s.

For a **gallery** to be **efficient** – with its inlet submerged – the following restriction for coefficient k_Q must be respected:

$$k_Q \geq 1.5$$

If coefficient k_Q is higher, the sluiceway dimensions will be smaller and the cofferdams will be higher.

Gallery Dimensions

The **number of galleries**, N_{ga}, can be calculated by the expression:

$$N_{ga} = \text{int}\left(\frac{Q_k}{100} + 0.99\right)$$

where:

int(x)	function that returns the integer part of x; and
Q_k	design flow in the diversion for a recurrence time of k years, in m^3/s.

The **width of one opening in the galleries**, B_{1ga} (m), is given by:

$$B_{1ga} = \left(\frac{Q_k}{1.3 \times k_Q \times N_{ga}}\right)^{0.4} \geq 1.5\ \text{m}$$

for:

k_Q	3.8, initially.

where:

Q_k	design flow in the diversion for a recurrence time of k years, in m^3/s;
k_Q	coefficient; and
N_{ga}	number of galleries.

The **height of a gallery opening**, H_{ga} (m), is given by:

$$H_{ga} = \left(\frac{Q_k}{k_Q \times N_{ga} \times B_{1ga}}\right)^{2/3} \geq 1.9\ \text{m}$$

for:

k_Q	3.8, initially.

where:

Q_k	design flow in the diversion for a recurrence time of k years, in m^3/s;
k_Q	coefficient;
N_{ga}	number of galleries; and
B_{1ga}	width of one opening in the galleries, in m.

The **mean velocity of discharge**, v_g (m/s), is given by:

$$v_g = \frac{Q_k}{N_{ga} \times B_{1ga} \times H_{ga}} \leq 15\ \text{m/s}$$

where:

Q_k	design flow in the diversion for a recurrence time of k years, in m^3/s;
N_{ga}	number of galleries;
B_{1ga}	width of one opening in the galleries, in m; and
H_{ga}	height of one opening in the galleries, in m.

The **thickness of the walls**, e_{pl} (m), is given by:

$$e_{pl} = 0.2 + 0.2 \times H_{ga}$$

where:

H_{ga}	height of one opening in the galleries, in m.

The **total width** of the galleries, B_{ga} (m), is given by:

$$B_{ga} = N_{ga} \times (B_{1ga} + e_{pl}) + e_{pl}$$

where:

e_{pl}	thickness of the gallery walls, in m;
N_{ga}	number of galleries; and
B_{1ga}	width of one opening in the galleries, in m.

Gallery Profile

The **elevation of the sill at the inlet** of the gallery, El_{de}, is given by:

$$El_{de} = El_{ca}$$

where:

El_{ca}	elevation of the bottom of the approach channel.

The **elevation of the sill at the outlet** of the gallery, El_{ds}, is given by:

$$El_{ds} = El_{de} - 0.005 \times L_{ga}$$

where:

El_{de}	elevation of the sill at the inlet of the gallery, in m; and
L_{ga}	total length of the galleries, in m.

The **elevation of the bottom of the downstream channel**, El_{cr}, is given by:

$$El_{cr} = El_{ds}$$

where:

El_{ds}	elevation of the sill at the outlet of the gallery.

Water level at the upstream cofferdam for a gallery with a submerged outlet

The outlet will be submerged if: $E_{dcr} \geq E_{ga}$

for:

$$E_{dcr} = NA_{dcr} + \frac{v_{cr}^2}{2 \times g} \qquad E_{ga} = El_{ds} + H_{ga} + \frac{v_g^2}{2 \times g}$$

$$v_{cr} = \frac{Q_k}{B_{ga} \times (NA_{dcr} - El_{cr})}$$

where:

E_{dcr}	height of the energy head line in the downstream channel, in m;
E_{ga}	height of the energy head line at the gallery outlet, in m;
NA_{dcr}	water level in the downstream channel of the tunnels for flow Q_k, in m;
v_{cr}	mean velocity of discharge in the downstream channel, in m/s;
g	9.81 m/s² – acceleration due to gravity;
El_{ds}	elevation of the sill at the gallery outlet, in m;
H_{ga}	height of the galleries, in m;
v_g	mean velocity of discharge in the gallery, in m/s;
Q_k	design flow in the diversion for a recurrence time of k years, in m³/s;
B_{ga}	total width of the galleries, in m; and
El_{cr}	elevation of the bottom of the downstream channel, in m.

The water level **at the upstream cofferdam** for a gallery with a submerged outlet, NA_{dm}, is given by:

$$NA_{dm} = E_{dcr} + h_p$$

for:

$$h_p = 0.2 \times \frac{v_g^2}{2 \times g} + L_{ga} \times \frac{n^2 \times v_g^2}{R_h^{4/3}}$$

$$R_h = \frac{B_{1ga} \times H_{ga}}{2 \times (B_{1ga} + H_{ga})}$$

where:

E_{dcr}	height of the energy head line in the downstream channel, in m;
h_p	head loss along the gallery, in m;
v_g	mean velocity of discharge in the gallery, in m/s;
g	9.81 m/s^2 – acceleration due to gravity;
L_{ga}	length of the galleries, in m;
n	0.013 – Manning's coefficient;
R_h	hydraulic radius of one opening in the galleries, in m;
B_{1ga}	width of one opening in the galleries, in m; and
H_{ga}	height of the galleries, in m.

Water level at the upstream cofferdam for galleries with a free-flowing outlet

The outlet will be free flowing if: $E_{dcr} < E_{ga}$

where:

E_{dcr}	height of the energy head line in the downstream channel, in m; and
E_{ga}	height of the energy head line at the gallery outlet, in m.

The water level **at the upstream cofferdam** for a gallery with a free-flowing outlet, NA_{dm}, is obtained with the help of Graph 5.7.3.01 (COPEL, 1977) or by:

$$NA_{dm} = El_{de} + H$$

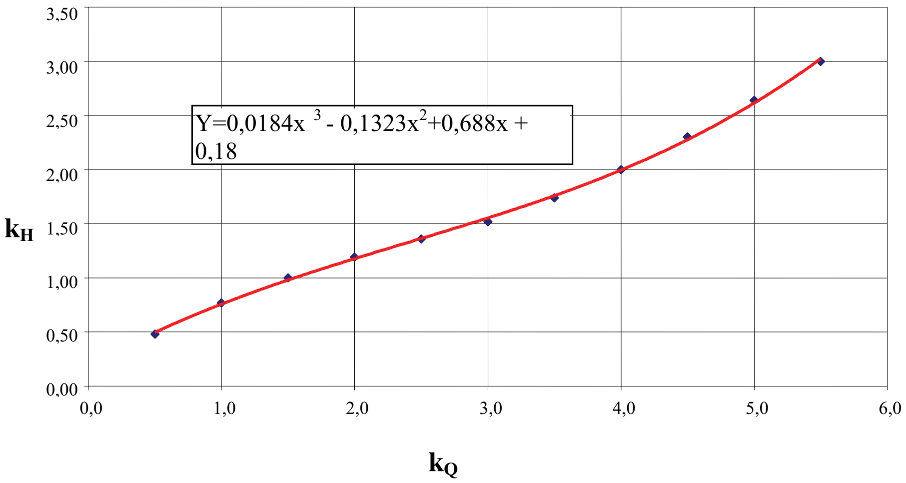
for:

$$H = k_H \times H_{ga}$$

$$k_H = 0.0184 \times k_Q^3 - 0.1323 \times k_Q^2 + 0.688 \times k_Q + 0.18$$

where:

El_{de}	elevation of the sill at the gallery inlet;
H	hydrostatic load upstream from the galleries, in m;
H_{ga}	height of the galleries, in m; and
k_Q, k_H	coefficients.



Graph 5.7.3.01 – Hydrostatic load at the sill of the gallery inlet.

Common excavation (account .12.16.24.12.10)

The **common excavation volume** for the galleries, V_{tga} (m³), is given by:

$$V_{tga} = V_{tca} + V_{tes} + V_{tcr}$$

for:

$$V_{tca} = \left(\frac{V_{ta0}}{2} + V_{ta1} + V_{ta2} \right) \times \frac{L_{ca}}{3}$$

$$V_{tai} = \left[B_{ca} - 6 + 2 \times \left(0.6 \times h_{rai} + e_{te} \right) \right] \times e_{te}$$

$$B_{ca} = B_{ga}$$

$$h_{rai} = El_{tai} - El_{ca} - e_{te}\text{ , i = 0, 1, 2}$$

$$V_{tes} = B_{ga} \times L_{ga} \times e_{te}$$

$$V_{tcr} = \left(\frac{V_{tr0}}{2} + V_{tr1} + V_{tr2} \right) \times \frac{L_{cr}}{3}$$

$$V_{tri} = \left[B_{cr} - 6 + 2 \times \left(0.6 \times h_{rri} + e_{te} \right) \right] \times e_{te}$$

$$B_{cr} = B_{ga}$$

$$h_{rri} = El_{tri} - El_{cr} - e_{te}\text{ , i = 0, 1, 2}$$

where:

V_{tca}	common excavation volume for the approach channel, in m³;
V_{tes}	common excavation volume for the gallery area, in m³;
V_{tcr}	common excavation volume for the downstream channel, in m³;
V_{tai}	common excavation volume per meter in section i of the approach channel, in m³/m;
L_{ca}	length of the approach channel, in m;
B_{ca}	width of the bottom of the approach channel, in m;
h_{rai}	depth of excavation in rock in section i of the approach channel, in m;
e_{te}	mean thickness of the layer of soil in the gallery area, in m;
El_{tai}	mean elevation of the land in section i of the approach channel, in m;
El_{ca}	elevation of the bottom of the approach channel, in m;

B_{ga}	total width of the galleries, in m;
L_{ga}	length of the galleries, in m;
V_{tri}	common excavation volume per meter in section i of the downstream channel, in m^3/m;
L_{cr}	length of the downstream channel, in m;
B_{cr}	width of the bottom of the downstream channel, in m;
h_{rri}	depth of excavation in rock in section i of the downstream channel, in m;
El_{tri}	mean elevation of the land in section i perpendicular to the longitudinal axis of the downstream channel, in m;
El_{cr}	elevation of the bottom of the downstream channel, in m.

The unit price of common excavation is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.16.24.12.11)

The **volume of excavation in rock** for the galleries, V_{rga} (m^3), is given by:

$$V_{rga} = V_{rca} + V_{res} + V_{rcr}$$

for:

$$V_{rca} = \left(\frac{V_{ra0}}{2} + V_{ra1} + V_{ra2}\right) \times \frac{L_{ca}}{3}$$

$$V_{rai} = \left[B_{ca} - 6 + 0.6 \times h_{rai}\right] \times h_{rai}$$

$$V_{res} = B_{ga} \times L_{ga} \times h_r$$

$$h_r = El_{te} - e_{te} - \left(El_{ca} - e_{pl}\right)$$

$$V_{rcr} = \left(\frac{V_{rr0}}{2} + V_{rr1} + V_{rr2}\right) \times \frac{L_{cr}}{3}$$

$$V_{rri} = \left[B_{cr} - 6 + 0.6 \times h_{rri}\right] \times h_{rri}$$

where:

V_{rca}	volume of excavation in rock for the approach channel, in m^3 (COPEL, 1996);
V_{res}	volume of excavation in rock in the gallery area, in m^3;
V_{rcr}	volume of excavation in rock for the downstream channel, in m^3 (COPEL, 1996);
V_{rai}	volume of excavation in rock per meter in section i of the approach channel, in m^3/m;
L_{ca}	length of the approach channel, in m;
B_{ca}	width of the bottom of the approach channel, in m;
h_{rai}	depth of excavation in rock in section i of the approach channel, in m;
B_{ga}	total width of the galleries, in m;

L_{ga}	length of the galleries, in m;
h_r	mean depth of excavation in rock in the gallery area, in m;
El_{te}	mean elevation of the land in the gallery area, in m;
e_{te}	mean thickness of the layer of soil in the gallery area, in m;
El_{ca}	elevation of the bottom of the approach channel, in m;
e_{pl}	thickness of the gallery walls, in m;
V_{rri}	volume of excavation in rock per meter in section i of the downstream channel, in m^3/m;
L_{cr}	length of the downstream channel, in m;
B_{cr}	width of the bottom of the downstream channel, in m; and
h_{rri}	depth of excavation in rock in section i of the downstream channel, in m.

The unit price of excavation in rock is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.16.24.13)

The **area of foundations** to be cleaned for the galleries, A_{lf} (m^2), is given by:

$$A_{lf} = B_{ga} \times L_{ga}$$

where:

B_{ga}	total width of the galleries, in m; and
L_{ga}	length of the galleries, in m.

The unit prices for foundation cleaning and treatment services, expressed in Brazilian Reais (valid for the December 2006 database), can be used for projects in the south, southeast, central west and northeast. They include the execution of the work, supply of inputs and equipment, and depend on the kind of surface and the equipment to be used. The unit prices are:

- cleaning and treatment of rock foundations: 39.70/m^2

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.16.24.14)

The **volume of concrete** for the galleries, V_{cga} (m^3), is given by:

$$V_{cga} = V_{cpl} + V_{cto} + V_{ctt}$$

for:

$$V_{cpl} = [\, B_{ga} \text{ x } (H_{ga} + 2 \text{ x } e_{pl}) - N_{ga} \text{ x } B_{1ga} \text{ x } H_{ga} \,] \text{ x } L_{ga}$$

$$V_{cto} = \left(B_{ga} \times 2.3 - 4 \times B_{1ga} \times 0.4\right) \times \left(NA_{max} - El_{ca} - H_{ga}\right)$$

$$V_{ctt} = N_{ga} \times 3 \times e_{pl} \times B_{1ga} \times H_{ga}$$

where:

V_{cpl}	volume of concrete for the walls and slabs, in m^3;
V_{cto}	volume of concrete for the gate tower, in m^3;
V_{ctt}	volume of concrete for the plug, in m^3;
B_{ga}	total width of the galleries, in m;
H_{ga}	height of a gallery opening, in m;
e_{pl}	thickness of the gallery walls, in m;
N_{ga}	number of galleries;
B_{1ga}	width of one opening in the galleries, in m;
L_{ga}	length of the galleries, in m;
NA_{max}	maximum normal water level in the reservoir, NA_{max}; and
El_{ca}	elevation of the bottom of the approach channel, in m.

The amounts of cement and reinforcement steel are as follows:

	cement (kg/m^3)	reinforcement steel (kg/m^3)
pillars and slabs	250	70
tower	300	70
plug	220	20

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the construction site, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the construction site, storage, preparation and installation.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of the powerhouse volume (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- concrete without cement for pillars and slabs: 174.00/m^3
- concrete without cement for the tower: 174.00/m^3

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Emergency gates for the river diversion (account .12.16.24.23.16)

The **acquisition cost of an emergency gate for the diversion** gallery, C_{cp} (R$), – FOB cost excluding transportation, insurance, assembly and testing costs and provisions for charges and taxes payable according to the applicable tax legislation – can be obtained from the expression below (or from Graph. B 23, annex B, as a function of its dimensions and hydrostatic load), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.13 \leq z \leq 9.17$: $C_{cp} = -4.399 \times z^2 + 124.8 \times z + 110$

valid for $9.17 < z \leq 126$: $C_{cp} = -0.128 \times z^2 + 57.3 \times z + 370$

for:

$$z = \frac{B_{1ga}^2 \times H_{ga} \times H_x}{1000} \qquad H_x = NA_{max} - El_{de}$$

where:

z	parameter, in m^4;
B_{1ga}	width of one opening in the galleries, in m;
H_{ga}	height of one opening in the galleries, in m;
H_x	maximum hydrostatic load on the gate sill, in m;
NA_{max}	maximum normal water level in the reservoir, in m; and
El_{de}	elevation of the sill at the inlet, in m.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for the taxes and charges payable on the equipment.

Gates to close the diversion gallery (account .12.16.24.23.17)

The **acquisition cost of the gate to close the diversion** gallery, C_{sl} (R$), – FOB cost – can be obtained from the expression below (or from Graph. B 25, annex B, as a function of its dimensions and hydrostatic load), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

for $0.16 \leq z \leq 54.5$: $C_{sl} = 72.9 \times z^{0.716}$

for: $z = \frac{B_{1ga}^2 \times H_{ga} \times H_x}{1000}$

where:

z	parameter, in m^4;
B_{1ga}	width of one opening in the galleries, in m;
H_{ga}	height of one opening in the galleries, in m; and
H_x	maximum hydrostatic load on the sill of the gate, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

The **total acquisition cost of fixed parts and parts embedded in the concrete** for the gates to close the diversion gallery, C_{gpf} (R$), – FOB cost – is given below, valid for the December 2006 database and for projects anywhere in Brazil:

$$C_{gpf} = 2 \times N_{ga} \times (H_x + H_{bl}) \times 2084.80$$

where:

N_{ga}	number of galleries;
H_x	maximum hydrostatic load on the sill of the gate, in m; and
H_{bl}	height of the dam freeboard, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

Crane (account .12.16.24.23.20)

Use a construction hoist.

DIVERSION SLUICEWAYS THROUGH CONCRETE DAMS (ACCOUNT .12.16.24)

The main **information required for dimensioning purposes** is:

- design flow in the diversion for a recurrence time of k years, Q_k in m^3/s, from item 5.1.2.;
- water level in the downstream channel of the sluiceways for the design flow in the diversion, NA_{dcr}, from item 5.1.2, in m;
- elevation of the bottom of the approach channel, El_{ca}, in m; and
- elevation of the bottom of the downstream channel, El_{cr}, in m.

The main **information required for quantification purposes** is:

- mean thickness of the layer of soil in the area of the structure, e_{te}, in m;
- slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m, from item 5.7.4.;
- maximum normal water level in the reservoir, NA_{max}, from item 4.6, in m;
- length of the dam in the section with sluiceways (direction of flow), L_{ba}, in m, from item 5.7.4.;
- mean elevation of the land in section i – 0, 1 and 2 as indicated in Fig. 5.7.5.05 – perpendicular to the longitudinal axis of the approach channel, El_{tai}, in m;
- mean elevation of the land in section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the downstream channel, El_{tri}, in m;
- length of the approach channel, L_{ca}, in m; and
- length of the downstream channel, L_{cr}, in m.

Considerations and recommendations

The text relates to sluiceways with a typical cross-section (see Fig. 5.7.3.07).

When the sluiceways are concreted, there may be no need to exclude water from the stretch downstream from the structure. Otherwise, gates can be used downstream from the structure or a cofferdam can be built in the downstream channel, always bearing in mind their respective costs.

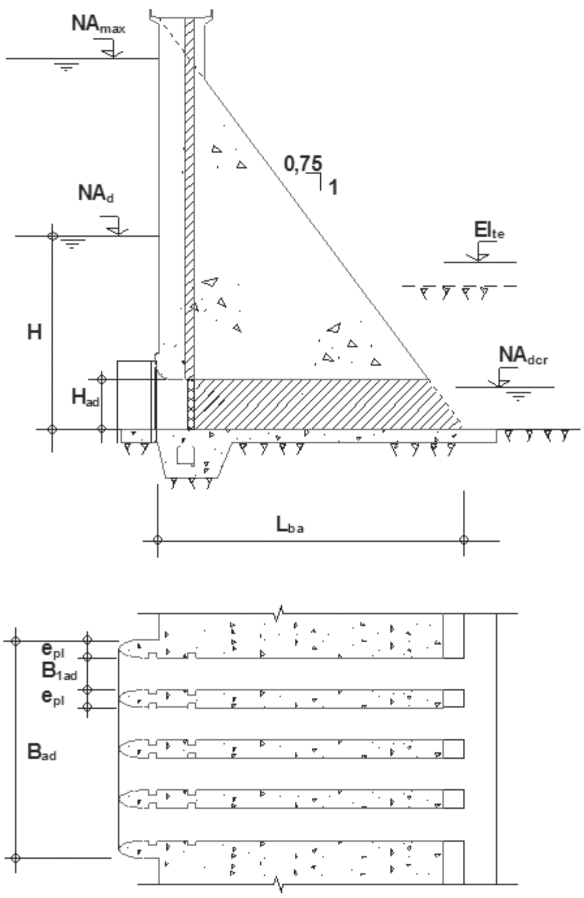


Fig. 5.7.3.07 – Typical cross-section and plan of sluiceways in a concrete gravity dam.

Coefficient k_Q can be reduced to meet the minimum **width or height** restrictions.

In order to stay within the **velocity limit,** the width or the number of openings can be increased, or coefficient k_Q can be reduced.

Whenever the **dimensions are altered**, the following ratio must be respected:

$$k_Q \times N_{ad} \times B_{1ad} \times H_{ad}^{3/2} = Q_k$$

where:

k_Q	coefficient;
N_{ad}	number of sluiceways;
B_{1ad}	width of sluiceway, in m;
H_{ad}	height of sluiceways, in m; and
Q_k	design flow in the diversion for a recurrence time of k years, in m^3/s.

For a **sluiceway** to be **efficient** – with its inlet submerged – the following restriction for coefficient k_Q must be respected:

$$k_Q \geq 1.5$$

When coefficient k_Q is higher, the sluiceway dimensions will be smaller and the cofferdams will be higher.

Generally speaking, the following must be true for the **elevation of the bottom of the downstream channel**:

$$El_{cr} = El_{ca}$$

where:

El_{ca}	elevation of the bottom of the approach channel, in m.

Dimensions of the sluiceways

Initially, the **number of sluiceways**, N_{ad}, is defined from the expression:

$$N_{ad} = \text{int}\left(\frac{Q_k}{1000} + 1.5\right)$$

where:

int(x)	function that returns the integer part of x; and
Q_k	design flow in the diversion for a recurrence time of k years, in m^3/s.

The width of sluiceway, B_{1ad} (m), is given by:

$$B_{1ad} = \left(\frac{Q_k}{4 \times k_Q \times N_{ad}}\right)^{0.4} \geq 1.5\,\text{m}$$

for:

k_Q	3.2 , initially.

where:

Q_k	design flow in the diversion for a recurrence time of k years, in m^3/s;
k_Q	coefficient; and
N_{ad}	number of sluiceways.

The **height of sluiceways**, H_{ad} (m), is given by:

$$H_{ad} = \left(\frac{Q_k}{k_Q \times N_{ad} \times B_{1ad}}\right)^{2/3} \geq 1.9\,\text{m}$$

for:

k_Q	3.2 , initially

where:

Q_k	design flow in the diversion for a recurrence time of k years, in m^3/s;
k_Q	coefficient;
N_{ad}	number of sluiceways; and
B_{1ad}	width of sluiceway, in m.

The **mean velocity of discharge**, v_a (m/s), is given by:

$$v_a = \frac{Q_k}{N_{ad} \times B_{1ad} \times H_{ad}} \leq 15\,\text{m/s}$$

where:

Q_k	design flow in the diversion for a recurrence time of k years, in m^3/s;
N_{ad}	number of sluiceways;
B_{1ad}	width of sluiceway, in m; and
H_{ad}	height of sluiceways, in m.

The thickness of the walls between two sluiceways, e_{pa} (m), is given by:

$$e_{pa} = 2{,}0 + 0.15 \times H_{ad}$$

where:

H_{ad}	height of sluiceways, in m.

The **total width of the sluiceways**, B_{ad} (m), is given by:

$$B_{ad} = N_{ad} \times (B_{1ad} + e_{pl}) + e_{pl}$$

where:

N_{ad}	number of sluiceways;
B_{1ad}	width of sluiceway, in m; and
e_{pl}	thickness of the spillway walls, in m.

Water level at the upstream cofferdam for sluiceways with a submerged outlet

The outlet will be submerged if:

$$E_{dcr} \geq E_{ad}$$

for:

$$E_{dcr} = NA_{dcr} + \frac{v_{cr}^2}{2 \times g}$$

$$E_{ad} = El_{ca} + H_{ad} + \frac{v_a^2}{2 \times g}$$

$$v_{cr} = \frac{Q_k}{B_{ad} \times (NA_{dcr} - El_{cr})}$$

where:

E_{dcr}	height of the energy head line in the downstream channel for the design flow in the diversion, in m;
E_{ad}	height of the energy head line at the sluiceway outlet for the design flow in the diversion, in m;
NA_{dcr}	water level in the downstream channel of the sluiceways for the design flow in the diversion, in m;
v_{cr}	mean velocity of discharge in the downstream channel, in m/s;
g	9.81 m/s^2 – acceleration due to gravity;
El_{ca}	elevation of the bottom of the approach channel, in m;
H_{ad}	height of sluiceways, in m;
v_a	mean velocity of discharge in the sluiceway, in m/s;
Q_k	design flow in the diversion for a recurrence time of k years, in m^3/s;
B_{ad}	total width of the sluiceways, in m; and
El_{cr}	elevation of the bottom of the downstream channel, in m.

The water level **at the upstream cofferdam** for sluiceways with submerged outlets, NA_{dm}, is given by:

for:

$$h_p = 0.2 \times \frac{v_a^2}{2 \times g} + L_{ba} \times \frac{n^2 \times v_a^2}{R_h^{4/3}}$$

$$R_h = \frac{B_{1ad} \times H_{ad}}{2 \times (B_{1ad} + H_{ad})}$$

where:

E_{dcr}	height of the energy head line in the downstream channel for the design flow in the diversion, in m;
h_p	head loss along the sluiceway, in m;
v_a	mean velocity of discharge in the sluiceway, in m/s;
g	9.81 m/s^2 – acceleration due to gravity;
L_{ba}	length of the dam, in m;

n	0.013 – Manning's coefficient;
R_h	hydraulic radius of one opening in the sluiceways, in m;
B_{1ad}	width of one opening in the sluiceways, in m; and
H_{ad}	height of sluiceways, in m.

Water level at the upstream cofferdam for sluiceways with a free-flowing outlet

The outlet will be free flowing if: $E_{dcr} < E_{ga}$

where:

E_{dcr}	height of the energy head line in the downstream channel for the design flow in the diversion, in m; and
E_{ga}	height of the energy head line at the sluiceway outlet for the design flow in the diversion, in m.

The water level **at the upstream cofferdam** for sluiceways with a free-flowing outlet, NA_{dm}, is obtained from Graph 5.7.3.01 (COPEL, 1977) or by:

$$NA_{dm} = El_{ca} + H$$

for:

$$H = k_H \times H_{ad}$$

$$k_H = 0.0184 \times k_Q^3 - 0.132 \times k_Q^2 + 0.688 \times k_Q + 0.18$$

where:

El_{ca}	elevation of the bottom of the approach channel, in m;
H	hydrostatic load on the upstream face of the dam, in m;
H_{ad}	height of sluiceways, in m; and
k_Q, k_H	coefficients.

Common excavation (account .12.16.24.12.10)

The **common excavation volume** for the approach and downstream channels – the volume in the area of the structure is included in the dam –, V_{tad} (m^3), is given by:

$$V_{tad} = V_{tca} + V_{tcr}$$

for:

$$V_{tca} = \left(\frac{V_{ta0}}{2} + V_{ta1} + V_{ta2} \right) \times \frac{L_{ca}}{3}$$

$$V_{tai} = \left[B_{ca} - 6 + 2 \times \left(0.6 \times h_{rai} + e_{te} \right) \right] \times e_{te}$$

$$B_{ca} = B_{ad}$$

$$h_{rai} = E_{tai} - E_{a} - e_{te}, \ i = 0, 1, 2$$

$$V_{tcr} = \left(\frac{V_{tr0}}{2} + V_{tr1} + V_{tr2} \right) \times \frac{L_{cr}}{3}$$

$$V_{tri} = \left[B_{cr} - 6 + 2 \times \left(0.6 \times h_{rri} + e_{te} \right) \right] \times e_{te}$$

$$B_{cr} = B_{ad}$$

$$h_{rri} = El_{tri} - El_{cr} - e_{te}, \ i = 0, 1, 2$$

where:

V_{tca}	common excavation volume for the approach channel, in m^3;
V_{tcr}	common excavation volume for the downstream channel, in m^3;
V_{tai}	common excavation volume per meter in section i of the approach channel, in m^3/m;
L_{ca}	length of the approach channel, in m;
B_{ca}	width of the bottom of the approach channel, in m;
B_{ad}	total width of the sluiceways, in m;
h_{rai}	depth of excavation in rock in section i of the approach channel, in m;
e_{te}	mean thickness of the layer of soil in the sluiceway area, in m;
El_{tai}	mean elevation of the land in section i of the approach channel, in m;
El_{ca}	elevation of the bottom of the approach channel, in m;
V_{tri}	common excavation volume per meter in section i of the downstream channel, in m^3/m;
L_{cr}	length of the downstream channel, in m;
B_{cr}	width of the bottom of the downstream channel, in m;
h_{rri}	depth of excavation in rock in section i of the downstream channel, in m;
El_{tri}	mean elevation of the land in section i perpendicular to the longitudinal axis of the downstream channel, in m;
El_{cr}	elevation of the bottom of the downstream channel, in m.

The **extra volume of excavation** in the dam for the sluiceways is negligible.

The unit price of common excavation is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.16.24.12.11)

The **volume of excavation in rock** for the approach and downstream channels – the volume in the area of the structure is included in the dam –, V_{rad} (m^3), is given by:

$$V_{rad} = V_{rca} + V_{rcr}$$

for:

$$V_{rca} = \left(\frac{V_{ra0}}{2} + V_{ra1} + V_{ra2} \right) \times \frac{L_{ca}}{3}$$

$$V_{rai} = (B_{ca} - 6 + 0.6 \times h_{rai}) \times h_{rai}$$

$$V_{rcr} = \left(\frac{V_{rr0}}{2} + V_{rr1} + V_{rr2} \right) \times \frac{L_{cr}}{3}$$

$$V_{rri} = (B_{cr} - 6 + 0.6 \times h_{rri}) \times h_{rri}$$

where:

V_{rca}	volume of surface rock excavation for the approach channel, in m^3;
V_{rcr}	volume of surface rock excavation for the downstream channel, in m^3;
V_{rai}	volume of excavation in rock per meter in section i of the approach channel, in m^3/m;
L_{ca}	length of the approach channel, in m;
B_{ca}	width of the bottom of the approach channel, in m;
h_{rai}	depth of excavation in rock in section i of the approach channel, in m;
V_{rri}	volume of excavation in rock per meter in section i of the downstream channel, in m^3/m;
L_{cr}	length of the downstream channel, in m;
B_{cr}	width of the bottom of the downstream channel, in m; and
h_{rri}	depth of excavation in rock in section i of the downstream channel, in m.

The unit price of excavation in rock is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the powerhouse. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.16.24.13)

The **extra foundation cleaning and treatment** is included in the dam.

Concrete (account .12.16.24.14)

The **extra volume of concrete** in the dam for the sluiceways is included in the dam.

Emergency gates for the diversion (account .12.16.24.23.16)

The **acquisition cost for a fixed-wheel emergency gate** for the diversion sluiceway, C_{cp} (R$), – FOB cost, without including transportation and insurance, assembly and testing, and provisions for taxes and charges payable, as per the current tax legislation – is given below (or from Graph. B 23, annex B, as a function of its dimensions and the maximum hydrostatic load), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.13 \leq z \leq 9.17$: $C_{cp} = -4.399 \times z^2 + 124.8 \times z + 110$

valid for $9.17 < z \leq 126$: $C_{cp} = -0.128 \times z^2 + 57.3 \times z + 370$

for:

$$z = \frac{B_{1ga}^2 \times H_{ga} \times H_x}{1000} \qquad H_x = NA_{max} - El_{de}$$

where:

z	parameter, in m^4;
B_{1ga}	width of one opening in the galleries, in m;
H_{ga}	height of one opening in the galleries, in m;
H_x	maximum hydrostatic load on the sill of the gate, in m;
NA_{max}	maximum normal water level in the reservoir, in m; and
El_{de}	elevation of the sill at the inlet, in m.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for the taxes and charges payable on the equipment.

Gates to close the diversion sluiceway (account .12.16.24.23.17)

The **acquisition cost of each gate to close a diversion sluiceway**, C_{sl} (R$), – FOB cost –, can be obtained from the expression below (or from Graph. B 25, annex B, as a function of its dimensions and the maximum hydrostatic load), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

for $0.16 \leq z \leq 54.5$: $C_{sl} = 72.9 \times z^{0.716}$

for: $z = \dfrac{B_{1ga}^2 \times H_{ga} \times H_x}{1000}$

where:

z	parameter, in m^4;
B_{1ga}	width of one opening in the galleries, in m;
H_{ga}	height of one opening in the galleries, in m; and
H_x	maximum hydrostatic load on the sill of the gate, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

The **total acquisition cost of fixed parts and parts embedded in the concrete** for the gates to close the diversion sluiceways, C_{gpf} (R$), – FOB cost – is given below, valid for the December 2006 database and for projects anywhere in Brazil:

$$C_{gpf} = 2 \times N_{ad} \times (H_x + H_{bl}) \times 2{,}084.80$$

where:

N_{ad}	number of sluiceways;
H_x	maximum hydrostatic load on the sill of the gate, in m; and
H_{bl}	height of the dam freeboard, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

Crane (account .12.16.24.20)

A construction hoist should be used.

DIVERSION SLUICEWAYS THROUGH A GATED SURFACE SPILLWAY (ACCOUNT .12.16.24)

The main **information required for dimensioning purposes** is:

- design flow in the diversion for a recurrence time of k years, Q_k in m³/s, from item 5.1.2.;
- number of spillway gates, N_{cp}, from item 5.7.5.;
- width of the spillway gates, B_{cp} in m, from item 5.7.5.;
- thickness of the spillway walls, e_{pl} in m, from item 5.7.5.;
- elevation of the bottom of the approach channel, El_{ca};
- elevation of the bottom of the downstream channel, El_{cr}; and
- water level in the downstream channel of the sluiceways for the project flow for the, NA_{dcr}, from item 5.1.2.

The main **information required for quantification purposes** is:

- height of the spillway gates, H_{cp} in m, from item 5.7.5.;
- height of the ogee crest above the bottom of the approach channel, p_v in m, from item 5.7.5.;
- length of the ogee crest of the spillway (direction of flow), L_{og}, in m, from item 5.7.5.;
- additional length of the spillway (direction of flow) for the ski jump, L_{se}, in m, from item 5.7.5., when applicable;
- radius of curvature of the ski jump, R_{se} in m, from item 5.7.5., when applicable; and
- elevation of the sill of the ski jump, El_{se}, in m, from item 5.7.5., when applicable.

Considerations and recommendations

This text applies to sluiceways with a typical cross-section, as shown in Fig. 5.7.3.08.

When the sluiceways are concreted, there may be no need to exclude water from the section downstream from the structure. Otherwise, gates can be used downstream from the structure or a cofferdam can be built in the downstream channel, always bearing in mind their respective costs.

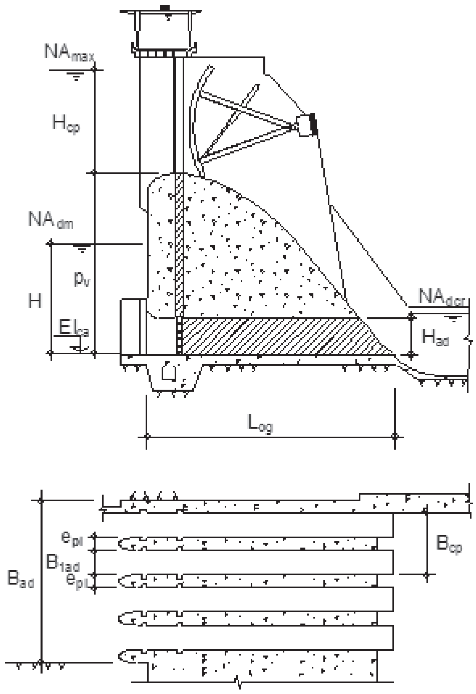


Fig. 5.7.3.08 – Typical cross-section and plan for sluiceways through a spillway with a high ogee crest and a stilling basin.

Coefficient k_Q can be reduced to meet the minimum **width or height** restrictions.

Ideally, the height of sluiceways should respect the following: $H_{ad} \leq 3.1 \times B_{1ad}$

where:

H_{ad}	height of sluiceways, in m; and
B_{1ad}	width of sluiceway, in m.

In order to stay within the **velocity limits,** the width or the number of openings can be increased, or coefficient k_Q can be reduced.

Whenever the **dimensions are altered**, the following ratio must be observed:

$$k_Q \times N_{ad} \times B_{1ad} \times H_{ad}^{3/2} = Q_k$$

where:

k_Q	coefficient;
N_{ad}	number of sluiceways;
B_{1ad}	width of sluiceway, in m;
H_{ad}	height of sluiceways, in m; and
Q_k	design flow in the diversion for a recurrence time of k years, in m^3/s.

For a **sluiceway** to be **efficient** – with its inlet submerged – the following restriction for coefficient k_Q must be respected:

$$k_Q \geq 1.5$$

When coefficient k_Q is higher, the sluiceway dimensions will be smaller and the cofferdams will be higher.

Generally speaking, for the **elevation of the bottom of the downstream channel**, the following must apply:

$$El_{cr} = El_{ca}$$

where:

El_{ca}	elevation of the bottom of the approach channel.

Dimensions of the sluiceways

Initially, the **number of sluiceways**, N_{ad}, is defined from the expression:

$$N_{ad} = 2 \times \text{int}\left(0.75 \times N_{cp}\right) + 1$$

where:

int(x)	function that returns the integer part of x; and
N_{cp}	number of spillway gates.

The width of sluiceway, B_{1ad} (m), is given by:

$$B_{1ad} = \frac{B_{cp} - e_{pl}}{2}$$

where:

B_{cp}	width of the spillway gates, in m; and
e_{pl}	thickness of the spillway walls, in m.

Ideally, the height of sluiceways should respect the following: $H_{ad} \leq 3.1 \times B_{1ad}$

where:

H_{ad}	height of sluiceways, in m; and
B_{1ad}	width of sluiceway, in m.

The **height of sluiceways**, H_{ad} (m), is given by:

$$H_{ad} = \left(\frac{Q_k}{k_Q \times N_{ad} \times B_{1ad}}\right)^{2/3} \geq 1.9\,\text{m}$$

for:

k_Q	3.2, initially.

and to meet the physical restriction due to the height of the ogee crest:

- gated surface spillways with a high ogee crest:

$$H_{ad} \leq NA_{MAX} \text{ x } H_{CP} - El_{ca}$$

- gated surface spillways with a high ogee crest:

$$H_{ad} \leq NA_{MAX} \times H_d - El_{ca}$$

where:

Q_k	design flow in the diversion for a recurrence time of k years, in m³/s;
k_Q	coefficient;
N_{ad}	number of sluiceways;
B_{1ad}	width of sluiceway, in m;
NA_{max}	maximum normal water level in the reservoir;
H_{cp}	height of the spillway gates;
El_{ca}	elevation from the bottom of the approach channel to the sluiceways; and
H_d	maximum energy head on the spillway crest, in m.

The **mean velocity of discharge**, v_a (m/s), is given by:

$$v_a = \frac{Q_k}{N_{ad} \times B_{1ad} \times H_{ad}} \leq 15 \text{ m/s}$$

where:

Q_k	design flow in the diversion for a recurrence time of k years, in m³/s;
N_{ad}	number of sluiceways;
B_{1ad}	width of sluiceway, in m; and
H_{ad}	height of sluiceways, in m.

The thickness of the walls between two sluiceways is the same as for the spillway.

A **total width of the sluiceways**, B_{ad} (m), is given by:

$$B_{ad} = N_{ad} \times (B_{1ad} + e_{pl}) + e_{pl}$$

where:

N_{ad}	number of sluiceways;
B_{1ad}	width of sluiceway, in m; and
e_{pl}	thickness of the spillway walls, in m.

Water level at the upstream cofferdam for sluiceways with a submerged outlet

The outlet will be submerged if: $E_{dcr} \geq E_{ad}$

for:

$$E_{dcr} = NA_{dcr} + \frac{v_{cr}^2}{2 \times g}$$

$$E_{ad} = El_{ca} + H_{ad} + \frac{v_a^2}{2 \times g}$$

$$v_{cr} = \frac{Q_k}{B_{ad} \times (NA_{dcr} - El_{cr})}$$

where:

E_{dcr}	height of the energy head line in the downstream channel of the sluiceways for the design flow in the diversion;
E_{ad}	height of the energy head line at the sluiceway outlet for the design flow in the diversion;
NA_{dcr}	water level in the downstream channel of the sluiceways for the design flow in the diversion;
v_{cr}	mean velocity of discharge in the downstream channel, in m/s;
g	9.81 m/s² – acceleration due to gravity;
El_{ca}	elevation of the bottom of the approach channel;

H_{ad}	height of sluiceways, in m;
v_a	mean velocity of discharge in a sluiceway, in m/s;
Q_k	design flow in the diversion for a recurrence time of k years, in m^3/s;
B_{ad}	total width of sluiceways, in m; and
El_{cr}	elevation of the bottom of the downstream channel.

The water level **at the upstream cofferdam** for sluiceways with a submerged outlet, NA_{dm}, is given by:

$$NA_{dm} = E_{dcr} + h_p$$

for:

$$h_p = 0.2 \times \frac{v_a^2}{2 \times g} + L_{og} \times \frac{n^2 \times v_a^2}{R_h^{4/3}}$$

$$R_h = \frac{B_{1ad} \times H_{ad}}{2 \times (B_{1ad} + H_{ad})}$$

where:

E_{dcr}	height of the energy head line in the downstream channel of the sluiceways for the design flow in the diversion, in m;
h_p	head loss along the sluiceway, in m;
v_a	mean velocity of discharge in the sluiceway, in m/s;
g	9.81 m/s^2 – acceleration due to gravity;
L_{og}	length of the ogee crest, in m;
n	0.013 – Manning's coefficient;
R_h	hydraulic radius of a sluiceway opening, in m;
B_{1ad}	width of a sluiceway opening, in m; and
H_{ad}	height of sluiceways, in m.

The water level at the upstream cofferdam for sluiceways with a free-flowing outlet

The outlet will be free flowing if: $E_{dcr} < E_{ga}$

where:

E_{dcr}	height of the energy head line in the downstream channel for the design flow in the diversion, in m; and
E_{ga}	height of the energy head line at the sluiceway outlet for the design flow in the diversion, in m.

The water level **at the upstream cofferdam** for sluiceways with a free-flowing outlet, NA_{dm}, can be obtained from Graph 5.7.3.01 (COPEL, 1977) or from:

$$NA_{dm} = El_{ca} + H$$

for:

$$H = k_H \times H_{ad}$$

$$k_H = 0.0184 \times k_Q^3 - 0.1323 \times k_Q^2 + 0.688 \times k_Q + 0.18$$

where:

El_{ca}	elevation of the bottom of the approach channel, in m;
H	hydrostatic load on the upstream face of the dam, in m;
H_{ad}	height of sluiceways, in m; and
k_Q, k_H	coefficients.

Common excavation (account .12.16.24.12.10)

Common excavation is included in the spillway.

Surface Rock Excavation (account .12.16.24.12.11)

Excavation in rock is included in the spillway.

Foundation Cleaning and Treatment (account .12.16.24.13)

Foundation cleaning and treatment is included in the spillway.

Concrete (account .12.16.24.14)

The **extra volume of concrete** for the sluiceways is included in the spillway.

Emergency Gates for the Diversion (account .12.16.24.23.16)

The **acquisition cost of a fixed-wheel emergency gate** for a diversion sluiceway, C_{cp} (R$), – FOB cost excluding transportation, insurance, assembly and testing costs and provisions for charges and taxes payable according to the applicable tax legislation – is given below (or obtained from Graph B 23, annex B, as a function of its dimensions and maximum hydrostatic load), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.13 \leq z \leq 9.17$: $C_{cp} = -4.399 \times z^2 + 124.8 \times z + 110$

valid for $9.17 < z \leq 126$: $C_{cp} = -0.128 \times z^2 + 57.3 \times z + 370$

for:

$$z = \frac{B_{1ga}^2 \times H_{ga} \times H_x}{1000} \qquad H_x = NA_{max} - El_{de}$$

where:

z	parameter, in m^4;
B_{1ga}	width of an opening for the galleries, in m;
H_{ga}	height of an opening for the galleries, in m;
H_x	maximum hydrostatic load on the sill of the gate, in m;
NA_{max}	maximum normal water level in the reservoir, in m; and
El_{de}	elevation of the sill at the inlet, in m.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for the taxes and charges payable on the equipment.

Gates to close diversion sluiceways (account .12.16.24.23.17)

The **acquisition cost of each gate to close the diversion sluiceway**, C_{sl} (R$), – FOB cost – can be obtained from the expression below (or from Graph B 25, annex B, as a function of its dimensions and maximum hydrostatic load), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.16 \leq z \leq 54.43$: $C_{sl} = 72.896 \times z^{0.716}$

for:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000} \qquad H_x = NA_{max} - El_{td}$$

where:

z	parameter, in m^4;
B_{cp}	width of the gates in the diversion sluiceway, in m;
H_{cp}	height of the gates in the diversion sluiceway, in m; and
H_x	maximum hydrostatic load on the sill of the gate in the diversion sluiceway, in m.

The **total acquisition cost of fixed parts and parts embedded in the concrete** for the gates to close the diversion sluiceway, C_{gpf} (R$), – FOB cost – is given below, valid for the December 2006 database and for projects anywhere in Brazil:

$$C_{gpf} = 2 \times N_{ad} \times (H_x + H_{bl}) \times 2{,}084.80$$

where:

N_{ad}	number of sluiceways;
H_x	maximum hydrostatic load on the sill of the gate, in m; and
H_{bl}	4.0 m – height of the spillway freeboard, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

Crane (account .12.16.24.20)

A construction hoist should be used.

DIVERSION SLUICEWAYS THROUGH AN UNGATED SURFACE SPILLWAY (ACCOUNT .12.16.24)

The main **information required for dimensioning purposes** is:

- design flow in the diversion for a recurrence time of k years, Q_k in m³/s, from item 5.1.2.;
- water level in the downstream channel for the sluiceways for the design flow in the diversion, NA_{dcr}, from item 5.1.2., in m;
- elevation of the bottom of the approach channel, El_{ca}, in m;
- elevation of the bottom of the downstream channel, El_{cr}, in m.

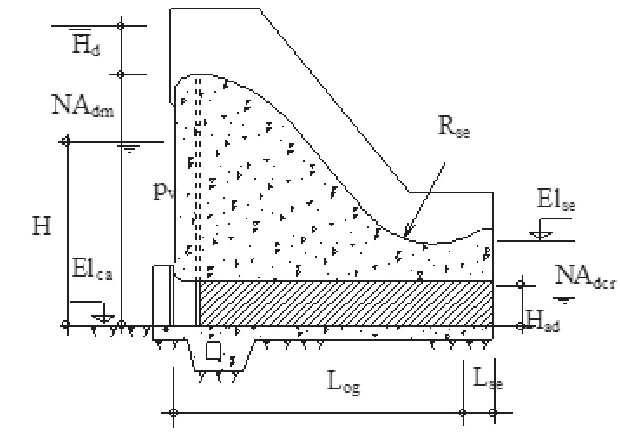
The main **information used for quantification** purposes is:

- height of the ogee above the bottom of the approach channel, p_v in m, from item 5.7.5.;
- hydrostatic load on the ogee crest, H_d in m, from item 5.7.5.;
- length of the spillway's ogee crest (direction of flow), L_{og}, in m, from item 5.7.5.;
- extra length of the spillway (direction of flow) for the ski jump, L_{se}, in m, from item 5.7.5., when applicable;
- radius of curvature of the ski jump, R_{se}, in m, from item 5.7.5., when applicable; and
- elevation of the ski jump sill, El_{se}, from item 5.7.5., when applicable.

Considerations and recommendations

This text relates to sluiceways with a typical cross-section, as shown in Fig. 5.7.3.09.

When the sluiceways are concreted, there may be no need to exclude water from the section downstream from the structure. Otherwise, gates can be used downstream from the structure or a cofferdam can be built in the downstream channel, always bearing in mind their respective costs.



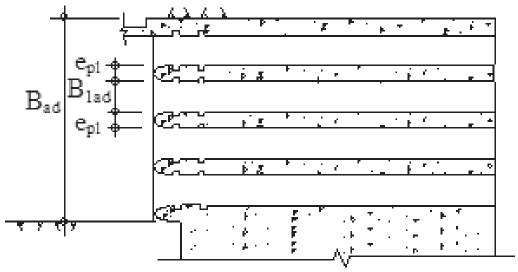


Fig. 5.7.3.09 – Typical cross-section and plan of sluiceways through spillways with a high ogee crest and a ski jump.

Coefficient k_Q can be reduced to meet the minimum **wigth or height** restrictions.

Ideally, the height of the sluiceways should respect the following: $V_{cdi} = m_j \times H_{bai} \times 0.5$

where:

H_{ad}	height of sluiceways, in m; and
B_{1ad}	width of sluiceway, in m.

In order to stay within the **velocity limits,** the width or the number of openings can be increased or coefficient k_Q can be reduced.

Whenever the **dimensions are altered**, the following ratio must be observed:

$$k_Q \times N_{ad} \times B_{1ad} \times H_{ad}^{3/2} = Q_k$$

where:

k_Q	coefficient;
N_{ad}	number of sluiceways;
B_{1ad}	width of sluiceway, in m;
H_{ad}	height of sluiceways, in m; and
Q_k	design flow in the diversion for a recurrence time of k years, in m^3/s.

For the **sluiceway** to be **efficient** – with a submerged inlet – the following restriction for coefficient k_Q must be respected:

$$k_Q \geq 1.5$$

When coefficient k_Q is higher, the sluiceway dimensions will be smaller and the cofferdams will be higher.

Generally speaking, for the **elevation of the bottom of the downstream channel**, the following must apply:

$$El_{cr} = El_{ca}$$

where:

El_{ca}	elevation of the bottom of the approach channel, in m.

Sluiceway Dimensions

First of all, the **number of sluiceways**, N_{ad}, is defined from the expression:

$$N_{ad} = \text{int}\left(\frac{Q_k}{1000} + 1.5\right)$$

where:

int(x)	function that returns the integer part of x; and
Q_k	design flow in the diversion for a recurrence time of k years, in m³/s.

The width of sluiceway, B_{1ad} (m), is given by:

$$B_{1ad} = \left(\frac{Q_k}{4 \times k_Q \times N_{ad}}\right)^{0.4} \geq 1.5\,\text{m}$$

for:

k_Q	3.2 , initially.

where:

Q_k	design flow in the diversion for a recurrence time of k years, in m³/s;
k_Q	coefficient; and
N_{ad}	number of sluiceways.

The **height of sluiceways**, H_{ad} (m), is given by:

$$H_{ad} = \left(\frac{Q_k}{k_Q \times N_{ad} \times B_{1ad}}\right)^{2/3} \geq 1.9\,\text{m}$$

for:

k_Q	3.2 , initially.

where:

Q_k	design flow in the diversion for a recurrence time of k years, in m³/s;
k_Q	coefficient;
N_{ad}	number of sluiceways; and
B_{1ad}	width of sluiceway, in m.

The **mean velocity of discharge**, v_a (m/s), is given by:

$$v_a = \frac{Q_k}{N_{ad} \times B_{1ad} \times H_{ad}} \leq 15\,\text{m/s}$$

where:

Q_k	design flow in the diversion for a recurrence time of k years, in m³/s;
N_{ad}	number of sluiceways;
B_{1ad}	width of sluiceway, in m; and
H_{ad}	height of a sluiceway, in m.

The thickness of the walls between two sluiceways, e_{pa} (m), is given by:

$$e_{pl} = 2.0 + 0.15 \times H_{ad}$$

where:

H_{ad}	height of sluiceways, in m.

The **total width of the sluiceways**, B_{ad} (m), is given by:

$$B_{ad} = N_{ad} \times (B_{1ad} + e_{pl}) + e_{pl}$$

where:

N_{ad}	number of sluiceways;
B_{1ad}	width of sluiceway, in m; and
e_{pl}	thickness of the spillway walls, in m.

Water level at the upstream cofferdam for sluiceways with a submerged outlet

The outlet will be free flowing if: $E_{dcr} \geq E_{ad}$

for:

$$E_{dcr} = NA_{dcr} + \frac{v_{cr}^2}{2 \times g} \qquad E_{ad} = El_{ca} + H_{ad} + \frac{v_a^2}{2 \times g}$$

$$v_{cr} = \frac{Q_k}{B_{ad} \times (NA_{dcr} - El_{cr})}$$

where:

E_{dcr}	height of the energy head line in the downstream channel of the sluiceways for the design flow in the diversion, in m;
E_{ad}	height of the energy head line at the sluiceway outlet for the design flow in the diversion, in m;
NA_{dcr}	water level in the downstream channel of the sluiceways for the design flow in the diversion, in m;
v_{cr}	mean velocity of discharge in the downstream channel, in m/s;
g	9.81 m/s² – acceleration due to gravity;
El_{ca}	elevation of the bottom of the approach channel, in m;
H_{ad}	height of sluiceways, in m;
v_a	mean velocity of discharge in the sluiceway, in m/s;
Q_k	design flow in the diversion for a recurrence time of k years, in m³/s;
B_{ad}	total width of the sluiceways, in m; and
El_{cr}	elevation of the bottom of the downstream channel, in m.

The water level **at the upstream cofferdam** for sluiceways with submerged outlets, NA_{dm}, is given by:

$$NA_{dm} = E_{dcr} + h_p$$

for:

$$h_p = 0.2 \times \frac{v_a^2}{2 \times g} + L_{og} \times \frac{n^2 \times v_a^2}{R_h^{4/3}} \qquad R_h = \frac{B_{1ad} \times H_{ad}}{2 \times (B_{1ad} + H_{ad})}$$

where:

E_{dcr}	height of the energy head line in the downstream channel for the design flow in the diversion, in m;
h_p	head loss along the sluiceway, in m;
v_a	mean velocity of discharge in the sluiceway, in m/s;
g	9.81 m/s² – acceleration due to gravity;
L_{og}	length of the ogee crest, in m;
n	0.013 – Manning's coefficient;
R_h	hydraulic radius of an opening in the sluiceways, in m;
B_{1ad}	width of an opening in the sluiceways, in m; and
H_{ad}	height of sluiceways, in m.

Water level at the upstream cofferdam for sluiceways with a free-flowing outlet

The outlet will be free flowing if: $E_{dcr} < E_{ga}$

where:

E_{dcr}	height of the energy head line in the downstream channel, in m; and
E_{ga}	height of the energy head line at the sluiceway outlet, in m.

The water level **at the upstream cofferdam** for sluiceways with a free-flowing outlet, NA_{dm}, can be obtained with the help of Graph 5.7.3.01 (COPEL, 1977) or by the expression:

$$NA_{dm} = El_{ca} + H$$

for:

$$H = k_H \times H_{ad}$$

$$k_H = 0.0184 \times k_Q^3 - 0.132 \times k_Q^2 + 0.688 \times k_Q + 0.18$$

where:

El_{ca}	elevation of the bottom of the approach channel, in m;
H	hydrostatic load upstream from the spillway, in m;
H_{ad}	height of sluiceways, in m; and
k_Q, k_H	coefficients.

Common excavation (account .12.16.24.12.10)

Common excavation is included in the spillway.

Surface Rock Excavation (account .12.16.24.12.11)

Excavation in rock is included in the spillway.

Foundation Cleaning and Treatment (account .12.16.24.13)

Foundation cleaning and treatment is included in the spillway.

Concrete (account .12.16.24.14)

The **extra volume of concrete** for the sluiceways is included in the spillway.

Emergency Gates for the Diversion (account .12.16.24.23.16)

The **acquisition cost of a fixed-wheel emergency gate** for the diversion sluiceway, C_{cp} (R$), – FOB cost, excluding transportation and insurance, assembly and testing and provisions for taxes and charges payable, depending on the current tax regime – is given by the expression below (or obtained from Graph B 23, annex B, as a function of its dimensions and the maximum hydrostatic load), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.13 \le z \le 9.17$: $C_{cp} = -4.399 \times z^2 + 124.8 \times z + 110$

valid for $9.17 < z \le 126$: $C_{cp} = -0.128 \times z^2 + 57.3 \times z + 370$

for:

$$z = \frac{B_{1ga}^2 \times H_{ga} \times H_x}{1000} \qquad H_x = NA_{max} - El_{de}$$

where:

z	parameter, in m^4;
B_{1ga}	width of an opening in the galleries, in m;
H_{ga}	height of an opening in the galleries, in m;

H_x	maximum hydrostatic load on the gate sill, in m;
NA_{max}	maximum normal water level in the reservoir, in m; and
El_{de}	elevation of the sill at the inlet, in m.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for the taxes and charges payable on the equipment.

Gates to close diversion sluiceways (account .12.16.24.23.17)

The **acquisition cost of a gate for a diversion sluiceway**, C_{sl} (R$), – FOB cost – can be obtained from the expression below (or from Graph. B 25, annex B, as a function of its dimensions and maximum hydrostatic load), valid for the December 2006 database and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.16 \le z \le 54.43$: $C_{sl} = 72.896 \times z^{0.716}$

for:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000} \qquad\qquad H_x = NA_{max} - El_{td}$$

where:

z	parameter, in m^4;
B_{cp}	width of the gates for the diversion tunnel, in m;
H_{cp}	height of the gates for the diversion tunnel, in m; and
H_x	maximum hydrostatic load on the sill of the gate in the diversion tunnel, in m.

The **total acquisition cost of fixed parts and parts embedded in the concrete** for the gates, C_{gpf} (R$), – FOB cost – is given by the expression below, valid for the December 2006 database and for projects anywhere in Brazil:

$$C_{gpf} = 2 \times N_{ad} \times (H_x + H_{bl}) \times 2{,}084.80$$

where:

N_{ad}	number of sluiceways;
H_x	maximum hydrostatic load on the sill of the gate for the diversion sluiceway, in m; and
H_{bl}	4.0 m – height of the spillway freeboard, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment should be added to the FOB cost.

Crane (account .12.16.24.23.20)

A construction hoist should be used.

5.7.4 Dams and Dikes (account .12.17)

EARTHFILL DAMS (ACCOUNT .12.17.25)

Basic Data

The main information required for dimensioning and quantification purposes can be obtained from the overall layout, as follows:

- maximum normal water level in the reservoir, NA_{max}, from item 4.6, in m;
- mean elevation of the land in section i perpendicular to the longitudinal dam axis, El_{tei}, in m;
- distance between sections i and i-1, ΔL_i in m;
- minimum water level in the reservoir, NA_{min}, from item 5.3, in m;
- depth of the cutoff trench, H_{tr} in m, when applicable.

Considerations and recommendations

This text relates to dams with a typical cross-section, as shown in Fig. 5.7.4.01.

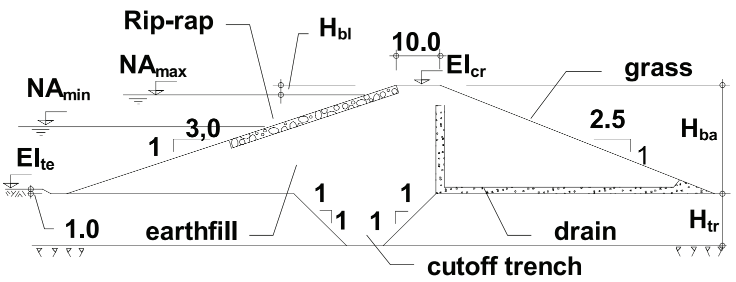


Fig. 5.7.4.01 – Typical cross-section of a homogeneous earthfill dam.

Sections should be selected over contour lines and at important features, like the bottom of saddles, tops of hills, river banks and at any points of contact, such as at intakes or spillways.

Quantities per type of service

The **quantities per type of service**, Qtd, are given by the general expression:

$$Qtd = \sum_i \frac{Sec_i + Sec_{i-1}}{2} \times \Delta L_i$$

where:

Sec_i	length, area or volume per meter of dam at section i, in m/m, m²/m or m³/m; and
ΔL_i	distance between sections i and i-1, in m.

The **height of the dam in section i**, H_{bai} (m), is given by:

$$H_{bai} = El_{cr} - (El_{tei} - 1.0)$$

where: $El_{cr} = NA_{max} + H_{bl}$

where:

El_{cr}	elevation of the dam crest, in m;
El_{tei}	mean elevation of the land in section i perpendicular to the longitudinal dam axis, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
H_{bl}	4.0m, height of the dam freeboard.

The **maximum reservoir drawdown**, d (m), is given by:

$$d = NA_{max} - NA_{min}$$

where:

NA_{max}	maximum normal water level in the reservoir, in m;
NA_{min}	minimum water level in the reservoir, in m.

The **width of the base of the cutoff trench** in section i, B_{tri} (m), required for permeable foundations, is given by:

$$B_{tri} = 0.3 \times (H_{bai} - H_{bl} + H_{tr}) \geq 6.0 \text{ m}$$

where:

H_{bai}	dam height at section i, in m;
H_{bl}	4.0m, height of the dam freeboard; and
H_{tr}	depth of the cutoff trench at section i, in m.

When an impervious blanket has to be used, the following dimensions should be assumed:

The **width of the impervious blanket at section i**, B_{tpi} (m), is given by:

for: $H_{tr} \geq 15.0$ m: $B_{tpi} = 10 \times (H_{bai} - H_{bl})$

for: $H_{tr} < 15.0$ m: $B_{tpi} = 0$

where:

H_{bai}	dam height at section i, in m;
H_{bl}	4.0m, height of the dam freeboard; and
H_{tr}	depth of the cutoff trench at section i, in m.

The **thickness of the impervious blanket at section i**, e_{tpi} (m), is given by:

$$e_{tpi} = 0.1 \times (H_{bai} - H_{bl})$$

where:

H_{bai}	dam height at section i, in m; and
H_{bl}	4.0m, height of the dam freeboard.

Common excavation in borrow areas (account .12.17.25.12.10)

The volume of common excavation per meter of dam at section i, V_{ti} (m^3/m), is given by:

$$V_{ti} = V_{tbi} + V_{tri} + V_{tpi}$$

where:

$$V_{tbi} = 5.5 \times H_{bai} + 30$$

$$V_{tri} = (B_{tri} + H_{tr}) \times H_{tr}$$

$$V_{tpi} = B_{tpi} \times 1.0$$

where:

V_{tbi}	volume of common excavation for the dam at section i, in m^3/m;
V_{tri}	volume of common excavation for the cutoff trench at section i, in m^3/m;
V_{tpi}	volume of common excavation for the impervious blanket at section, in m^3/m;
H_{bai}	dam height at section i, in m;
B_{tri}	width of the base of the cutoff trench at section i, in m;
H_{tr}	depth of the cutoff trench at section i, in m; and
B_{tpi}	width of the impervious blanket at section i, in m.

The **volume of common excavation in borrow areas**, V_{tp} (m^3), is given by:

$$V_{tp} = \sum V_{aj} - 0.9 \times \sum V_{tj} \geq 0$$

where:

V_{aj}	volume of landfill for structures j, in m^3; and
V_{tj}	volume of common excavation for structures j, in m^3.

The unit price of common excavation is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.17.25.13)

The area of foundation to be cleaned **per meter of dam at section i**, A_{lfi} (m^2/m), is given by:

$$A_{lfi} = 5.5 \times H_{bai} + 10$$

where:

H_{bai}	dam height at section i, in m.

The **volume of excavation and concrete leveling** (without reinforcement steel but with 200 kg cement per m^3) at the base of the cutoff trench, **per meter of dam at section i**, V_{TFi} (m^3/m), is given by:

$$V_{TFI} = 0.3 \times (H_{bai} - H_{bl} + H_{tr}) \geq 6.0 \text{ m}$$

where:

H_{bai}	dam height at section i, in m;
H_{bl}	4.0m, height of the dam freeboard; and
H_{tr}	depth of the cutoff trench at section i, in m.

The **length of the grout curtain per meter of dam at section i**, L_{tf} (m/m), is given by:

$$L_{tf} = \frac{1}{3.0} \times L_{1tf}$$

$$L_{1tf} = H_{bai} - H_{bl} \quad \leq \; 40 \text{ m}$$

where:

L_{1tf}	length of one grout hole, in m;
H_{bai}	dam height at section i, in m; and
H_{bl}	height of the dam freeboard, in m.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- leveling (dental) concrete: 113.00/m^3
- cleaning of rock surface: 39.70/m^2

- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

When they are needed, the **length of relief holes per meter of dam at section i**, L_{tf} (m/m), is given by:

$$L_{tf} = \frac{1}{10.0} \times (H_{bai} - H_{bl})$$

where:

H_{bai}	dam height at section i, in m; and
H_{bl}	4.0m, height of the dam freeboard.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of earth surface: 4.96/m^2
- cleaning of rock surface: 39.70/m^2
- rotary percussive drilling: 168.00/m
- dental concrete: 113.00/m^3

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Compacted Earthfill (account .12.17.25.24)

The **volume of compacted earthfill per meter of dam at section i**, V_{ai} (m^3/m), is given by:

$$V_{ai} = V_{abi} + V_{ari} + V_{api}$$

where:

$$V_{abi} = 2.75 \times H_{bai}^2 + 4.25 \times H_{bai} + 10 - 4.74 \times d_p$$

$$V_{ari} = (B_{tri} + H_{tr}) \times H_{tr} \qquad V_{api} = B_{tpi} \times e_{tpi}$$

$$d_p = H_{bl} + d + 4 \leq H_{bai}$$

where:

V_{abi}	volume of earthfill for the dam at section i, in m^3/m;
V_{ari}	volume of earthfill for the cutoff trench at section i, in m^3/m;
V_{api}	volume of earthfill for the impervious blanket at section i, in m^3/m;
H_{bai}	dam height at section i, in m;
d_p	effective distance, in m;
B_{tri}	width of the base of the cutoff trench at section i, in m;
H_{tr}	depth of the cutoff trench at section i, in m;
B_{tpi}	width of the impervious blanket at section i, in m;
e_{imi}	thickness of the impervious blanket at section i, in m;
H_{bl}	4.0m, height of the dam freeboard; and
d	maximum reservoir drawdown, in m.

The unit price of **compacted earthfill** is R$ 2.69/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the earthfill as defined by the design lines of the dam, and includes only spreading and compacting services and basically depends on the source of the material.

Below are the unit prices for earthfill-related services (in Reais, from December 2006 database) which can be used for projects in the south, southeast, central west and northeast regions of Brazil:

- stock handling: 6.98/m^3
- excavation of soil in borrow areas: 8.66/m^3
- extra for earth transportation: 2.29/m^3.km

Whenever an intermediate stockpile has to be used, the cost of the volume in question will relate to the replenishment and transportation services. If there should be a shortage of material, soil should be excavated from a borrow area, to which the cost of clearing the vegetation from the area, excavation, loading, transportation up to 1.5 km and unloading will be added. When the mandatory excavation sites are further away than 1.5 km, the cost of the transportation will be added to the volume cost.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Transitions, filters and drains (account .12.17.25.29)

The **volume of vertical filter per meter of dam at section i**, V_{vi} (m^3/m), is given by:

$$V_{vi} = 2 \times (H_{bai} - 5)$$

where:

H_{bai}	dam height at section i, in m.

The **volume of horizontal drain per meter of dam at section i**, V_{hi} (m^3/m), is given by:

$$V_{hi} = 3.75 \times H_{bai}$$

where:

H_{bai}	dam height at section i, in m.

The unit price of **drains and filters** is R$ 26.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the earth or rockfill as defined by the design lines of the dam, and includes producing artificial sand and gravel by processing the excavated rock, as well as transportation, spreading and compacting. Should natural sand have to be used, a specific market survey should be done or unit prices already charged in the region should be used, adding R$ 9.86/m^3 to the cost for the loading, local transportation, unloading and compacting services, based on the same database.

In order to obtain the final price of the compacted material and in view of the fact that the proportion of its component parts and services will considerably affect the unit price of the earthfill, an estimate should be made on a case-by-case basis of the proportion of volumes per type of service and origin of materials, which will give the mean weighted unit price.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Rip-Rap for the Upstream Face (account .12.17.25.32.18)

The **volume of rip-rap per meter of dam at section i**, V_{pi} (m^3/m), is given by:

$$V_{pi} = 4.74 \times d_p$$

where:

$$d_p = H_{bl} + d + 4 \leq H_{bai}$$

where:

d_p	effective distance, in m;
H_{bl}	4.0m, height of the dam freeboard;
d	maximum reservoir drawdown, in m; and
H_{bai}	dam height at section i, in m.

The unit price of **rip-rap** is R$ 10.40/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the rip-rap as defined by the design lines of the dam, and includes only spreading and compacting services and basically depends on the source of the material.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Protection of the downstream face (account .12.17.25.32.19)

The **area of grass per meter of dam at section i**, A_{gi} (m^2/m), is given by:

$$A_{gi} = 2.69 \times H_{bai} - 4$$

where:

H_{bai}	dam height at section i, in m.

The unit price for **grass** is R$ 5.90/m² (December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per square meter measured using the cross-section as defined by the design lines of the dam, and includes supplying and laying the grass.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Other Costs (account .12.17.25.17)

Other costs are estimated as accounting for 2% of the total cost of the corresponding structure.

ROCKFILL DAMS WITH A CENTRAL CLAY CORE (ACCOUNT .12.17.25)

Basic Data

The main information required for dimensioning and quantification purposes can be obtained from the overall layout, as follows:

- maximum normal water level in the reservoir, NA_{max}, from item 4.6;
- mean elevation of the land in section i perpendicular to the longitudinal dam axis, El_{tei}, in m;
- mean thickness of the layer of soil in the dam area, e_{tei} in m; slope of the upstream face, horizontal distance for a 1.0 m difference in level, m_m, in m;
- slope of the downstream face, horizontal distance for a 1.0 m difference in level, m_j, in m; and
- distance between sections i and i-1, ΔL_i, in m.

Considerations and recommendations

This text relates to dams with a typical cross-section, as shown in Fig. 5.7.4.02.

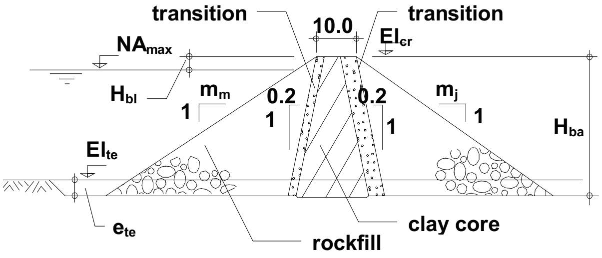


Fig. 5.7.4.02 – Typical cross-section of a rockfill dam with a central clay core.

Sections should be selected over contour lines and at important features, like the bottom of saddles, tops of hills, river banks and at any points of contact, such as at intakes or spillways.

The thickness of the **layer of soil** on the river bed may be different from the mean thickness on the abutments, or there may be no soil at all.

Quantities per type of service

The **quantities per type of service**, Qtd, are given by the general expression:

$$Qtd = \sum_i \frac{Sec_i + Sec_{i-1}}{2} \times \Delta L_i$$

where:

Sec_i	length, area or volume per meter of dam at section i, in m/m, m²/m or m³/m; and
ΔL_i	distance between sections i and i-1, in m.

The **height of the dam in section i**, H_{bai} (m), is given by:

$$H_{bai} = El_{cr} - (El_{tei} - e_{tei})$$

where:

$$El_{cr} = NA_{max} + H_{bl}$$

H_{bl}	For
3.0	dams with a maximum height of less than 20 m and a reservoir of less than 50 km²
4.0	all other cases

where:

El_{cr}	elevation of the dam crest;
El_{tei}	mean elevation of the land in section i perpendicular to the longitudinal dam axis;
e_{tei}	thickness of the layer of soil at section i of the dam, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
H_{bl}	height of the dam freeboard, in m.

Common excavation in borrow areas (account .12.17.25.12.10)

The **volume of common excavation per meter of dam at section i**, V_{ti} (m³/m), is given by:

$$V_{ti} = \left[(m_m + m_j) \times H_{bai} + 30 + e_{tei} \right] \times e_{tei}$$

where:

m	for
1.30	low dam at a site with good quality foundations and with no intermediate berms
1.75	very high dam at a site with poor quality foundations and with intermediate berms

where:

m_m	slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m;
m_j	slope of the downstream face, in m;
H_{bai}	dam height at section i, in m; and
e_{tei}	thickness of the layer of soil at section i of the dam, in m.

The **volume of common excavation in borrow areas**, V_{tp} (m^3), is given by:

$$V_{tp} = V_n - 0.9 \times \sum V_{tnj} \geq 0$$

where:

V_n	volume of clay core, in m^3; and
V_{tnj}	volume of common excavation for structures j that is suitable for the core, in m^3.

The unit price of common excavation is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface rock excavation (account .12.17.25.12.11)

The volume necessary for leveling is calculated as part of foundation treatment.

If there is any need to excavate below the shells or if cutoff trenches are required, this volume should be estimated and added to this account.

The volume of rock excavated from quarries, V_{rp} (m^3), is given by:

$$V_{rp} = \frac{\sum V_{ej}}{1.3} + \sum V_{cj} - 0.9 \times \left(\sum V_{rj} + \sum V_{sj} \right) \geq 0$$

where:

V_{ej}	volume of rockfill for structures j, in m^3;
1.3	coefficient assuming 30% increase in volume of rock after excavation;
V_{cj}	volume of concrete for structures j, in m^3;
0.9	coefficient considering a 10% loss;
V_{rj}	volume of excavated rock for structures j, in m^3; and
V_{sj}	volume of rock excavated for underground structures j, in m^3.

The unit price of **excavation in rock** is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.17.25.13)

The area of foundation to be cleaned **per meter of dam at section i**, A_{lfi} (m^2/m), is given by:

$$A_{lfi} = (m_m + m_j) \times H_{bai} + 10$$

where:

m_m	slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m;
m_j	slope of the downstream face, in m; and
H_{bai}	dam height at section i, in m.

The **volume of exavation and concrete leveling** (without reinforcement steel but with 200 kg cement per m^3) **under the core, per meter of dam at section i**, V_{TFi} (m^3/m), is given by:

$$V_{TFi} = 0.2 \times H_{bai} + 2$$

where:

H_{bai}	dam height at section i, in m.

The **length of the grout curtain per meter of dam at section i**, L_{tf} (m/m), is given by:

$$L_{tf} = \frac{1}{3.0} \times L_{1tf}$$

$$L_{1tf} = H_{bai} - H_{bl} \quad \leq 40\text{m}$$

where:

L_{1tf}	length of one grout hole, in m;
H_{bai}	dam height at section i, in m; and
H_{bl}	height of the dam freeboard, in m.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- surface rock excavation: 21.00/ m^3
- leveling (dental) concrete: 113.00/m^3
- cleaning of rock surface: 39.70/m^2
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Rockfill (account .12.17.25.25)

The **volume of rockfill per meter of dam at section i**, V_{ei} (m^3/m), is given by:

$$V_{ei} = \left(\frac{m_m + m_j - 0.4}{2}\right) \times H_{bai}^2$$

where:

m_m	slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m;
m_j	slope of the downstream face, in m; and
H_{bai}	dam height at section i, in m.

The unit price of **rockfill** is R$ 10.40/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the rip-rap as defined by the design lines of the dam, and includes only spreading and compacting services and basically depends on the source of the material.

Below are the unit prices for rockfill-related services (in Reais, from December 2006 database) which can be used for projects in the south, southeast, central west and northeast regions of Brazil:

- stock handling: 6.98/m³
- excavation in quarries: 14.43/m³
- rockfill transportation: 2.21/m³.km

Whenever an intermediate stockpile has to be used, the cost of the volume in question will relate to the replenishment and transportation services. If there should be a shortage of material, rockfill should be excavated in a quarry, to which the cost of clearing the vegetation from the area, excavation, loading, transportation up to 1.5 km and unloading will be added. When the mandatory excavation sites are further away than 1.5 km, the cost of the transportation will be added to the volume cost.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Clay Core (account .12.17.25.26)

The **volume of clay core per meter of dam at section i**, V_{ni} (m^3/m), is given by:

$$V_{ni} = 0.2 \times H_{bai}^2 + 4 \times H_{bai}$$

where:

H_{bai}	dam height at section i, in m.

The unit price of the **clay core** is R$ 11.10/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the earthfill as defined by the design lines of the dam, and includes only spreading and compacting services and basically depends on the source of the material.

Below are the unit prices for earthfill-related services (in Reais, from December 2006 database) which can be used for projects in the south, southeast, central west and northeast regions of Brazil:

- stock handling: 6.98/m³
- excavation of soil in borrow areas: 8.66/m³
- transportation of soil: 2.29/m³.km

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Transitions, Drains and Filters (account .12.17.25.29)

The **volume of transitions, drains and filters per meter of dam at section i**, V_{vi} (m^3/m), is given by:

$$V_{vi} = 6 \times H_{bai}$$

where:

H_{bai}	dam height at section i, in m.

The unit price of **transitions, drains and filters** is R$ 26.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the earth or rockfill as defined by the design lines of the dam, and includes producing artificial sand and gravel by processing the excavated rock, as well as transportation, spreading and compacting. Should natural sand have to be used, a specific market survey should be done or unit prices already charged in the region should be used, adding R$ 9.86/m^3 to the cost for the loading, local transportation, unloading and compacting services, based on the same database.

In order to obtain the final price of the compacted material and in view of the fact that the proportion of its component parts and services will considerably affect the unit price of the earthfill, an estimate should be made on a case-by-case basis of the proportion of volumes per type of service and origin of materials, which will give the mean weighted unit price.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Other Costs (account .12.17.25.17)

Other costs are estimated as accounting for 2% of the total cost of the corresponding structure.

ROCKFILL DAMS WITH AN INCLINED CLAY CORE (ACCOUNT .12.17.25)

Basic Data

The main information required for dimensioning and quantification purposes can be obtained from the overall layout, as follows:

- maximum normal water level in the reservoir, NA_{max}, from item 4.6, in m;
- mean elevation of the land in section i perpendicular to the longitudinal dam axis, El_{tei}, in m;
- thickness of the layer of soil at section i of the dam, e_{tei}, in m;
- slope of the upstream face, horizontal distance for a 1.0 m difference in level, m_m, in m;
- slope of the downstream face, horizontal distance for a 1.0 m difference in level, m_j, in m; and
- distance between sections i and i-1, ΔL_i, in m.

Considerations and recommendations

This text relates to dams with a typical cross-section, as shown in Fig. 5.7.4.03.

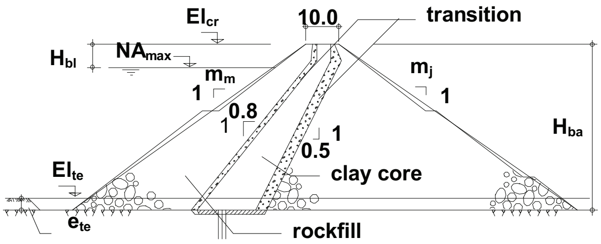


Fig. 5.7.4.03 – Typical cross-section of a rockfill dam with inclined clay core.

Sections should be selected over contour lines and at important features, like the bottom of saddles, tops of hills, river banks and at any points of contact, such as at intakes or spillways.

The thickness of the layer of soil on the river bed may be different from the mean thickness on the abutments, or there may be no soil at all.

Quantities per type of service

The **quantities per type of service**, Qtd, are given by the general expression:

$$Qtd = \sum_i \frac{Sec_i + Sec_{i-1}}{2} \times \Delta L_i$$

where:

Sec_i	length, area or volume per meter of dam at section i, in m/m, m^2/m or m^3/m; and
ΔL_i	distance between sections i and i-1, in m.

The height of the dam in section i, H_{bai} (m), is given by:

$$H_{bai} = El_{cr} - (El_{tei} - e_{tei})$$

where: $El_{cr} = NA_{max} + H_{bl}$

H_{bl}	For
3.0	dams with a maximum height of less than 20 m and a reservoir of less than 50 km^2
4.0	all other cases

where:

El_{cr}	elevation of the dam crest, in m;
El_{tei}	mean elevation of the land in section i perpendicular to the longitudinal dam axis, in m;
e_{tei}	thickness of the layer of soil at section i of the dam, in m;
NA_{max}	maximum normal water level in the reservoir; and
H_{bl}	height of the dam freeboard, in m.

Common excavation in borrow areas (account .12.17.25.12.10)

The **volume of common excavation per meter of dam at section i**, V_{ti} (m^3/m), is given by:

$$V_{ti} = \left[(m_m + m_j) \times H_{bai} + 30 + e_{tei} \right] \times e_{tei}$$

where:

m	For
1.30	low dam at a site with good quality foundations and with no intermediate berms
1.75	very high dam at a site with poor quality foundations and with intermediate berms

where:

m_m	slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m;
m_j	slope of the downstream face, in m;
H_{bai}	dam height at section i, in m; and
e_{tei}	thickness of the layer of soil at section i of the dam, in m.

The **volume of common excavation in borrow areas**, V_{tp} (m^3), is given by:

$$V_{tp} = V_n - 0.9 \times \sum V_{tnj} \geq 0$$

where:

V_n	volume of clay core, in m^3; and
V_{tnj}	volume of common excavation for structures j with material that is suitable for the core, in m^3.

The unit price of common excavation is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.17.25.12.11)

The volume necessary for leveling is calculated as part of the foundation treatment.

If there is any need to excavate below the shells or if cutoff trenches are required, this volume should be estimated and added to this account.

The **volume of rock excavated from quarries**, V_{rp} (m^3), is given by:

$$V_{rp} = \frac{\sum V_{ej}}{1.3} + \sum V_{cj} - 0.9 \times \left(\sum V_{rj} + \sum V_{sj}\right) \geq 0$$

where:

V_{ej}	volume of rockfill for structures j, in m^3;
1.3	coefficient assuming 30% increase in volume of rock after excavation;
V_{cj}	volume of concrete for structures j, in m^3;
0.9	coefficient considering a 10% loss;
V_{rj}	volume of excavated rock for structures j, in m^3; and
V_{sj}	volume of rock excavated for underground structures j, in m^3.

The unit price for **excavation in rock** is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter of excavated material and includes clearing the vegetation from the area, excavating, loading, transporting up to 1.5 km and unloading.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.17.25.13)

The **area of foundation to be cleaned per meter of dam at section i**, A_{lfi} (m^2/m), is given by:

$$A_{lfi} = (m_m + m_j) \times H_{bai} + 10$$

where:

m_m	slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m;
m_j	slope of the downstream face, in m; and
H_{bai}	dam height at section i, in m.

The **volume of excavated material and concrete leveling** (without reinforcement steel but with 200 kg cement per m³) **under the core, per meter of dam at section i**, V_{TFi} (m³/m), is given by:

$$V_{TFi} = 0.15 \times H_{bai} + 4.8$$

where:

H_{bai}	dam height at section i, in m.

The **length of grout curtain per meter of dam at section i**, L_{tf} (m/m), is given by:

$$L_{tf} = \frac{1}{3.0} \times (H_{bai} - H_{bl})$$

where:

H_{bai}	dam height at section i, in m; and
H_{bl}	height of the dam freeboard, in m.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- surface rock excavation: 21.00/ m³
- leveling (dental) concrete: 113.00/m³
- cleaning of rock surface: 39.70/m²
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount as identified in market research.

Rockfill (account .12.17.25.25)

The **volume of rockfill per meter of dam at section i**, V_{ei} (m³/m), is given by:

$$V_{ei} = \left(\frac{m_m + m_j - 0.3}{2} \right) \times H_{bai}^2 + 0.45 \times H_{bai} + 5$$

where:

m_m	slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m;
m_j	slope of the downstream face, in m; and
H_{bai}	dam height at section i, in m.

The unit price of **rockfill** is R$ 10.40/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the rockfill as defined by the design lines of the dam, and includes only spreading and compacting services and basically depends on the source of the material.

Below are the unit prices for rockfill-related services (in Reais, from December 2006 database) which can be used for projects in the south, southeast, central west and northeast regions of Brazil:

- stock handling: 6.98/m³
- excavation in quarries: 14.43/m³
- rockfill transportation: 2.21/m³.km

Whenever an intermediate stockpile has to be used, the cost of the volume in question will relate to the replenishment and transportation services. If there should be a shortage of material, rockfill should be excavated in a quarry, to which the cost of clearing the vegetation from the area, excavation, loading, transportation up to 1.5 km and unloading will be added. When the mandatory excavation sites are further away than 1.5 km, the cost of the transportation will be added to the volume cost.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Clay Core (account .12.17.25.26)

The **volume of clay core per meter of dam at section i**, V_{ni} (m^3/m), is given by:

$$V_{ni} = 0.15 \times H_{bai}^2 + 3.55 \times H_{bai} + 2$$

where:

H_{bai}	dam height at section i, in m.

The unit price of the **clay core** is R$ 11.10/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the earthfill as defined by the design lines of the dam, and includes only spreading and compacting services and basically depends on the source of the material.

Below are the unit prices for earthfill-related services (in Reais, from December 2006 database) which can be used for projects in the south, southeast, central west and northeast regions of Brazil:

- stock handling: 6.98/m³
- excavation of soil in borrow areas: 8.66/m³
- transportation of soil: 2.29/m³.km

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Transitions, Filters and Drains (account .12.17.25.29)

The **volume of transitions, filters and drains per meter of dam at section i**, V_{vi} (m^3/m), is given by:

$$V_{vi} = 6 \times H_{bai} - 7$$

where:

H_{bai}	dam height at section i, in m.

The unit price of **filters and drains** is R$ 26.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the earth or rockfill as defined by the design lines of the dam, and includes producing artificial sand and gravel by processing the excavated rock, as well as transportation, spreading and compacting. Should natural sand have to be used, a specific market survey should be done or unit prices already charged in the region should be used, adding R$ 9.86/m³ to the cost for the loading, local transportation, unloading and compacting services, based on the same database.

In order to obtain the final price of the compacted material and in view of the fact that the proportion of its component parts and services will considerably affect the unit price of the earthfill, an estimate should be made on a case-by-case basis of the proportion of volumes per type of service and origin of materials, which will give the mean weighted unit price.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Other Costs (account .12.17.25.17)

Other costs are estimated as accounting for 2% of the total cost of the corresponding structure.

CONCRETE-FACED ROCKFILL DAMS (ACCOUNT .12.17.25)

Basic Data

The main information required for dimensioning and quantification purposes can be obtained from the overall layout, as follows:

- maximum normal water level in the reservoir, NA_{max}, from item 4.6, in m;
- mean elevation of the land in section i perpendicular to the longitudinal dam axis, El_{tei}, in m;
- thickness of the layer of soil at section i of the dam, e_{tei}, in m;
- slope of the upstream face, horizontal distance for a 1.0 m difference in level, m_m in m;
- slope of the downstream face, horizontal distance for a 1.0 m difference in level, m_j in m; and
- distance between sections i and i-1, ΔL_i in m.

Considerations and recommendations

This text relates to dams with a typical cross-section, as shown in Fig. 5.7.4.04.

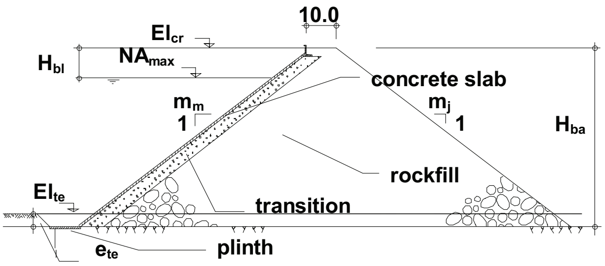


Fig. 5.7.4.04 – Typical cross-section of a concrete-faced rockfill dam.

Sections should be selected over contour lines and at important features, like the bottom of saddles, tops of hills, river banks and at any points of contact, such as at intakes or spillways.

The thickness of the layer of soil on the river bed may be different from the mean thickness on the abutments, or there may be no soil at all.

Quantities per type of service

The **quantities per type of service**, Qtd, are given by the general expression:

$$Qtd = \sum_i \frac{Sec_i + Sec_{i-1}}{2} \times \Delta L_i$$

where:

Sec_i	length, area or volume per meter of dam at section i, in m/m, m^2/m or m^3/m; and
ΔL_i	distance between sections i and i-1, in m.

The height of the dam in section i, H_{bai} (m), is given by:

$$H_{bai} = El_{cr} - (El_{tei} - e_{tei})$$

where: $El_{cr} = NA_{max} + H_{bl}$

H_{bl}	For
3.0	dams with a maximum height of less than 20 m and a reservoir of less than 50 km^2
4.0	all other cases

where:

El_{cr}	elevation of the dam crest;
El_{tei}	mean elevation of the land in section i perpendicular to the longitudinal dam axis;
e_{tei}	mean thickness of the layer of soil in the dam area, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
H_{bl}	height of the dam freeboard, in m.

Common excavation in borrow areas (account .12.17.25.12.10)

The **volume of common excavation per meter of dam at section i**, V_{ti} (m^3/m), is given by:

$$V_{ti} = \left[\left(m_m + m_j\right) \times H_{bai} + 33 + e_{tei}\right] \times e_{tei}$$

where:

m	For
1.3	low dam at a site with good quality foundations
1.5	very high dam at a site with poor quality foundations

where:

m_m	slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m;
m_j	slope of the downstream face, in m;
H_{bai}	dam height at section i, in m; and
e_{tei}	thickness of the layer of soil at section i of the dam, in m.

The unit price of common excavation is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface rock excavation (account .12.17.25.12.11)

If there is any need to excavate below the shells or if cutoff trenches are required, this volume should be estimated and added to this account.

The **volume of rock excavated from quarries**, V_{rp} (m^3), is given by:

$$V_{rp} = \frac{\sum V_{ej}}{1.3} + \sum V_{cj} - 0.9 \times \left(\sum V_{rj} + \sum V_{sj}\right) \geq 0$$

where:

V_{ej}	volume of rockfill for structures j, in m³;
1.3	coefficient assuming 30% increase in volume of rock after excavation;
V_{cj}	volume of concrete for structures j, in m³;
0.9	coefficient considering a 10% loss;
V_{rj}	volume of excavated rock for structures j, in m³; and
V_{sj}	volume of rock excavated for underground structures j, in m³.

The unit price for **excavation in rock** is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter of excavated material and includes clearing the vegetation from the area, excavating, loading, transporting up to 1.5 km and unloading.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.17.25.13)

The area of foundation to be cleaned **per meter of dam at section i**, A_{lfi} (m^2/m), is given by:

$$A_{lfi} = (m_m + m_j) \times H_{bai} + 13$$

where:

m_m	slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m;
m_j	slope of the downstream face, in m; and
H_{bai}	dam height at section i, in m.

The **length of rock anchors per meter of dam at section i**, L_{tfci} (m/m), is given by:

$$L_{tfci} = 13.3$$

The **length of the grout holes per meter of dam at section i**, L_{tfi} (m/m), is given by:

$$L_{tfi} = \frac{1}{3.0} \times (H_{bai} - H_{bl})$$

where:

H_{bai}	dam height at section i, in m; and
H_{bl}	height of the dam freeboard, in m.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- surface rock excavation: 21.00/ m^3
- leveling (dental) concrete: 113.00/m^3
- cleaning of rock surface: 39.70/m^2
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Rockfill (account .12.17.25.25)

The **volume of rockfill per meter of dam at section i**, V_{ei} (m^3/m), is given by:

$$V_{ei} = \left(\frac{m_m + m_j - 0.035}{2}\right) \times H_{bai}^2 + 2.47 \times H_{bai} - 12$$

where:

m_m	slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m;
m_j	slope of the downstream face, in m; and
H_{bai}	dam height at section i, in m.

The unit price of **rockfill** is R$ 10.40/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the rockfill as defined by the design lines of the dam, and includes only spreading and compacting services and basically depends on the source of the material.

Below are the unit prices for rockfill-related services (in Reais, from December 2006 database) which can be used for projects in the south, southeast, central west and northeast regions of Brazil:

- stock handling: 6.98/m^3
- excavation in quarries: 14.43/m^3
- extra transportation of rockfill: 2.29/m^3.km

Whenever an intermediate stockpile has to be used, the cost of the volume in question will relate to the replenishment and transportation services. If there should be a shortage of material, rockfill should be excavated from a quarry, to which the cost of clearing the vegetation from the area, excavation, loading, transportation up to 1.5 km and unloading will be added. When the mandatory excavation sites are further away than 1.5 km, the cost of the transportation will be added to the volume cost.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.17.25.14)

The **volume of concrete per meter of dam at section i**, V_{cbi} (m^3/m), is given by:

$$V_{cbi} = V_{cti} + V_{cli} + V_{cni} + V_{cdi}$$

where

$$V_{cti} = 3.19$$

$$V_{cli} = \sqrt{1+m_m^2} \times \left(0.00179 \times H_{bai}^2 + 0.29 \times H_{bai} - 0.8\right)$$

$$V_{cni} = 3.85 \qquad V_{cdi} = 2.75$$

where:

V_{cti}	volume of concrete for the parapet, in m^3/m;
V_{cli}	volume of concrete for slabs, in m^3/m;
V_{cni}	volume of concrete for the plinth, in m^3/m;
V_{cdi}	volume of leveling concrete, in m^3/m;
m_m	slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m; and
H_{bai}	dam height at section i, in m.

The amounts of cement and reinforcement steel are:

	cement (kg/m^3)	reinforcement steel (kg/m^3)
parapet	300	100
slab and plinth	250	80
leveling	200	0

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- parapet: 474.00/m^3
- slab and plinth: 234.00/m^3
- leveling: 113.00/m^3

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Transitions, Filters and Drains (account .12.17.25.29)

The **volume of the bed of crushed rock per meter of dam at section i**, V_{vi} (m^3/m), is given by:

$$V_{vi} = 0.0175 \times H_{bai}^2 + 4.9 \times H_{bai} - 14$$

where:

H_{bai}	dam height at section i, in m.

The unit price of **transitions** is R$ 26.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the earth or rockfill as defined by the design lines of the dam, and includes producing artificial sand and gravel by processing the excavated rock, as well as transportation, spreading and compacting. Should natural sand have to be used, a specific market survey should be done or unit prices already charged in the region should be used, adding R$ 9.86/m^3 to the cost for the loading, local transportation, unloading and compacting services, based on the same database.

In order to obtain the final price of the compacted material and in view of the fact that the proportion of its component parts and services will considerably affect the unit price of the earthfill, an estimate should be made on a case-by-case basis of the proportion of volumes per type of service and origin of materials, which will give the mean weighted unit price.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Other Costs (account .12.17.25.17)

Other costs are estimated as accounting for 2% of the total cost of the corresponding structure.

CONVENTIONAL CONCRETE GRAVITY DAMS (ACCOUNT .12.17.26)

Basic Data

The main information required for dimensioning and quantification purposes can be obtained from the overall layout, as follows:

- maximum normal water level in the reservoir, NA_{max}, from item 4.6, in m;
- mean elevation of the land in section i perpendicular to the longitudinal dam axis, El_{tei}, in m;
- thickness of the layer of soil at section i of the dam, e_{tei}, in m;
- slope of the downstream face, horizontal distance for a 1.0 m difference in level, m_j, in m;
- distance between sections i and i-1, ΔL_i, in m;
- elevation of the bottom of the approach channel to the diversion sluiceways, El_{ca}, from item 5.7.3, when applicable, in m;
- mean elevation of the land in the sluiceway area, El_{te}, when applicable, in m;
- height of sluiceways, H_{ad}, in m, from item 5.7.3, when applicable;
- width of one sluiceway, B_{1ad} in m, from item 5.7.3, when applicable;
- number of sluiceways, N_{ad}, from item 5.7.3, when applicable; and
- total width of sluiceways, B_{ad} in m, from item 5.7.3, when applicable.

Considerations and recommendations

This text relates to dams with a typical cross-section, as shown in Fig. 5.7.4.05.

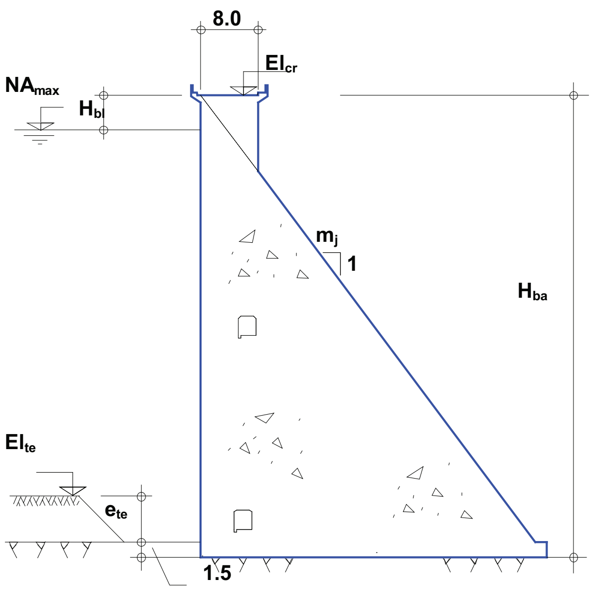


Fig. 5.7.4.05 – Typical cross-section of a conventional concrete gravity dam.

Sections should be selected over contour lines and at important features, like the bottom of saddles, tops of hills, river banks and at any points of contact, such as at intakes or spillways.

The thickness of the **layer of soil** on the river bed may be different from the mean thickness on the abutments, or there may be no soil at all.

Quantities per type of service

The **quantities per type of service**, Qtd, are given by the general expression:

$$Qtd = \sum_i \frac{Sec_i + Sec_{i-1}}{2} \times \Delta L_i$$

where:

Sec_i	length, area or volume per meter of dam at section i, in m/m, m²/m ou m³/m; and
ΔL_i	distance between sections i and i-1, in m.

The height of the dam in section i, H_{bai} (m), is given by:

$$H_{bai} = El_{cr} - (El_{tei} - e_{tei} - 1.5)$$

where: $El_{cr} = NA_{max} + H_{bl}$

where:

El_{cr}	elevation of the dam crest, in m;
El_{tei}	mean elevation of the land in section i perpendicular to the longitudinal dam axis, in m;
e_{te}	mean thickness of the layer of soil in the dam area, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
H_{bl}	3.0 m – height of the dam freeboard.

The **height of the dam in the section with the sluiceways**, H_{ba} (m), when applicable, is given by:

$$H_{ba} = El_{cr} - (El_{ca} - 1.5)$$

where:

El_{cr}	elevation of the dam crest, in m;
El_{ca}	mean elevation of the bottom of the approach channel to the sluiceways, in m;

Common Excavation (account .12.17.26.12.10)

The **volume of common excavation per meter of dam at section i**, V_{ti} (m^3/m), is given by:

$$V_{ti} = (m_j \times H_{bai} + 20 + e_{tei}) \times e_{tei}$$

where:

m_j	0.75 – slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m, for good quality foundations;
H_{bai}	dam height at section i, in m; and
e_{te}	mean thickness of the layer of soil in the dam area, in m.

The unit price of common excavation is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.17.26.12.11)

The **volume of excavated rock per meter of dam at section i**, V_{ri} (m^3/m), is given by:

$$V_{ri} = m_j \times H_{bai} \times 1.5$$

where:

m_j	slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; and
H_{bai}	dam height at section i, in m.

If there are sluiceways through the body of the dam, the **extra volume of material excavated in rock**, V_{res} (m^3), should be defined by the expression:

$$V_{res} = \left[m_j \times H_{ba} \times \left(El_{te} - e_{te} - El_{ca} \right) + 17 \right] \times B_{ad}$$

where:

m_j	slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m;
H_{ba}	height of the dam at the section with the sluiceway, in m;
El_{te}	mean elevation of the land in the sluiceway area, in m;
e_{te}	mean thickness of the layer of soil in the dam area, in m;
El_{ca}	mean elevation of the bottom of the approach channel to the sluiceways, in m;
B_{ad}	total width of sluiceways, in m.

The unit price of **excavation in rock** is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.17.26.13)

The **area of foundation to be cleaned per meter of dam at section i**, A_{lfi} (m^2/m), is given by:

$$A_{lfi} = m_j \times H_{bai}$$

where:

m_j	slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; and
H_{bai}	dam height at section i, in m.

The **length of the grout holes per meter of dam at section i**, L_{tfi} (m/m), is given by:

$$L_{tfi} = \frac{1}{3.0} \times \left(H_{bai} - H_{bl} \right)$$

where:

H_{bai}	dam height at section i, in m; and
H_{bl}	3.0 m – height of the dam freeboard.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m^2
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.17.26.14)

The **volume of concrete per meter of dam at section i**, V_{cbi} (m^3/m), is given by:

$$V_{cbi} = V_{cmi} + V_{cti}$$

where

$$V_{cmi} = \frac{m_j}{2} \times H_{bai}{}^2 + \frac{32}{m_j} + 1.13 \times m_j - 6.7$$

$$V_{cti} = 2.5$$

where:

V_{cmi}	volume of concrete for the body of the dam, in m^3/m;
V_{cti}	volume of concrete for the parapet, in m^3/m;
m_j	slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; and
H_{bai}	dam height at section i, in m.

If there are sluiceways through the body of the dam, the **extra volume of concrete for the sluiceways**, V_{cad} (m^3), should be defined by the expression:

$$V_{cad} = V_{cac} + V_{cpl} - V_{cae} + V_{cat}$$

where:

$$V_{cac} = (0.24 \times H_{ad} + 15) \times B_{ad}$$

$$V_{cpl} = (0.16 \times H_{ad}^2 + 2.7 \times H_{ad} + 8) \times (N_{ad} + 1) \times e_{pa}$$

$$V_{cae} = (0.38 \times H_{ad} + 0.2) \times H_{ad} \times N_{ad} \times B_{1ad}$$

$$V_{cat} = \frac{\left[NA_{max} - \left(EL_{ca} + \frac{H_{ad}}{2}\right)\right] \times B_{1ad} \times H_{ad}}{(2 \times H_{ad} + B_{1ad}) \times 6} \times H_{ad} \times N_{ad} \times B_{1ad}$$

where:

V_{cac}	volume of concrete for part of the sluiceway sills, in m^3;
V_{cpl}	volume of concrete for the sluiceway walls upstream from the face of the dam, in m^3;
V_{cae}	volume of concrete for the sluiceway inlets, in m^3;
V_{cat}	volume of concrete for the sluiceway plugs, in m^3
H_{ad}	height of sluiceways, in m;
B_{ad}	total width of sluiceways, in m;
e_{pa}	thickness of the sluiceway walls, in m;
N_{ad}	number of sluiceways; and
B_{1ad}	width of one sluiceway, in m.

The calculation for subtracting the **volume of empty space for the sluiceways** (V_{adu}), is given by:

$$V_{adu} = N_{ad} \times H_{ad} \times B_{1ad} \times m_j \times \left(H_{ba} - \frac{H_{ad}}{2}\right)$$

where:

H_{ad}	height of sluiceways, in m;
m_j	slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m;
H_{ba}	height of the dam, in m;
N_{ad}	number of sluiceways; and
B_{1ad}	width of one sluiceway, in m.

In the case in question, when the river is diverted through the dam, the **volume of concrete with quantities of cement and reinforcement steel greater** than the quantities for the dam, V_{cen} (m^3), is given by:

$$V_{cen} = V_{cet} + V_{ces} + V_{cep}$$

where:

$$V_{cet} = m_j \times (H_{ba} - H_{ad} - 1.5) \times 0.25 \times H_{ad} \times B_{ad}$$

$$V_{ces} = 1.5 \times m_j \times H_{ba} \times B_{ad}$$

$$V_{cep} = m_j \times \left(H_{ba} - \frac{H_{ad}}{2} - 1{,}5 \right) \times H_{ad} \times (N_{ad} + 1) \times e_{pa} - V_{cae}$$

where:

V_{cet}	volume of concrete for slabs above the sluiceways, in m^3;
V_{ces}	volume of concrete for the sluiceway sills, in m^3;
V_{cep}	volume of concrete for the walls between the sluiceways, in m^3;
m_j	slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m;
H_{ba}	height of the dam in the sluiceway area, in m;
H_{ad}	height of sluiceways, in m;
B_{ad}	Total width of sluiceways, in m;
e_{pa}	thickness of the sluiceway walls, in m;
N_{ad}	number of sluiceways; and
V_{cae}	volume of concrete for the sluiceway inlets, in m^3.

The amounts of cement and reinforcement steel are:

	cement (kg/m³)	reinforcement steel (kg/m³)
parapet	300	70
body of the dam and sluiceway inlet	200	10
sill and walls for the sluiceways	250	80
plugs	220	20

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- parapet: 474.00/m³
- body of the dam and sluiceway inlet: 113.00/m³
- sill and walls for the sluiceways: 174.00/m³
- plugs: 174.00/m³

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Other Costs (account .12.17.26.17)

Other costs are estimated as accounting for 2% of the total cost of the corresponding structure.

ROLLER COMPACTED CONCRETE DAMS (ACCOUNT .12.17.26)

Basic Data

The main information required for dimensioning and quantification purposes can be obtained from the overall layout, as follows:

- slope of the downstream face, horizontal distance for a 1.0 m difference in level, m_j in m; maximum normal water level in the reservoir, NA_{max}, from item 4.6, in m;
- mean elevation of the land in section i perpendicular to the longitudinal dam axis, El_{tei}, in m;
- thickness of the layer of soil at section i of the dam, e_{tei} in m;
- distance between sections i and i-1, ΔL_i in m;
- elevation of the bottom of the approach channel to the diversion sluiceways, El_{ca}, from item 5.7.3, when applicable, in m;
- mean elevation of the land in the sluiceway area, El_{te}, when applicable, in m;
- height of sluiceways, H_{ad} in m, from item 5.7.3, when applicable;
- width of one sluiceway, B_{1ad} in m, from item 5.7.3, when applicable;
- number of sluiceways, N_{ad}, from item 5.7.3, when applicable; and
- total width of sluiceways, B_{ad} in m, from item 5.7.3, when applicable.

Considerations and recommendations

This text relates to dams with a typical cross-section, as shown in Fig. 5.7.4.06.

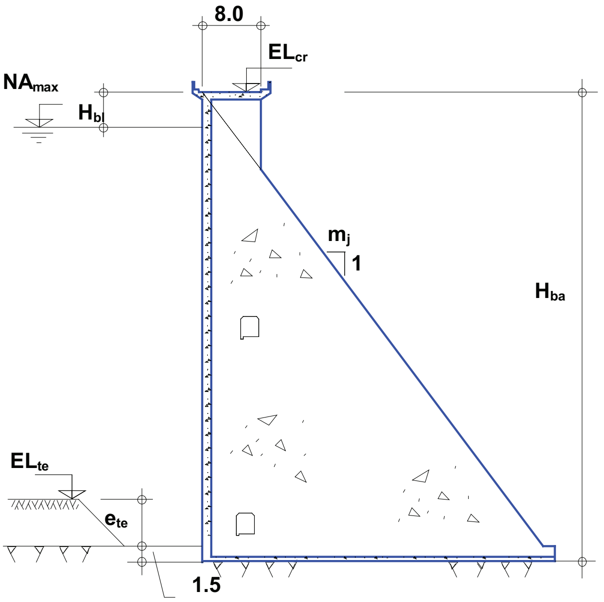


Fig. 5.7.4.06 – Typical cross-section of a roller compacted concrete dam.

Sections should be selected over contour lines and at important features, like the bottom of saddles, tops of hills, river banks and at any points of contact, such as at intakes or spillways.

The thickness of the **layer of soil** on the river bed may be different from the mean thickness on the abutments, or there may be no soil at all.

Quantities per type of service

The **quantities per type of service**, Qtd, are given by the general expression:

$$Qtd = \sum_i \frac{Sec_i + Sec_{i-1}}{2} \times \Delta L_i$$

where:

Sec_i	length, area or volume per meter of dam at section i, in m/m, m^2/m or m^3/m; and
ΔL_i	distance between sections i and i-1, in m.

The **height of the dam in section i**, H_{bai} (m), is given by:

$$H_{bai} = El_{cr} - (El_{tei} - e_{tei} - 1.5)$$

where: $El_{cr} = NA_{max} + H_{bl}$

where:

El_{cr}	elevation of the dam crest;
El_{tei}	mean elevation of the land in section i perpendicular to the longitudinal dam axis;
e_{te}	mean thickness of the layer of soil in the dam area, in m;
NA_{max}	maximum normal water level in the reservoir; and
H_{bl}	3.0 m – height of the dam freeboard.

The **height of the dam in the section with the sluiceways**, H_{ba} (m), when applicable, is given by:

$$H_{ba} = El_{cr} - (El_{ca} - 1.5)$$

where:

El_{cr}	elevation of the dam crest; and
El_{ca}	mean elevation of the bottom of the approach channel to the sluiceways.

Common Excavation (account .12.17.26.12.10)

The **volume of common excavation per meter of dam at section i**, V_{ti} (m^3/m), is given by:

$$V_{ti} = \left(m_j \times H_{bai} + 20 + e_{tei}\right) \times e_{tei}$$

where:

m_j	slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m;
H_{bai}	dam height at section i, in m; and
e_{tei}	thickness of the layer of soil at section i of the dam, in m.

The unit price of **common excavation** is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.17.26.12.11)

The **volume of excavated rock per meter of dam at section i**, V_{ri} (m^3/m), is given by:

$$V_{ri} = m_j \times H_{bai} \times 1.5$$

where:

m_j	slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; and
H_{bai}	dam height at section i, in m.

If there are sluiceways through the body of the dam, the **extra volume of material excavated in rock**, V_{res} (m^3), should be taken into account as shown below:

$$V_{res} = \left[m_j \times H_{ba} \times (El_{te} - e_{te} - El_{ca}) + 17 \right] \times B_{ad}$$

where:

m_j	slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m;
H_{ba}	height of the dam in the section with the sluiceway, in m;
El_{te}	mean elevation of the land in the sluiceway area;
e_{te}	mean thickness of the layer of soil in the dam area, in m;
El_{ca}	mean elevation of the bottom of the approach channel to the sluiceways; and
B_{ad}	total width of sluiceways, in m.

The unit price of **excavation in rock** is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.17.26.13)

The area of foundation to be cleaned **per meter of dam at section i**, A_{lfi} (m^2/m), is given by:

$$A_{lfi} = m_j \times H_{bai}$$

where:

m_j	slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m;
H_{bai}	dam height at section i, in m.

The **length of the grout holes per meter of dam at section i**, L_{tfi} (m/m), is given by:

$$L_{tfi} = \frac{1}{3.0} \times (H_{bai} - H_{bl})$$

where:

H_{bai}	dam height at section i, in m; and
H_{bl}	3.0 m – height of the dam freeboard.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- leveling (dental) concrete: 113.00/m^3
- cleaning of rock surface: 39.70/m^2
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.17.26.14)

The **volume of concrete per meter of dam at section i**, V_{cbi} (m^3/m), is given by:

$$V_{cbi} = V_{cdi} + V_{cpi} + V_{cci} + V_{cti} + V_{cri}$$

where

$$V_{cdi} = m_j \times H_{bai} \times 0.5$$

$$V_{cpi} = H_{bai} \times 0.0794 \times \sqrt{H_{bai} - H_{bl}} \geq 0$$

$$V_{cci} = 8.0 \qquad V_{cti} = 2.5$$

$$V_{cri} = \frac{m_j}{2} \times H_{bai}^{\ 2} - \left(0.5 \times m_j + 0.0794 \times \sqrt{H_{bai} - H_{bl}}\right) \times H_{bai} + \frac{32}{m_j} + 1.13 \times m_j - 14.7$$

where:

V_{cdi}	volume of leveling concrete, in m^3/m;
V_{cpi}	volume of concrete for the face, in m^3/m;
V_{cci}	volume of concrete for the crest, in m^3/m;
V_{cti}	volume of concrete for the parapet, in m^3/m;
V_{cri}	volume of roller compacted concrete, in m^3/m;
m_j	slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m;
H_{bl}	height of the dam freeboard, in m;
H_{bai}	dam height at section i, in m.

If there are sluiceways through the body of the dam, the **extra volume of concrete for the sluiceways**, V_{cad} (m^3), should be taken into account as shown below:

$$V_{cad} = V_{cac} + V_{cpl} - V_{cae} + V_{cat} - V_{adu}$$

where:

$$V_{cac} = (0.24 \times H_{ad} + 15) \times B_{ad}$$

$$V_{cpl} = \left(0.16 \times H_{ad}^2 + 2.7 \times H_{ad} + 8\right) \times (N_{ad} + 1) \times e_{pa}$$

$$V_{cae} = (0.38 \times H_{ad} + 0.2) \times H_{ad} \times N_{ad} \times B_{1ad}$$

$$V_{cat} = \frac{\left[NA_{max} - \left(EL_{ca} + \frac{H_{ad}}{2}\right)\right] \times B_{1ad} \times H_{ad}}{\left(2 \times H_{ad} + B_{1ad}\right) \times 6} \times H_{ad} \times N_{ad} \times B_{1ad}$$

$$V_{adu} = N_{ad} \times H_{ad} \times B_{1ad} \times m_j \times \left(H_{ba} - \frac{H_{ad}}{2}\right)$$

where:

V_{cac}	Volume of concrete for part of the sluiceway sill, in m^3;
V_{cpl}	Volume of concrete for the sluiceway walls upstream from the face of the dam, in m^3;
V_{cae}	Volume of concrete for the sluiceway inlets, in m^3;
V_{cat}	Volume of concrete for the sluiceway plugs, in m^3;
V_{adu}	Volume of empty space in the sluiceways (before plugging), in m^3;
H_{ad}	height of sluiceways, in m;
B_{ad}	total width of sluiceways, in m;
e_{pa}	thickness of the sluiceway walls, in m;
N_{ad}	number of sluiceways; and
B_{1ad}	width of one sluiceway, in m.

In the case in question, when the river is diverted through the dam, the **volume of concrete with quantities of cement and reinforcement steel greater** than the quantities for the dam, V_{cen} (m^3), is given by:

$$V_{cen} = V_{cet} + V_{ces} + V_{cep}$$

where:

$$V_{cet} = m_j \times \left(H_{ba} - H_{ad} - 1.5\right) \times 0.25 \times H_{ad} \times B_{ad}$$

$$V_{ces} = 1{,}5 \times m_j \times H_{ba} \times B_{ad}$$

$$V_{cep} = m_j \times \left(H_{ba} - \frac{H_{ad}}{2} - 1.5\right) \times H_{ad} \times \left(N_{ad} + 1\right) \times e_{pa} - V_{cae}$$

where:

V_{cet}	volume of concrete for slabs above the sluiceways, in m^3;
V_{ces}	volume of concrete for the sluiceway sills, in m^3;
V_{cep}	volume of concrete for the walls between the sluiceways, in m^3;
m_j	slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m;
H_{ba}	height of the dam in the sluiceway area, in m;
H_{ad}	height of sluiceways, in m;
B_{ad}	Total width of sluiceways, in m;
e_{pa}	thickness of the sluiceway walls, in m;
N_{ad}	number of sluiceways; and
V_{cae}	volume of concrete for the sluiceway inlets, in m^3.

The volume of **conventional concrete to replace roller-compacted concrete** for the dam, V_{ccs} (m^3), is given by:

$$V_{ccs} = m_j \times (H_{ba} \times d_6 - \frac{d_6^2}{2} - \frac{H_{ba}}{2}) \times B_{ad} - V_{cae}$$

where:

$d_6 = 1.25 \times H_{ad} + 1.5$

where:

m_j	slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m;
H_{ba}	height of the dam in the sluiceway area, in m;
B_{ad}	total width of sluiceways, in m;
V_{cae}	volume of concrete for the sluiceway inlets to be subtracted from the dam, in m^3;
H_{ad}	height of sluiceways, in m; and
d_6	secondary dimension, in m.

The amounts of cement and reinforcement steel are:

	cement (kg/m³)	reinforcement steel (kg/m³)
parapet	300	70
face	250	
crest	250	50
leveling	200	
roller compacted	100	
sluiceway sills and inlets	280	60
plugs	220	20

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- parapet: 474.00/m³
- face: 234.00/m³
- crest: 113.00/m³
- leveling: 113.00/m³
- roller compacted: 71.00/m³
- sluiceway sills and inlets: 174.00/m³
- sluiceway walls: 174.00/m³
- plugs: 128.00/m³

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Other Costs (account .12.17.26.17)

Other costs are estimated as accounting for 2% of the total cost of the corresponding structure.

CONCRETE RETAINING AND TRANSITION WALLS (ACCOUNT .12.17.27)

Retaining Walls

The **basic data required for dimensioning purposes** are:

- topographic information;
- geological information;
- mean elevation of the land in the wall area, El_{te};
- mean thickness of the layer of soil in the wall area, e_{te}, in m;
- maximum normal water level in the reservoir, NA_{max}, from item 4.4;
- height of the dam freeboard, H_{bl} in m, from item 5.7.4;
- slope of the upstream face, m_m, in m, and downstream face, m_j, in m, horizontal distance for a 1.0 m difference in level, from item 5.7.4.

Considerations and recommendations

This text relates to retaining walls with a typical cross-section, as shown in Fig. 5.7.4.07.

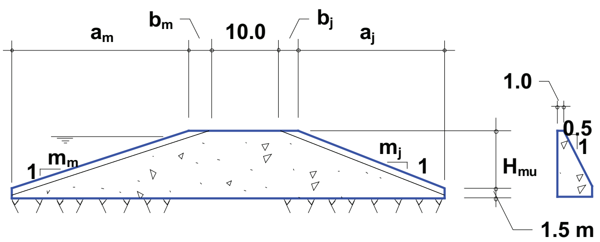


Fig. 5.7.4.07 – Typical cross-section of a retaining wall.

Retaining walls are only recommended for **heights** lower than 11m. For heights greater than 11m, the use of transition walls is recommended.

Height of the wall

The **height of the wall**, H_{mu} (m), is given by:

$$H_{mu} = NA_{max} + H_{bl} - (El_{te} - e_{te})$$

where:

NA_{max}	maximum normal water level in the reservoir;
H_{bl}	height of the dam freeboard, in m;
El_{te}	mean elevation of the land in the wall area; and
e_{te}	mean thickness of the layer of soil in the wall area, in m.

Common Excavation (account .12.17.27.12.10)

The **volume of common excavation**, V_{tmu} (m^3), is included in the dam account, except for earthfill dams, where it is given by:

$$V_{tmu} = (1.375 \times H_{mu}^2 + 13.25 \times H_{mu} + 15.5) \times e_{te}$$

where:

H_{mu}	height of the wall, in m; and
e_{te}	mean thickness of the layer of soil in the wall area, in m.

The unit price of **common excavation** is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the wall. The price includes clearing the

vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.17.27.12.11)

The **volume excavated in rock**, V_{rmu} (m^3), is given by:

$$V_{rmu} = 0.375 \times m \times H_{mu}^2 + (2.25 \times m + 7.5) \times H_{mu} + 1.5 \times m + 15$$

where: $m = m_m + m_j$

where:

m	secondary variable;
m_m	slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m;
m_j	slope of the downstream face, in m; and
H_{mu}	height of the wall, in m.

The unit price of **excavation in rock** is R\$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the wall. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.17.27.13)

The **area of foundation to be cleaned**, A_{lf} (m^2), is included in the dam account, except for earthfill dams, where it is given by:

$$A_{lf} = 0.25 \times m \times H_{mu}^2 + (1.5 \times m + 5) \times H_{mu} + m + 10$$

where: $m = m_m + m_j$

where:

m	secondary variable;
m_m	slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m;
m_j	slope of the downstream face, in m; and
H_{mu}	height of the wall, in m.

Foundation treatment, L_{tf} (m), is considered as part of the dam.

The unit price for **foundation cleaning** is R$ 39.70/m² (December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per square meter measured using the design line, and includes the service per se and the supply of inputs and equipment, which will depend on the kind of surface in question.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.17.27.14)

The **volume of concrete for a retaining wall**, V_{cmu} (m³), is given by:

$$V_{cmu} = m \times \frac{H_{mu}^3}{12} + (0.875 \times m + 2.5) \times H_{mu}^2 + (1.5 \times m + 12.5) \times H_{mu} + 15$$

where: $m = m_m + m_j$

where:

m	secondary variable;
m_m	slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m;
m_j	slope of the downstream face, in m; and
H_{mu}	height of the wall, in m.

The amounts of cement and reinforcement steel are:

	cement (kg/m³)	reinforcement steel (kg/m³)
structural concrete	280	50

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit price for **concrete without cement** is R$ 113.00/m³ (December 2006 database) and is valid for projects in the south, southeast, central west and northeast regions of Brazil. This is the unit price per cubic meter of volume of the wall and includes all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment.

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Transition Walls

The basic data used for this item are:

- topographic information;
- geological information;
- mean elevation of the land in the wall area, El_{te};

- mean thickness of the layer of soil in the wall area, e_{te} in m;
- maximum normal water level in the reservoir, NA_{max}, from item 4.6;
- height of the dam freeboard, H_{bl} in m, from item 5.7.4;
- slope of the upstream face, m_m in m, and downstream face, m_j in m, horizontal distance for a 1.0 m difference in level, from item 5.7.4.

Considerations and recommendations

This text relates to transition walls with a typical cross-section, as shown in Fig. 5.7.4.08.

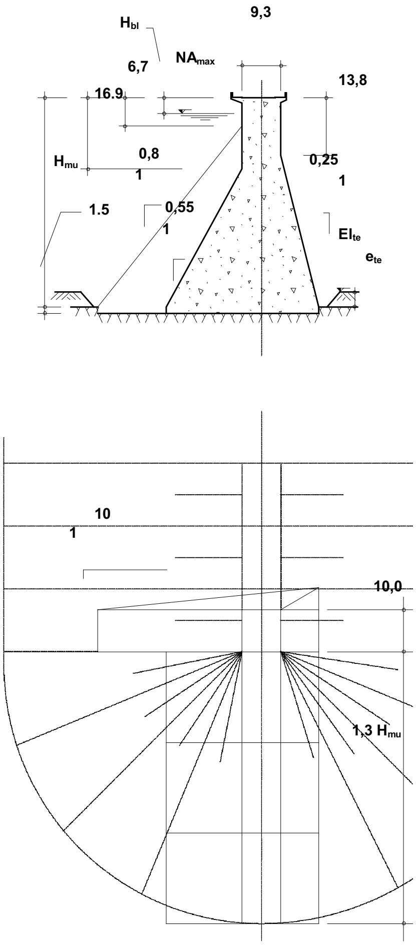


Fig. 5.7.4.08 – Typical cross-section of a transition wall.

Transition walls are recommended for heights of 11m or over. When the height is less than 11m, the use of retaining walls is recommended. However, when the wall is adjoining an intake, a transition wall should be used no matter how high it is.

Height of the wall

The **height of the wall**, H_{mu} (m), is given by:

$$H_{mu} = NA_{max} + H_{bl} - (El_{te} - e_{te})$$

where:

NA_{max}	maximum normal water level in the reservoir;
H_{bl}	height of the dam freeboard, in m;
El_{te}	mean elevation of the land in the wall area, in m; and
e_{te}	mean thickness of the layer of soil in the wall area, in m.

Common Excavation (account .12.17.27.12.10)

The **volume of common excavation**, V_{tmu} (m³), is included in the dam account, except for earthfill dams, where it is given by:

$$V_{tmu} = \left(1.1 \times H_{mu}^2 + 6.1 \times H_{mu} + 5\right) \times e_{te}$$

where:

H_{mu}	height of the wall, in m; and
e_{te}	mean thickness of the layer of soil in the wall area, in m;

The unit price of **common excavation** is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the wall. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.17.27.12.11)

The **volume excavated in rock**, V_{rmu} (m³), is given by:

$$V_{rmu} = \left(1.1 \times H_{mu}^2 + 6.1 \times H_{mu} + 5\right) \times 1.5$$

where:

H_{mu}	height of the wall, in m.

The unit price of **excavation in rock** is R$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the wall. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.17.27.13)

The **area of foundation to be cleaned**, A_{lf} (m²), is included in the dam account, except for earthfill dams, where it is given by:

$$A_{lf} = 1.1 \times H_{mu}^2 + 6.1 \times H_{mu} + 5$$

where:

H_{mu}	height of the wall, in m.

Foundation treatment, L_{tf} (m), is part of the dam account.

The unit price for **foundation cleaning** is R$ 39.70/m^2 (December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per square meter measured using the design line, and includes the service per se and the supply of inputs and equipment, which will depend on the kind of surface in question.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.17.27.14)

The **volume of concrete for transition walls**, V_{cmu} (m^3), is given by:

$$V_{cmu} = 0.53837 \times H_{mu}^3 + 2.1778 \times H_{mu}^2 + 146.8 \times H_{mu} + 425$$

where:

H_{mu}	height of the wall, in m.

The amounts of cement and reinforcement steel are:

	cement (kg/m^3)	reinforcement steel (kg/m^3)
structural concrete	210	5

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit price for **concrete without cement** is R$ 113.00/m^3 (December 2006 database) and is valid for projects in the south, southeast, central west and northeast regions of Brazil. This is the unit price per cubic meter of volume of the wall and includes all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

5.7.5 Spillways

GATED SURFACE SPILLWAYS WITH A HIGH OGEE CREST (ACCOUNT .12.18.28)

The main **information required for dimensioning purposes** can be obtained from the overall layout and item 5.1.2. (Hydrometeorological Data) as shown below:

- coefficient for determining the initial height of the gates, k_v;
- height of the gates, H_{cp} in m;
- design flood through the spillway, Q_v, in m^3/s, from item 5.1.2.;

- 100-year flood flow, Q_c, in m^3/s, from item 5.1.2.;
- maximum normal water level in the reservoir, NA_{max}, from item 4.6.;
- elevation of the bottom of the approach channel to the sluiceways, El_{ca}, from item 5.7.3., when applicable;
- elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, El_{cv};
- maximum water level in the downstream channel, NA_{xcr}, from item 5.1.2.;
- water level in the downstream channel for a 100-year flood, NA_{ccr}, from item 5.1.2.;
- elevation of the bottom of the downstream channel, El_{cr}; and
- height of sluiceways, H_{ad} in m, from item 5.7.3., when applicable.

The **information required for quantification purposes** is:

- mean elevation of the land in the area of the spillway, including the energy dissipator, El_{te}, in m;
- mean elevation of the land in the stilling basin or ski jump area, exclusively, El_{tde}, in m;
- mean thickness of the layer of soil in the area of the spillway per se, e_{te}, in m;
- mean elevation of the land at section i – 0, 1 and 2 from Fig. 5.7.5.05 – perpendicular to the longitudinal axis of the approach channel, El_{tai}, in m;
- mean elevation of the land at section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the downstream channel, El_{tri}, in m;
- mean length of the approach channel in the part with no sluiceways, L_{ca}, in m;
- mean length of the approach channel in the part with sluiceways, L_{cad}, in m, when applicable;
- mean length of the downstream channel, L_{cr}, in m;
- thickness of the concrete lining for the stilling basin sill, e_c, in m, when applicable;
- width of one sluiceway, B_{1ad} in m, from item 5.7.3., when applicable;
- total width of sluiceways, B_{ad} in m, from item 5.7.3., when applicable; and
- number of sluiceways, N_{ad}, from item 5.7.3., when applicable.

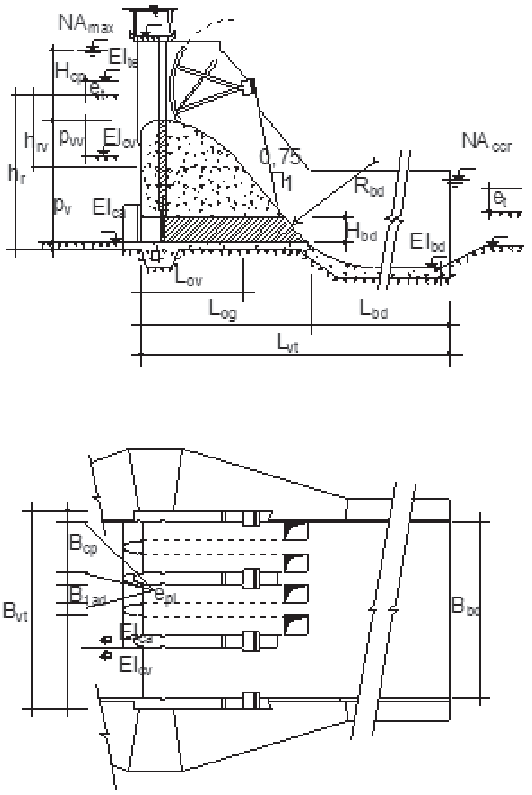


Fig. 5.7.5.01 – Typical cross-section and plan of a surface spillway with a high ogee crest and a stilling basin.

Considerations and recommendations

This text relates to spillways with a typical cross-section as shown in Fig. 5.7.5.01 and 5.7.5.02.

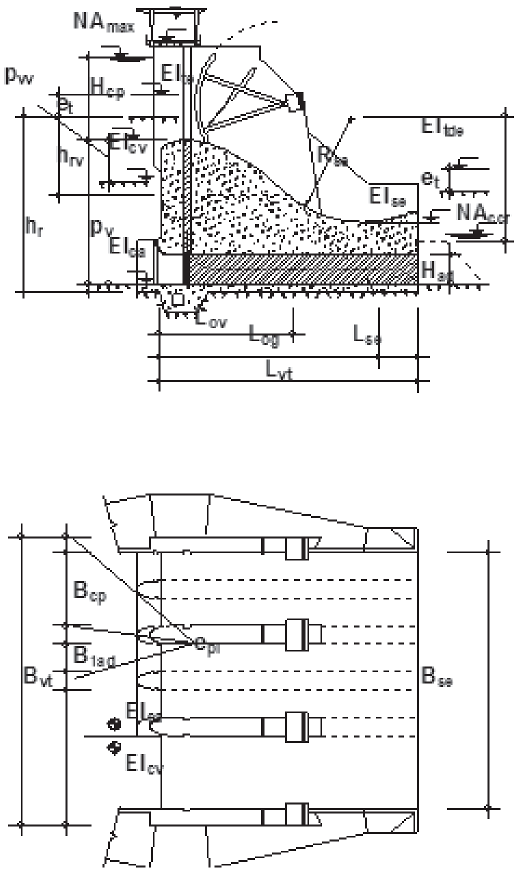


Fig. 5.7.5.02 – Typical cross-section and plan of a gated spillway with a high ogee crest and a ski jump.

An initial estimate of the **height of the gates**, H_{cp} (m), is given by (COPEL, 1996):

$$H_{cp} = k_v \times Q_v^{0.4} \leq 21.0m$$

where:

k_v	For spillways with
0.6	two gates
0.5	three gates
0.4	five gates
0.3	ten gates

where:

k_v	coefficient for determining the initial height of the gates; and
Q_v	design flow through the spillway, in m^3/s.

Intermediate k_v values can be used. In spillways when it is known that the ogee crest will be submerged, this coefficient should be raised.

When the spillway is not all on the river bed, the **elevation of the foundations** at the two ends of the structure are often different, in which case an mean of these elevations should be used for the elevation of the bottom of the approach channel to the spillway, excluding the sluiceways.

The following applies for **spillways with no sluiceways**:

$$El_{ca} = El_{cv}$$

where:

El_{ca}	elevation of the bottom of the approach channel to the sluiceways, in m; and
El_{cv}	elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m.

The thickness of the **layer of soil** on the river bed may be different from the thickness on the abutments, and there may often be no soil at all.

When **dimensioning the stilling basin**, the recommended range for the Froude number before the hydraulic jump is betwen 4.5 and 9.0, in order to ensure its stability. One way of raising the Froude number to 4.5 would be to lower the bottom of the stilling basin beyond the result given by the calculation. Likewise, one way to reduce the Froude number to 9.0 would be to reduce the width of the stilling basin.

In the absence of more accurate information, use 1.0 m as the **thickness of the concrete lining for the bottom of the stilling basin**.

Discharge Coefficient

The **initial discharge coefficient**, C'_d, for the typical cross-section recommended can be obtained from Graph 5.7.5.01 (BuRec, 1977) or from the equivalent expressions:

for: $z \leq 0.475$:

$$C'_d = 2.535 \times z^3 - 3.61 \times z^2 + 1.96 \times z + 1.702$$

for: $0.475 < z \leq 1.2$:

$$C'_d = 0.145 \times z^3 - 0.475 \times z^2 + 0.559 \times z + 1.916$$

for: $1.2 < z \leq 3.0$:

$$C'_d = -0.0072 \times z^2 + 0.0442 \times z + 2.112$$

for: $z > 3.0$:

$$C'_d = 2.18$$

where:

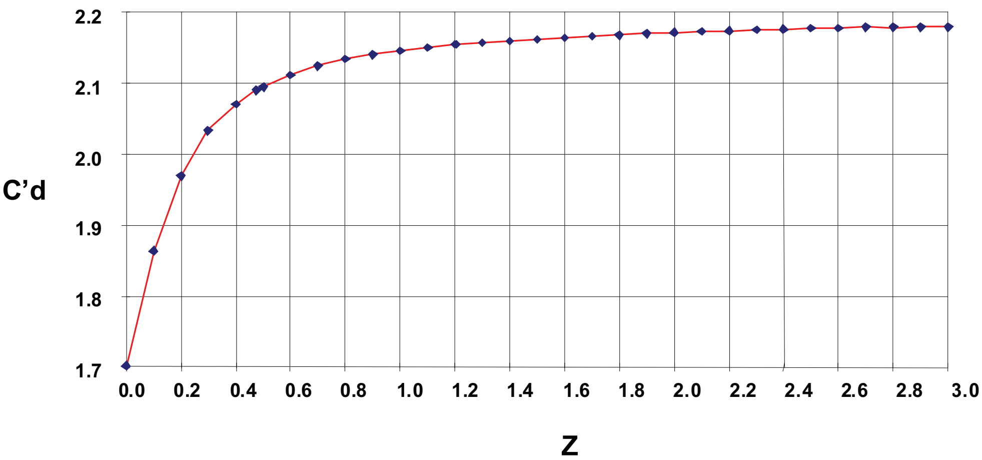
$$z = \frac{0.7 \times p_v + 0.3 \times p_{vv}}{H_{cp}}$$

$$p_v = NA_{max} - H_{cp} - El_{ca}$$

$$p_{vv} = NA_{max} - H_{cp} - El_{cv}$$

where:

z	adimensional parameter;
p_v	vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m;
p_{vv}	vertical distance between the ogee sill and the bottom of the spillway approach channel, sluiceways excluded, in m;
H_{cp}	height of the gates, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
El_{ca}	elevation of the bottom of the approach channel to the sluiceways, in m; and
El_{cv}	elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m.



Graph 5.7.5.01 – Initial Discharge Coefficient.

The **reduction coefficient for the discharge coefficient when the outlet is submerged**, k_c, can be obtained from Graph 5.7.5.02 (Bureau of Reclamation, 1977) or from the equivalent expressions:

for: $-4 \times u + 7 \times w + 2.6 \geq 0$ (section I):

$$k_c = -0.952 \times \left(\frac{1}{u}\right)^2 + 0.956 \times \left(\frac{1}{u}\right) + 0.767 \leq 1$$

for: $u < 3.6 \text{ e } -4 \times u + 7 \times w + 2.6 < 0$ (section II):

$$k_c = 1.058 - \frac{4 \times (u + 5)}{860 \times w} \leq 1$$

for: $u \geq 3.6$ (section III):

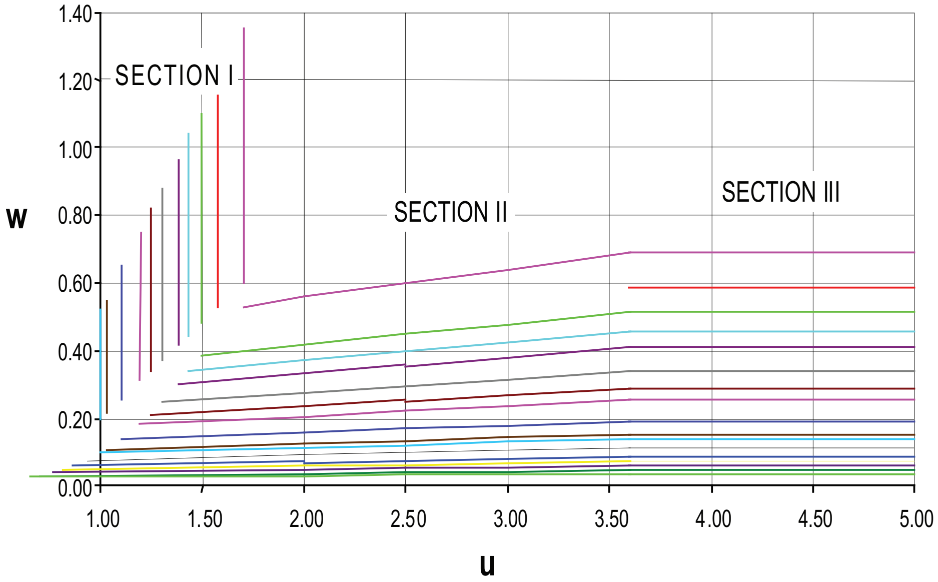
$$k_c = 1.058 - \frac{4}{100 \times w} \leq 1$$

where:

$$u = \frac{NA_{max} - El_{cr}}{H_{cp}} \qquad w = \frac{NA_{max} - NA_{xcr}}{H_{cp}}$$

where:

u, w	adimensional parameters;
NA_{max}	maximum normal water level in the reservoir, in m;
El_{cr}	elevation of the bottom of the downstream channel; , in m;
H_{cp}	height of the gates, in m; and
NA_{xcr}	maximum water level in the downstream channel, in m.



Graph 5.7.5.02 – Reduction coefficient for the discharge coefficient when the outlet is submerged.

The **corrected discharge coefficient**, C_d, is given by:

$$C_d = k_c \times C'_d$$

where:

k_c	reduction coefficient for the discharge coefficient; and
C'_d	initial discharge coefficient.

Dimensions of the gates and spillway

The **useful width of the openings**, B_{uvt} (m), is given by:

$$B_{uvt} = \frac{Q_v}{C_d \times H_{cp}^{3/2}}$$

where:

Q_v	design flood through the spillway, in m³/s;
C_d	discharge coefficient; and
H_{cp}	height of the gates, in m.

The **real width of the openings**, B_{rvt} (m), is given by:

$$B_{rvt} = B_{uvt} + 0.2 \times H_{cp}$$

where:

B_{uvt}	useful width of the openings, in m;
0.2	contraction coefficient at the end pillars; and
H_{cp}	height of the gates, in m.

The **number of gates**, N_{cp}, is given by:

$$N_{cp} = \text{int}\left(\frac{B_{rvt}}{H_{cp}} + 0.999\right)$$

where:

B_{rvt}	real width of the openings, in m;
H_{cp}	height of the gates, in m; and
int(x)	function that returns the integer part of x.

The **width of the gates**, B_{cp} (m), is given by:

$$B_{cp} = 0.05 \times \text{int}\left(\frac{1}{0.05} \times \frac{B_{rvt}}{N_{cp}} + 0.5\right)$$

where:

B_{rvt}	real width of the openings, in m; and
N_{cp}	number of gates.

The **thickness of the pillars**, e_{pl} (m), is given by:

$$e_{pl} = 0.12 \times H_{cp} + 2.4$$

where:

H_{cp}	height of the gates, in m.

The **total width of the spillway**, B_{vt} (m), perpendicular to flow, is given by:

$$B_{vt} = (N_{cp} + 1) \times e_{pl} + N_{cp} \times B_{cp}$$

where:

N_{cp}	number of gates;
e_{pl}	thickness of the pillars, in m; and
B_{cp}	width of the gates, in m.

The **length of the ogee crest** in the direction of flow is given by:

L_{ov} (m), for the part with no sluiceways:

$$L_{ov} = 1.46 \times H_{cp}^{0.46} \times (p_{vv} + 1.5)^{0.54} + 0.27 \times H_{cp}$$

ou L_{og} (m), for the part with sluiceways:

$$L_{og} = 1.46 \times H_{cp}^{0.46} \times (p_{v} + 1.5)^{0.54} + 0.27 \times H_{cp}$$

where:

H_{cp}	height of the gates, in m; and
p_v	vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m; and
p_{vv}	vertical distance between the ogee sill and the bottom of the spillway approach channel, sluiceways excluded, in m.

The **total length** of the spillway, L_{vt} (m), can be obtained from one of the following:

- for spillways with a stilling basin and sluiceways: $L_{vt} = L_{og} + L_{bd}$
- for spillways with a stilling basin and no sluiceways: $L_{vt} = L_{ov} + L_{bd}$
- for spillways with a ski jump and sluiceways: $L_{vt} = L_{og} + L_{se}$
- for spillways with a ski jump and no sluiceways: $L_{vt} = L_{ov} + L_{se}$

where:

L_{og}	length of the ogee crest with sluiceways, in m;
L_{ov}	length of the ogee crest with no sluiceways, in m;
L_{bd}	length of the stilling basin, defined below, in m; and
L_{se}	length of the ski jump at the foundations, defined below, in m.

Stilling Basin

The **width of the stilling basin**, B_{bd} (m), is given by:

$$B_{bd} = (N_{cp} - 1) \times e_{pl} + N_{cp} \times B_{cp}$$

where:

N_{cp}	number of gates;
B_{cp}	width of the gates, in m; and
e_{pl}	thickness of the pillars, in m.

The **stilling basin depth** is determined iteratively, based on the Froude number before the hydraulic jump, in section one of Fig. 5.7.5.03, for the 100-year flood flow .

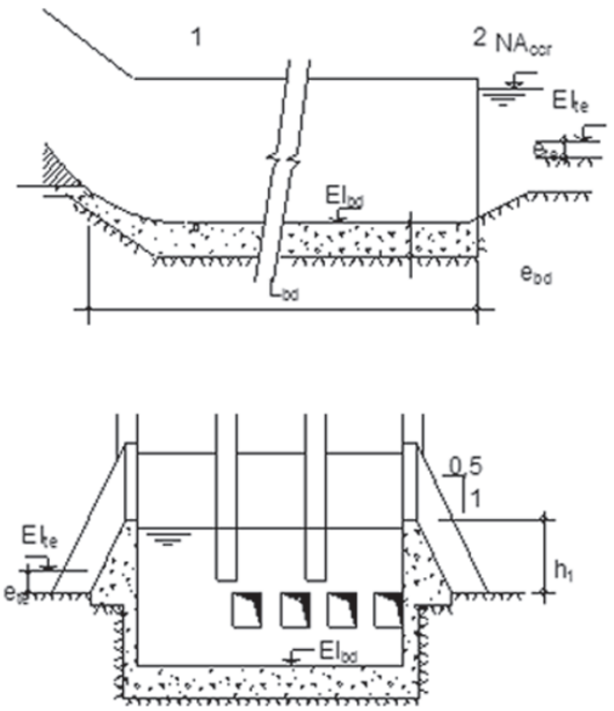


Fig. 5.7.5.03 – Typical cross-section of a stilling basin.

An elevation is picked for the bottom of the stilling basin and its suitability is checked by calculating the **velocity**, v_1 (m/s), the **depth of discharge**, y_1 (m), and the **Froude number**, Fr_1, before the hydraulic jump, **depth of discharge** after the hydraulic jump, y_2 (m), and finally the elevation of the **bottom of the stilling basin**.

Should the elevation of the bottom of the stilling basin be different from the figure first picked, the calculations should be redone until the level of precision required is reached.

$$v_1 = \sqrt{k \times 2 \times g \times (NA_{max} - El_{bd})} \qquad y_1 = \frac{Q_c}{B_{bd} \times v_1}$$

$$Fr_1 = \frac{v_1}{\sqrt{g \times y_1}} \qquad y_2 = \left(\frac{y_1}{2}\right) \times \left(\sqrt{1 + 8 \times Fr_1^2} - 1\right)$$

$$El_{bd} = NA_{ccr} - y_2$$

where:

v_1	mean velocity of discharge before the hydraulic jump, in m/s;
k	0.9 – reduction coefficient for the energy head;
g	9.81 m/s^2 – acceleration due to gravity;
NA_{max}	maximum normal water level in the reservoir, in m;
El_{bd}	elevation of the bottom of the stilling basin, in m;
y_1	depth of discharge before the hydraulic jump, in m;
Q_c	100-year flood flow, in m^3/s;
B_{bd}	width of the stilling basin, in m;
Fr_1	Froude number before the hydraulic jump;
y_2	depth of discharge after the hydraulic jump, in m; and
NA_{ccr}	water level in the downstream channel for a 100-year flood, in m.

The **radius of curvature at the stilling basin inlet**, R_{bd} (m), is given by:

$R_{bd} = 6 \times y_1$ (Peterka)

where:

y_1	depth of discharge before the hydraulic jump, in m.

The **length of the stilling basin**, L_{bd} (m), is given by:

$$L_{bd} = 6 \times y_2 + 0.75 \times (El_{ca} - El_{bd}) + 0.5 \times R_{bd} - 1.1$$

where:

y_2	depth of discharge after the hydraulic jump, in m;
El_{ca}	elevation of the bottom of the approach channel, in m;
El_{bd}	elevation of the bottom of the stilling basin, in m;
R_{bd}	radius of curvature at the stilling basin inlet, in m.

Ski Jump

The **width of the ski jump**, B_{se} (m), is given by:

$$B_{se} = (N_{cp} - 1) \times e_{pl} + N_{cp} \times B_{cp}$$

where:

N_{cp}	number of gates;
B_{cp}	width of the gates, in m; and
e_{pl}	thickness of the pillars, in m.

The **elevation of the ski jump sill**, El_{se}, is given by:

$$El_{se} = NA_{ccr} + 1.0 \geq El_{cr}$$

where:

NA_{ccr}	water level in the downstream channel for a 100-year flood, in m;
El_{cr}	elevation of the bottom of the downstream channel, in m.

If the sluiceways are built in the body of the spillway, the **elevation of the sill of the ski jump**, El_{se}, should be higher than the top of the sluiceways, as shown in Fig. 5.7.5.04:

$$El_{se} \geq El_{ca} + 1.25 \times H_{ad}$$

where:

El_{ca}	elevation of the bottom of the approach channel, in m;
H_{ad}	height of sluiceways, in m.

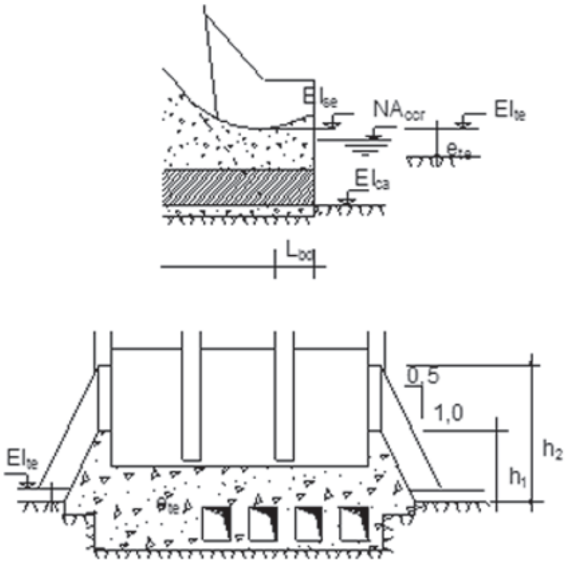


Fig. 5.7.5.04 – Typical cross-section of a ski jump.

The **radius of curvature of the ski jump**, R_{se} (m), is given by:

$$R_{se} = 6 \times y$$

where:

$$y = \frac{Q_c}{B_{se} \times v} \qquad v = \sqrt{k \times 2 \times g \times (NA_{max} - El_{se})}$$

where:

y	depth of the water column at the ski jump, in m;
Q_c	100-year flood flow, in m^3/s;
B_{se}	width of the ski jump, in m;
v	velocity of the water column at the ski jump, in m/s;
k	0.9 – reduction coefficient for the energy head;
g	9.81 m/s^2 – acceleration due to gravity;
NA_{max}	maximum normal water level in the reservoir, in m;
El_{se}	elevation of the ski jump sill, in m.

The **length of the ski jump at the foundations**, L_{se} (m), is given by:

$$L_{se} = d_{16} + 1.286 \times R_{se} - d_{12} \geq 0$$

where:

$$d_{16} = 1.46 \times H_{cp}^{0.46} \times (p_v - h_5)^{0.54}$$

$$d_{12} = 1.46 \times H_{cp}^{0.46} \times (p_v - 1.5)^{0.54}$$

$$h_5 = El_{se} + 0.6 \times R_{se} - El_{cr}$$

where:

R_{se}	radius of curvature of the ski jump, in m;
H_{cp}	height of the gates, in m;
p_v	vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m;
El_{se}	elevation of the ski jump sill, in m;
El_{cr}	elevation of the bottom of the downstream channel, in m; and
d_i	effective distances, in m.

Common Excavation (account .12.19.28.12.10)

The **volume of common excavation** for the spillway, V_{tvt} (m^3), is given by:

$$V_{tvt} = V_{tca} + V_{tes} + V_{tcr}$$

where:

$$V_{tca} = \left(\frac{V_{ta0}}{2} + V_{ta1} + V_{ta2}\right) \times \frac{L_{ca}}{3} + V_{tad}$$

$$V_{tes} = L_{vt} \times e_{te} \times B_{vt}$$

$$V_{tcr} = \left(\frac{V_{tr0}}{2} + V_{tr1} + V_{tr2}\right) \times \frac{L_{cr}}{3}$$

$$V_{tai} = \left[B_{ca} - 6 + 2 \times (0.6 \times h_{rai} + e_{te})\right] \times e_{te}$$

$$h_{rai} = El_{tai} - El_{cv} - e_{te}, \ i = 0, 1, 2$$

$$B_{ca} = B_{vt} - 2 \times (e_{pl} - 1.0)$$

$$V_{tad} = (L_{cad} - L_{ca}) \times e_{te} \times (B_{ad} - e_{pl})$$

$$V_{tri} = \left[B_{cr} - 6 + 2 \times (0.6 \times h_{rri} + e_{te})\right] \times e_{te}$$

$$h_{rri} = El_{tri} - El_{cr} - e_{te}, \ i = 0, 1, 2$$

for the stilling basin: $B_{cr} = B_{bd} + 2 \times 1.0$

for the ski jump: $B_{cr} = B_{se} + 2 \times 1.0$

where:

V_{tca}	volume of common excavation for the approach channel, in m^3;
V_{tes}	volume of common excavation for the structure, in m^3;
V_{tcr}	volume of common excavation for the downstream channel, in m^3;
V_{tai}	volume of common excavation per meter at section i of the approach channel, in m^3/m;
L_{ca}	mean length of the approach channel at the part with no sluiceways, in m;
V_{tad}	extra volume of common excavation for the approach channel because of the sluiceways, in m^3;
B_{vt}	total width of the spillway, in m;
e_{te}	mean thickness of the layer of soil in the area of the spillway per se, in m;
L_{vt}	total length of the spillway, in m;
V_{tri}	volume of common excavation per meter at section i for the downstream channel, in m^3/m;
L_{cr}	mean length of the downstream channel, in m;
B_{ca}	width of the bottom of the approach channel, in m;
h_{rai}	mean depth of excavation in rock at section i of the approach channel, in m;
El_{tai}	mean elevation of the land at section i of the approach channel;
El_{cv}	elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m;
e_{pl}	thickness of the pillars, in m;
L_{cad}	mean length of the approach channel in the part with sluiceways, in m;
B_{ad}	total width of sluiceways, in m;
B_{cr}	width of the bottom of the downstream channel, in m;
h_{rri}	mean depth of excavation in rock at section i of the downstream channel, in m;
El_{tri}	mean elevation of the land at section i perpendicular to the longitudinal axis of the downstream channel, in m;
El_{cr}	elevation of the bottom of the downstream channel, in m;
B_{bd}	width of the stilling basin, in m; and
B_{se}	width of the ski jump, in m.

The unit price of **common excavation** is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price

per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.18.28.12.11)

Fig. 5.7.5.05 shows typical cross-sections for the excavation of approach and downstream channels.

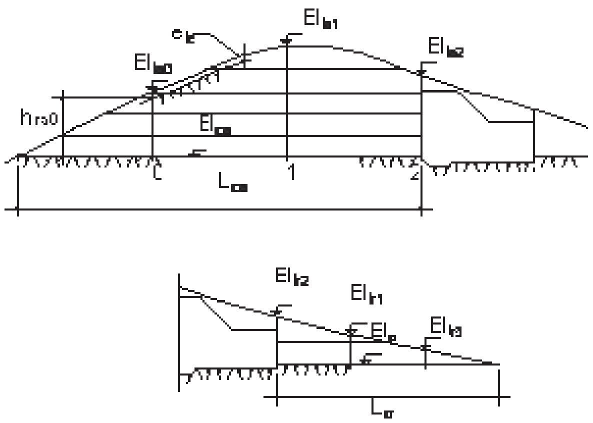


Fig. 5.7.5.05 – Excavation for approach and downstream channels.

The **volume of excavated rock** for the spillway, V_{rvt} (m^3), is given by:

$$V_{rvt} = V_{rca} + V_{rog} + V_{rpj} + V_{rde} + V_{rcr}$$

where:

$$V_{rca} = \left(\frac{V_{ra0}}{2} + V_{ra1} + V_{ra2}\right) \times \frac{L_{ca}}{3} + V_{rad}$$

$$V_{rcr} = \left(\frac{V_{rr0}}{2} + V_{rr1} + V_{rr2}\right) \times \frac{L_{cr}}{3}$$

$$V_{rai} = (B_{ca} - 6 + 0.6 \times h_{rai}) \times h_{rai}$$

$$V_{rri} = (B_{cr} - 6 + 0.6 \times h_{rri}) \times h_{rri}$$

$$h_r = El_{te} - e_{te} - (El_{ca} - 1.5)$$

$$h_{rv} = El_{te} - e_{te} - (El_{cv} - 1.5)$$

for spillways without sluiceways:

$$V_{rog} = (L_{ov} \times h_{rv} + 23) \times B_{vt}$$

$$V_{rad} = 0$$

and with a stilling basin:

$$V_{rpj} = d_1 \times \left(\frac{h_{rv} + h_{rb}}{2} + 0.167 \times H_{cp} \right) \times (B_{bd} + 2.0)$$

and with a ski jump:

$$V_{rpj} = d_7 \times \left(\frac{h_{rv} + h_{rs}}{2} + 0.167 \times H_{cp} \right) \times (B_{se} + 2.0)$$

$$V_{rde} = d_8 \times h_{rs} \times (B_{se} + 2.0)$$

for spillways with sluiceways:

$$V_{rog} = (L_{ov} \times h_{rv} + 23) \times (B_{vt} - B_{ad}) + (L_{og} \times h_r + 23) \times B_{ad}$$

$$V_{rad} = \frac{L_{cad} + L_{ca}}{2} \times (El_{cv} - El_{ca}) \times (B_{ad} - e_{pl}) - V_{tad}$$

and with a stilling basin:

$$V_{rpj} = d_1 \times \left(\frac{h_{rv} + h_{rb}}{2} + 0.167 \times H_{cp} \right) \times (B_{bd} + 2.0 - B_{ad}) + d_2 \times \left(\frac{h_r + h_{rb}}{2} + 0.167 \times H_{cp} \right) \times B_{ad}$$

$$d_2 = 0.75 \times \left[El_{ca} - 1.5 - (El_{bd} - e_c) \right]$$

and with a ski jump:

$$V_{rpj} = d_7 \times \left(\frac{h_{rv} + h_{rs}}{2} + 0.167 \times H_{cp} \right) \times (B_{se} + 2.0 - B_{ad})$$

$$V_{rsd} = d_8 \times h_{rs} \times (B_{se} + 2.0 - B_{ad}) + L_{se} \times h_r \times B_{ad}$$

for any spillway with a stilling basin:

$$V_{rde} = V_{rbd} + V_{rmc} + V_{rbe}$$

$$V_{rbd} = L_{bd} \times h_{rb} \times (B_{bd} + 2.0)$$

$$V_{rmc} = 2 \times (L_{bd} + d_4) \times 0.5 \times h_1 \times 1.5 + 2 \times (d_3 - d_4) \times 0.5 \times \frac{h_1 + h_2}{2} \times 1.5$$

$$V_{rbe} = 2 \times (L_{bd} + d_4) \times 0.3 \times h_{re}^2$$

$$h_{rb} = El_{te} - e_{te} - (El_{bd} - e_c)$$

$$h_1 = NA_{ccr} + 2.0 - (El_{tde} - e_{te}) \geq 0$$

$$h_2 = NA_{max} - 0.88 \times H_{cp} - (El_{tde} - e_{te}) \geq 0$$

$$d_1 = 0.75 \times \left[El_{cv} - 1.5 - (El_{bd} - e_c) \right]$$

$$d_3 = 0.75 \times \left[NA_{max} - 1.83 \times H_{cp} - (El_{bd} - e_c) \right]$$

$$d_4 = 0.75 \times (y_2 + 2.0 + e_c - 0.95 \times H_{cp}) \leq d_3$$

$$h_{re} = El_{tde} - e_{te} - (NA_{ccr} - 5.0) \geq 0$$

for any spillway with a ski jump:

$$h_{rs} = El_{te} - e_{te} - (El_{se} - 0.25 \times H_{cp})$$

$$d_7 = 1.46 \times H_{cp}^{0.46} \times \left[(p_v - h_4)^{0.54} - (p_{vv} + 1.5)^{0.54} \right]$$

$$h_4 = El_{se} - 0.25 \times H_{cp} - El_{ca}$$

$$d_8 = 0.986 \times R_{se} - 0.188 \times H_{cp}$$

where:

V_{rca}	volume of excavated rock for the approach channel, in m^3;
V_{rog}	volume of excavated rock in the ogee crest area, in m^3;
V_{rpj}	volume of excavated rock in the area of the downstream face of the ogee crest, in m^3;
V_{rde}	volume of excavated rock in the area of the stilling basin or ski jump, in m^3;
V_{rcr}	volume of excavated rock in the downstream channel, in m^3;
V_{rai}	volume of excavated rock per meter at section i of the approach channel, in m^3/m;
L_{cad}	mean length of the approach channel in the part with sluiceways, in m;
V_{rad}	extra volume of excavation in rock for the approach channel due to the sluiceways, in m^3;
V_{rri}	volume of excavated rock per meter at section i for the downstream channel, in m^3/m;
L_{cr}	mean length of the downstream channel, in m;
B_{ca}	width of the bottom of the approach channel, in m;
h_{rai}	mean depth of excavation in rock at section i of the approach channel, in m;
B_{cr}	width of the bottom of the downstream channel, in m;
h_{rri}	mean depth of excavation in rock at section i of the downstream channel, in m;
h_r	mean depth of the excavation in rock for the ogee crest in the part with sluiceways, in m;
El_{te}	mean elevation of the land in the area of the spillway per se, in m;
e_{te}	mean thickness of the layer of soil in the area of the spillway per se, in m;
El_{ca}	elevation of the bottom of the approach channel to the sluiceways, in m;
h_{rv}	mean depth of excavation in rock at the part of the ogee crest with no sluiceways, in m;
El_{cv}	elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m;
L_{ov}	length of the ogee crest at the part with no sluiceways, in m;
B_{vt}	total width of the spillway, in m;
h_{rb}	mean depth of excavation in rock for the stilling basin, in m;
H_{cp}	height of the gates, in m;
B_{bd}	width of the stilling basin, in m; and
L_{bd}	length of the stilling basin, in m;
h_{rs}	mean depth of excavation in rock for the ski jump at the part with no sluiceways, in m;
B_{se}	width of the ski jump, in m;
B_{ad}	total width of sluiceways, in m;
L_{og}	length of the ogee crest at the part with sluiceways, in m;
L_{ca}	mean length of the approach channel at the part with no sluiceways, in m;
e_{pl}	thickness of the pillars, in m;
V_{tad}	extra volume of common excavation for the approach channel because of the sluiceways, in m^3;
El_{bd}	elevation of the bottom of the stilling basin, in m;
e_c	thickness of the concrete lining for the stilling basin sill, in m;
L_{se}	length of the ski jump at the foundations, in m;
V_{rbd}	volume of excavated rock in the stilling basin area, in m^3;
V_{rmc}	volume of excavated rock in the area of the buttress walls for the stilling basin or ski jump, in m^3;
V_{rbe}	volume of excavated rock for berms in the stilling basin area, in m^3;
h_{re}	mean depth of excavation in rock in berms in the stilling basin area, in m;
NA_{ccr}	water level in the downstream channel for a 100-year flood, in m;
El_{tde}	mean elevation of the land in the stilling basin or ski jump area, in m;

NA_{max}	maximum normal water level in the reservoir, in m;
y_2	depth of discharge after the hydraulic jump, in m;
El_{se}	elevation of the ski jump sill, in m;
p_v	vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m;
p_{vv}	vertical distance between the ogee sill and the bottom of the spillway approach channel, sluiceways excluded, in m;
R_{se}	radius of curvature of the ski jump, in m; and
d_i, h_i	secondary dimensions, in m.

The unit price of **excavation in rock** is R\$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.18.28.13)

The **area of foundation to be cleaned** for the spillway, A_{lf} (m²), is given by:

$$A_{lf} = B_{vt} \times L_{vt}$$

where:

B_{vt}	total width of the spillway, in m; and
L_{vt}	total length of the spillway, in m.

The **length of the grout holes** and **drainage line** for the spillway, L_{tf} (m), is given by:

$$L_{tf} = \frac{B_{vt}}{3.0} \times L_{1tf}$$

$$L_{1tf} = 1.5 \times \left(NA_{max} - El_{ca}\right) \leq 40\text{m}$$

where:

B_{vt}	total width of the spillway, in m;
L_{1tf}	length of one grout hole, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
El_{ca}	elevation of the bottom of the approach channel; and
3.0	space between the grout holes, in m.

The **total length of the rock anchors** in the stilling basin, L_{tfc} (m), when necessary, is given by:

$$L_{tfc} = B_{bd} \times L_{bd}$$

where:

B_{bd}	width of the stilling basin, in m; and
L_{bd}	length of the stilling basin, in m.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m²
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.18.28.14)

The **volume of concrete** for the spillway, V_{cvt} (m³), is given by (COPEL, 1983 and 1996):

$$V_{cvt} = V_{cog} + V_{cpj} + V_{cpl} + V_{cpo} + V_{cde} + V_{cmv} + V_{cmc}$$

where:

$$V_{cpl} = (1.98 \times H_{cp}^2 + 6.0 \times H_{cp} + 6) \times (N_{cp} + 1) \times e_{pl}$$

$$V_{cpo} = 6.0 \text{ x } B_{vt}$$

for spillways without sluiceways:

$$V_{cog} = [0.944 \times H_{cp}^{0.46} \times (p_{vv} + 1.5)^{1.54} + 0.27 \times p_{vv} \times H_{cp} - 0.007 \times H_{cp}^2 + 0.40 \times H_{cp} + 18] \times B_{vt}$$

and with a stilling basin:

$$V_{cpj} = d_1 \times 0.167 \times H_{cp} \times (B_{bd} + 2.0)$$

and with a ski jump:

$$V_{cpj} = 1.25 \times H_{cp} \times h_5 \times (B_{se} + 2.0)$$

$$V_{csb} = 0$$

$$h_5 = El_{cv} - 1.5 - (El_{se} - 0.25 \times H_{cp}) \geq 0$$

for spillways with sluiceways:

$$V_{cog} = [0.944 \times H_{cp}^{0.46} \times (p_{vv} + 1.5)^{1.54} + 0.27 \times p_{vv} \times H_{cp}] \times (B_{vt} - B_{ad}) + [0.944 \times H_{cp}^{0.46} \times (p_v + 1.5)^{1.54} +$$
$$+ 0.27 \times p_v \times H_{cp}] \times B_{ad} + (-0.007 \times H_{cp}^2 + 0.40 \times H_{cp} + 18) \times B_{vt}$$

and with a stilling basin:

$$V_{cpj} = d_1 \times 0.167 \times H_{cp} \times (B_{bd} + 2.0 - B_{ad}) + d_2 \times 0.167 \times H_{cp} \times B_{ad}$$

$$d_2 = 0.75 \times [El_{ca} - 1.5 - (El_{bd} - e_c)]$$

and with a ski jump:

$$V_{cpj} = 1.25 \times H_{cp} \times h_5 \times (B_{se} + 2.0 - B_{ad})$$

$$V_{csb} = \left(\frac{d_9}{2} + L_{se}\right) \times (h_4 + 1.5) \times (B_{se} + 2.0)$$

$$d_9 = 1.46 \times H_{cp}^{0.46} \times \left[(p_v + 1.5)^{0.54} - (p_v - h_4)^{0.54} \right]$$

$$h_4 = El_{se} - 0.25 \times H_{cp} - El_{ca}$$

for any spillway with a stilling basin:

$$V_{cde} = \left(L_{bd} \times e_c + 0.036 \times R_{bd}^2 + 0.375 \times e_c^2 \right) \times (B_{bd} + 2.0)$$

$$V_{cmv} = 2 \times \left[\left(d_5 \times 0.95 \times H_{cp} \times 1.0 \right) + \frac{d_6^2}{2 \times 0.75} \right] + 2 \times L_{bd} \times (y_2 + 2.0) \times 1.0$$

$$V_{cmc} = 2 \times \left(L_{bd} + \frac{d_3 + d_4}{2} \right) \times \left(0.25 \times h_1^2 + 0.75 \times h_1 \right) + 2 \times \frac{d_3 - d_4}{2} \times \left(0.25 \times h_2^2 + 0.75 \times h_2 \right)$$

$$d_1 = 0.75 \times \left[El_{cv} - 1.5 - (El_{bd} - e_c) \right]$$

$$d_5 = 0.75 \times \left[NA_{max} - 2.0 \times H_{cp} - (El_{bd} - e_c) \right]$$

$$d_6 = 0.75 \times \left(y_2 + 2.0 + e_c - 0.95 \times H_{cp} \right)$$

$$d_3 = 0.75 \times \left[NA_{max} - 1.83 \times H_{cp} - (El_{bd} - e_c) \right]$$

$$d_4 = 0.75 \times \left(y_2 + 2.0 + e_c - 0.95 \times H_{cp} \right) \leq d_3$$

$$h_1 = NA_{ccr} + 2.0 - (El_{tde} - e_{te}) \geq 0$$

$$h_2 = NA_{max} - 0.88 \times H_{cp} - (El_{tde} - e_{te}) \geq 0$$

for any spillway with a ski jump:

$$V_{cde} = V_{csd} + V_{csb}$$

$$V_{csd} = (0.116 \times R_{se}^2 + 0.247 \times H_{cp} \times R_{se} - 0.023 \times H_{cp}^2) \times (B_{se} + 2.0)$$

$$V_{cmv} = 2 \times (d_{11} \times 1.6 \times y \times 1.0 + d_{13} \times 0.95 \times H_{cp} \times 1.0)$$

$$V_{cmc} = 2 \times \left(d_{11} + \frac{d_{10}}{2} \right) \times \left(0.25 \times h_3^2 + 0.75 \times h_3 \right) + 2 \times \frac{d_{10}}{2} \times \left(0.25 \times h_2^2 + 0.75 \times h_2 \right)$$

$$d_{10} = 0.75 \times [NA_{max} - 0.878 \times H_{cp} - (El_{se} - 1.6 \times y)] \geq 0$$

$$d_{11} = d_{12} + L_{se} - 1.46 \times H_{cp} - d_{13}$$

$$d_{12} = 1.46 \times H_{cp}^{0.46} \times (p_v + 1.5)^{0.54}$$

$$d_{13} = 0.75 \times [NA_{max} - 1.05 \times H_{cp} - (El_{se} - 1.6 \times y)] \geq 0$$

$$h_3 = El_{se} - 1.6 \times y - (El_{tde} - e_{te}) \geq 0$$

$$h_2 = NA_{max} - 0.88 \times H_{cp} - (El_{tde} - e_{te}) \geq 0$$

where:

V_{cog}	volume of concrete for the ogee crest, in m^3;
V_{cpj}	volume of concrete for the downstream face of the ogee crest, in m^3;
V_{cpl}	volume of concrete for the pillars, in m^3;
V_{cpo}	volume of concrete for the bridge, in m^3;
V_{cde}	volume of concrete for the stilling basin or ski jump, in m^3;
V_{cmv}	volume of concrete for the vertical lining of the stilling basin or ski jump, in m^3;
V_{cmc}	volume of concrete for the walls of the stilling basin or ski jump, in m^3;
H_{cp}	height of the gates, in m;
N_{cp}	number of gates;
e_{pl}	thickness of the pillars, in m;
B_{vt}	total width of the spillway, in m;
p_{vv}	vertical distance between the ogee sill and the bottom of the spillway approach channel, sluiceways excluded, in m;
B_{bd}	width of the stilling basin, in m;
B_{se}	width of the ski jump, in m;
B_{ad}	total width of sluiceways, in m;
p_v	vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m;
El_{ca}	elevation of the bottom of the approach channel to the sluiceways, in m;
El_{bd}	elevation of the bottom of the stilling basin, in m;
e_c	thickness of the concrete lining for the stilling basin sill, in m;
L_{se}	length of the ski jump at the foundations, in m;
El_{se}	elevation of the ski jump sill, in m;
L_{bd}	length of the stilling basin, in m;
R_{bd}	radius of curvature at the stilling basin inlet, in m;
y_2	depth of discharge after the hydraulic jump, in m;
El_{cv}	elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
NA_{ccr}	water level in the downstream channel for a 100-year flood, in m;
El_{tde}	mean elevation of the land in the stilling basin or ski jump area, in m;
e_{te}	mean thickness of the layer of soil in the area of the spillway per se, in m;
V_{csd}	volume of concrete for the ski jump deflector baffle, in m^3;
V_{csb}	volume of concrete below the deflector baffle, in m^3;
R_{se}	radius of curvature of the ski jump, in m;
y	flow depth in the ski jump, in m; and
d_i, h_i	secondary dimensions, in m.

If there are sluiceways through the body of the spillway, the **extra volume of concrete for the sluiceways**, V_{cad} (m^3), should be taken into account as shown below:

$$V_{cad} = V_{cac} + V_{cpl} - V_{cae}$$

where:

$$V_{cac} = (0.24 \times H_{ad} + 2) \times B_{ad}$$

$$V_{cpl} = (0.16 \times H_{ad}^2 + 2.7 \times H_{ad} + 8) \times (N_{ad} + 1) \times e_{pl}$$

$$V_{cae} = (0.38 \times H_{ad} + 0.2) \times H_{ad} \times N_{ad} \times B_{1ad}$$

where:

V_{cac}	volume of concrete for part of the sluiceway sills, in m^3;
V_{cpl}	volume of concrete for the sluiceway walls upstream from the face of the dam, in m^3;
V_{cae}	volume of concrete for the sluiceway inlets, in m^3;

H_{ad}	height of sluiceways, in m;
B_{ad}	total width of sluiceways, in m;
N_{ad}	number of sluiceways;
e_{pl}	thickness of the walls, in m; and
B_{1ad}	width of one sluiceway, in m.

Still considering the case where the diversion is through sluiceways in the spillway, the **volume of concrete for larger amounts of cement and reinforcement steel** than for the ogee crest of the spillway, V_{cen} (m^3), is given by:

$$V_{cen} = V_{cet} + V_{ces} + V_{cep}$$

in the case of a stilling basin:

$$V_{cet} = (0.27 \times H_{cp} + d_{14}) \times 0.25 \times H_{ad} \times B_{ad}$$

$$V_{ces} = (0.27 \times H_{cp} + d_{15}) \times 1.5 \times B_{ad}$$

$$V_{cep} = \left(0.27 \times H_{cp} + \frac{d_{14} + d_{15}}{2}\right) \times H_{ad} \times (N_{ad} + 1) \times e_{pl}$$

$$d_{14} = 1.46 \times H_{cp}^{0.46} \times (p_v - H_{ad})^{0.54}$$

$$d_{15} = 1.46 \times H_{cp}^{0.46} \times p_v^{\ 0.54}$$

and for a ski jump:

$$V_{cet} = (L_{vt} - d_0) \times 0.25 \times H_{ad} \times B_{ad}$$

$$V_{ces} = L_{vt} \times 1.5 \times B_{ad}$$

$$V_{cep} = \left(L_{vt} - \frac{d_0}{2}\right) \times H_{ad} \times (N_{ad} + 1) \times e_{pl}$$

$$d_0 = (L_{vt} - 0.27 \times H_{cp} - d_{16} - 0.836 \times R_{se} + 0.15 \times H_{cp}) \times \frac{H_{ad}}{h_4}$$

$$d_{16} = 1.46 \times H_{cp}^{0.46} \times (p_v - h_4)^{0.54}$$

$$h_4 = El_{se} - 0.25 \times H_{ad} - El_{ca}$$

where:

V_{cet}	volume of concrete for the sluiceway roofs, in m^3;
V_{ces}	volume of concrete for the sluiceway sills, in m^3;
V_{cep}	volume of concrete for the sluiceway walls, in m^3;
H_{cp}	height of the spillway gates, in m;
H_{ad}	height of sluiceways, in m;
B_{ad}	total width of sluiceways, in m;
N_{ad}	number of sluiceways;
e_{pl}	thickness of the walls, in m;
p_v	vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m;
L_{vt}	total length of the spillway, in m;
R_{se}	radius of curvature of the ski jump, in m;
El_{se}	elevation of the ski jump sill, in m;
El_{ca}	elevation of the bottom of the approach channel to the sluiceways, in m;
d_i, h_i	secondary dimensions, in m.

The amounts of cement and reinforcement steel are:

	cement (kg/m^3)	reinforcement steel (kg/m^3)
ogee crest, buttress, beneath the deflector baffle, sluiceway sills and inlets	200	20
downstream face of the ogee crest, stilling basin and deflector baffle	250	50
walls and walls	250	80
bridge	300	100
cement and reinforcement steel with greater unit quantities	250	80

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of spillway (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- ogee crest, buttress, beneath the deflector baffle: 113.00/m^3
- walls, walls, stilling basin and deflector baffle: 200.00/m^3
- bridge: 474.00/m^3
- sluiceway sills and inlets and concrete with greater unit quantities: 174.00/m^3

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

Tainter Gates (account .12.18.28.23.16)

The **acquisition cost of each surface tainter gate** for the spillway, C_{cp} (R$), – FOB cost excluding tranportation and insurance, assembly and testing and provisions for taxes payable, depending on the current tax regime – can be obtained from the expression below (or from Graph B.21, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $2.2 \leq z \leq 178$: $C_{cp} = 193.95 \times z^{0.5406}$

where:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$$

$H_x = H_{cp}$

where:

z	parameter, in m^4;
B_{cp}	width of the gates, in m;
H_{cp}	height of the gates, in m; and
H_x	maximum hydrostatic load on the gate sill, in m.

The following percentages must be added to the FOB price:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for taxes and charges payable on the equipment.

Stoplogs

The **acquisition cost of each stoplog** for the spillway, C_{sl} (R$), – FOB cost – can be obtained from the expression below (or from Graph B.24, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.34 \le z \le 177$: $C_{sl} = 72.9 \times z^{0.716}$

where: $z = \dfrac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$

where:

z	parameter, in m^4;
B_{cp}	width of the gates, in m;
H_{cp}	height of the gates, in m; and
H_x	maximum hydrostatic load on the gate sill, in m.

The cost of transportation and insurance, assembly and testing, taxes and charges payable on the equipment must be added to the FOB cost.

The **overall acquisition cost of fixed parts and parts embedded in the concrete** of the stoplogs for the spillway, C_{gpf} (R$), – FOB cost – is given by the expression below. The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

$$C_{gpf} = N_{cp} \times \left[2 \times \left(H_{cp} + H_{bl}\right) + B_{cp}\right] \times 2084.80$$

where:

N_{cp}	number of gates;
H_{cp}	height of the gates, in m;
H_{bl}	height of the spillway freeboard, in m; and
B_{cp}	width of the gates, in m.

The cost of transportation and insurance, assembly and testing, taxes and charges payable on the equipment must be added to the FOB cost.

Crane (account .12.18.28.23.20)

The **acquisition cost of the gantry crane** for the spillway, C_{pcr} (R$), – FOB cost – can be obtained from the expressions below (or from Graph B.26, annex B, as a function of the dimensions of the stoplog and the maximum hydrostatic load on its sill). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.34 \le z \le 13.15$: $C_{pcr} = 141.34 \times z^{0.3555}$

valid for $13.15 < z \le 176.43$: $C_{pcr} = -0.0082 \times z^2 + 6.8982 \times z + 263.93$

where: $z = \dfrac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$

where:

z	parameter, in m^4;
B_{cp}	width of the gates, in m;
H_{cp}	height of the gates, in m; and
H_x	maximum hydrostatic load on the gate sill, in m.

The cost of transportation and insurance, assembly and testing, taxes and charges payable on the equipment must be added to the FOB cost.

Other Costs (account .12.18.28.17)

Other costs are estimated as accounting for 2% of the total cost of the corresponding structure.

GATED SURFACE ABUTMENT SPILLWAYS (ACCOUNT .12.18.28)

Basic Data

The main **information required for dimensioning purposes** can be obtained from the overall layout and from item 5.1.2. (Hydrometeorological Data) as follows:

- coefficient for determining the initial height of the gates, k_v;
- height of the gates H_{cp}, in m;
- design flood through the spillway, Q_v, in m^3/s, from item 5.1.2.;
- 100-year flood flow, Q_c, in m^3/s, from item 5.1.2.;
- maximum normal water level in the reservoir, NA_{max}, from item 4.6.;
- elevation of the bottom of the approach channel, El_{ca}, from item 5.7.3. , in m;
- slope of the upstream face of the ogee crest, horizontal distance for a 1.0 m difference in level, m_m in m;
- slope of the chute, tangent of the absolute value of the angle with the horizontal, i_{cl};, in m/m;
- water level in the downstream channel for a 100-year flood, NA_{ccr}, from item 5.1.2, in m.

The **main information used for quantification** purposes is:

- mean elevation of the land in the area of the spillway per se, including the energy dissipator, El_{te}, in m;
- mean elevation of the land in the chute area, exclusively, El_{tc}, in m;
- mean elevation of the land in the stilling basin or ski jump area, exclusively, El_{tde}, in m;
- mean thickness of the layer of soil in the area of the spillway per se, e_{te}, in m;
- mean elevation of the land at section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the approach channel, El_{tai}, in m;
- mean elevation of the land at section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the downstream channel, El_{tri}, in m;
- elevation of the bottom of the downstream channel, El_{cr}, in m;
- mean length of the approach channel, L_{ca}, in m;
- mean length of the downstream channel, L_{cr}, in m; and
- thickness of the concrete lining for the stilling basin sill, e_c, in m, when applicable.

Considerations and recommendations

This text relates to spillways with a typical cross-section as shown in Fig. 5.7.5.06 and Fig. 5.7.5.07.

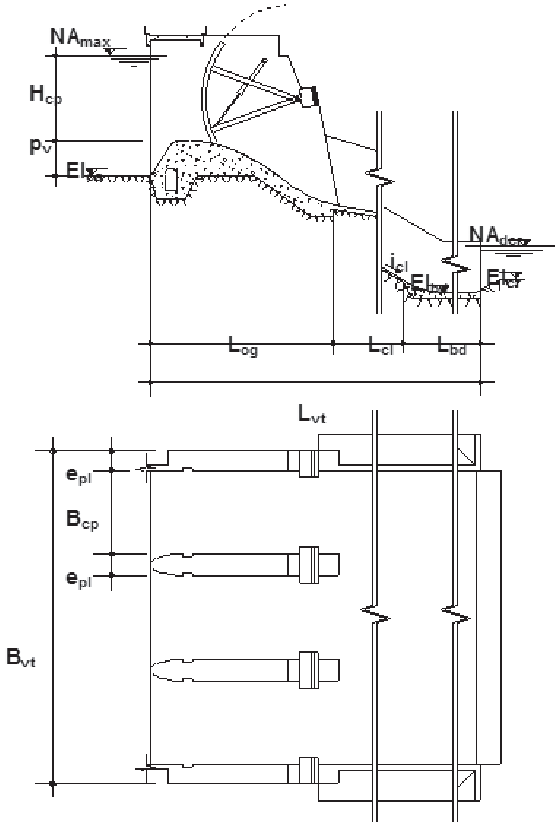


Fig. 5.7.5.06 – Typical cross-section of a gated surface abutment spillway with a stilling basin.

An initial estimate of the **height of the gates**, H_{cp} (m), is given by:

$$H_{cp} = k_v \times Q_v^{0.4} \leq 21.0m$$

where:

k_v	For spillways with
0.65	two gates
0.55	three gates
0.45	five gates
0.35	ten gates

where:

k_v	coefficient for determining the initial height of the gates; and
Q_v	design flow through the spillway, in m³/s.

Intermediate values can be used for k_v.

In the absence of more accurate information, 10% is the recommended **slope of the chute**.

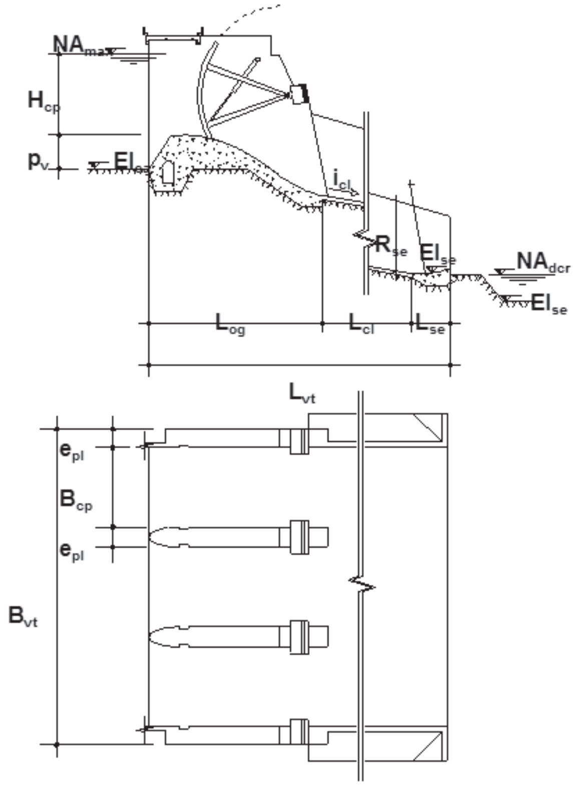


Fig. 5.7.5.07 – Typical cross-section of a gated surface abutment spillway with a ski jump.

When **dimensioning the stilling basin**, the recommended range for the Froude number before the hydraulic jump is betwen 4.5 and 9.0, in order to ensure its stability. One way of raising the Froude number to 4.5 would be to lower the bottom of the stilling basin beyond the result given by the calculation. Likewise, one way to reduce the Froude number to 9.0 would be to reduce the width of the stilling basin.

In the absence of more accurate information, use 1.0 m as the **thickness of the concrete lining for the bottom of the stilling basin**.

Discharge Coefficient

The **discharge coefficient**, C_d, for the typical cross-section recommended, can be obtained from Graph 5.7.5.03 (Bureau of Reclamation 1977) or from the equivalent expressions:

for m_m=1.0 and 0.100 < z ≤ 0.524:

$$C_d = 1.9507 \times z^3 - 2.9011 \times z^2 + 1.5498 \times z + 1.8274$$

for m_m=1.0 and 0.524 < z ≤ 0.813:

$$C_d = 0.1592 \times z^3 - 0.4409 \times z^2 + 0.4248 \times z + 1.9984$$

for m_m=1.0 and 0.813 < z ≤ 1.800:

$$C_d = 0.0159 \times z + 2.1256$$

for m_m=0.67 and 0.100 < z ≤ 0.497:

$$C_d = 2.5495 \times z^3 - 3.6032 \times z^2 + 1.8832 \times z + 1.7678$$

m_m=0.67 and 0.497 < z ≤ 0.759:

$$C_d = 0.2261 \times z^3 - 0.6256 \times z^2 + 0.6137 \times z + 1.9481$$

m_m=0.67 and 0.759 < z ≤ 1.800:

$$C_d = 0.0242 \times z^3 - 0.1143 \times z^2 + 0.1775 \times z + 2.0734$$

m_m=0.33 and 0.100 < z ≤ 0.505:

$$C_d = 2.4283 \times z^3 - 3.5181 \times z^2 + 1.9125 \times z + 1.7265$$

m_m=0.33 and 0.505 < z ≤ 0.755:

$$C_d = 0.2514 \times z^3 - 0.6927 \times z^2 + 0.6896 \times z + 1.9033$$

m_m=0.33 and 0.755 < z ≤ 1.800:

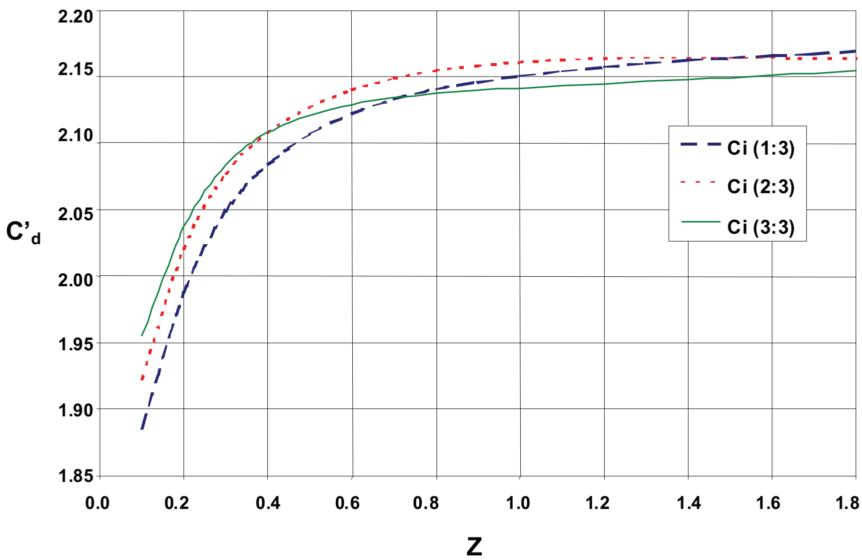
$$C_d = 0.02 \times z^3 - 0.0985 \times z^2 + 0.1782 \times z + 2.0508$$

where:

$$z = \frac{p_v}{H_{cp}} \qquad p_v = NA_{max} - H_{cp} - El_{ca}$$

where:

z	adimensional parameter;
p_v	difference in height between the ogee crest and the bottom of the approach channel, in m;
H_{cp}	height of the gates, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
El_{ca}	elevation of the bottom of the approach channel, in m.



Graph 5.7.5.03 – Discharge coefficient for spillways with an inclined upstream face.

Dimensions of the gates and spillway

The **real width of the openings**, B_{rvt} (m), is given by:

$$B_{rvt} = \frac{Q_v}{C_d \times H_{cp}^{3/2}}$$

where:

Q_v	design flood through the spillway, in m³/s;
C_d	discharge coefficient; and
H_{cp}	height of the gates, in m.

The **number of gates**, N_{cp}, is given by:

$$N_{cp} = \text{int}\left(\frac{B_{rvt}}{H_{cp}} + 0{,}999\right)$$

where:

B_{rvt}	real width of the openings, in m;
H_{cp}	height of the gates, in m; and
int(x)	function that returns the integer part of x.

The **width of the gates**, B_{cp} (m), is given by:

$$B_{cp} = 0.05 \times \text{int}\left(\frac{1}{0.05} \times \frac{B_{rvt}}{N_{cp}} + 0.5\right)$$

where:

B_{rvt}	real width of the openings, in m;
N_{cp}	number of gates; and
int(x)	function that returns the integer part of x.

The **thickness of the pillars**, e_{pl} (m), is given by:

$$e_{pl} = 0.12 \times H_{cp} + 2.4$$

where:

H_{cp}	height of the gates, in m.

The **total width of the spillway**, B_{vt} (m), perpendicular to flow, is given by:

$$B_{vt} = (N_{cp} + 1) \times e_{pl} + N_{cp} \times B_{cp}$$

where:

N_{cp}	number of gates;
e_{pl}	thickness of the pillars, in m; and
B_{cp}	width of the gates, in m.

The **width of the chute**, B_{cl} (m), is given by:

$$B_{cl} = (N_{cp} - 1) \times e_{pl} + N_{cp} \times B_{cp}$$

where:

N_{cp}	number of gates;
e_{pl}	thickness of the pillars, in m; and
B_{cp}	width of the gates, in m.

The **length of the ogee crest** in the direction of flow, L_{og} (m), is given by:

$$L_{og} = 1.66 \times H_{cp} + m_m \times p_v$$

where:

H_{cp}	height of the gates, in m;
m_m	slope of the upstream face of the ogee crest, horizontal distance for a 1.0 m difference in level; and
p_v	difference in height between the ogee crest and the bottom of the approach channel, in m.

The **length of the chute**, L_{cl} (m), is given by:

for the stilling basin: $L_{cl} = \dfrac{NA_{max} - 1.69 \times H_{cp} - El_{bd}}{i_{cl}}$

for the ski jump: $L_{cl} = \dfrac{NA_{max} - 1.69 \times H_{cp} - El_{se} - 0.03 \times R_{se}}{i_{cl}}$

where:

NA_{max}	maximum normal water level in the reservoir, in m;
H_{cp}	height of the gates, in m;
El_{bd}	elevation of the bottom of the stilling basin, defined below, in m;
El_{se}	elevation of the ski jump sill, defined below, in m;
R_{se}	radius of curvature of the ski jump, defined below, in m; and
i_{cl}	slope of the chute.

The **total length** of the spillway, L_{vt} (m), is given by:

for the stilling basin: $L_{vt} = L_{og} + L_{cl} + L_{bd}$

for the ski jump: $L_{vt} = L_{og} + L_{cl} + L_{se}$

where:

L_{og}	length of the ogee crest, in m;
L_{cl}	length of the chute, in m;
L_{bd}	length of the stilling basin, defined below, in m; and
L_{se}	length of the ski jump at the foundations, defined below, in m.

Stilling Basin

The **width of the stilling basin**, B_{bd} (m), is given by:

$B_{bd} = B_{cl}$

where:

B_{cl}	width of the chute, in m.

The **stilling basin depth** is determined iteratively, based on the Froude number before the hydraulic jump, in section one of Fig. 5.7.5.03, for the 100-year flood flow .

An elevation is picked for the bottom of the stilling basin and its suitability is checked by calculating the **velocity**, v_1 (m/s), the **depth of discharge**, y_1 (m), and the **Froude number**, Fr_1, before the hydraulic jump, **depth of discharge** after the hydraulic jump, y_2 (m), and finally the elevation of the **bottom of the stilling basin**.

Should the elevation of the bottom of the stilling basin be different from the figure first picked, the calculations should be redone until the level of precision required is reached.

$$v_1 = \sqrt{k \times 2 \times g \times (NA_{max} - El_{bd})}$$

$$y_1 = \frac{Q_c}{B_{bd} \times v_1} \qquad Fr_1 = \frac{v_1}{\sqrt{g \times y_1}}$$

$$y_2 = \left(\frac{y_1}{2}\right) \times \left(\sqrt{1 + 8 \times Fr_1^2} - 1\right)$$

$$El_{bd} = NA_{ccr} - y_2$$

where:

v_1	mean velocity of discharge before the hydraulic jump, in m/s;
k	0.9 – reduction coefficient for the energy head;

g	9.81 m/s² – acceleration due to gravity;
NA_{max}	maximum normal water level in the reservoir, in m;
El_{bd}	elevation of the bottom of the stilling basin, in m;
y_1	depth of discharge before the hydraulic jump, in m;
Q_c	100-year flood flow, in m³/s;
B_{bd}	width of the stilling basin, in m;
Fr_1	Froude number before the hydraulic jump;
y_2	depth of discharge after the hydraulic jump, in m; and
NA_{ccr}	water level in the downstream channel for a 100-year flood, in m.

The **radius of curvature at the stilling basin inlet**, R_{bd} (m), is given by (PETERKA):

$$R_{bd} = 6 \text{ x } y_1$$

where:

y_1	depth of discharge before the hydraulic jump, in m.

The **length of the stilling basin**, L_{bd} (m), is given by:

$$L_{bd} = 6 \times y_2 + R_{bd} \times \tan\left[\frac{a\tan(i_{cl})}{2}\right]$$

where:

y_2	depth of discharge after the hydraulic jump, in m;
i_{cl}	slope of the chute; and
R_{bd}	radius of curvature at the stilling basin inlet, in m.

Ski Jump

The **width of the ski jump**, B_{se} (m), is given by:

$$B_{se} = B_{cl}$$

where:

B_{cl}	width of the chute, in m.

The **elevation of the ski jump sill**, El_{se}, is given by:

$$El_{se} = NA_{ccr} + 1.0 \geq El_{cr}$$

where:

NA_{ccr}	water level in the downstream channel for a 100-year flood, in m;
El_{cr}	elevation of the bottom of the downstream channel, in m.

The **radius of curvature of the ski jump**, R_{se} (m), is given by (PETERKA):

$$R_{se} = 6 \text{ x } y$$

where:

$$y = \frac{Q_c}{B_{se} \times v}$$

$$v = \sqrt{k \times 2 \times g \times (NA_{max} - El_{se})}$$

where:

y	depth of the water column at the ski jump, in m;
Q_c	100-year flood flow, in m³/s;
B_{se}	width of the ski jump, in m;

v	velocity of the water column at the ski jump, in m/s;
k	0.9 – reduction coefficient for the energy head;
g	9.81 m/s² – acceleration due to gravity;
NA_{max}	maximum normal water level in the reservoir, in m; and
El_{se}	elevation of the ski jump sill, in m.

The **length of the ski jump**, L_{se} (m), is given by:

$$L_{se} = 0.80 \times R_{se} + 1.5$$

where:

R_{se}	radius of curvature of the ski jump, in m.

Common Excavation (account .12.19.30.12.10)

The **volume of common excavation** for the spillway, V_{tvt} (m³), is given by:

$$V_{tvt} = V_{tca} + V_{tes} + V_{tcr}$$

where:

$$V_{tca} = \left(\frac{V_{ta0}}{2} + V_{ta1} + V_{ta2} \right) \times \frac{L_{ca}}{3}$$

$$V_{tai} = \left[B_{ca} - 6 + 2 \times (0.6 \times h_{rai} + e_{te}) \right] \times e_{te}$$

$$h_{rai} = El_{tai} - El_{ca} - e_{te}, \ i = 0,\ 1,\ 2$$

$$B_{ca} = B_{vt} - 2 \times (e_{pl} - 1.0)$$

$$V_{tes} = L_{vt} \times e_{te} \times B_{vt}$$

$$V_{tcr} = \left(\frac{V_{tr0}}{2} + V_{tr1} + V_{tr2} \right) \times \frac{L_{cr}}{3}$$

$$V_{tri} = \left[B_{cr} - 6 + 2 \times (0.6 \times h_{rri} + e_{te}) \right] \times e_{te}$$

$$h_{rri} = El_{tri} - El_{cr} - e_{te}, \ i = 0,\ 1,\ 2$$

for the stilling basin: $B_{cr} = B_{bd} + 2 \times 1.0$

for the ski jump: $B_{cr} = B_{se} + 2 \times 1.0$

where:

V_{tca}	volume of common excavation for the approach channel, in m³;
V_{tes}	volume of common excavation for the structure, in m³;
V_{tcr}	volume of common excavation for the downstream channel, in m³;
V_{tai}	volume of common excavation per meter at section i of the approach channel, in m³/m;
L_{ca}	length of the approach channel, in m;
B_{ca}	width of the bottom of the approach channel, in m;
h_{rai}	depth of excavation in rock at section i of the approach channel, in m;
e_{te}	mean thickness of the layer of soil in the area of the spillway per se, in m;
El_{tai}	mean elevation of the land at section i of the approach channel, in m;
El_{ca}	elevation of the bottom of the approach channel, in m;
B_{vt}	total width of the spillway, in m;
e_{pl}	thickness of the pillars, in m;
L_{vt}	total length of the spillway, in m;

V_{tri}	volume of common excavation per meter at section i for the downstream channel, in m^3/m;
L_{cr}	length of the downstream channel, in m;
B_{cr}	width of the bottom of the downstream channel, in m;
h_{rri}	depth of excavation in rock at section i of the downstream channel, in m;
El_{tri}	mean elevation of the land at section i perpendicular to the longitudinal axis of the downstream channel, in m;
El_{cr}	elevation of the bottom of the downstream channel, in m;
B_{bd}	width of the stilling basin, in m; and
B_{se}	width of the ski jump, in m.

The unit price of **common excavation** is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.19.30.12.11)

Figure 5.7.5.05 shows typical cross-sections for the excavation of approach and downstream channels.

The **volume excavated in rock**, V_{rvt} (m^3), is given by:

$$V_{rvt} = V_{rca} + V_{rog} + V_{rcl} + V_{rbc} + V_{rde} + V_{rbe} + V_{rcr}$$

where:

$$V_{rca} = \left(\frac{V_{ra0}}{2} + V_{ra1} + V_{ra2} \right) \times \frac{L_{ca}}{3}$$

$$V_{rai} = (B_{ca} - 6 + 0.6 \times h_{rai}) \times h_{rai}$$

$$V_{rog} = L_{og} \times \left[El_{te} - e_{te} - (El_{ca} - 2) \right] \times B_{vt}$$

$$V_{rcl} = L_{cl} \times \left[El_{tc} - e_{te} - (El_{tc} - 0.7) \right] \times (B_{cl} + 2)$$

$$V_{rbc} = 2 \times L_{cl} \times 0.3 \times h_{rc}^2$$

$$V_{rcr} = \left(\frac{V_{rr0}}{2} + V_{rr1} + V_{rr2} \right) \times \frac{L_{cr}}{3}$$

$$V_{rri} = (B_{cr} - 6 + 0.6 \times h_{rri}) \times h_{rri}$$

$$h_{rc} = El_{tc} - e_{te} - (El_{cm} - 0.7) \geq 0$$

for the stilling basin:

$$V_{rde} = L_{bd} \times \left[El_{te} - e_{te} - (El_{bd} - e_c) \right] \times (B_{bd} + 2)$$

$$V_{rbe} = 2 \times L_{bd} \times 0.3 \times h_{re}^2$$

$$El_{cm} = \frac{NA_{max} - 1.69 \times H_{cp} + El_{bd}}{2}$$

$$h_{re} = El_{tde} - e_{te} - (NA_{ccr} - 5.0) \geq 0$$

for the ski jump:

$$V_{rde} = L_{se} \times [El_{te} - e_{te} - (El_{se} - 2)] \times (B_{se} + 2)$$

$$V_{rbe} = 2 \times L_{se} \times 0.3 \times h_{rs}^2$$

$$El_{cm} = \frac{NA_{max} - 1.69 \times H_{cp} + El_{se}}{2}$$

$$h_{rs} = El_{tde} - e_{te} - (El_{se} - 2.0) \geq 0$$

where:

V_{rca}	volume of rock excavated for the approach channel, in m^3;
V_{rog}	volume of rock excavated in the ogee crest area, in m^3;
V_{rcl}	volume of rock excavated in the chute area, in m^3;
V_{rbc}	volume of rock excavated for berms in the chute section, in m^3;
V_{rde}	volume of rock excavated in the area of the stilling basin or ski jump, in m^3;
V_{rbe}	volume of rock excavated for berms in the stilling basin or ski jump section, in m^3;
V_{rcr}	volume of rock excavated for the downstream channel, in m^3;
V_{rai}	volume of excavated rock per meter at section i of the approach channel, in m^3/m;
L_{ca}	length of the approach channel, in m;
B_{ca}	width of the bottom of the approach channel, in m;
h_{rai}	depth of excavation in rock at section i of the approach channel, in m;
L_{og}	length of the ogee crest, in m;
El_{te}	mean elevation of the land in the area of the spillway per se, in m;
e_{te}	mean thickness of the layer of soil in the area of the spillway per se, in m;
El_{ca}	elevation of the bottom of the approach channel, in m;
B_{vt}	total width of the spillway, in m;
L_{cl}	length of the chute, in m;
El_{cm}	mean elevation of the chute, in m;
B_{cl}	width of the chute, in m;
h_{rc}	depth of excavation in rock in the chute area, in m;
V_{rri}	volume of excavated rock per meter at section i for the downstream channel, in m^3/m;
L_{cr}	length of the downstream channel, in m;
B_{cr}	width of the bottom of the downstream channel, in m;
h_{rri}	depth of excavation in rock at section i of the downstream channel, in m;
El_{tc}	mean elevation of the land in the chute area, exclusively, in m;
L_{bd}	length of the stilling basin, in m;
El_{bd}	elevation of the bottom of the stilling basin, in m;
e_c	Thickness of the concrete lining for the stilling basin sill, in m;
B_{bd}	width of the stilling basin, in m;
h_{re}	depth of excavation in rock in the stiling basin area, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
H_{cp}	height of the gates, in m;
El_{tde}	mean elevation of the land in the stilling basin or ski jump area, exclusively, in m;
NA_{ccr}	water level in the downstream channel for a 100-year flood, in m;

L_{se}	length of the ski jump, in m;
El_{se}	elevation of the ski jump sill, in m;
B_{se}	width of the ski jump, in m; and
h_{rs}	depth of excavation in rock in the ski jump area, in m.

The unit price of **excavation in rock** is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.19.30.13)

The area of foundation to be cleaned, A_{lf} (m^2), is given by:

$$A_{lf} = B_{vt} \times L_{vt}$$

where:

B_{vt}	total width of the spillway, in m; and
L_{vt}	total length of the spillway, in m.

The length of the **grout holes** and the **drainage line**, L_{tf} (m), is given by:

$$L_{tf} = \frac{B_{vt}}{3.0} \times L_{1tf}$$

$$L_{1tf} = 1.5 \times \left(NA_{max} - El_{ca}\right) \leq 40\text{m}$$

where:

B_{vt}	total width of the spillway, in m;
L_{1tf}	length of one grout hole, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
El_{ca}	elevation of the bottom of the approach channel, in m;
3.0	space between the grout holes, in m.

The **total length of the rock anchors** in the stilling basin, L_{tfc}(m), when necessary, is given by:

$$L_{tfc} = B_{bd} \text{ x } L_{bd}$$

where:

B_{bd}	width of the stilling basin, in m; and
L_{bd}	length of the stilling basin, in m.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m^2
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.19.30.14)

The **volume of concrete** for the spillway, V_{cvt} (m^3), is given by:

$$V_{cvt} = V_{cog} + V_{cpl} + V_{cpo} + V_{ccl} + V_{cde} + V_{cmv}$$

where:

$$V_{cog} = (0.165 \times H_{cp}^2 + 0.67 \times p_v \times H_{cp} + 0.84 \times p_v^2 + 32) \times B_{vt}$$

$$V_{cpl} = (N_{cp} + 1) \times e_{pl} \times (1.85 \times H_{cp}^2 + 7.1 \times H_{cp} + 15)$$

$$V_{cpo} = 6.0 \times B_{vt}$$

$$V_{ccl} = L_{cl} \times [0.7 \times B_{cl} + 2 \times (H_{cl} + 0.7) \times 1.0]$$

$$H_{cl} = 0.95 \times H_{cp}$$

for the stilling basin:

$$V_{cde} = L_{bd} \times e_c \times (B_{bd} + 2.0)$$

$$V_{cmv} = 2 \times \left[L_{bd} \times (2.0 + y_2 + e_c) + \frac{d_1^2}{2 \times i_{cl}} \right] \times 1.0$$

$$d_1 = 2.0 + y_2 - H_{cl}$$

for the ski jump:

$$V_{cde} = (0.12 \times R_{se}^2 + 0.93 \times R_{se} + 0.53) \times (B_{se} + 2.0)$$

$$V_{cmv} = 2 \times L_{se} \times H_{cl} \times 1.0$$

where:

V_{cog}	volume of concrete for the ogee crest, in m^3;
V_{cpl}	volume of concrete for the pillars, in m^3;
V_{cpo}	volume of concrete for the bridge, in m^3;
V_{ccl}	volume of concrete for the chute, including walls, in m^3;
V_{cde}	volume of concrete for the stilling basin or ski jump, including walls, in m^3;
H_{cp}	height of the gates, in m;
p_v	Difference in height between the ogee crest and the bottom of the approach channel, in m;
B_{vt}	total width of the spillway, in m;
N_{cp}	number of gates;
e_{pl}	Thickness of the pillars, in m;
L_{cl}	length of the chute, in m;
B_{cl}	width of the chute, in m;

H_{cl}	height of the wall of the chute, in m;
L_{bd}	length of the stilling basin, in m;
e_c	Thickness of the concrete lining for the stilling basin sill, in m;
B_{bd}	width of the stilling basin, in m;
y_2	depth of discharge after the hydraulic jump, in m;
i_{cl}	slope of the chute;
R_{se}	radius of curvature of the ski jump, in m;
B_{se}	Width of the ski jump, in m;
L_{se}	length of the ski jump, in m; and
d_i	secondary dimension, in m.

The amounts of cement and reinforcement steel are:

	cement (kg/m³)	reinforcement steel (kg/m³)
ogee crest with stilling basin	200	20
ogee crest with ski jump	250	50
pillars, chute, stilling basin and walls	270	80
pillars, chute, ski jump and walls	250	80
bridge	300	100

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of spillway (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- ogee crest: 113.00/m³
- pillars, chute, stilling basin, ski jump and walls: 200.00/m³
- bridge: 474.00/m³

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Tainter Gates (account .12.18.28.23.16)

The **acquisition cost of each surface tainter gate** for the spillway, C_{cp} (R$), – FOB cost excluding tranportation and insurance, assembly and testing and provisions for taxes payable, depending on the current tax regime – can be obtained from the expression below (or from Graph B.21, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $2.2 \leq z \leq 178$: $C_{cp} = 193.95 \times z^{0.5406}$

where:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000} \qquad\qquad H_x = H_{cp}$$

where:

z	parameter, in m^4;
B_{cp}	width of the gates, in m;
H_{cp}	height of the gates, in m; and
H_x	maximum hydrostatic load on the gate sill, in m.

The following percentages must be added to the FOB price:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for taxes and charges payable on the equipment.

Stoplogs (account .12.18.28.23.17)

The **acquisition cost of each stoplog** for the spillway, C_{sl} (R$),– FOB cost – can be obtained from the expression below (or from Graph B.24, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.34 \le z \le 177$: $C_{sl} = 72.9 \times z^{0.716}$

where: $z = \dfrac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$

where:

z	parameter, in m^4;
B_{cp}	width of the gates, in m;
H_{cp}	height of the gates, in m; and
H_x	maximum hydrostatic load on the gate sill, in m.

It is recommended that one stoplog be used for every ten tainter gates.

The cost of transportation and insurance, assembly and testing, taxes and charges payable on the equipment must be added to the FOB cost.

The **total acquisition cost of fixed parts and parts embedded in the concrete** for the spillway's stoplogs, C_{gpf} (R$), – FOB cost – is given by the expression below. The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

$$C_{gpf} = N_{cp} \times \left[2 \times \left(H_{cp} + H_{bl}\right) + B_{cp}\right] \times 2084.80$$

where:

N_{cp}	number of gates;
H_{cp}	height of the gates, in m;
H_{bl}	height of the spillway freeboard, in m; and
B_{cp}	width of the gates, in m.

The cost of transportation and insurance, assembly and testing, taxes and charges payable on the equipment must be added to the FOB cost.

Crane (account .12.18.28.23.20)

The **acquisition cost of the gantry crane** for the spillway, C_{pcr} (R$), – FOB cost – can be obtained from the expressions below (or from Graph B.26, annex B, as a function of the dimensions of the stoplog and the maximum hydrostatic load on its sill). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

for $0.34 \leq z \leq 13.15$: $C_{pcr} = 141.34 \times z^{0.3555}$

for $13.15 < z \leq 176.43$: $C_{pcr} = -0.0082 \times z^2 + 6.8982 \times z + 263.93$

where: $z = \dfrac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$

where:

z	parameter, in m^4;
B_{cp}	width of the gates, in m;
H_{cp}	height of the gates, in m; and
H_x	maximum hydrostatic load on the gate sill, in m.

The cost of transportation and insurance, assembly and testing, taxes and charges payable on the equipment must be added to the FOB cost.

Other Costs (account .12.18.28.17)

Other costs are estimated as accounting for 2% of the total cost of the corresponding structure.

UNGATED SURFACE SPILLWAYS WITH A HIGH OGEE CREST (ACCOUNT .12.18.28)

Basic Data

The main **information required for dimensioning** purposes can be obtained from the overall layout and from item 5.1.2. (Hydrometeorological Data), as follows:

- design flood through the spillway, Q_v in m^3/s, from item 5.1.2.;
- 100-year flood flow, Q_c in m^3/s, from item 5.1.2.;
- maximum water level of the reservoir under design flood conditions, NA_{xmx}, from item 5.1.2.;
- maximum normal water level in the reservoir, NA_{max}, from item 4.6.;
- elevation of the bottom of the approach channel to the sluiceways, El_{ca}, from item 5.7.3., when applicable;
- elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, El_{cv};
- maximum water level in the downstream channel, NA_{xcr}, from item 5.1.2.;
- water level in the downstream channel for a 100-year flood, NA_{ccr}, from item 5.1.2.;
- elevation of the bottom of the downstream channel, El_{cr};
- number of gaps in the bridge, when applicable; and
- height of sluiceways, H_{ad} in m, from item 5.7.3., when applicable.

The **main information required for quantification** purposes is:

- mean elevation of the land in the area of the spillway per se, including the energy dissipator, El_{te}, in m;
- mean elevation of the land in the stilling basin or ski jump area, exclusively, El_{tde}, in m;
- mean thickness of the layer of soil in the area of the spillway per se, e_{te}, in m;
- mean elevation of the land at section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the approach channel, El_{tai}, in m;
- mean elevation of the land at section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the downstream channel, El_{tri}, in m;
- mean length of the approach channel at the part with no sluiceways, L_{ca}, in m;
- mean length of the approach channel in the part with sluiceways, L_{cad}, in m, when applicable;
- mean length of the downstream channel, L_{cr}, in m;
- thickness of the concrete lining for the stilling basin sill, e_c, in m, when applicable;
- width of one sluiceway, B_{1ad} in m, from item 5.7.3., when applicable;
- total width of sluiceways, B_{ad} in m, from item 5.7.3., when applicable;
- number of sluiceways, N_{ad}, from item 5.7.3., when applicable;
- thickness of the sluiceway walls, e_{pl}, in m, from item 5.7.3., when applicable;
- existence of a bridge.

Considerations and recommendations

This text relates to spillways with a typical cross-section, as shown in Fig. 5.7.5.08 and 5.7.5.09.

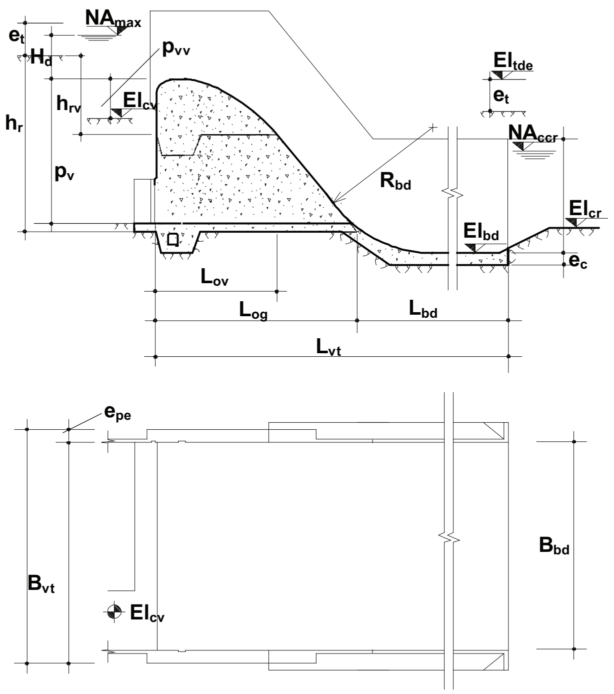


Fig. 5.7.5.08 – Typical cross-section and plan of an ungated surface spillway with a high ogee crest and a stilling basin.

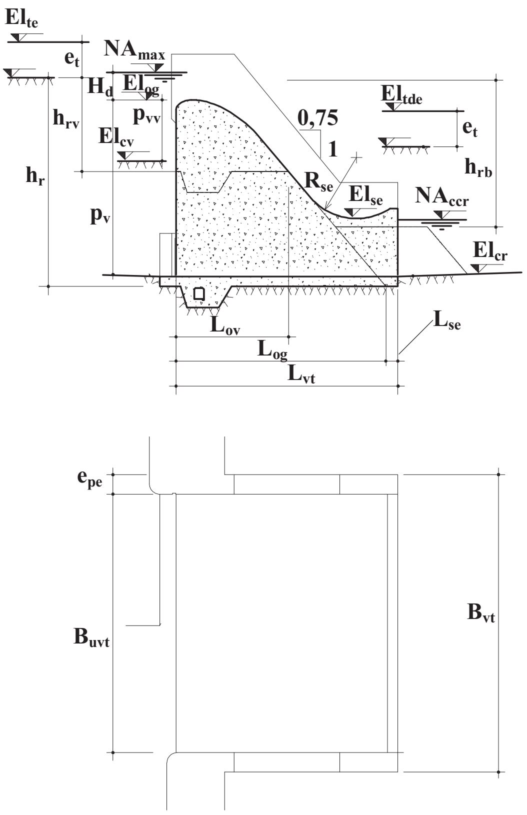


Fig. 5.7.5.09 – Typical cross-section and plan of an ungated surface spillway with a high ogee crest and a ski jump.

When the spillway is not all on the river bed, the **elevation of the foundations** at the two ends of the structure are often different, in which case an mean of these elevations should be used for the elevation of the bottom of the approach channel to the spillway, excluding the sluiceways.

The following should be true for **spillways with no sluiceways**: $El_{ca} = El_{cv}$

where:

El_{ca}	elevation of the bottom of the approach channel to the sluiceways, in m; and
El_{cv}	elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m.

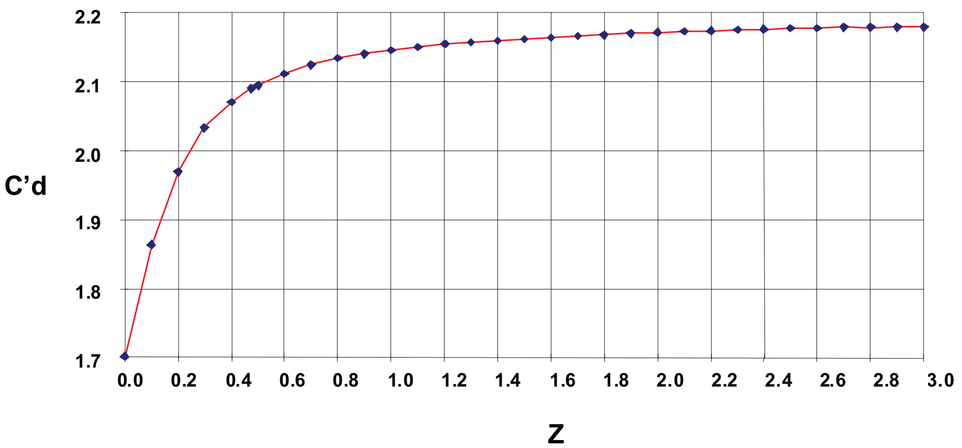
The thickness of the **layer of soil** on the river bed may be different from the thickness on the abutments, and there may often be no soil at all.

When **dimensioning the stilling basin**, the recommended range for the Froude number before the hydraulic jump is betwen 4.5 and 9.0, in order to ensure its stability. One way of raising the Froude number to 4.5 would be to lower the bottom of the stilling basin beyond the result given by the calculation. Likewise, one way to reduce the Froude number to 9.0 would be to reduce the width of the stilling basin.

In the absence of more accurate information, use 1.0 m as the **thickness of the concrete lining for the bottom of the stilling basin**.

Discharge Coefficient

The **initial discharge coefficient**, C'_d, for the recommended typical cross-section, can be obtained from Graph 5.7.5.01 or from the equivalent expressions (Bureau of Reclamation, 1977):



Graph 5.7.5.01 – Initial Discharge Coefficient.

for $z \leq 0.475$: $C'_d = 2.535 \times z^3 - 3.61 \times z^2 + 1.96 \times z + 1.702$

for $0.475 < z \leq 1.2$: $C'_d = 0.145 \times z^3 - 0.475 \times z^2 + 0.559 \times z + 1.916$

for $1.2 < z \leq 3.0$: $C'_d = -0.0072 \times z^2 + 0.0442 \times z + 2.112$

for $z > 3.0$: $C'_d = 2.18$

where:

$$z = \frac{0.7 \times p_v + 0.3 \times p_{vv}}{H_d}$$

$p_v = NA_{max} - El_{ca}$ $\qquad$ $p_{vv} = NA_{max} - El_{cv}$

$H_d = NA_{xmx} - NA_{max}$

where:

z	adimensional parameter;
p_v	vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m;
p_{vv}	vertical distance between the ogee sill and the bottom of the spillway approach channel, sluiceways excluded, in m;
H_d	maximum energy head on the spillway crest, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
El_{ca}	elevation of the bottom of the approach channel to the sluiceways, in m;
El_{cv}	elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m; and
NA_{xmx}	maximum water level of the reservoir under design flood conditions, in m.

The **reduction coefficient for the discharge coefficient when the outlet is submerged**, k_c, can be obtained from Graph 5.7.5.02 (Bureau of Reclamation, 1977) or from the equivalent expressions:

for Section I: $-4 \times u + 7 \times w + 2.6 \geq 0$:

$$k_c = -0.952 \times \left(\frac{1}{u}\right)^2 + 0.956 \times \left(\frac{1}{u}\right) + 0.767 \leq 1$$

for Section II $u < 3.6\ e - 4 \times u + 7 \times w + 2.6 < 0$:

$$k_c = 1.058 - \frac{4 \times (u + 5)}{860 \times w} \leq 1$$

for Section III $u \geq 3.6$:

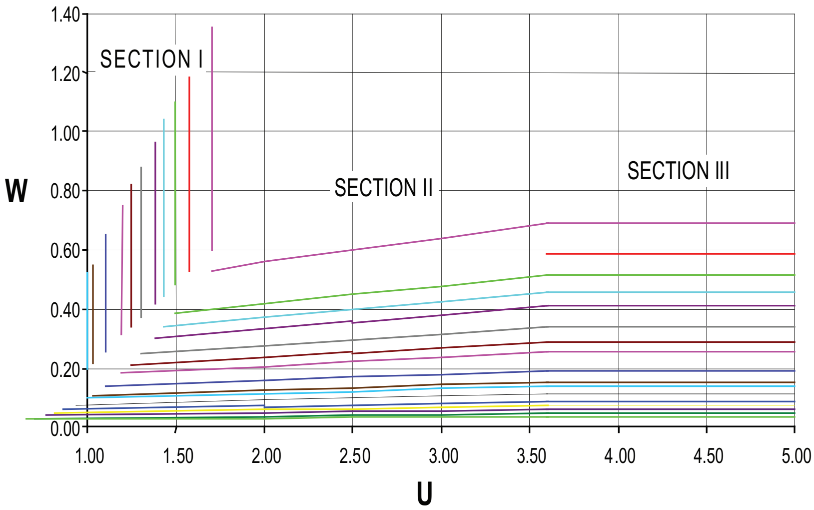
$$k_c = 1.058 - \frac{4}{100 \times w} \leq 1$$

where:

$$u = \frac{NA_{xmx} - El_{cr}}{H_d} \qquad w = \frac{NA_{xmx} - NA_{xcr}}{H_d}$$

where:

u, w	adimensional parameters;
NA_{xmx}	maximum water level of the reservoir under design flood conditions, in m;
El_{ca}	elevation of the bottom of the approach channel, in m;
H_d	maximum energy head on the spillway crest, in m; and
NA_{xcr}	maximum water level in the downstream channel, in m.



Graph 5.7.5.02 – Reduction coefficient for the discharge coefficient when the outlet is submerged.

The **corrected discharge coefficient**, C_d, is given by:

$$C_d = k_c \times C'_d$$

where:

k_c	reduction coefficient for the discharge coefficient; and
C'_d	initial discharge coefficient.

Spillway Dimensions

The **useful width** of the spillway perpendicular to flow, B_{uvt} (m), is given by:

$$B_{uvt} = \frac{Q_v}{C_d \times H_d^{3/2}}$$

where:

Q_v	design flood through the spillway, in m³/s;
C_d	discharge coefficient; and
H_d	maximum energy head on the spillway crest, in m.

The **thickness of the end pillars**, e_{pe} (m), is given by:

$$e_{pe} = 0.12 \times H_d + 2.4$$

where:

H_d	maximum energy head on the spillway crest, in m.

The **total width of the spillway**, B_{vt} (m), perpendicular to flow, is given by:

$$B_{vt} = 0.05 \times \text{int}\left[(B_{uvt} + 2 \times e_{pe}) \times \frac{1}{0.05} + 0.5\right]$$

where:

$N_{vãos}$	number of openings in the spillway;
2.0	thickness of the intermediate pillars, in m;
e_{pe}	thickness of the end pillars, in m;
$B_{vãos}$	width of an opening, in m.

The length of the ogee crest in the direction of flow, L_{og} (m), is given by:

for the part with no sluiceways: $L_{ov} = 1.46 \times H_d^{0.46} \times (p_{vv} + 1.5)^{0.54} + 0.27 \times H_d$

for the part with sluiceways: $L_{og} = 1.46 \times H_d^{0.46} \times (p_v + 1.5)^{0.54} + 0.27 \times H_d$

where:

H_d	maximum energy head on the spillway crest, in m; and
p_v	difference in height between the ogee crest and the bottom of the approach channel, in m.

The **total length** of the spillway, L_{vt} (m), is given by:

for spillways with a stilling basin and sluiceways: $L_{vt} = L_{og} + L_{bd}$

for spillways with a stilling basin and no sluiceways: $L_{vt} = L_{ov} + L_{bd}$

for spillways with a ski jump and sluiceways: $L_{vt} = L_{og} + L_{se}$

for spillways with a ski jump and no sluiceways: $L_{vt} = L_{ov} + L_{se}$

where:

L_{og}	length of the ogee crest with sluiceways, in m;
L_{ov}	length of the ogee crest with no sluiceways, in m;
L_{bd}	length of the stilling basin, defined below, in m; and
L_{se}	length of the ski jump at the foundations, defined below, in m.

Stilling Basin

The **width of the stilling basin**, B_{bd} (m), is given by:

$$B_{bd} = B_{uvt}$$

where:

B_{uvt}	useful width of the spillway, in m.

The **stilling basin depth** is determined iteratively, based on the Froude number before the hydraulic jump, in section one of Fig. 5.7.5.03, for the 100-year flood flow .

An elevation is picked for the bottom of the stilling basin and its suitability is checked by calculating the **velocity**, v_1 (m/s), the **depth of discharge**, y_1 (m), and the **Froude number**, Fr_1, before the hydraulic jump, **depth of discharge** after the hydraulic jump, y_2 (m), and finally the elevation of the **bottom of the stilling basin**.

Should the elevation of the bottom of the stilling basin be different from the figure first picked, the calculations should be redone until the level of precision required is reached.

$$v_1 = \sqrt{k \times 2 \times g \times (NA_{xmx} - El_{bd})}$$

$$y_1 = \frac{Q_c}{B_{bd} \times v_1}$$

$$Fr_1 = \frac{v_1}{\sqrt{g \times y_1}}$$

$$y_2 = \left(\frac{y_1}{2}\right) \times \left(\sqrt{1 + 8 \times Fr_1^2} - 1\right)$$

$$El_{bd} = NA_{ccr} - y_2$$

where:

v_1	mean velocity of discharge before the hydraulic jump, in m/s;
k	0.9 – reduction coefficient for the energy head;
g	9.81 m/s^2 – acceleration due to gravity;
NA_{xmx}	maximum water level of the reservoir under design flood conditions, in m;
El_{bd}	elevation of the bottom of the stilling basin, in m;
y_1	depth of discharge before the hydraulic jump, in m;
Q_c	100-year flood flow, in m^3/s;
B_{bd}	width of the stilling basin, in m;
Fr_1	Froude number before the hydraulic jump;
y_2	depth of discharge after the hydraulic jump, in m; and
NA_{ccr}	water level in the downstream channel for a 100-year flood, in m.

The **radius of curvature at the stilling basin inlet**, R_{bd} (m), is given by (PETERKA):

$$R_{bd} = 6 \times y_1$$

where:

y_1	depth of discharge before the hydraulic jump, in m.

The **length of the stilling basin**, L_{bd} (m), is given by:

$$L_{bd} = 6 \times y_2 + 0.75 \times (El_{ca} - El_{bd}) + 0.5 \times R_{bd} - 1.1$$

where:

y_2	depth of discharge after the hydraulic jump, in m;
El_{ca}	elevation of the bottom of the approach channel, in m;
El_{bd}	elevation of the bottom of the stilling basin, in m; and
R_{bd}	radius of curvature at the stilling basin inlet, in m.

Ski Jump

The **width of the ski jump**, B_{se} (m), is given by:

$$B_{se} = B_{uvt}$$

where:

B_{uvt}	useful width of the spillway, in m.

The **elevation of the ski jump sill**, El_{se}, is given by:

$$El_{se} = NA_{ccr} + 1.0 \geq El_{cr}$$

where:

NA_{ccr}	water level in the downstream channel for a 100-year flood, in m; and
El_{cr}	elevation of the bottom of the downstream channel, in m.

If the sluiceways are built in the body of the spillway, the **elevation of the sill of the ski jump**, El_{se}, should be higher than the top of the sluiceways, as shown in Fig. 5.7.5.04:

$$El_{se} \geq El_{ca} + 1.25 \times H_{ad}$$

where:

El_{ca}	elevation of the bottom of the approach channel, in m;
H_{ad}	height of sluiceways, in m.

The **radius of curvature of the ski jump**, R_{se} (m), is given by (PETERKA):

$$R_{se} = 6 \text{ x } y$$

where:

$$y = \frac{Q_c}{B_{se} \times v}$$

$$v = \sqrt{k \times 2 \times g \times (NA_{xmx} - El_{se})}$$

where:

y	depth of the water column at the ski jump, in m;
Q_c	100-year flood flow, in m^3/s;
B_{se}	width of the ski jump, in m;
v	velocity of the water column at the ski jump, in m/s;
k	0.9 – reduction coefficient for the energy head;
g	9.81 m/s^2 – acceleration due to gravity;
NA_{xmx}	maximum water level of the reservoir under design flood conditions, in m; and
El_{se}	elevation of the ski jump sill, in m;

The **length of the ski jump in the foundations**, L_{se} (m), is given by:

$$L_{se} = d_{16} + 1.286 \times R_{se} - d_{12} \quad \geq 0$$

where:

$$d_{16} = 1.46 \times H_d^{0.46} \times (p_v - h_5)^{0.54}$$

$$d_{12} = 1.46 \times H_d^{0.46} \times (p_v + 1.5)^{0.54}$$

$$h_5 = El_{se} + 0.6 \times R_{se} - El_{ca}$$

where:

R_{se}	radius of curvature of the ski jump, in m;
H_d	maximum energy head on the spillway crest, in m;
p_v	difference in height between the ogee crest and the bottom of the approach channel, in m;
El_{se}	elevation of the ski jump sill, in m;
El_{ca}	elevation of the bottom of the approach channel, in m; and
d_i, h_i	secondary dimensions, in m.

Common Excavation (account .12.19.30.12.10)

The **volume of common excavation**, V_{tvt} (m^3), is given by:

$$V_{tvt} = V_{tca} + V_{tes} + V_{tcr}$$

where:

$$V_{tca} = \left(\frac{V_{ta0}}{2} + V_{ta1} + V_{ta2} \right) \times \frac{L_{ca}}{3} + V_{tad}$$

$$V_{tes} = L_{vt} \times e_{te} \times B_{vt}$$

$$V_{tcr} = \left(\frac{V_{tr0}}{2} + V_{tr1} + V_{tr2} \right) \times \frac{L_{cr}}{3}$$

$$V_{tai} = \left[B_{ca} - 6 + 2 \times \left(0.6 \times h_{rai} + e_{te} \right) \right] \times e_{te}$$

$$h_{rai} = El_{tai} - El_{ca} - e_{te}, \; i = 0, 1, 2$$

$$B_{ca} = B_{vt} - 2 \times \left(e_{pe} - 1.0 \right)$$

$$V_{tad} = \left(L_{cad} - L_{ca} \right) \times e_{te} \times \left(B_{ad} - e_{pl} \right)$$

$$V_{tri} = \left[B_{cr} - 6 + 2 \times \left(0.6 \times h_{rri} + e_{te} \right) \right] \times e_{te}$$

$$h_{rri} = El_{tri} - El_{cr} - e_{te}, \; i = 0, 1, 2$$

for the stilling basin: $B_{cr} = B_{bd} + 2 \times 1.0$

for the ski jump: $B_{cr} = B_{se} + 2 \times 1.0$

where:

V_{tca}	volume of common excavation for the approach channel, in m^3;
V_{tes}	volume of common excavation for the structure, in m^3;
V_{tcr}	volume of common excavation for the downstream channel, in m^3;
V_{tai}	volume of common excavation per meter at section i of the approach channel, in m^3/m;
L_{ca}	length of the approach channel in the part with no sluiceways, in m;
V_{tad}	extra volume of common excavation for the approach channel because of the sluiceways, in m^3;
B_{vt}	total width of the spillway, in m;
e_{te}	mean thickness of the layer of soil in the area of the spillway per se, in m;
L_{vt}	total length of the spillway, in m;
V_{tri}	volume of common excavation per meter at section i for the downstream channel, in m^3/m;
L_{cr}	length of the downstream channel, in m;
B_{ca}	width of the bottom of the approach channel, in m;
h_{rai}	depth of excavation in rock at section i of the approach channel, in m;
El_{tai}	mean elevation of the land at section i of the approach channel;
El_{cv}	elevation of the bottom of the spillway approach channel, sluiceways excluded;
e_{pe}	thickness of the end pillars of the spillway, in m;
L_{cad}	mean length of the approach channel in the part with sluiceways, in m;
B_{ad}	total width of sluiceways, in m;
e_{pl}	thickness of the sluiceway walls, in m;
B_{cr}	width of the bottom of the downstream channel, in m;
h_{rri}	depth of excavation in rock at section i of the downstream channel, in m;
El_{tri}	mean elevation of the land at section i perpendicular to the longitudinal axis of the downstream channel, in m;
El_{cr}	elevation of the bottom of the downstream channel, in m;
B_{bd}	width of the stilling basin, in m; and
B_{se}	width of the ski jump, in m.

The unit price of **common excavation** is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price

per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.19.30.12.11)

Fig. 5.7.5.10 shows typical cross-sections with excavation for the approach and downstream channels.

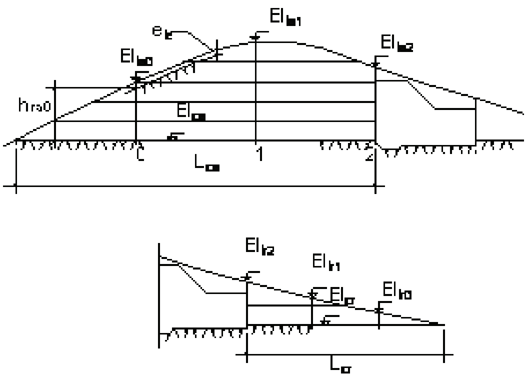


Fig. 5.7.5.10 – Excavation for approach and downstream channels.

The **volume of excavated rock** for the spillway, V_{rvt} (m^3), is given by:

$$V_{rvt} = V_{rca} + V_{rog} + V_{rpj} + V_{rde} + V_{rcr}$$

where:

$$V_{rca} = \left(\frac{V_{ra0}}{2} + V_{ra1} + V_{ra2}\right) \times \frac{L_{ca}}{3} + V_{rad}$$

$$V_{rcr} = \left(\frac{V_{rr0}}{2} + V_{rr1} + V_{rr2}\right) \times \frac{L_{cr}}{3}$$

$$V_{rai} = (B_{ca} - 6 + 0{,}6 \times h_{rai}) \times h_{rai}$$

$$V_{rri} = (B_{cr} - 6 + 0{,}6 \times h_{rri}) \times h_{rri}$$

$$h_r = El_{te} - e_{te} - (El_{ca} - 1.5)$$

$$h_{rv} = El_{te} - e_{te} - (El_{cv} - 1.5)$$

for spillways without sluiceways:

$$V_{rog} = (L_{ov} \times h_{rv} + 23) \times B_{vt}$$

$$V_{rad} = 0$$

and with a stilling basin:

$$V_{rpj} = d_1 \times \left(\frac{h_{rv} + h_{rb}}{2} + 0.167 \times H_d \right) \times \left(B_{bd} + 2.0 \right)$$

and with a ski jump:

$$V_{rpj} = d_7 \times \left(\frac{h_{rv} + h_{rs}}{2} + 0.167 \times H_d \right) \times \left(B_{se} + 2.0 \right)$$

$$V_{rde} = d_8 \times h_{rs} \times \left(B_{se} + 2.0 \right)$$

for spillways with sluiceways:

$$V_{rog} = \left(L_{ov} \times h_{rv} + 23 \right) \times \left(B_{vt} - B_{ad} \right) + \left(L_{og} \times h_r + 23 \right) \times B_{ad}$$

$$V_{rad} = \frac{L_{cad} + L_{ca}}{2} \times \left(El_{cv} - El_{ca} \right) \times \left(B_{ad} - e_{pl} \right) - V_{tad}$$

and with a stilling basin:

$$V_{rpj} = d_1 \times \left(\frac{h_{rv} + h_{rb}}{2} + 0.167 \times H_d \right) \times \left(B_{bd} + 2.0 - B_{ad} \right) + d_2 \times \left(\frac{h_r + h_{rb}}{2} + 0.167 \times H_d \right) \times B_{ad}$$

$$d_2 = 0.75 \times \left[El_{ca} - 1.5 - \left(El_{bd} - e_c \right) \right]$$

and with a ski jump:

$$V_{rpj} = d_7 \times \left(\frac{h_{rv} + h_{rs}}{2} + 0.167 \times H_d \right) \times \left(B_{se} + 2.0 - B_{ad} \right)$$

$$V_{rde} = d_8 \times h_{rs} \times \left(B_{se} + 2.0 - B_{ad} \right) + L_{se} \times h_r \times B_{ad}$$

for any spillway with a stilling basin:

$$V_{rde} = V_{rbd} + V_{rmc} + V_{rbe}$$

$$V_{rbd} = L_{bd} \times h_{rb} \times \left(B_{bd} + 2.0 \right)$$

$$V_{rmc} = 2 \times \left(L_{bd} + d_4 \right) \times 0.5 \times h_1 \times 1.5 + 2 \times \left(d_3 - d_4 \right) \times 0.5 \times \frac{h_1 + h_2}{2} \times 1.5$$

$$V_{rbe} = 2 \times \left(L_{bd} + d_4 \right) \times 0.3 \times h_{re}^2$$

$$h_{rb} = El_{te} - e_{te} - \left(El_{bd} - e_c \right)$$

$$h_1 = NA_{ccr} + 2.0 - \left(El_{tde} - e_{te} \right) \geq 0$$

$$h_2 = NA_{max} + 0.12 \times H_d - \left(El_{tde} - e_{te} \right) \geq 0$$

$$d_1 = 0.75 \times \left[El_{cv} - 1.5 - \left(El_{bd} - e_c \right) \right]$$

$$d_3 = 0.75 \times \left[NA_{max} - 0.83 \times H_d - \left(El_{bd} - e_c \right) \right]$$

$$d_4 = 0.75 \times \left(y_2 + 2.0 + e_c - 0.95 \times H_d \right) \leq d_3$$

$$h_{re} = El_{tde} - e_{te} - \left(NA_{ccr} - 5.0 \right) \geq 0$$

for any spillway with a ski jump:

$$h_{rs} = El_{te} - e_{te} - (El_{se} - 0.25 \times H_d)$$

$$d_7 = 1.46 \times H_d^{0.46} \times \left[(p_v - h_4)^{0.54} - (p_{vv} + 1.5)^{0.54} \right]$$

$$h_4 = El_{se} - 0.25 \times H_d - El_{ca}$$

$$d_8 = 0.986 \times R_{se} - 0.188 \times H_d$$

where:

V_{rca}	volume of rock excavated for the approach channel, in m³;
V_{rog}	volume of rock excavated in the ogee crest area, in m³;
V_{rpj}	volume of rock excavated in the area of the downstream face of the ogee crest, in m³;
V_{rde}	volume of rock excavated in the area of the stilling basin or ski jump, in m³;
V_{rcr}	volume of rock excavated for the downstream channel, in m³;
V_{rai}	volume of excavated rock per meter at section i of the approach channel, in m³/m;
L_{cad}	mean length of the approach channel in the part with sluiceways, in m;
V_{rad}	extra volume of excavation in rock for the approach channel due to the sluiceways, in m³;
V_{rri}	volume of excavated rock per meter at section i for the downstream channel, in m³/m;
L_{cr}	mean length of the downstream channel, in m;
B_{ca}	width of the bottom of the approach channel, in m;
h_{rai}	depth of excavation in rock at section i of the approach channel, in m;
B_{cr}	width of the bottom of the downstream channel, in m;
h_{rri}	depth of excavation in rock at section i of the downstream channel, in m;
h_r	mean depth of the excavation in rock for the ogee crest in the part with sluiceways, in m;
El_{te}	mean elevation of the land in the area of the spillway per se, in m;
e_{te}	mean thickness of the layer of soil in the spillway area, in m;
El_{ca}	elevation of the bottom of the approach channel, in m;
h_{rv}	mean depth of excavation in rock at the part of the ogee crest with no sluiceways, in m;
El_{cv}	elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m;
L_{ov}	length of the ogee crest at the part with no sluiceways, in m;
B_{vt}	total width of the spillway, in m;
h_{rb}	mean depth of excavation in rock for the stilling basin, in m;
H_d	maximum energy head on the spillway crest, in m;
B_{bd}	width of the stilling basin, in m;
L_{bd}	length of the stilling basin, in m;
h_{rs}	mean depth of excavation in rock for the ski jump at the part with no sluiceways, in m;
B_{se}	width of the ski jump, in m;
B_{ad}	total width of sluiceways, in m;
L_{og}	length of the ogee crest at the part with sluiceways, in m;
L_{ca}	mean length of the approach channel at the part with no sluiceways, in m;
e_{pl}	thickness of the sluiceway walls, in m;
V_{tad}	extra volume of common excavation for the approach channel because of the sluiceways, in m³;
El_{bd}	elevation of the bottom of the stilling basin, in m;
e_c	thickness of the concrete lining for the stilling basin sill, in m;
L_{se}	length of the ski jump at the foundations, in m;
V_{rbd}	volume of excavated rock in the stilling basin area, in m³;
V_{rmc}	volume of excavated rock in the area of the buttress walls for the stilling basin or ski jump, in m³;
V_{rbe}	volume of excavated rock for berms in the stilling basin area, in m³;
h_{re}	mean depth of excavation in rock in berms in the stilling basin area, in m;
NA_{ccr}	water level in the downstream channel for a 100-year flood, in m;
El_{tde}	mean elevation of the land in the stilling basin or ski jump area, in m;

NA_{max}	maximum normal water level in the reservoir, in m;
y_2	depth of discharge after the hydraulic jump, in m;
El_{se}	elevation of the ski jump sill, in m;
p_v	vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m;
p_{vv}	vertical distance between the ogee sill and the bottom of the spillway approach channel, in m;
R_{se}	radius of curvature of the ski jump, in m; and
d_i, h_i	secondary dimensions, in m.

The unit price of **excavation in rock** is R$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.19.30.13)

The **area of foundation to be cleaned**, A_{lf} (m²), is given by:

$$A_{lf} = B_{vt} \times L_{vt}$$

where:

B_{vt}	total width of the spillway, in m; and
L_{vt}	total length of the spillway, in m.

The **length of the grout holes** and the **drainage line** of the spillway, L_{tf} (m), is given by:

$$L_{tf} = \frac{B_{vt}}{3.0} \times L_{1tf}$$

$$L_{1tf} = 1.5 \times (NA_{xmx} - El_{cv}) \ \leq \ 40m$$

where:

B_{vt}	total width of the spillway, in m;
L_{1tf}	length of one grout hole, in m;
NA_{xmx}	maximum water level of the reservoir under design flood conditions, in m;
El_{cv}	elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m; and
3.0	space between the grout holes, in m.

The **total length of the rock anchors** in the stilling basin, L_{tfc} (m), when necessary, is given by:

$$L_{tfc} = B_{bd} \times L_{bd}$$

where:

B_{bd}	width of the stilling basin, in m; and
L_{bd}	length of the stilling basin, in m.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m^2
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.19.30.14)

The **volume of concrete** for the spillway, V_{cvt} (m^3), is given by (COPEL, 1983 and 1996):

$$V_{cvt} = V_{cog} + V_{cpj} + V_{cpl} + V_{cpo} + V_{cde} + V_{cmv} + V_{cmc}$$

where:

$$V_{cpl} = 2 \times (1.21 \times H_d^2 + 18.4 \times H_d + 25) \times e_{pe}$$

$$V_{cpo} = 6 \times B_{vt}$$

for spillways without sluiceways:

$$V_{cog} = [0.944 \times H_d^{0.46} \times (p_{vv} + 1.5)^{1.54} + 0.27 \times p_{vv} \times H_d - 0.007 \times H_d^2 + 0.40 \times H_d + 18] \times B_{vt}$$

and with a stilling basin:

$$V_{cpj} = d_1 \times 0.167 \times H_d \times (B_{bd} + 2.0)$$

and with a ski jump:

$$V_{cpj} = 1.25 \times H_d \times h_5 \times (B_{se} + 2.0)$$

$$V_{csb} = 0$$

for spillways with sluiceways:

$$\begin{aligned} V_{cog} = {} & [0.944 \times H_d^{0.46} \times (p_{vv} + 1.5)^{1.54} + 0.27 \times p_{vv} \times H_d] \times (B_{vt} - B_{ad}) + \\ & + [0.944 \times H_d^{0.46} \times (p_v + 1.5)^{1.54} + 0.27 \times p_v \times H_d] \times B_{ad} + \\ & + (-0.007 \times H_d^2 + 0.40 \times H_d + 18) \times B_{vt} \end{aligned}$$

and with a stilling basin:

$$V_{cpj} = d_1 \times 0.167 \times H_d \times (B_{bd} + 2.0 - B_{ad}) + d_2 \times 0.167 \times H_d \times B_{ad}$$

$$d_2 = 0.75 \times [El_{ca} - 1.5 - (El_{bd} - e_c)]$$

and with a ski jump:

$$V_{cpj} = 1.25 \times H_d \times h_5 \times (B_{se} + 2.0 - B_{ad})$$

$$V_{csb} = \left(\frac{d_9}{2} + L_{se}\right) \times (h_4 + 1.5) \times (B_{se} + 2.0)$$

$$d_9 = 1{,}46 \times H_d^{0.46} \times \left[(p_v + 1.5)^{0.54} - (p_v - h_4)^{0.54} \right]$$

$$h_4 = El_{se} - 0.25 \times H_d - El_{ca}$$

for any spillway with a stilling basin:

$$V_{cde} = \left(L_{bd} \times e_c + 0.036 \times R_{bd}^2 - 0.375 \times e_c^2 \right) \times (B_{bd} + 2.0)$$

$$V_{cmv} = 2 \times \left[(d_5 \times 0.95 \times H_d \times 1.0) + \frac{d_6^2}{2 \times 0.75} \right] + 2 \times L_{bd} \times (y_2 + 2.0) \times 1.0$$

$$V_{cmc} = 2 \times \left(L_{bd} + \frac{d_3 + d_4}{2} \right) \times \left(0.25 \times h_1^2 + 0.75 \times h_1 \right) + 2 \times \frac{d_3 - d_4}{2} \times \left(0.25 \times h_2^2 + 0.75 \times h_2 \right)$$

$$d_1 = 0.75 \times \left[El_{cv} - 1.5 - (El_{bd} - e_c) \right]$$

$$d_5 = 0.75 \times \left[NA_{max} - 1.0 \times H_d - (El_{bd} - e_c) \right]$$

$$d_6 = 0.75 \times (y_2 + 2.0 + e_c - 0.95 \times H_d)$$

$$d_3 = 0.75 \times \left[NA_{max} - 0.83 \times H_d - (El_{bd} - e_c) \right]$$

$$d_4 = 0.75 \times (y_2 + 2.0 + e_c - 0.95 \times H_d) \leq d_3$$

$$h_1 = NA_{ccr} + 2.0 - (El_{tde} - e_{te}) \geq 0$$

$$h_2 = NA_{max} + 0.12 \times H_d - (El_{tde} - e_{te}) \geq 0$$

for any spillway with a ski jump:

$$V_{cde} = V_{csd} + V_{csb}$$

$$V_{csd} = (0.116 \times R_{se}^2 + 0.247 \times H_d \times R_{se} - 0.023 \times H_d^2) \times (B_{se} + 2.0)$$

$$V_{cmv} = 2 \times (d_{11} \times 1.6 \times y \times 1.0 + d_{13} \times 0.95 \times H_d \times 1.0)$$

$$V_{cmc} = 2 \times \left(d_{11} + \frac{d_{10}}{2} \right) \times \left(0.25 \times h_3^2 + 0.75 \times h_3 \right) + 2 \times \frac{d_{10}}{2} \times \left(0.25 \times h_2^2 + 0.75 \times h_2 \right)$$

$$h_5 = El_{cv} - 1.5 - (El_{se} - 0.25 \times H_d) \geq 0$$

$$d_{10} = 0.75 \times [NA_{max} + 0.12 \times H_d - (El_{se} - 1.6 \times y)] \geq 0$$

$$d_{11} = d_{12} + L_{se} - 1.46 \times H_d - d_{13}$$

$$d_{12} = 1.46 \times H_d^{0.46} \times (p_v + 1.5)^{0.54}$$

$$d_{13} = 0.75 \times [NA_{max} - 0.05 \times H_d - (El_{se} - 1.6 \times y)] \geq 0$$

$$h_3 = El_{se} - 1.6 \times y - (El_{tde} - e_{te}) \geq 0$$

$$h_2 = NA_{max} + 0.12 \times H_d - (El_{tde} - e_{te}) \geq 0$$

where:

V_{cog}	volume of concrete for the ogee crest, in m^3;
V_{cpo}	volume of concrete for the bridge, in m^3;
V_{cpj}	volume of concrete for the downstream face of the ogee crest, in m^3;
V_{cpl}	volume of concrete for the pillars, in m^3;
V_{cde}	volume of concrete for the stilling basin or ski jump, in m^3;
V_{cmv}	volume of concrete for the vertical lining of the stilling basin or ski jump, in m^3;
V_{cmc}	volume of concrete for the walls of the stilling basin or ski jump, in m^3;
H_d	maximum energy head on the spillway crest, in m;
e_{pe}	thickness of the end pillars, in m;
B_{vt}	total width of the spillway, in m;
p_{vv}	vertical distance between the ogee sill and the bottom of the spillway approach channel, in m;
B_{bd}	width of the stilling basin, in m;
B_{se}	width of the ski jump, in m;
B_{ad}	total width of sluiceways, in m;
p_v	difference in height between the ogee crest and the bottom of the approach channel, in m;
El_{ca}	elevation of the bottom of the approach channel to the sluiceways, in m;
El_{bd}	elevation of the bottom of the stilling basin, in m;
e_c	thickness of the concrete lining for the stilling basin sill, in m;
L_{se}	length of the ski jump at the foundations, in m;
El_{se}	elevation of the ski jump sill, in m;
L_{bd}	length of the stilling basin, in m;
R_{bd}	radius of curvature at the stilling basin inlet, in m;
y_2	depth of discharge after the hydraulic jump, in m;
El_{cv}	elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
NA_{ccr}	water level in the downstream channel for a 100-year flood, in m;
El_{tde}	mean elevation of the land in the stilling basin or ski jump area, in m;
e_{te}	mean thickness of the layer of soil in the area of the spillway per se, in m;
V_{csd}	volume of concrete for the ski jump deflector baffle, in m^3;
V_{csb}	volume of concrete beneath the deflector baffle, in m^3;
R_{se}	radius of curvature of the ski jump, in m;
y	flow depth at the ski jump, in m; and
d_i, h_i	secondary dimensions, in m.

If there are sluiceways through the body of the spillway, the **extra volume of concrete for the sluiceways**, V_{cad} (m^3), should be taken into account as shown below:

$$V_{cad} = V_{cac} + V_{cpl} - V_{cae}$$

where:

$$V_{cac} = \left(0.24 \times H_{ad} + 2\right) \times B_{ad}$$

$$V_{cpl} = \left(0.16 \times H_{ad}^2 + 2.7 \times H_{ad} + 8\right) \times \left(N_{ad} + 1\right) \times e_{pa}$$

$$V_{cae} = \left(0.38 \times H_{ad} + 0.2\right) \times H_{ad} \times N_{ad} \times B_{1ad}$$

where:

V_{cac}	volume of concrete for part of the sluiceway sills, in m^3;
V_{cpl}	volume of concrete for the sluiceway walls upstream from the face of the dam, in m^3;
V_{cae}	volume of concrete for the sluiceway inlets, in m^3;
H_{ad}	height of sluiceways, in m;
B_{ad}	total width of sluiceways, in m;
N_{ad}	number of sluiceways;
e_{pa}	thickness of the sluiceway walls, in m; and
B_{1ad}	width of one sluiceway, in m.

Still considering the case where the diversion is through sluiceways in the spillway, the **volume of concrete for larger amounts of cement and reinforcement steel** than for the ogee crest of the spillway, V_{cen} (m^3), is given by:

$$V_{cen} = V_{cet} + V_{ces} + V_{cep}$$

for a stilling basin:

$$V_{cet} = (0.27 \times H_d + d_{14}) \times 0.25 \times H_{ad} \times B_{ad}$$

$$V_{ces} = (0.27 \times H_d + d_{15}) \times 1.5 \times B_{ad}$$

$$V_{cep} = \left(0.27 \times H_d + \frac{d_{14} + d_{15}}{2}\right) \times H_{ad} \times (N_{ad} + 1) \times e_{pa}$$

$$d_{14} = 1.46 \times H_d^{0.46} \times (p_v - H_{ad})^{0.54}$$

$$d_{15} = 1.46 \times H_d^{0.46} \times p_v^{\ 0.54}$$

for a ski jump:

$$V_{cet} = (L_{vt} - d_0) \times 0.25 \times H_{ad} \times B_{ad}$$

$$V_{ces} = L_{vt} \times 1.5 \times B_{ad}$$

$$V_{cep} = \left(L_{vt} - \frac{d_0}{2}\right) \times H_{ad} \times (N_{ad} + 1) \times e_{pa}$$

$$d_0 = (L_{vt} - 0.27 \times H_d - d_{16} - 0.836 \times R_{se} + 0.15 \times H_d) \times \frac{H_{ad}}{h_4}$$

$$d_{16} = 1.46 \times H_d^{0.46} \times (p_v - h_4)^{0.54}$$

$$h_4 = El_{se} - 0.25 \times H_{ad} - El_{ca}$$

where:

V_{cet}	volume of concrete for the sluiceway roofs, in m^3;
V_{ces}	volume of concrete for the sluiceway sills, in m^3;
V_{cep}	volume of concrete for the sluiceway walls, in m^3;
H_d	hydrostatic load on the ogee crest, in m;
H_{ad}	height of sluiceways, in m;
B_{ad}	total width of sluiceways, in m;
N_{ad}	number of sluiceways;
e_{pa}	thickness of the end pillars of the spillway, in m;
p_v	vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m;
L_{vt}	total length of the spillway, in m;
R_{se}	radius of curvature of the ski jump, in m;
El_{se}	elevation of the ski jump sill, in m;
El_{ca}	elevation of the bottom of the approach channel to the sluiceways, in m;
d_i, h_i	secondary dimensions, in m.

The amounts of cement and reinforcement steel are:

	cement (kg/m^3)	reinforcement steel (kg/m^3)
ogee crest, buttress, beneath the deflector baffle, sluiceway sills and inlets	200	20
plugs	220	20
stilling basin and deflector baffle	270	50
pillars and walls	270	80
bridge	300	100

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of spillway (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- ogee crest, buttress, beneath the deflector baffle: 113.00/m^3
- pillars, walls, stilling basin and deflector baffle: 200.00/m^3
- sluiceway sills and inlets and concrete with greater unit quantities: 113.00/m^3

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

UNGATED SURFACE ABUTMENT SPILLWAYS (ACCOUNT .12.18.28)

Basic Data

The main **information required for dimensioning** purposes can be obtained from the overall layout and from item 5.1.2. (Hydrometeorological Data), as follows:

- design flood through the spillway, Q_v, in m^3/s, from item 5.1.2.;
- 100-year flood flow, Q_c, in m^3/s, from item 5.1.2.;
- maximum water level of the reservoir under design flood conditions, NA_{xmx}, from item 5.1.2.;
- maximum normal water level in the reservoir, NA_{max}, from item 4.6.;
- elevation of the bottom of the approach channel, El_{ca}, from item 5.7.3.;
- slope of the upstream face of the ogee crest, horizontal distance for a 1.0 m difference in level, m_m in m;

- slope of the chute, tangent of the absolute value of the angle with the horizontal, i_{cl}; and
- water level in the downstream channel for a 100-year flood flow, NA_{ccr}, from item 5.1.2.
- elevation of the ski jump sill, El_{se}, when applicable;

The **main information used for quantification** purposes is:

- mean elevation of the land in the area of the spillway per se, including the energy dissipator, El_{te}, in m;
- mean elevation of the land in the chute area, exclusively, El_{tc}, in m;
- mean elevation of the land in the stilling basin or ski jump area, exclusively, El_{tde}, in m;
- mean thickness of the layer of soil in the area of the spillway per se, e_{te} in m;
- mean elevation of the land at section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the approach channel, El_{tai}, in m;
- mean elevation of the land at section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the downstream channel, El_{tri}, in m;
- elevation of the bottom of the downstream channel, El_{cr}, in m;
- mean length of the approach channel, L_{ca}, in m;
- mean length of the downstream channel, L_{cr}, in m; and
- thickness of the concrete lining for the stilling basin sill, e_c, in m, when applicable.

Considerations and recommendations

This text relates to spillways with a typical cross-section as shown in Fig. 5.7.5.11 or 5.7.5.12.

Use 10% as the **slope of the chute** in the absence of more accurate information.

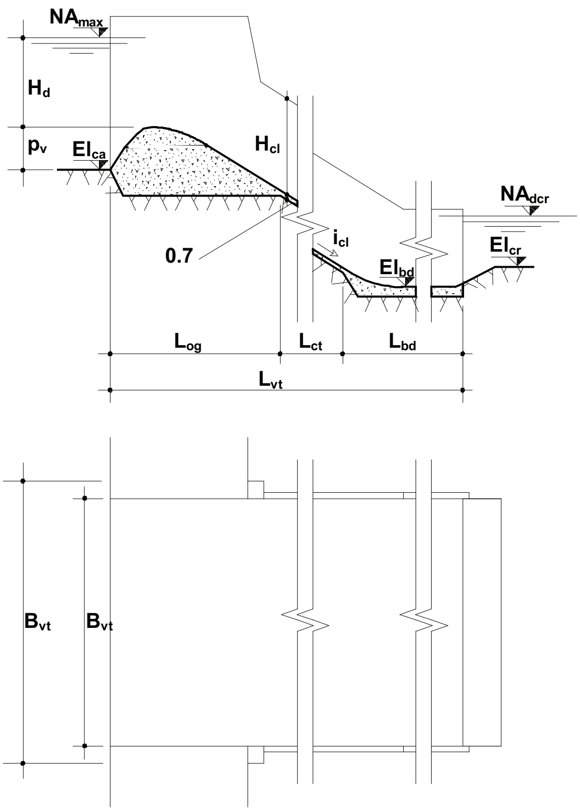


Fig. 5.7.5.11 – Typical cross-section and plan of an ungated abutment spillway with a stilling basin.

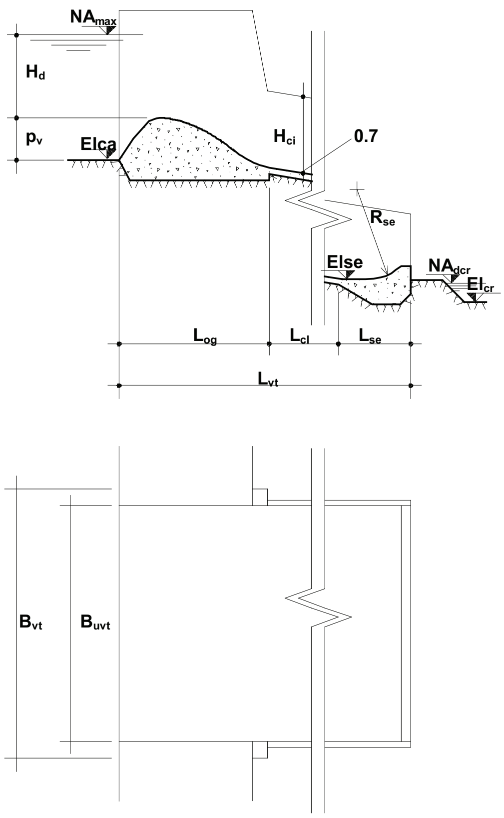


Fig. 5.7.5.12 – Typical cross-section of an ungated abutment spillway with a ski jump.

When **dimensioning the stilling basin**, the recommended range for the Froude number before the hydraulic jump is betwen 4.5 and 9.0, in order to ensure its stability. One way of raising the Froude number to 4.5 would be to lower the bottom of the stilling basin beyond the result given by the calculation. Likewise, one way to reduce the Froude number to 9.0 would be to reduce the width of the stilling basin.

In the absence of more accurate information, use 1.0 m as the **thickness of the concrete lining for the bottom of the stilling basin**.

Discharge Coefficient

The **discharge coefficient**, C_d, for the typical cross-section recommended can be obtained from Graph 5.7.5.03 (Bureau of Reclamarion, 1977) or from the equivalent expressions:

for m_m=1.0 and $0.100 < z \le 0.524$:

$$C_d = 1.9507 \times z^3 - 2.9011 \times z^2 + 1.5498 \times z + 1.8274$$

for m_m=1.0 and $0.524 < z \le 0.813$:

$$C_d = 0.1592 \times z^3 - 0.4409 \times z^2 + 0.4248 \times z + 1.9984$$

for m_m=1.0 and $0.813 < z \le 1.800$:

$$C_d = 0.0159 \times z + 2.1256$$

for m_m=0.67 and $0.100 < z \le 0.497$:

$$C_d = 2.5495 \times z^3 - 3.6032 \times z^2 + 1.8832 \times z + 1.7678$$

m_m=0.67 and $0.497 < z \le 0.759$:

$$C_d = 0.2261 \times z^3 - 0.6256 \times z^2 + 0.6137 \times z + 1.9481$$

for m_m=0.67 and 0.759 < z ≤ 1.800:

$$C_d = 0.0242 \times z^3 - 0.1143 \times z^2 + 0.1775 \times z + 2.0734$$

for m_m=0.33 and 0.100 < z ≤ 0.505:

$$C_d = 2.4283 \times z^3 - 3.5181 \times z^2 + 1.9125 \times z + 1.7265$$

for m_m=0.33 and 0.505 < z ≤ 0.755:

$$C_d = 0.2514 \times z^3 - 0.6927 \times z^2 + 0.6896 \times z + 1.9033$$

m_m=0.33 and 0.755 < z ≤ 1.800:

$$C_d = 0.02 \times z^3 - 0.0985 \times z^2 + 0.1782 \times z + 2.0508$$

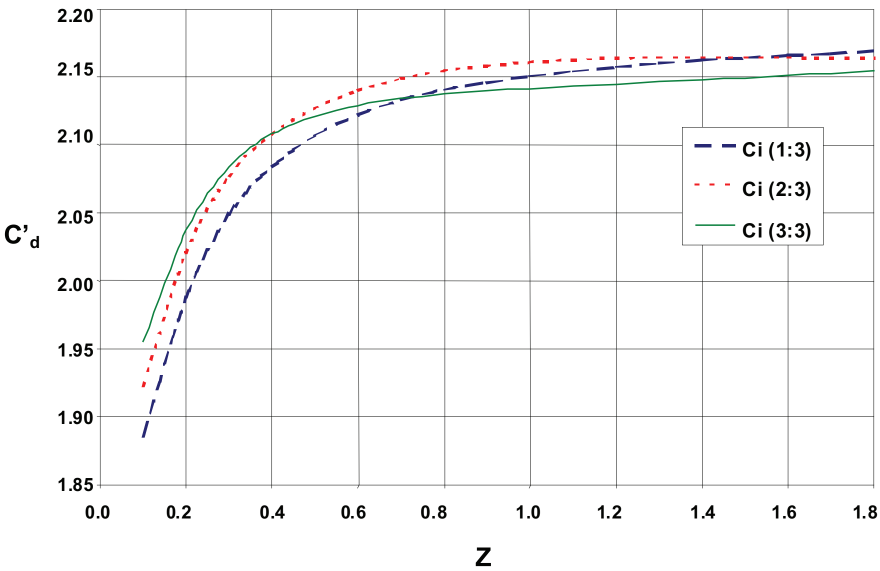
where:

$$z = \frac{p_v}{H_d}$$

$$p_v = NA_{max} - El_{ca} \qquad H_d = NA_{xmx} - NA_{max}$$

where:

z	adimensional parameter;
p_v	difference in height between the ogee crest and the bottom of the approach channel, in m;
H_d	energy head on the spillway crest, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
El_{ca}	elevation of the bottom of the approach channel, in m; and
NA_{xmx}	maximum water level of the reservoir under design flood conditions, in m.



Graph 5.7.5.03 – Discharge coefficient for spillways with an inclined upstream face.

Spillway Dimensions

The **useful width** of the spillway, perpendicular to flow, B_{uvt} (m), is given by:

$$B_{uvt} = \frac{Q_v}{C_d \times H_d^{3/2}}$$

where:

Q_v	design flood through the spillway, in m³/s;
C_d	discharge coefficient; and
H_d	maximum energy head on the spillway crest, in m.

The **thickness of the end pillars**, e_{pe} (m), is given by:

$$e_{pe} = 0.12 \times H_d + 2.4$$

where:

H_d	maximum energy head on the spillway crest, in m.

The **total width of the spillway**, B_{vt} (m), perpendicular to flow, is given by:

$$B_{vt} = 0.05 \times \text{int}\left[(B_{uvt} + 2 \times e_{pe}) \times \frac{1}{0.05} + 0.5\right]$$

where:

B_{uvt}	useful width of the spillway, in m;
e_{pe}	thickness of the end pillars, in m; and
int(x)	function that returns the integer part of x.

The **width of the chute**, B_{cl} (m), is given by:

$$B_{cl} = B_{uvt}$$

where:

B_{uvt}	useful width of the spillway, in m.

The **length of the ogee crest** in the direction of flow, L_{og} (m), is given by:

$$L_{og} = 1.66 \times H_d + m_m \times p_v$$

where:

H_d	maximum energy head on the spillway crest, in m;
m_m	slope of the upstream face of the ogee crest, horizontal distance for a 1.0 m difference in level; and
p_v	difference in height between the ogee crest and the bottom of the approach channel, in m.

The **length of the chute**, L_{cl} (m), is given by:

for the stilling basin:

$$L_{cl} = \frac{NA_{max} - 0.69 \times H_d - El_{bd}}{i_{cl}}$$

for the ski jump:

$$L_{cl} = \frac{NA_{max} - 0.69 \times H_d - El_{se} - 0.03 \times R_{se}}{i_{cl}}$$

where:

NA_{max}	maximum normal water level in the reservoir, in m;
H_d	maximum energy head on the spillway crest, in m;
El_{bd}	elevation of the bottom of the stilling basin, defined below, in m;
El_{se}	elevation of the ski jump sill, defined below, in m;
R_{se}	radius of curvature of the ski jump, defined below, in m; and
i_{cl}	slope of the chute.

The **total length** of the spillway, L_{vt} (m), is given by:

for the stilling basin: $L_{vt} = L_{og} + L_{cl} + L_{bd}$

for the ski jump: $L_{vt} = L_{og} + L_{cl} + L_{se}$

where:

L_{og}	length of the ogee crest, in m;
L_{cl}	length of the chute, in m;
L_{bd}	length of the stilling basin, defined below, in m; and
L_{se}	length of the ski jump at the foundations, defined below, in m.

Stilling Basin

The **width of the stilling basin**, B_{bd} (m), is given by:

$$B_{bd} = B_{uvt}$$

where:

B_{uvt}	useful width of the spillway, in m.

The **stilling basin depth** is determined iteratively, based on the Froude number before the hydraulic jump, in section one of Fig. 5.7.5.03, for the 100-year flood flow .

An elevation is picked for the bottom of the stilling basin and its suitability is checked by calculating the **velocity**, v_1 (m/s), the **depth of discharge**, y_1 (m), and the **Froude number**, Fr_1, before the hydraulic jump, **depth of discharge** after the hydraulic jump, y_2 (m), and finally the elevation of the **bottom of the stilling basin**.

Should the elevation of the bottom of the stilling basin be different from the figure first picked, the calculations should be redone until the level of precision required is reached.

$$v_1 = \sqrt{k \times 2 \times g \times (NA_{xmx} - El_{bd})}$$

$$y_1 = \frac{Q_c}{B_{bd} \times v_1}$$

$$Fr_1 = \frac{v_1}{\sqrt{g \times y_1}}$$

$$y_2 = \left(\frac{y_1}{2}\right) \times \left(\sqrt{1 + 8 \times Fr_1^2} - 1\right)$$

where:

v_1	mean velocity of discharge before the hydraulic jump, in m/s;
k	0.9 – reduction coefficient for the energy head;
g	9.81 m/s^2 – acceleration due to gravity;
NA_{xmx}	maximum water level of the reservoir under design flood conditions, in m;
El_{bd}	elevation of the bottom of the stilling basin, in m;
y_1	mean depth of discharge in section 1, before the hydraulic jump, in m;
Q_c	100-year flood flow, in m^3/s;
B_{bd}	width of the stilling basin, in m;
Fr_1	Froude number before the hydraulic jump;
y_2	depth of discharge after the hydraulic jump, in m; and
NA_{ccr}	water level in the downstream channel for a 100-year flood, in m.

The **radius of curvature at the stilling basin inlet**, R_{bd} (m), is given by (PETERKA):

$$R_{bd} = 6 \times y_1$$

where:

y_1	depth of discharge before the hydraulic jump, in m.

The length **of the stilling basin**, L_{bd} (m), is given by:

$$L_{bd} = 6 \times y_2 + R_{bd} \times \tan\left[\frac{\arctan(i_{cl})}{2}\right]$$

where:

y_2	depth of discharge after the hydraulic jump, in m;
i_{cl}	slope of the chute, tangent of the absolute value of the angle with the horizontal; and
R_{bd}	radius of curvature at the stilling basin inlet, in m.

Ski Jump

The **width of the ski jump**, B_{se} (m), is given by:

$$B_{se} = B_{cl}$$

where:

B_{cl}	width of the chute, in m.

The **elevation of the ski jump sill**, El_{se}, is given by:

$$El_{se} = NA_{ccr} + 1.0 \geq El_{cr}$$

where:

NA_{ccr}	water level in the downstream channel for a 100-year flood, in m; and
El_{cr}	elevation of the bottom of the downstream channel, in m.

The **radius of curvature of the ski jump**, R_{se} (m), is given by:

$$R_{se} = 6 \times y$$

where:

$$y = \frac{Q_c}{B_{se} \times v} \qquad v = \sqrt{k \times 2 \times g \times (NA_{xmx} - El_{se})}$$

where:

y	depth of the water column at the ski jump, in m;
Q_c	100-year flood flow, in m^3/s;
B_{se}	width of the ski jump, in m;
v	velocity of the water column at the ski jump, in m/s;
k	0.9 – reduction coefficient for the energy head;
g	9.81 m/s^2 – acceleration due to gravity;
NA_{xmx}	maximum water level of the reservoir under design flood conditions, in m; and
El_{se}	elevation of the ski jump sill, in m.

The **length of the ski jump**, L_{se} (m), is given by:

$$L_{se} = 0.80 \times R_{se} + 1.5$$

where:

R_{se}	radius of curvature of the ski jump, in m.

Common Excavation (account .12.19.30.12.10)

The **volume of common excavation** for the spillway, V_{tvt} (m^3), is given by:

$$V_{tvt} = V_{tca} + V_{tes} + V_{tcr}$$

where:

$$V_{tca} = \left(\frac{V_{ta0}}{2} + V_{ta1} + V_{ta2} \right) \times \frac{L_{ca}}{3}$$

$$V_{tai} = \left[B_{ca} - 6 + 2 \times \left(0.6 \times h_{rai} + e_{te} \right) \right] \times e_{te}$$

$$h_{rai} = El_{tai} - El_{ca} - e_{te}\text{, } i = 0, 1, 2$$

$$B_{ca} = B_{vt} - 2 \times \left(e_{pl} - 1.0 \right)$$

$$V_{tes} = L_{vt} \times e_{te} \times B_{vt}$$

$$V_{tcr} = \left(\frac{V_{tr0}}{2} + V_{tr1} + V_{tr2} \right) \times \frac{L_{cr}}{3}$$

$$V_{tri} = \left[B_{cr} - 6 + 2 \times \left(0.6 \times h_{rri} + e_{te} \right) \right] \times e_{te}$$

$$h_{rri} = El_{tri} - El_{cr} - e_{te}\text{, } i = 0, 1, 2$$

for the stilling basin: $B_{cr} = B_{bd} + 2 \times 1.0$

for the ski jump: $B_{cr} = B_{se} + 2 \times 1.0$

where:

V_{tca}	volume of common excavation for the approach channel, in m^3;
V_{tes}	volume of common excavation for the structure, in m^3;
V_{tcr}	volume of common excavation for the downstream channel, in m^3;
V_{tai}	volume of common excavation per meter at section i of the approach channel, in m^3/m;
L_{ca}	length of the approach channel, in m;
B_{ca}	width of the bottom of the approach channel, in m;
h_{rai}	depth of excavation in rock at section i of the approach channel, in m;
e_{te}	mean thickness of the layer of soil in the area of the spillway, in m;
El_{tai}	mean elevation of the land at section i of the approach channel, in m;
El_{ca}	elevation of the bottom of the approach channel, in m;
B_{vt}	total width of the spillway, in m;
e_{pl}	thickness of the pillars, in m;
L_{vt}	total length of the spillway, in m;
V_{tri}	volume of common excavation per meter at section i for the downstream channel, in m^3/m;
L_{cr}	length of the downstream channel, in m;
B_{cr}	width of the bottom of the downstream channel, in m;
h_{rri}	depth of excavation in rock at section i of the downstream channel, in m;
El_{tri}	mean elevation of the land at section i perpendicular to the longitudinal axis of the downstream channel, in m;
El_{cr}	elevation of the bottom of the downstream channel, in m;
B_{bd}	width of the stilling basin, in m; and
B_{se}	width of the ski jump, in m.

The unit price of **common excavation** is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.19.30.12.11)

Figure 5.7.5.10 shows typical cross-sections for the excavation of approach and downstream channels.

The **volume excavated in rock**, V_{rtv} (m³), is given by:

$$V_{rvt} = V_{rca} + V_{rog} + V_{rcl} + V_{rbc} + V_{rde} + V_{rbe} + V_{rcr}$$

where:

$$V_{rca} = \left(\frac{V_{ra0}}{2} + V_{ra1} + V_{ra2} \right) \times \frac{L_{ca}}{3}$$

$$V_{rai} = \left(B_{ca} - 6 + 0.6 \times h_{rai} \right) \times h_{rai}$$

$$V_{rog} = L_{og} \times \left[El_{te} - e_{te} - \left(El_{ca} - 2 \right) \right] \times B_{vt}$$

$$V_{rcl} = L_{cl} \times \left[El_{tc} - e_{te} - \left(El_{tc} - 0.7 \right) \right] \times \left(B_{cl} + 2 \right)$$

$$V_{rbc} = 2 \times L_{cl} \times 0.3 \times h_{rc}^2$$

$$V_{rcr} = \left(\frac{V_{rr0}}{2} + V_{rr1} + V_{rr2} \right) \times \frac{L_{cr}}{3}$$

$$V_{rri} = \left(B_{cr} - 6 + 0.6 \times h_{rri} \right) \times h_{rri}$$

$$h_{rc} = El_{tc} - e_{te} - \left(El_{cm} - 0.7 \right) \ \geq \ 0$$

for the stilling basin:

$$V_{rde} = L_{bd} \times \left[El_{te} - e_{te} - \left(El_{bd} - e_c \right) \right] \times \left(B_{bd} + 2 \right)$$

$$V_{rbe} = 2 \times L_{bd} \times 0.3 \times h_{re}^2$$

$$El_{cm} = \frac{NA_{max} - 0.69 \times H_d + El_{bd}}{2}$$

$$h_{re} = El_{tde} - e_{te} - \left(NA_{ccr} - 5.0 \right) \ \geq \ 0$$

for the ski jump:

$$V_{rde} = L_{se} \times \left[El_{te} - e_{te} - (El_{se} - 2)\right] \times (B_{se} + 2)$$

$$V_{rbe} = 2 \times L_{se} \times 0.3 \times h_{rs}^2$$

$$El_{cm} = \frac{NA_{max} - 0.69 \times H_d + El_{se}}{2}$$

$$h_{rs} = El_{tde} - e_{te} - (El_{se} - 2.0) \geq 0$$

where:

V_{rca}	volume of rock excavated for the approach channel, in m^3;
V_{rog}	volume of rock excavated in the ogee crest area, in m^3;
V_{rcl}	volume of rock excavated in the chute area, in m^3;
V_{rbc}	volume of rock excavated for berms in the chute section, in m^3;
V_{rde}	volume of rock excavated in the area of the stilling basin or ski jump, in m^3;
V_{rbe}	volume of rock excavated for berms in the stilling basin or ski jump section, in m^3;
V_{rcr}	volume of rock excavated for the downstream channel, in m^3;
V_{rai}	volume of excavated rock per meter at section i of the approach channel, in m^3/m;
L_{ca}	Length of the approach channel, in m;
B_{ca}	width of the bottom of the approach channel, in m;
h_{rai}	Depth of excavation in rock at section i of the approach channel, in m;
L_{og}	length of the ogee crest, in m;
El_{te}	mean elevation of the land in the area of the spillway per se, in m;
e_{te}	mean thickness of the layer of soil in the area of the spillway per se, in m;
El_{ca}	elevation of the bottom of the approach channel, in m;
B_{vt}	total width of the spillway, in m;
L_{cl}	length of the chute, in m;
El_{cm}	mean elevation of the chute, in m;
B_{cl}	width of the chute, in m;
h_{rc}	Depth of excavation in rock in the chute area, in m;
V_{rri}	volume of excavated rock per meter at section i for the downstream channel, in m^3/m;
L_{cr}	Length of the downstream channel, in m;
B_{cr}	width of the bottom of the downstream channel, in m;
h_{rri}	Depth of excavation in rock at section i of the downstream channel, in m;
El_{tc}	mean elevation of the land in the chute area, exclusively, in m;
L_{bd}	length of the stilling basin, in m;
El_{bd}	elevation of the bottom of the stilling basin, in m;
e_c	Thickness of the concrete lining for the stilling basin sill, in m;
B_{bd}	width of the stilling basin, in m;
h_{re}	depth of excavation in rock in the stiling basin area, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
H_d	maximum energy head on the spillway crest, in m;
El_{tde}	mean elevation of the land in the stilling basin or ski jump area, exclusively, in m;
NA_{ccr}	water level in the downstream channel for a 100-year flood, in m;
L_{se}	length of the ski jump, in m;
El_{se}	elevation of the ski jump sill, in m;
B_{se}	width of the ski jump, in m; and
h_{rs}	depth of excavation in rock in the ski jump area, in m.

The unit price of **excavation in rock** is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price

per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.19.30.13)

The **area of foundation to be cleaned**, A_{lf} (m^2), is given by:

$$A_{lf} = B_{vt} \times L_{vt}$$

where:

B_{vt}	total width of the spillway, in m; and
L_{vt}	total length of the spillway, in m.

The **length of the grout holes** and the **drainage line**, L_{tf} (m), is given by:

$$L_{tf} = \frac{B_{vt}}{3.0} \times L_{1tf}$$

$$L_{1tf} = 1.5 \times (NA_{xmx} - El_{ca}) \leq 40m$$

where:

B_{vt}	total width of the spillway, in m;
L_{1tf}	length of one grout hole, in m;
NA_{xmx}	maximum water level of the reservoir under design flood conditions, in m;
El_{ca}	elevation of the bottom of the approach channel, in m; and
3.0	space between the grout holes, in m.

The **total length of the rock anchors** in the stilling basin, L_{tfc}(m), when necessary, is given by:

$$L_{tfc} = B_{bd} \times L_{bd}$$

where:

B_{bd}	width of the stilling basin, in m; and
L_{bd}	length of the stilling basin, in m.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m^2
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.19.30.14)

The **volume of concrete** for the spillway, V_{cvt} (m^3), is given by:

$$V_{cvt} = V_{cog} + V_{cpl} + V_{cpo} + V_{ccl} + V_{cde} + V_{cmv}$$

where:

$$V_{cog} = (0.165 \times H_d^2 + 0.67 \times p_v \times H_d + 0.84 \times p_v^2 + 32) \times B_{vt}$$

$$V_{cpl} = 2 \times \left(1.21 \times H_d^2 + 18.4 \times H_d + 25\right) \times e_{pe}$$

$$V_{cpo} = 6.0 \times B_{vt}$$

$$V_{ccl} = L_{cl} \times \left[0.7 \times B_{cl} + 2 \times \left(H_{cl} + 0.7\right) \times 1.0\right]$$

$$H_{cl} = 0.95 \times H_d$$

for the stilling basin:

$$V_{cde} = L_{bd} \times e_c \times \left(B_{bd} + 2.0\right)$$

$$V_{cmv} = 2 \times \left[L_{bd} \times \left(2.0 + y_2 + e_c\right) + \frac{d_1^2}{2 \times i_{cl}}\right] \times 1.0$$

$$d_1 = 2.0 + y_2 - H_{cl}$$

for the ski jump:

$$V_{cde} = \left(0.12 \times R_{se}^2 + 0.93 \times R_{se} + 0.53\right) \times \left(B_{se} + 2.0\right)$$

$$V_{cmv} = 2 \times L_{se} \times H_{cl} \times 1.0$$

where:

V_{cog}	volume of concrete for the ogee crest, in m^3;
V_{cpl}	volume of concrete for the pillars, in m^3;
V_{cpo}	volume of concrete for the bridge, in m^3;
V_{ccl}	Volume of concrete for the chute, including walls, in m^3;
V_{cde}	Volume of concrete for the stilling basin or ski jump, including walls, in m^3;
H_d	maximum energy head on the spillway crest, in m;
p_v	difference in height between the ogee crest and the bottom of the approach channel, in m;
B_{vt}	total width of the spillway, in m;
e_{pe}	thickness of the end pillars, in m;
L_{cl}	length of the chute, in m;
B_{cl}	width of the chute, in m;
H_{cl}	height of the wall for the chute, in m;
L_{bd}	length of the stilling basin, in m;
e_c	thickness of the concrete lining for the stilling basin sill, in m;
B_{bd}	width of the stilling basin, in m;
y_2	depth of discharge after the hydraulic jump, in m;
i_{cl}	slope of the chute;
R_{se}	radius of curvature of the ski jump, in m;
B_{se}	Width of the ski jump, in m;
L_{se}	length of the ski jump, in m; and
d_i	secondary dimensions, in m.

The amounts of cement and reinforcement steel are:

	cement (kg/m³)	reinforcement steel (kg/m³)
ogee crest	200	40
pillars, chute, stilling basin, ski jump and walls	250	80
bridge	300	100

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of spillway (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- ogee crest: 113.00/m³
- pillars, chute , walls and stilling basin: 200.00/m³
- bridge: 474.00/m

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

5.7.6 Intake (account .12.19)

HEADRACE CANAL (.12.19.31)

Basic Data

The main **information required for dimensioning** purposes is:

- length of the canal, L_{cn} in m;
- length of the section lined with concrete, L_c in m;
- total maximum turbine flow, Q_t, in m³/s, from item 5.7.2.;
- mean elevation of the land along the canal axis, El_{te};
- mean thickness of the layer of soil, e_{te}, in m;
- maximum normal water level in the reservoir, NA_{max}, from item 4.4.; and
- minimum water level in the reservoir, NA_{min}, from item 5.3.

The **information required for quantification** purposes is:

- thickness of the concrete lining, e_c, in m.

Considerations and Recommendations

Three typical cross-sections are used for design purposes:

- canal with a **compound** trapezoidal cross-section excavated in soil and rock and with the minimum water level **above** the elevation of the top of the bedrock, as per Fig. 5.7.6.01. (Case 1)
- canal with a **compound** trapezoidal cross-section excavated in soil and rock and with the minimum water level **below** the elevation of the top of the bedrock, as per Fig. 5.7.6.02. (Case 2)
- canal with a **simple** trapezoidal cross-section excavated in soil only, as per Fig. 5.7.6.03. (Case 3)

The freeboard of the canal is set at 2.0 m.

The canals may or may not be lined. The lining serves two main purposes: to reduce seepage along the canal and to increase the velocity of discharge, allowing the cross-section of the canal to be smaller. Lining the canal tends to be economically advantageous when the canal is long.

When the canal is excavated through rock, a concrete lining should be used. When the canal is excavated through soil, it can be lined with a clay blanket.

Basic Cross-Section

Maximum reservoir drawdown, d (m), is given by:

$$d = NA_{max} - NA_{min}$$

where:

NA_{max}	maximum normal water level in the reservoir, in m; and
NA_{min}	minimum water level in the reservoir, in m.

The **depth of flow**, y_m (m), is given by:

$$y_m = \sqrt{\frac{Q_t}{3}}$$

where:

Q_t	total maximum turbine flow, in m³/s.

The **elevation of the canal bottom**, El_{cn}, is given by:

$$El_{cn} = NA_{min} - y_m$$

where:

NA_{min}	minimum water level in the reservoir, in m; and
y_m	depth of flow, in m.

The **width of the canal bottom**, B_{cn} (m), is given by:

$$B_{cn} = 1.5 \times y_m$$

where:

y_m	depth of flow, in m.

Canal excavated through soil and rock (Case 1)

Flow is through a compound cross-section of soil and rock when:

$$El_{cn} < El_{te} - e_{te}$$

$$NA_{min} > El_{te} - e_{te}$$

where:

El_{cn}	elevation of the canal bottom, in m;
El_{te}	mean elevation of the land along the canal axis, in m;
e_{te}	thickness of the layer of soil, in m; and
NA_{min}	minimum water level in the reservoir, in m.

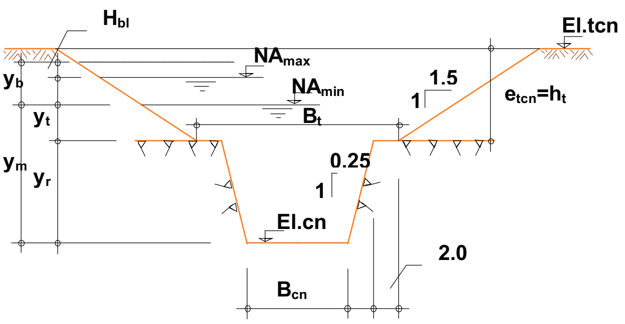


Fig. 5.7.6.01 – Typical cross-section of a headrace canal excavated through soil and rock (case 1).

The **height of the water column in the part of the canal cut through soil**, y_t (m), is given by:

$$y_t = e_{te} - (El_{te} - NA_{min})$$

where:

e_{te}	thickness of the layer of soil, in m;
El_{te}	elevation of the land along the canal axis, in m; and
NA_{min}	minimum water level in the reservoir, in m.

The **depth of excavation in soil for the canal**, h_t (m), is given by:

$$h_t = e_{te}$$

where:

e_{te}	thickness of the layer of soil, in m.

The **depth of the water in the part of the canal cut through rock**, y_r (m), is given by:

$$y_r = y_m - y_t$$

where:

y_m	depth of flow, in m; and
y_t	height of the water column in the part of the canal cut through soil, in m.

The **total area of the canal cross-section**, A_{cn} (m^2), is given by:

$$A_{cn} = A_{rcn} + A_{tcn}$$

where:

$$A_{rcn} = (1.5 \times y_m + 0.25 \times y_r) \times y_r$$

$$A_{tcn} = (B_{cn} + 0.5 \times y_r + 1.5 \times y_t + 4) \times y_t$$

where:

A_{rcn}	flow cross-section through rock, in m^2;
A_{tcn}	flow cross-section through soil, in m^2;
y_m	depth of flow, in m;
y_r	depth of the water in the part of the canal cut through rock, in m;
B_{cn}	width of the canal bottom, in m; and
y_t	depth of the water in the part of the canal cut through soil, in m.

The head loss in the headrace canal, h_c (m), is given by:

$$h_c = \left[(L_{cn} - L_c) \times n^2 + L_c \times n_c^2 \right] \times \frac{v_{cn}^2}{R_h^{4/3}}$$

where:

$$v_{cn} = \frac{Q_t}{A_{cn}}$$

$$R_h = \frac{A_{cn}}{B_{cn} + 2.06 \times y_r + 4 + 3.61 \times y_t}$$

n	Type of Lining
0.035	for canals cut through rock
0.025	for canals cut through soil
0.014	for canals lined with concrete
0.010	for canals lined with a clay blanket

where:

n	Manning's coefficient for the unlined section;
n_c	Manning's coefficient for the lined section;
L_{cn}	length of the canal, in m.
L_c	length of the lined section of the canal, in m.
v_{cn}	mean velocity of discharge in the canal, in m/s;
R_h	hydraulic radius, in m;
Q_t	total maximum turbine flow, in m^3/s;
A_{cn}	total flow cross-section of the canal, in m^2;
B_{cn}	width of the canal bottom, in m;
y_r	depth of the water in the part of the canal cut through rock, in m; and
y_t	depth of the water in the part of the canal cut through soil, in m.

The **slope of the canal bottom**, i (m/m), is given by:

$$i = \frac{h_c}{L_{cn}}$$

where:

h_c	head loss in the canal, in m; and
L_{cn}	length of the canal, in m.

For long headrace canals (over 3.0 km long), a forebay of volume V_{cg} (m^3) is included at the intake, given by:

$$V_{cg} = \frac{300 \times Q_t}{N_g}$$

where:

Q_t	total maximum turbine flow, in m^3/s; and
N_g	number of generating units in a plant.

Common Excavation (account .12.19.31.10)

The **volume of common excavation**, V_{tcn} (m^3), is given by:

$$V_{tcn} = (B_{cn} + 0.5 \times h_r + 1.5 \times h_t + 4) \times h_t \times L_{cn}$$

where: $h_r = y_r$

where:

B_{cn}	width of the canal bottom, in m;
y_r	depth of the water in the part of the canal cut through rock, in m;
h_t	depth of excavation in soil, in m;
h_r	depth of excavation in rock, in m; and
L_{cn}	length of the canal, in m.

The unit price of **common excavation** is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the canal. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation(account .12.19.31.12.11).

The **volume of excavation in rock**, V_{rcn} (m^3), is given by:

$$V_{rcn} = (B_{cn} + 0.25 \times h_r) \times h_r \times L_{cn}$$

where: $h_r = y_r$

where:

B_{cn}	width of the canal bottom, in m;
y_r	depth of the water in the part of the canal cut through rock, in m;
h_r	depth of excavation in rock; and
L_{cn}	length of the canal, in m.

The unit price of **excavation in rock** is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the canal. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account.12.19.31.14)

The **volume of concrete**, V_{ccn} (m^3), is given by:

$$V_{ccn} = \left[B_{cn} + 2.06 \times y_r + 4.0 + 3.61 \times (d + y_t + 2)\right] \times e_c \times L_c$$

where:

B_{cn}	width of the canal bottom, in m;
y_r	depth of the water in the part of the canal cut through rock, in m;
d	maximum reservoir drawdown, in m;
y_t	depth of the water in the part of the canal cut through soil, in m;
e_c	thickness of the concrete lining, in m; and
L_c	length of the section of canal lined with concrete, in m.

The amounts of cement and reinforcement steel are:

	cement (kg/m^3)	reinforcement steel (kg/m^3)
conventional concrete	275	50
shotcrete	300	70

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- conventional concrete: 234.00/m³
- shotcrete: 128.00/m³

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Canal excavated through rock (Case 2)

The flow is through a compound cross-section, but predominantly of rock, when:

$$NA_{min} \leq El_{te} - e_{te}$$

where:

NA_{min}	minimum water level in the reservoir, in m;
El_{te}	elevation of the land along the canal axis, in m; and
e_{te}	thickness of the layer of soil, in m.

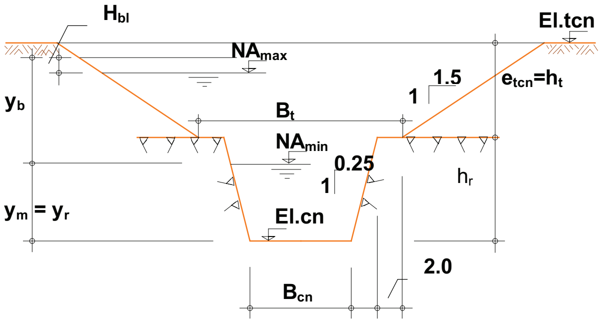


Fig. 5.7.6.02 – Typical cross-section of a headrace canal through rock (case 2).

The **depth of the water in the part of the canal cut through soil**, y_t (m), is given by:

$$y_t = 0$$

The **depth of excavation in soil for the canal**, h_t (m), is given by:

$$h_t = e_{te}$$

where:

e_{te}	thickness of the layer of soil, in m.

The **depth of the water in the part of the canal cut through rock**, y_r (m), is given by:

$$y_r = y_m$$

where:

y_m	depth of flow, in m.

The **depth of excavation in rock**, h_r (m), is given by:

$$h_r = y_r + El_{te} - e_{te} - NA_{min}$$

where:

y_r	depth of the water in the part of the canal cut through rock, in m;
El_{te}	elevation of the land along the canal axis, in m;
e_{te}	thickness of the layer of soil, in m; and
NA_{min}	minimum water level in the reservoir, in m.

The **total flow cross-section through the canal**, A_{cn} (m^2), is given by:

$$A_{cn} = (1.5 \times y_m + 0.25 \times y_r) \times y_r$$

where:

y_m	depth of flow, in m; and
y_r	depth of the water in the part of the canal cut through rock, in m.

The **head loss in the headrace canal**, h_c (m), is given by:

$$h_c = \left[(L_{cn} - L_c) \times n^2 + L_c \times n_c^2\right] \times \frac{v_{cn}^2}{R_h^{4/3}}$$

where:

$$v_{cn} = \frac{Q_t}{A_{cn}}$$

$$R_h = \frac{A_{cn}}{B_{cn} + 2.06 \times y_r} \text{ v}$$

where:

n	Type of Lining
0.035	for canals cut through rock
0.025	for canals cut through soil
0.014	for canals lined with concrete
0.010	for canals lined with a clay blanket

where:

n	Manning's coefficient for the unlined section;
n_c	Manning's coefficient for the lined section;
L_{cn}	length of the canal, in m;
L_c	length of the lined section of the canal, in m;
v_{cn}	mean velocity of discharge in the canal, in m/s;
R_h	hydraulic radius, in m;
Q_t	total maximum turbine flow, in m^3/s;
A_{cn}	total flow cross-section of the canal, in m^2;
B_{cn}	width of the canal bottom, in m; and
y_r	depth of the water in the part of the canal cut through rock, in m.

The **slope of the canal bottom**, i (m/m), is given by:

$$i = \frac{h_c}{L_{cn}}$$

where:

h_c	head loss in the canal, in m; and
L_{cn}	length of the canal, in m.

For long headrace canals (over 3.0 km long), a forebay of volume V_{cg} (m^3) is included at the intake, given by:

$$V_{cg} = \frac{300 \times Q_t}{N_g}$$

where:

Q_t	total maximum turbine flow, in m^3/s; and
N_g	number of generating units in a plant.

Common Excavation (account .12.19.31.12.10)

The **volume of common excavation**, V_{tcn} (m^3), is given by:

$$V_{tcn} = (B_{cn} + 0.5 \times h_r + 1.5 \times h_t + 4) \times h_t \times L_{cn}$$

where:

B_{cn}	width of the canal bottom, in m;
h_r	depth of excavation in rock, in m;
h_t	depth of excavation in soil, in m; and
L_{cn}	length of the canal, in m.

The unit price of **common excavation** is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the canal. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves a favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower%.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.19.31.12.11)

The **volume of excavation in rock** , V_{rcn} (m^3), is given by:

$$V_{rcn} = (B_{cn} + 0.25 \times h_r) \times h_r \times L_{cn}$$

where:

B_{cn}	width of the canal bottom, in m;
h_r	depth of excavation in rock, in m; and
L_{cn}	length of the canal, in m.

The unit price of **excavation in rock** is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the canal. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves a favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account.12.19.31.14)

The **volume of concrete**, V_{ccn} (m^3), is given by:

$$V_{ccn} = \left[B_{cn} + 2.06 \times h_r + 4.0 + 3.61 \times (d_1 + y_t)\right] \times e_c \times L_c$$

where:

$$d_1 = NA_{max} - (El_{te} - e_{te}) + 2$$

where:

B_{cn}	width of the canal bottom, in m;
h_r	depth of excavation in rock, in m;
d_1	secondary variable, in m;
y_t	depth of the water in the part of the canal cut through soil; in m;
e_c	thickness of the concrete lining, in m;
L_c	length of the section of canal lined with concrete, in m;
NA_{max}	maximum level of the reservoir, in m;
El_{te}	elevation of the land along the canal axis, in m; and
e_{te}	thickness of the layer of soil, in m.

The **amounts of cement and reinforcement steel** are:

	cement (kg/m^3)	reinforcement steel (kg/m^3)
conventional concrete	275	50
shotcrete	300	70

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- conventional concrete: 234.00/m^3
- shotcrete: 128.00/m^3

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Canal excavated through soil (Case 3)

The canal is excavated through soil only when:

$$El_{cn} \geq El_{te} - e_{te}$$

where:

El_{cn}	elevation of the canal bottom;
El_{te}	elevation of the land along the canal axis; and
e_{te}	thickness of the layer of soil, in m.

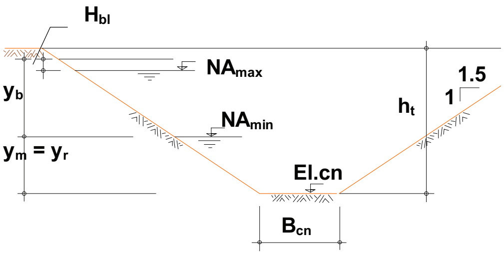


Fig. 5.7.6.03 – Typical cross-section of a headrace canal cut through soil (Case 3).

The **depth of the water in a canal cut through soil**, y_t (m), is given by:

$$y_t = y_m$$

where:

y_m	depth of flow, in m.

The **depth of excavation for the canal**, h_t (m), is given by:

$$h_t = El_{te} - El_{cn}$$

where:

El_{te}	elevation of the land along the canal axis, in m; and
El_{cn}	elevation of the canal bottom, in m.

The total flow cross-section of the canal, A_{cn} (m²), is given by:

$$A_{cn} = (B_{cn} + 1.5 \times y_t) \times y_t$$

where:

B_{cn}	width of the canal bottom, in m; and
y_t	depth of the water in the part of the canal cut through soil, in m.

The head loss in the headrace canal, h_c (m), is given by:

$$h_c = \left[(L_{cn} - L_c) \times n^2 + L_c \times n_c^2\right] \times \frac{v_{cn}^2}{R_h^{4/3}}$$

where:

$$v_{cn} = \frac{Q_t}{A_{cn}}$$

$$R_h = \frac{A_{cn}}{B_{cn} + 3.61 \times y_t}$$

n	Type of Lining
0.025	for canals cut through soil
0.014	for canals lined with concrete
0.010	for canals lined with a clay blanket

where:

n	Manning's coefficient for the unlined section;
n_c	Manning's coefficient for the lined section;
L_{cn}	length of the canal, in m.
L_c	length of the lined section of the canal, in m.
v_{cn}	mean velocity of discharge in the canal, in m/s;
R_h	hydraulic radius, in m;
Q_t	total turbine flow, in m³/s;
A_{cn}	total flow cross-section of the canal, in m²;
B_{cn}	width of the canal bottom, in m; and
y_t	depth of the water in the part of the canal cut through soil, in m.

The **slope of the canal bottom**, i (m/m), is given by:

$$i = \frac{h_c}{L_{cn}}$$

where:

h_c	head loss in the canal, in m; and
L_{cn}	length of the canal, in m.

For long headrace canals (over 3.0 km long), a forebay of volume V_{cg} (m^3) is included at the intake, given by:

$$V_{cg} = \frac{300 \times Q_t}{N_g}$$

where:

Q_t	total turbine flow, in m³/s; and
N_g	number of generating units in a plant.

Common Excavation (account .12.19.31.12.10)

The **volume of common excavation**, V_{tcn} (m^3), is given by:

$$V_{tcn} = (B_{cn} + 1.5 \times h_t) \times h_t \times L_{cn}$$

where:

B_{cn}	width of the canal bottom, in m;
h_t	depth of excavation in soil, in m; and
L_{cn}	length of the canal, in m.

The unit price of **common excavation** is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the canal. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account.12.19.31.14)

The **volume of concrete**, V_{ccn} (m^3), is given by:

$$V_{ccn} = \left[B_{cn} + 3.61 \times \left(d + y_t + 2\right)\right] \times e_c \times L_c$$

where:

B_{cn}	width of the canal bottom, in m;
d	maximum reservoir drawdown, in m;
y_t	depth of the water in the part of the canal cut through soil; in m;
e_c	thickness of the concrete lining, in m; and
L_c	length of the section of canal lined with concrete, in m.

The amounts of **cement and reinforcement steel** are:

	cement (kg/m³)	reinforcement steel (kg/m³)
conventional concrete	275	50
shotcrete	300	70

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- conventional concrete: 234.00/m^3
- shotcrete: 128.00/m^3

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

GRAVITY INTAKE (ACCOUNT .12.19.30)

This item applies for projects with penstocks, whether they be pressure penstocks or not, and with Pelton, Francis or Kaplan turbines with a steel spiral casing.

The intake recommended for this item is shown in Fig. 5.7.6.04.

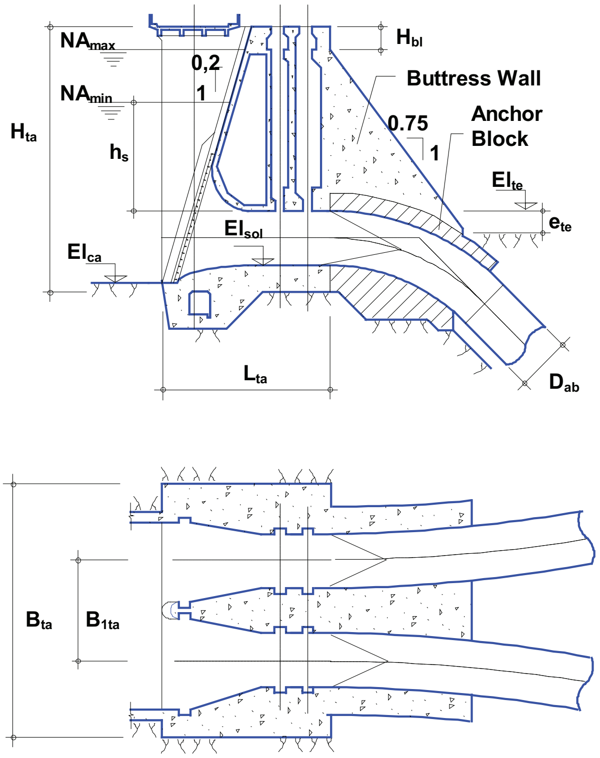


Fig 5.7.6.04 – Typical cross-section and plan of a gravity intake.

Basic Data

The main **information required for dimensioning** purposes is as follows:

- number of generating units, N_g, from item 5.7.2.;
- number of generating units per pressure penstock or tunnel, N_f, from item 5.7.2., when applicable;
- internal diameter of the conduit associated to the intake, D_{ab}, in m;
- maximum normal water level in the reservoir, NA_{max}, from item 4.4; and
- elevation of the intake sill, El_{sol}.

The main **information required for quantification** purposes is as follows:

- total maximum turbine flow, Q_t, in m³/s, from item 5.7.2.;
- mean elevation of the land in the structure area, El_{te}, in m; and
- mean thickness of the layer of soil in the intake area, e_{te}, in m.

Dimensions of the intake

The **number of openings in the intake**, N_{at}, is:

for pressure penstocks or tunnels: $N_{at} = \frac{N_g}{N_f}$

for intake penstocks: $N_{at} = 1$

where:

N_g	number of generating units; and
N_f	number of generating units per intake conduit.

The **height of the intake**, H_{ta} (m), is given by:

$$H_{ta} = NA_{max} - El_{sol} + H_{bl} + 2.5$$

where:

NA_{max}	maximum normal water level in the reservoir, in m;
H_{bl}	4.0 m – freeboard of the intake; and
El_{sol}	elevation of the intake sill, in m.

The **width of the block of a unit perpendicular to flow**, B_{1ta} (m), is given by:

$$B_{1ta} = 1.2 \times D_{ab} + 1.2$$

where:

D_{ab}	internal diameter of the intake conduit, in m.

The **total width**, B_{ta} (m), is given by:

$$B_{ta} = N_{at} \times B_{1ta} + 2 \times 2.0$$

where:

N_{at}	number of openings in the intake;
B_{1ta}	width of the block of a unit, in m; and
2.0	extra thickness for the end pillars, in m.

The **length of the intake at its base**, in the direction of flow, L_{ta} (m), is given by:

$$L_{ta} = 9.2 + 0.20 \times H_{ta}$$

where:

H_{ta}	height of the intake, in m; and
0.2	slope of the upstream face.

Common Excavation (account .12.19.30.12.10)

The volume of common excavation, V_{tta} (m^3), is given by:

$$V_{tta} = B_{ta} \times L_{ta} \times e_{te}$$

where:

B_{ta}	total width, in m;
L_{ta}	length of the intake at the foundations, in m; and
e_{te}	mean thickness of the layer of soil in the intake area, in m.

The unit price of **common excavation** is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the intake. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.19.30.12.11)

The volume of excavation in rock, V_{rta} (m^3), is given by:

$$V_{rta} = L_{ta} \times \left[El_{te} - e_{te} - (El_{sol} - 2.5) \right] \times B_{ta}$$

where:

B_{ta}	total width, in m;
L_{ta}	length of the intake at the foundations, in m;
e_{te}	mean thickness of the layer of soil in the intake area, in m;
El_{te}	mean elevation of the land in the intake area, in m; and
El_{sol}	elevation of the intake sill, in m.

The unit price of **excavation in rock** is R$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the intake. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.19.30.13)

The **area of foundation to be cleaned**, A_{lf} (m^2), is given by:

$$A_{lf} = B_{ta} \times L_{ta}$$

where:

B_{ta}	total width, in m; and
L_{ta}	length of the intake at the foundations, in m.

The **foundation treatment** entails a drainage line immediately downstream from the grout curtain, with the **total length of each grout hole**, L_{tf} (m), being given by:

$$L_{tf} = \frac{B_{ta}}{3.0} \times L_{1tf}$$

$$L_{1tf} = 1.5 \times (NA_{max} - El_{sol}) \leq 40m$$

where:

B_{ta}	total width of the intake, in m;
L_{1tf}	length of the grout holes, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
El_{sol}	elevation of the sill, in m;
3.0	space between the grout holes, in m.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m²
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.19.30.14)

The **volume of concrete for the intake**, V_{cta} (m^3), is given by:

$$V_{cta} = V_{ctp} + N_{at} \times V_{ctb} + V_{ctc}$$

where:

$$V_{ctp} = 2 \times (2.0 \times L_{ta} + 10.0) \times H_{ta}$$

$$V_{ctc} = 0.375 \times (H_{ta} - D_{ab} - 9.3)^2 \times B_{ta}$$

$$V_{ctb} = 1{,}3 \times e^z$$

$$z = (0.0460 - 0.00167 \times D_{ab}) \times (H_{ta} - 104.0) + 10.16$$

where:

V_{ctp}	volume of concrete for the end walls, in m³;
V_{ctb}	volume of concrete for the block of the unit, in m³;
V_{ctc}	volume of concrete for the downstream buttress wall, in m³;
L_{ta}	length of the intake at the foundations, in m;
z_i	parameter, in m³;
N_{at}	number of openings, in m;
D_{ab}	internal diameter of the intake conduit, in m; and
H_{ta}	height of the intake, in m.

The amounts of cement and reinforcement steel are:

	cement (kg/m³)	reinforcement steel (kg/m³)
external wall	200	40
block of the unit	300	60
buttress wall	200	20

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- external wall: 128.00/m³
- block of the unit: 174.00/m³
- buttress wall: 129.00/m³

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Emergency Gates (account .12.19.30.23.16)

The **acquisition cost of each emergency gate** for the intake, C_{cp} (R$), including the respective operating system and fixed and embedded parts – FOB cost excluding tranportation and insurance, assembly and testing and provisions for taxes payable, depending on the current tax regime – can be obtained from the expression below (or from Graph B.23, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

for $0.13 \leq z \leq 9.17$:

$$C_{cp} = -4.3986 \times z^2 + 124.79 \times z + 110.2$$

for $9.17 < z \leq 125.39$:

$$C_{cp} = -0.128 \times z^2 + 57.311 \times z + 369.83$$

where:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000} \qquad H_x = NA_{max} - El_{sol}$$

where:

z	parameter, in m^4;
B_{cp}	width of the gate, in m;
H_{cp}	height of the gate, in m;
H_x	maximum hydrostatic load on the gate sill, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
El_{sol}	elevation of the intake sill, in m.

The following percentages must be added to the FOB price:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for taxes and charges payable on the equipment.

Stoplogs (account .12.19.30.23.17)

The **acquisition cost of each stoplog** for the intake, C_{sl} (R$), – FOB cost – can be obtained from the expression below (or from Graph B.25, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil:

for $0.16 \leq z \leq 54.43$:

$$C_{sl} = 72.896 \times z^{0.716}$$

where: $z = \dfrac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$

where:

z	parameter, in m^4;
B_{cp}	width of the gate, in m;
H_{cp}	height of the gate, in m; and
H_x	maximum hydrostatic load on the gate sill, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

The **overall acquisition cost of fixed parts and parts embedded in the concrete** of the stoplogs for the inlet, C_{gpf} (R$), – FOB cost – is given by the expression below. The figures are valid for December 2006 and for projects anywhere in Brazil:

$$C_{gpf} = 2 \times N_{at} \times (H_{ta} - 1.0) \times 2084.80$$

where:

N_{at}	number of openings in the intake; and
H_{ta}	height of the intake, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

Crane (account .12.19.30.23.20)

The **acquisition cost of the gantry crane** for the intake, C_{pcr} (R$), – FOB cost – is given by the expression below (or from Graph B.27, annex B, as a function of the dimensions of the emergency fixed-wheel gate and the hydrostatic load on its sill). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

for $0.13 \leq z \leq 54.35$:

$$C_{pcr} = -0.71 \times z^2 + 97.3 \times z + 57.78$$

where: $z = \dfrac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$

where:

z	parameter, in m^4;
B_{cp}	width of the gate, in m;
H_{cp}	height of the gate, in m; and
H_x	maximum hydrostatic load on the bottom of the gate, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost.

Trash Racks and Trash Rack Cleaners (account .12.19.30.23.21)

The **overall acquisition cost of the set of trash racks** and respective embedded parts, C_{gr} (R$), – FOB cost – can be obtained from Graph B28, annex B, as a function of their dimensions or by the equivalent expression below. The figures are valid for December 2006 and for projects anywhere in Brazil:

valid for $2 \leq (B \times H) \leq 750$: $C_{gr} = 5.35 \times B \times H$

where:

B	width of the trash racks, in m;
H	height of the trash racks, in m;
1.0	velocity of discharge in the section with the trash racks, in m/s.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost.

Other Costs (account .12.19.30.17)

The cost of other services is estimated as being 2% of the total cost of the intake.

PROJECT WITH INTEGRAL INTAKE POWERHOUSE (ACCOUNT .12.19.30)

This item applies to projects equipped with Kaplan turbines with a semi-spiral casing made of concrete.

The intake recommended for this item can be seen in Fig. 5.8.6.04a.

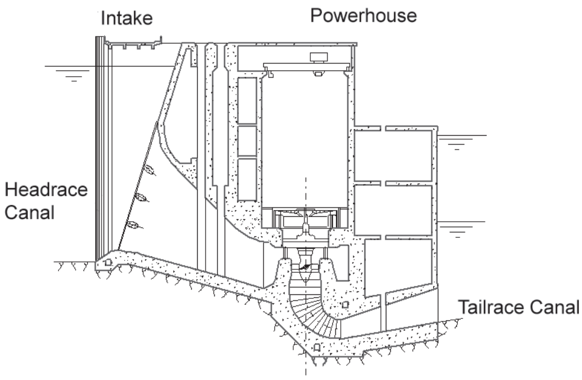


Fig. 5.7.6.04a – Typical cross-section of a project with an integral intake powerhouse.

Basic Data

The main **information required for dimensioning** purposes is as follows:

- number of generating units, N_g, from item 5.7.2.;
- maximum normal water level in the reservoir, NA_{max}, from item 4.4.;
- elevation of the intake sill, El_{sol}; and
- width of the block of a unit for the powerhouse, B_{1cf}, in m, from item 5.7.2.

The main **information required for quantification** purposes is as follows:

- total maximum turbine flow, Q_t, in m³/s, from item 5.7.2.;
- volume of soil excavated, V_{tta}, in m³;
- volume of surface rock excavation, V_{rta}, in m³;
- volume of concrete, V_{cta}, in m³;
- the area of foundation to be cleaned, A_{lf}, in m²;
- width of the emergency gate, L_{cp} in m;
- height of the emergency gate, H_{cp} in m;
- maximum hydrostatic load on the sill of the emergency gate, H_x in m; and
- height of the stoplog, H_{sl}, in m.

Dimensions of the intake

The **number of openings in the intake**, N_{at}, is given by:

$$N_{at} = N_g$$

where:

N_g	number of generating units.

The **height of the intake**, H_{ta} (m), is given by:

$$H_{ta} = NA_{max} - El_{sol} + H_{bl} + 2.5$$

where:

NA_{max}	maximum normal water level in the reservoir;
H_{bl}	4.0m, height of the intake freeboard, in m; and
El_{sol}	elevation of the intake sill, in m.

The **width of the block of a unit perpendicular to flow**, B_{1ta} (m), is given by:

$$B_{1ta} = B_{1cf}$$

where:

B_{1cf}	width of the block of a unit for the powerhouse, in m.

The **total width**, B_{ta} (m), is given by:

$$B_{ta} = N_g \times B_{1ta} + 2 \times 2.0$$

where:

N_g	number of generating units;
B_{1ta}	width of the block of a unit, in m; and
2.0	extra thickness for the end pillars, in m.

The lengthwise horizontal projection **of the base of the intake**, in the direction of flow, L_{ta} (m), is given by:

$$L_{ta} = 9.2 + 0.20 \times H_{ta}$$

where:

H_{ta}	height of the intake, in m; and
0.2	slope of the upstream face.

Common Excavation (account .12.19.30.12.10)

The volume of common excavation should be determined from the project design.

The unit price of **common excavation** is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the intake. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.19.30.12.11)

The volume of excavation in rock should be determined from the project design.

The unit price of **excavation in rock** is R$ 21.00/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the intake. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.19.30.13)

The area of foundation to be cleaned should be determined from the project design.

The **foundation treatment** should include a drainage line immediately downstream from a grout curtain, with the **length of each grout hole**, L_{tf} (m), given by:

$$L_{tf} = \frac{B_{ta}}{3.0} \times L_{1tf}$$

$$L_{1tf} = 1.5 \times (NA_{max} - El_{sol}) \leq 40m$$

where:

B_{ta}	total width of the intake, in m;
L_{1tf}	length of the grout holes, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
El_{sol}	elevation of the sill, in m; and
3.0	space between the grout holes, in m.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m^2
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.19.30.14)

The **volume of concrete for the intake**, V_{cta} (m^3), should be defined from the project design.

The amounts of cement and reinforcement steel are:

	cement (kg/m³)	reinforcement steel (kg/m³)
external wall	200	40
block of the unit	300	60

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- intake: 214.00/m³
- external walls: 128.00/m³

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Emergency Gates (account .12.19.30.23.16)

The **acquisition cost of each emergency gate** for the intake, C_{cp} (R$), including the respective operating system and fixed and embedded parts – FOB cost excluding tranportation and insurance, assembly and testing and provisions for taxes payable, depending on the current tax regime – can be obtained from the expression below (or from Graph B.23, annex B, as a function of its dimensions and maximum hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil

for: $0.13 \le z \le 9.17$: $C_{cp} = -4.3986 \times z^2 + 124.79 \times z + 110.2$ and

for: $9.17 \le z \le 125.39$: $C_{cp} = -0.128 \times z^2 + 57.311 \times z + 369.83$

where:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_{xe}}{1000} \qquad H_{xe} = \frac{NA_{max} - El_{td}}{3}$$

$$B_{cp} = \frac{0.88 \times D_{td}}{n_v} \qquad H_{cp} = 1.13 \times D_{td}$$

$$n_v = \text{int}\left(\frac{D_{td}}{4.5} + 0.9\right)$$

where:

z	parameter, in m^4;
B_{cp}	width of the gates in the diversion tunnel, in m;
H_{cp}	height of the gates in the diversion tunnel, in m;
H_x	maximum hydrostatic load on the sill of the diversion tunnel gate, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
D_{td}	internal diameter of the tunnels, in m;
n_v	number of openings in the inlet to each diversion tunnel; and
int(x)	function that returns the integer part of x.

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for taxes and charges payable on the equipment.

Stoplogs (account .12.19.30.23.17)

The **acquisition cost of each stoplog** for the intake, C_{sl} (R$), – FOB cost – can be obtained from the expression below (or from Graph B.25, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil:

valid for $0.16 \leq z \leq 54.43$: $C_{sl} = 72.896 \times z^{0.716}$

where:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000} \qquad H_x = NA_{max} - El_{td}$$

where:

z	parameter, in m^4;
B_{cp}	width of the gates in the diversion tunnel, in m;
H_{cp}	height of the gates in the diversion tunnel, in m; and
H_x	maximum hydrostatic load on the sill of the diversion tunnel gate, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

The **overall acquisition cost of fixed parts and parts embedded in the concrete** of the stoplogs for the inlet, C_{gpf} (R$), – FOB cost – is given by the expression below. The figures are valid for December 2006 and for projects anywhere in Brazil:

$$C_{gpf} = 2 \times N_v \times N_{td} \times H_{td} \times 2084.80$$

where: $H_{td} = 2.5 \times H_{cp}$

where:

N_v	number of openings in the inlet to each diversion tunnel;
N_{td}	number of diversion tunnels;
H_{td}	height of the inlet structure from the sill, in m; and
H_{cp}	height of the gates in the diversion tunnel, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

Crane (account .12.19.30.23.20)

The **acquisition cost of the gantry crane**, C_{pcr} (R$), for the intake – FOB cost – is given by the expression below (or from Graph B.27, annex B, as a function of the dimensions of the emergency fixed-wheel gate and the hydrostatic load on its sill). The figures are valid for December 2006 and for projects anywhere in Brazil:

for $0.13 \leq z \leq 54.35$: $C_{pcr} = -0.71 \times z^2 + 97.3 \times z + 57.78$

where: $z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$

where:

z	parameter, in m^4;
B_{cp}	width of the gate, in m;
H_{cp}	height of the gate, in m; and
H_x	maximum hydrostatic load on the bottom of the gate, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

Trash Racks and Trash Rack Cleaners (account .12.19.30.23.21)

The **overall acquisition cost of the set of trash racks** and respective embedded parts, C_{gr} (R$), – FOB cost – can be obtained from Graph B28, annex B, as a function of their dimensions or by the equivalent expression below. The figures are valid for December 2006 and for projects anywhere in Brazil:

Valid for $2 \leq (B \times H) \leq 750$: $C_{gr} = 5.35 \times B \times H$

where:

B	width of the trash racks, in m;
H	height of the trash racks, in m; and
1.0	velocity of discharge in the section with the trash racks, in m/s.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

Other Costs (account .12.19.30.17)

The cost of other services is estimated as being 2% of the total cost of the intake.

HEADRACE TUNNEL

The main **information required for dimensioning** purposes is as follows:

- maximum turbine flow, Q_t, in m^3/s, from item 5.7.2.;
- length of the tunnel, L_{ad}, in m;
- length of the section lined with structural concrete, L_c, in m;
- length of the section lined with shotcrete, L_{cp}, in m;

The **information required for quantification** purposes is as follows:

- geological conditions of the area crossed by the tunnels;
- maximum normal water level in the reservoir, NA_{max}, from item 4.4.;

- elevation of the intake sill, El_{sol}; and
- length of the section where the rock is to be treated, L_{pt}, in m.

Considerations and Recommendations

This text relates to headrace tunnels with a typical cross-section, as shown in Fig. 5.7.6.05.

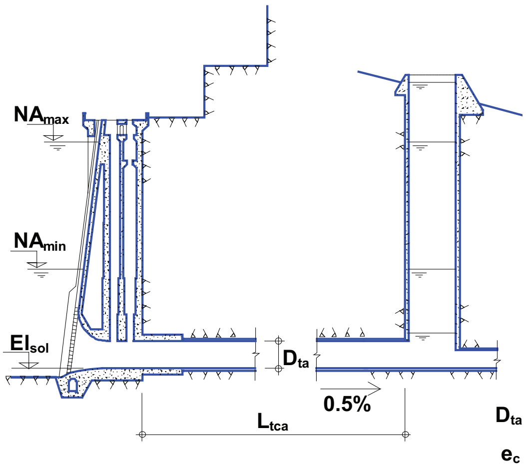


Fig. 5.7.6.05 Typical longitudinal section of a headrace tunnel.

When the diameter of the headrace tunnels exceeds the **maximum diameter**, the mean velocity of discharge must be increased to the limit, and if necessary the tunnel should be lined with shotcrete to raise the velocity limit.

When the diameter of the headrace tunnels is lower than the **minimum diameter**, the mean velocity can be reduced or the tunnel can be partially replaced by a canal or a surface penstock.

The **slope of the tunnel** should be no more than 0.005 m/m.

Diameter of the headrace tunnel

Initially, the **mean velocity of discharge**, v_{ad} (m/s), can be taken as:

$$v_{ad} = 0{,}8 \times v_{max}$$

where:

v_{max}	Type of Lining
2.2	for an unlined tunnel;
3.0	for a tunnel lined with shotcrete; and
4.5	for a tunnel lined with structural concrete.

where:

v_{max}	mean velocity of discharge limit for the tunnel, in m/s.

The **internal diameter of the headrace tunnel**, D_{ad} (m), is given by:

$$D_{ad} = \sqrt{\frac{Q_t}{0.8927 \times v_{ad}}}$$

where:

$$2.5 \leq D_{ad} \leq 15.0\,m$$

where:

Q_t	maximum turbine flow, in m^3/s; and
v_{ad}	mean velocity of discharge, in m/s.

The **total head loss along the tunnel**, h_a (m), is given by:

$$h_a = h_o + h_f$$

where:

$$h_o = \Sigma k_{oi} \times \frac{v_{ad}^2}{2 \times g}$$

$$h_f = 6.23 \times [(L_{ad} - L_c - L_{cp}) \times n^2 + L_c \times n_{cr}^2 + L_{cp} \times n_{cp}^2] \times \frac{v_{ad}^2}{D_{ad}^{4/3}}$$

For $(r_i/D_{ad}) < 5$

$$k_{oi} = 0.2147 \times \left(\frac{r_i}{D_{ad}}\right)^{-0.5718} \times (0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i)$$

For $(r_i/D_{ad}) \geq 5$

$$k_{oi} = \left(0.08 - 0.002 \times \left(\frac{r_i}{D_{ad}} - 5\right)\right) \times (0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i)$$

where:

h_o	head loss at bends, in m;
h_f	continuous head loss, in m;
v_{ad}	mean velocity of discharge in the tunnel, in m/s;
g	9.81 m/s^2 – acceleration due to gravity;
k_{oi}	coefficient of head loss at bends in the tunnel;
r_i	radius of curvature of the section, in meters;
L_{ad}	length of the tunnel, in m;
L_c	length of the section lined with structural concrete, in m;
L_{cp}	length of the section lined with shotcrete, in m;
n	0.035, Manning's coefficient for the unlined section;
n_{cr}	0.012, Manning's coefficient for the section lined with structural concrete;
n_{cp}	0.022, Manning's coefficient for the section lined with shotcrete;
D_{ad}	internal diameter of the tunnel, in m; and
θ	deflection of the tunnel axis, in radians.

Underground Excavation of Rock (account .12.19.32.12.12)

The **volume rock excavated underground** for the headrace tunnel, V_{sad} (m^3), is given by:

$$V_{sad} = V_{sae} + V_{san}$$

where:

$$V_{sae} = 0.8927 \times (D_{ad} + 2 \times e_c)^2 \times L_c$$

$$V_{san} = 0.8927 \times D_{ad}^2 \times (L_{ad} - L_c)$$

$$e_c = k_g \times [0.091 \times D_{ad}^{0,62} + 0.0034 \times (H - 30)]$$

$$H = NA_{max} - El_{sol}$$

k_g	geological conditions
1.0	good
1.4	average
2.0	poor or no information

where:

V_{sae}	volume of rock excavated underground for the section lined with structural concrete, in m^3;
V_{san}	volume of rock excavated underground for the unlined section and the section lined with shotcrete, in m^3;
D_{ad}	internal diameter of the tunnel, in m;
e_c	thickness of the structural concrete lining, in m;
k_g	coefficient to represent geological conditions;
H	hydrostatic load in the tunnel, in m;
e_{cp}	0.05 m, thickness of the shotcrete lining;
L_{ad}	length of the tunnel, in m;
L_c	length of the tunnel lined with structural concrete, in m;
L_{cp}	length of the tunnel lined with shotcrete, in m;
NA_{max}	maximum normal water level in the reservoir, in m; and
El_{sol}	elevation of the intake sill, in m.

The unit price of **underground excavation**, P_{us} (R$/m³), (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil, can be obtained from the expression below (or from Graph B33, annex B, as a function of the excavated area). This is the price per cubic meter measured using the excavation line of the project design and includes excavating, loading, transportation up to 1.5 km and unloading:

valid for $4 \leq A_{se} \leq 300$: $P_{us} = 474.08 \times A_{se}^{-0.3987}$

where: $A_{se} = 0.8927 \times D_{ad}^2$

where:

A_{se}	area of the excavated area, in m^2.

A careful appraisal should be made of the different situations where the tunnels form a representative part of the cost estimate, paying special attention to the geological conditions in the region and particularly long tunnels.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Rock Cleaning and Treatment (account .12.19.32.13)

The **total length of the rock anchors**, L_{tfp} (m), is given by:

$$L_{tfp} = 11.9 \times D_{ad} \times L_{pt}$$

where:

D_{ad}	internal diameter of the tunnel, in m; and
L_{pt}	length of the section where the rock is to be treated, in m.

The unit price of **rock anchors is** R$ 241.00/m (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per meter measured using the excavation line and includes the service per se and the supply of inputs and equipment.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.19.32.14)

The **volume of concrete**, V_{cad} (m^3), is given by:

$$V_{cad} = V_{cap} + V_{cae}$$

where:

$$V_{cap} = 2.57 \times D_{ad} \times L_{cp} \times e_{cp}$$

$$V_{cae} = 0.8927 \times \left[(D_{ad} + 2 \times e_c)^2 - D_{ad}^2 \right] \times L_c$$

where:

V_{cap}	volume of shotcrete, in m^3;
V_{cae}	volume of structural concrete for lining, in m^3;
D_{ad}	internal diameter of the tunnel, in m;
L_{cp}	length of the tunnel lined with shotcrete, in m;
e_{cp}	0.05, mean thickness of the shotcrete, in m;
e_c	thickness of the structural concrete lining, in m; and
L_c	length of tunnel lined with structural concrete, in m.

The amounts of cement and reinforcement steel are:

	cement (kg/m^3)	reinforcement steel (kg/m^3)
conventional concrete	250	50
shotcrete	300	70

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- conventional concrete: 234.00/m^3
- shotcrete: 378.00/m^3

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

SURGE TANK (ACCOUNT .12.19.33)

The use of surge tanks should fulfill the criteria set out in item 5.5.2.

Basic Data

The main **information required for dimensioning** this item can be obtained from the overall layout and item 4.4, Formulation of Cascade Options and 5.3 Energy Studies, as follows:

- mean velocity of discharge in the headrace tunnel, v_{ad}, in m/s, from item 5.7.6;
- length of the headrace tunnel, L_{ad}, in m, from item 5.7.6;
- diameter of the headrace tunnel, D_{ad}, in m, from item 5.7.6;
- head loss in the headrace tunnel, h_a, in m, from item 5.7.6;
- maximum normal water level in the reservoir, NA_{max}, from item 4.4, in m;
- minimum water level in the reservoir, NA_{min}, from item 5.3, in m;
- elevation of the center line of the turbine distributor, El_d, from item 5.7.2, in m;
- elevation of the intake sill, El_{sol}, in m;
- elevation of the land in the surge tank area, El_{te}, in m;
- mean thickness of the layer of soil in the surge tank area, e_{te} in m.

Considerations

This text relates to surge tanks with a typical cross-section, as shown in Fig. 5.7.6.06 or 5.7.6.07.

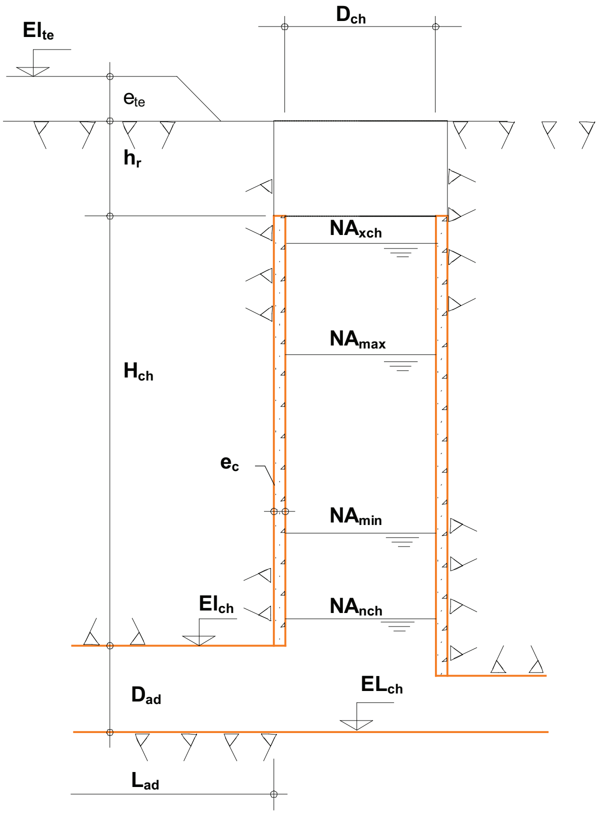


Fig. 5.7.6.06 – Typical cross-section of a surge tank.

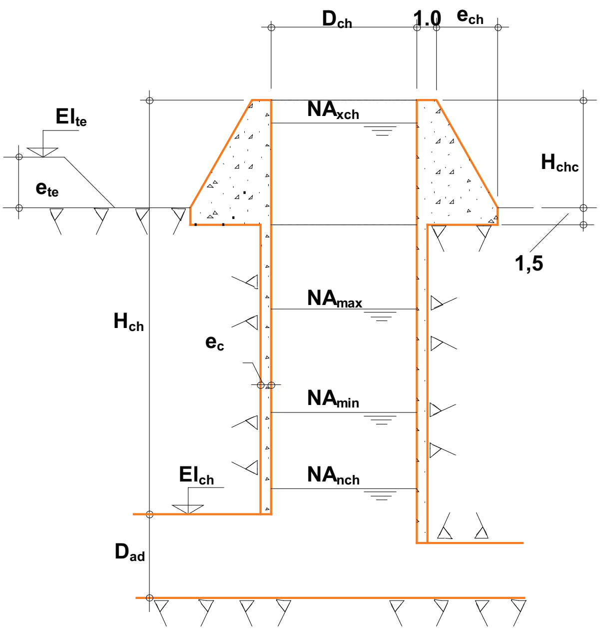


Fig. 5.7.6.07 – Typical cross-section of a surge tank with maximum oscillation exceeding the top of the bedrock.

Surge Tank Dimensions

The **cross-sectional area of the surge tank**, A_{ch} (m^2), is given by:

$$A_{ch} = 1.25 \times \frac{v_{ad}^2}{2 \times g} \times \frac{L_{ad} \times A_{ad}}{(H_d - h_e - h_a) \times (h_e + h_a)}$$

where:

$$A_{ad} = 0.8927 \times D_{ad}^2 \qquad H_d = NA_{min} - El_d$$

$$h_e = 0.20 \times \frac{v_{ad}^2}{2 \times g}$$

where:

v_{ad}	mean velocity of flow in the headrace tunnel, in m/s;
g	9.81 m/s^2, acceleration due to gravity;
L_{ad}	length of the headrace tunnel, in m;
A_{ad}	cross-sectional area of the headrace tunnel, in m^2;
H_d	minimum static head, in m;
h_e	head loss at the intake inlet, in m;
h_a	head loss in the headrace tunnel, in m;
D_{ad}	diameter of the headrace tunnel, in m;
NA_{min}	minimum water level in the reservoir, in m; and
El_d	elevation of the center line of the turbine distributor, in m.

The **internal diameter of the surge tank**, D_{ch} (m), is given by:

$$D_{ch} = \sqrt{\frac{4 \times A_{ch}}{\pi}}$$

where:

A_{ch}	cross-sectional area of the surge tank, in m^2.

The **maximum oscillation in the surge tank**, Y_{max} (m), is given by:

$$Y_{max} = v_{ad} \times \sqrt{\frac{L_{ad} \times A_{ad}}{g \times A_{ch}}}$$

where:

v_{ad}	mean velocity of flow in the headrace tunnel, in m/s;
L_{ad}	length of the headrace tunnel, in m;
A_{ad}	cross-sectional area of the headrace tunnel, in m^2;
g	9.81 m/s^2, acceleration due to gravity; and
A_{ch}	cross-sectional area of the surge tank, in m^2.

The **maximum and minimum water levels in the surge tank**, NA_{xch} and NA_{nch}, are given by:

$$NA_{xch} = NA_{max} - \frac{2}{3} \times (h_e + h_a) + Y_{max}$$

$$NA_{nch} = NA_{min} + 2 \times (h_e + h_a) - Y_{max}$$

where:

NA_{max}	maximum normal water level in the reservoir, in m;
h_e	head loss at the intake inlet, in m;
h_a	head loss in the headrace tunnel, in m;
Y_{max}	maximum oscillation in the surge tank, in m; and
NA_{min}	minimum water level in the reservoir, in m.

The **elevation of the bottom of the surge tank**, El_{ch}, is given by:

$$El_{ch} = El_{sol} - 0.005 \times L_{ad} + D_{ad} \quad \leq \; NA_{nch} - 1{,}0$$

where:

El_{sol}	elevation of the intake sill, in m;
L_{ad}	length of the headrace tunnel, in m;
D_{ad}	diameter of the headrace tunnel, in m; and
NA_{nch}	minimum water level in the surge tank, in m.

The **height of the surge tank**, H_{ch} (m), is given by:

$$H_{ch} = NA_{xch} + 1.0 - El_{ch}$$

where:

NA_{xch}	maximum water level in the surge tank, in m;
El_{ch}	elevation of the bottom of the surge tank, in m; and
1.0	freeboard, in m.

When the maximum water level in the surge tank exceeds the top of the bedrock (Fig. 5.7.6.07), the **height of the surge tank above the top of the bedrock**, H_{chc} (m), is given by:

For $NA_{xch} + 1.0 \; > \; El_{te} - e_{te} : \; H_{chc} = NA_{xch} + 1.0 - (El_{te} - e_{te})$

where:

NA_{xch}	maximum water level in the surge tank;
El_{te}	elevation of the land along the surge tank axis, in m; and
e_{te}	mean thickness of the layer of soil in the surge tank area, in m.

In the same situation, the **extra thickness of concrete at the base of the buttress wall**, e_{ch} (m), is given by:

$$e_{ch} = \frac{D_{ch}}{2}\left(\sqrt{\frac{\sigma_c + p_s}{\sigma_c - p_s}} - 1\right)$$

where:

$$p_s = \frac{NA_{xch} - (El_{te} - e_{te})}{10}$$

where:

D_{ch}	internal diameter of the surge tank, in m;
σ_c	10 kg/cm^2 – tensile stress capacity of concrete;
p_s	working pressure, in kg/cm^2;
NA_{xch}	maximum water level in the surge tank, in m;
El_{te}	elevation of the land along the surge tank axis, in m; and
e_{te}	mean thickness of the layer of soil in the surge tank area, in m.

When the maximum water level in the surge tank does not exceed the top of the bedrock (Fig. 5.8.6.06), the **mean depth of rock above the top of the surge tank**, h_r (m), is given by:

For $NA_{xch} + 1.0 \le El_{te} - e_{te}$: $h_r = El_{te} - e_{te} - (NA_{xch} + 1.0)$

where:

El_{te}	elevation of the land in the surge tank area;
e_{te}	thickness of the layer of soil in the surge tank area, in m; and
NA_{xch}	maximum water level in the surge tank.

In either case, the **thickness of the concrete lining**, e_c (m), is given by:

$$e_c = 0.00274 \times D_{ch}^2 + 0.018 \times D_{ch} + 0.10$$

where:

D_{ch}	internal diameter of the surge tank, in m.

Common Excavation (account .12.19.33.12.10)

The **volume of common excavation**, V_{tch} (m^3), is given by one of the expressions below:

$$\text{if } H_{chc} > 0: \; V_{tch} = \frac{\pi}{4} \times \left[D_{ch} + 2 \times (1 + e_{ch} + e_{te})\right]^2 \times e_{te}$$

$$\text{if } H_{chc} \le 0: \; V_{tch} = \frac{\pi}{4} \times \left[D_{ch} + 2 \times (e_c + e_{te})\right]^2 \times e_{te}$$

where:

D_{ch}	internal diameter of the surge tank, in m;
e_{ch}	extra thickness of concrete at the base of the buttress wall, in m;
e_{te}	thickness of the layer of soil in the surge tank area, in m; and
e_c	thickness of the concrete lining, in m.

The unit price of **common excavation** is R$ 7.60/m^3 (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the surge tank. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Underground Excavation of Rock (account .12.19.33.12.12)

The **volume of rock excavated underground**, V_{sch} (m^3), is given by one of the expressions below:

if $H_{chc} > 0$:

$$V_{sch} = \frac{\pi}{4} \times (D_{ch} + 2 \times e_c)^2 \times (H_{ch} - H_{chc}) + \pi \times (D_{ch} + e_c + e_{ch} + 1) \times (e_{ch} + 1 - e_c) \times 1.5$$

if $H_{chc} \leq 0$:

$$V_{sch} = \frac{\pi}{4} \times (D_{ch} + 2 \times e_c)^2 \times (h_r + H_{ch})$$

where:

D_{ch}	internal diameter of the surge tank, in m;
e_c	thickness of the concrete lining, in m;
H_{ch}	height of the surge tank, in m;
H_{chc}	height of the surge tank above the top of the bedrock, in m;
e_{ch}	extra thickness of concrete at the base of the buttress wall, in m; and
h_r	mean depth of rock above the top of the surge tank, in m.

The unit price of **underground excavation**, P_{us} (R\$/m^3), (from December 2006 database), including excavating, loading, transportation up to 1.5 km and unloading, can be obtained from the expression below (or from Graph B33, annex B, as a function of the excavated area) and can be used for projects in the south, southeast, central west and northeast regions of Brazil:

valid for $4 \leq A_{se} \leq 300$: $P_{us} = 474.08 \times A_{se}^{-0.3987}$

where: $A_{se} = \frac{\pi}{4} \times (D_{ch} + 2 \times e_c)^2$

where:

A_{se}	excavation cross-sectional area, in m^2;
D_{ch}	internal diameter of the surge tank, in m; and
e_c	thickness of the concrete lining, in m.

A careful appraisal should be made of the different situations where the tunnels form a representative part of the cost estimate, paying special attention to the geological conditions in the region and particularly long tunnels.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.19.33.13)

The **area of foundation to be cleaned**, A_{lf} (m^2), is given by one of the expressions below:

if $H_{chc} > 0$:

$$A_{lf} = \pi \times (D_{ch} + 2 \times e_c) \times (H_{ch} - H_{chc}) + \pi \times (D_{ch} + e_c + e_{ch} + 1) \times (e_{ch} + 1 - e_c)$$

if $H_{chc} \leq 0$:

$$A_{lf} = \pi \times (D_{ch} + 2 \times e_c) \times H_{ch}$$

where:

D_{ch}	internal diameter of the surge tank, in m;
e_c	thickness of the concrete lining, in m;
H_{ch}	height of the surge tank, in m;
H_{chc}	height of the surge tank above the top of the bedrock, in m; and
e_{ch}	extra thickness of concrete at the base of the buttress wall, in m.

The **length of contact grouting and consolidation grouting holes**, L_{tf} (m), is given by one of the expressions below:

if $H_{chc} > 0$: $L_{tf} = 2 \times \pi \times (D_{ch} + 2 \times e_c) \times (H_{ch} - H_{chc})$

if $H_{chc} \leq 0$: $L_{tf} = 2 \times \pi \times (D_{ch} + 2 \times e_c) \times H_{ch}$

where:

D_{ch}	internal diameter of the surge tank, in m;
e_c	thickness of the concrete lining, in m;
H_{ch}	height of the surge tank, in m; and
H_{chc}	height of the surge tank above the top of the bedrock, in m.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m^2
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.19.33.14)

The **volumes of structural concrete**, V_{cch} (m^3), and **shotcrete**, V_{cp} (m^3), are given by one of the expressions below:

if $H_{chc} > 0$:

$$V_{cch} = \left(\frac{A_1 + A_2}{2} - A_{ch}\right) \times H_{chc} + (A_0 - A_{ch}) \times H_{ch} + (A_2 - A_0) \times 1.5$$

where:

$$A_0 = \frac{\pi}{4} \times (D_{ch} + 2 \times e_c)^2$$

$$A_1 = \frac{\pi}{4} \times (D_{ch} + 2.0)^2$$

$$A_2 = \frac{\pi}{4} \times (D_{ch} + 2.0 + 2 \times e_{ch})^2$$

$$V_{cp} = 0$$

if $H_{chc} \leq 0$:

$$V_{cch} = \pi \times (D_{ch} + e_c) \times e_c \times H_{ch}$$

$$V_{cp} = \pi \times (D_{ch} + e_c) \times e_c \times h_r$$

The recommended thickness of shotcrete lining (e_c) is 0.10 m.

where:

A_i	secondary areas, in m²;
A_{ch}	cross-sectional area of the surge tank, in m²;
D_{ch}	internal diameter of the surge tank, in m;
H_{chc}	height of the surge tank above the top of the bedrock, in m;
e_{ch}	extra thickness of concrete at the base of the buttress wall, in m;
e_c	thickness of the concrete lining, in m;
H_{ch}	height of the surge tank, in m; and
h_r	mean depth of rock above the top of the surge tank, in m.

The amounts of cement and reinforcement steel are:

	cement (kg/m³)	reinforcement steel (kg/m³)
conventional concrete	250	50
shotcrete	300	70

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- conventional concrete: 214.00/m³
- shotcrete: 378.00/m³

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

SURFACE PRESSURE PENSTOCKS (ACCOUNT .12.19.34)

Basic Data

The main **information required for dimensioning** this item can be obtained from the overall layout of the project and the energy studies, as follows:

- horizontal projection of each section, L_{h1}, L_{h2} and L_h, in m;
- slope of each section, α_1, α_2 and α_3, in degrees;
- length of the headrace tunnel, L_{ad}, in m, when applicable;
- capacity of one generating unit, P_1 in MW, from item 5.7.2.;
- coefficient k, product of the yield of the generating unit by acceleration due to gravity, from item 2.1.;
- number of generating units, N_g, from item 5.7.2.;
- number of generating units per pressure penstock, N_f;
- maximum normal water level in the reservoir, NA_{max}, from item 4.4.;
- minimum water level in the reservoir, NA_{min}, from item 5.3.;
- lowest elevation of the land in the intake area, El_{ten};
- mean thickness of the layer of soil in the pressure penstock area, e_{te} in m;
- normal water level in the tailrace canal, NA_{fu}, from item 4.4.;
- elevation of the center line of the turbine distributor, El_d, from item 5.7.2.;
- maximum net head, H_1, in m, from item 5.3.;
- head loss in the headrace canal, h_c, in m, from item 5.7.6., when applicable;
- head loss in the headrace tunnel, h_a, in m, from item 5.7.6., when applicable; and
- diameter of the inlet to the spiral casing, A, in m, from item 5.7.2., when applicable.

The main **information required for quantification** purposes is as follows:

- maximum water level in the tailrace canal, NA_{xfu}, from item 5.1.2.;
- maximum water level in the surge tank, NA_{xch}, from item 5.7.6., when applicable;
- length of the section through a tunnel, L_t, in m, when applicable;
- volume of common excavation, V_{ttf}, in m^3; and
- volume of surface rock excavation, V_{rtf}, in m^3.

Considerations and Recommendations

A **section** is understood as being the stretch between two deflections in the vertical plane, counting from the outlet of the intake, point 0, to the inlet of the powerhouse, point 4, as shown in Figure 5.7.6.08.

For methodological reasons, the **length of section 3**, L_3, is obtained from the other data.

In this text, the term **bifurcation** is also used for sections that split into three and for separators.

The **horizontal projection** of sections 1 and 4, L_{h1} and L_{h4}, must be long enough to match the curve.

$$L_{h1} \geq 4 \times D_b \times tg\frac{\alpha_2 - \alpha_1}{2} \qquad\qquad L_{h4} \geq 4 \times D_b \times tg\frac{\alpha_3}{2}$$

where:

D_b	internal diameter of the pressure penstock defined below, in m; and
α_i	slope of section i, in degrees.

For **penstocks with just three sections**, the following must be true:

$\alpha_3 = \alpha_2$ and $L_{h2} = 1.0$ m

where:

L_{h2}	horizontal projection of section 2, in m; and
α_i	slope of section i, in degrees.

Internal diameter and mean velocity of discharge

The **number of pressure penstocks**, N_t, is given by:

$$N_t = \frac{N_g}{N_f}$$

where:

N_g	number of generating units; and
N_f	number of generating units per pressure penstock.

The **internal diameter of the pressure penstock**, D_b (m), can be determined by:

$$D_b = 14.2 \times \frac{(N_f \times P_1)^{0.43}}{H_{b1}^{0.65}}$$

where: $H_{b1} = NA_{max} - NA_{fu}$

where:

N_f	number of generating units per pressure penstock;
P_1	capacity of one generating unit, in MW;
H_{b1}	maximum gross head, in m;
NA_{max}	maximum normal water level in the reservoir; and
NA_{fu}	normal water level in the tailrace canal.

The **maximum turbine flow of each turbine**, Q_1 (m³/s), is given by:

$$Q_1 = \frac{10^6 \times P_1}{k \times H_1}$$

where:

$$k = \rho \times g \times \eta_{t1} \times \eta_{g1} \qquad P_2 = \frac{P_1}{f_p}$$

for Francis turbines:

$$\eta_{t1} = 0.856 \times Q_1^{0.013} \qquad \eta_{g1} = 0.92 \times P_2^{0.01}$$

for Pelton and Kaplan turbines:

$\eta_{t1} = 0.96$ $\qquad$ $\eta_{g1} = 0.98$

where:

P_1	capacity of one generating unit, in MW;
k	coefficient;
H_1	maximum net head, in m;
ρ	1000 kg/m³ – specific mass of water;
η_{t1}	turbine output for maximum net head;
η_{g1}	generator output for maximum net head;

g	9.81 m/s² – acceleration due to gravity;
P_2	capacity of one generator, in MVA; and
f_p	capacity factor.

The **maximum flow through each pressure penstock**, Q_{1f} (m^3/s), is given by:

$$Q_{1f} = N_f \times Q_1$$

where:

N_f	number of generating units per pressure penstock; and
Q_1	maximum turbine flow of each turbine, in m^3/s.

The **mean velocity of discharge** through the pressure penstock, v_b (m/s), is given by:

$$v_b = \frac{4 \times Q_{1f}}{\pi \times D_b^2} \leq 7 m/s$$

where:

Q_{1f}	maximum flow through each pressure penstock, in m^3/s; and
D_b	internal diameter of the pressure penstock, in m.

If this restriction is not fulfilled, the velocity limit should be adopted and the diameter should be recalculated using:

$$D_b = \sqrt{\frac{4}{\pi} \times \frac{Q_{1f}}{7}}$$

where:

Q_{1f}	maximum flow through each pressure penstock, in m^3/s.

The **internal diameter of the pressure penstock after a bifurcation** or **the lateral diameter after the separators**, D_{1b} (m), can be determined by:

$$D_{1b} = \frac{D_b}{N_f^{3/8}}$$

where:

D_b	internal diameter of the pressure penstock, in m; and
N_f	number of generating units per pressure penstock.

Profile of the pressure penstock

The **elevation of points 0 to 4**, El_0 to El_4, is given by:

$$El_0 = El_{sol} - i \times L_{ad} + \frac{D_b}{2}$$

$$El_1 = El_0 - L_{h1} \times tg\alpha_1$$

$$El_2 = El_1 - L_{h2} \times tg\alpha_2$$

$$El_3 = E_4 = E_d$$

where:

$$El_{sol} = 0.5 \times int\left(\frac{NA_{min} - h_c - h_s - H_{cp}}{0.5}\right)$$

$$El_{sol} \leq 0.5 \times int\left(\frac{El_{ten} - e_{te}}{0.5}\right) + 1.0$$

$$h_s = 0.8 \times v_{cp} \times \sqrt{H_{cp}}$$

$$v_{cp} = \frac{Q_{1f}}{B_{cp} \times H_{cp}}$$

for projects with no headrace tunnel:

$B_{cp} = D_b$ $\qquad$ $H_{cp} = D_b$

for projects with a headrace tunnel:

$B_{cp} = D_{ad}$ $\qquad$ $H_{cp} = D_{ad}$

where:

El_{sol}	elevation of the intake sill, in m;
L_{ad}	length of the headrace tunnel, in m;
D_b	internal diameter of the pressure penstock, in m;
L_{hi}	horizontal projection of section i, in m;
α_i	slope of section i, in degrees;
i	slope of the headrace tunnel, in degrees;
El_d	elevation of the center line of the turbine distributor, in m;
NA_{min}	normal minimum water level in the reservoir, in m;
h_c	head loss in the headrace canal, in m, when applicable;
h_s	minimum submergence of the intake (Gordon, 1970), in m;
H_{cp}	height of the intake gate, in m;
El_{ten}	lowest elevation of the land in the intake area, in m;
e_{te}	mean thickness of the layer of soil in the pressure penstock area, in m;
v_{cp}	velocity of discharge at the intake gate, in m/s;
Q_{1f}	maximum flow through each pressure penstock, in m^3/s;
B_{cp}	width of the intake gate, in m;
Q_t	total maximum turbine flow, in m^3/s; and
D_{ad}	internal diameter of the headrace tunnel, in m.

The **horizontal projection of section 3**, L_{h3} (m), is given by:

$$L_{h3} = \frac{El_2 - El_3}{tg\alpha_3}$$

where:

El_i	elevation of point i, in m; and
α_3	slope of section 3, in degrees.

The length of the penstock is determined from its profile.

The **length of each section**, L_i (m), of penstock is given by:

$$L_i = \frac{L_h}{\cos \alpha_i}, \; i = 1, 2, 3$$

where:

L_{hi}	horizontal projection of section i, in m; and
α_i	slope of section i, in degrees.

The **total length** of the penstock, L_b (m), is given by:

$$L_b = L_1 + L_2 + L_3 + L_{h4}$$

where:

L_{h4}	horizontal projection of section 4, in m; and
L_i	length of section i, in m.

The **horizontal projection** of the penstock, L_{hb} (m), is given by:

$$L_{hb} = L_{h1} + L_{h2} + L_{h3} + L_{h4}$$

where:

L_{hi}	the horizontal projection of section i, in m.

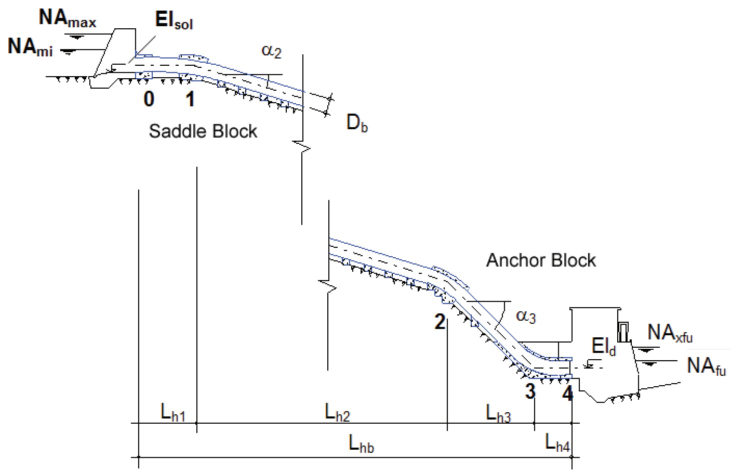


Fig. 5.7.6.08 – Schematic profile of a pressure penstock with four sections.

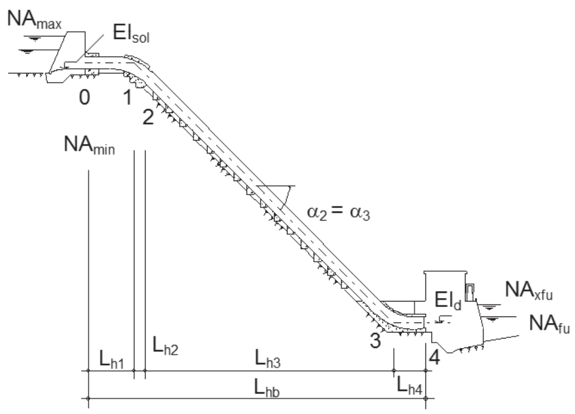


Fig. 5.7.6.09 – Schematic profile of a pressure penstock with three sections.

Surge Pressure

Maximum surge pressure due to water hammer, h_{sx} (m), is given by:

$$h_{sx} = \frac{2 \times L_b \times v_b}{g \times T_c} \leq 0.30 \times H_d$$

where: $H_d = NA_{max} - El_d$

T_c	For
6 s	short penstocks ($L_b \leq 3 \times H_{b1}$)
10 s	long penstocks ($L_b > 3 \times H_{b1}$)

where:

L_b	total length of the penstock, in m;
v_b	mean velocity of flow in the penstock, in m/s;
g	9.81 m/s^2 – acceleration due to gravity;
T_c	time taken to close the valve or the turbine distributor, in s;
H_d	static head, in m;
NA_{max}	maximum normal water level in the reservoir; and
El_d	elevation of the center line of the turbine distributor, in m.

When the maximum surge pressure exceeds the limit, the diameter must be redimensioned using:

$$D_b = \sqrt{\frac{4}{\pi} \times \frac{Q_{1f}}{v_b}}$$

with:

$$v_b = \frac{0.30 \times H_{b1} \times g \times T_c}{2 \times L_b}$$

where:

Q_{1f}	maximum flow through each pressure penstock, in m^3/s;
v_b	mean velocity of flow through the pressure penstock, in m/s;
H_{b1}	maximum gross head, in m.
g	9.81 m/s^2 – acceleration due to gravity;
T_c	time taken to close the valve or distributor, in s; and
L_b	total length of the pressure penstock, in m.

The **energy gradient** for maximum surge pressure, i_s, is given by:

$$i_s = \frac{h_{sx}}{L_{hb}}$$

where:

h_{sx}	maximum surge pressure due to water hammer, in m; and
L_{hb}	horizontal projection of the pressure penstock, in m.

The whole penstock should be below the minimum energy gradient. This can be checked at critical points 1 and 2:

$$El_1 \le NA_{min} - L_{h1} \times i_s - \frac{D_b}{2}$$

$$El_2 \le NA_{min} - (L_{h1} + L_{h2}) \times i_s - \frac{D_b}{2}$$

where:

El_i	elevation of points 1 and 2, in m;
NA_{min}	minimum water level in the reservoir, in m;
L_{hi}	horizontal projection of section i, in m;
i_s	energy gradient for maximum surge pressure, in m/m; and
D_b	internal diameter of the pressure penstock, in m.

If any one of these criteria is not fulfilled, the elevation of the critical point should be lowered and the slope of the adjacent sections should be adapted.

Head Loss

The **total head loss**, h_p (m), from the intake to the pressure penstock, is given by:

$$h_p = h_e + h_a + h_o + h_r + h_b + h_v + h_f$$

where:

$$h_o = \sum k_{oi} \times \frac{v_b^2}{2 \times g}$$

$$h_r = 0.10 \times \frac{(v_a - v_b)^2}{2 \times g}$$

$$h_b = 0.10 \times \frac{v_b^2}{2 \times g}$$

$$h_f = 6.35 \times L_b \times \frac{n^2 \times v_b^2}{D_b^{4/3}}$$

$$v_a = \frac{4}{\pi} \times \frac{Q_1}{A^2}$$

For $(r_i/D_b)<5$

$$k_{oi} = 0.2147 \times \left(\frac{r_i}{D_b}\right)^{-0.5718} \times \left(0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i\right)$$

For $(r_i/ D_b)\geq 5$

$$k_{oi} = \left(0.08 - 0.002 \times \left(\frac{r_i}{D_b} - 5\right)\right) \times \left(0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i\right)$$

$$\delta_i = \text{abs}(\alpha_{i+1} - \alpha_i),\ i = 1, 2, 3$$

for projects with no headrace tunnel: $h_e = 0.20 \times \frac{v_b^2}{2 \times g}$

for projects with a headrace tunnel: $h_e = 0.20 \times \frac{v_{ad}^2}{2 \times g}$

with a butterfly valve at the beginning of the pressure penstock: $h_v = 0.20 \times \frac{v_b^2}{2 \times g}$

with a butterfly valve at the end of the pressure penstock: $h_v = 0.20 \times \frac{v_a^2}{2 \times g}$

with a spherical valve: $h_v = 0.03 \times \frac{v_a^2}{2 \times g}$

where:

h_e	head loss at the intake, in m;
h_a	head loss in the headrace tunnel, in m, when applicable;
h_o	head loss at the bends, in m;

h_r	head loss where the diameter is reduced, in m;
h_b	head loss at the bifurcation, in m, when applicable;
h_v	head loss at the valve, in m, when applicable;
h_f	continuous head loss, in m;
k_o	coefficient for head loss at bends;
v_b	mean velocity of flow in the penstock, in m/s;
g	9.81 m/s² – acceleration due to gravity;
L_b	total length of the penstock, in m;
n	0.010 – Manning's coefficient for steel;
D_b	internal diameter of the pressure penstock, in m;
r_i	radius of curvature of the section, in m;
θ	deflection of the tunnel axis, in radians.
Q_1	maximum turbine flow of each turbine, in m³/s;
v_a	mean velocity of flow at the inlet to the spiral casing, in m/s;
A	diameter of the inlet to the spiral casing, in m;
δ_i	angle of vertical deflection at points 0 to 3 of the penstock, in degrees;
α_i	slope of section i, in degrees;
v_{ad}	mean velocity of flow in the headrace tunnel, in m/s; and
abs(x)	function that returns the absolute value of x.

Common Excavation (account .12.19.34.12.10)

The volume of **common excavation**, V_{ttf} (m³), should be determined from the project design.

The unit price of **common excavation** is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the penstock. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.19.34.12.11)

The **volume of excavation in rock**, V_{rtf} (m³), should be determined from the project design.

The unit price of **excavation in rock** is R$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the penstock. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Underground Excavation of Rock (account .12.19.34.12.12)

The **volume of rock excavated underground** for pressure penstocks, V_{stf} (m^3), is given by:

$$V_{stf} = V_{sb} + V_{st}$$

where:

$$V_{sb} = N_t \times 4 \times (D_b + 2 \times e_c)^3$$

$$V_{st} = N_t \times \frac{\pi}{4} \times [(D_b + 2 \times e_c)^2 \times L_t]$$

where:

V_{sb}	volume of rock excavated underground for the butterfly valve housing, in m^3, when installed in a separate cavity;
V_{st}	volume of rock excavated underground for tunnels, in m^3, when applicable;
N_t	number of pressure penstocks;
D_b	internal diameter of the pressure penstock, in m;
e_c	thickness of the concrete lining of the section through a tunnel, in m, as defined below; and
L_t	length of the section through a tunnel, in m.

The unit price of **underground excavation**, P_{us} (R\$/m^3), (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil, can be obtained from the expression below (or from Graph B33, annex B, as a function of the excavated area). This is the price per cubic meter measured using the excavation line of the project design and includes excavating, loading, transportation up to 1.5 km and unloading:

valid for $4 \leq A_{se} \leq 300$: $P_{us} = 474.08 \times A_{se}^{-0.3987}$

where: $A_{se} = \frac{\pi}{4} \times (D_b + 2 \times e_c)^2$

where:

A_{se}	area of the excavated area, in m^2;
D_b	internal diameter of the pressure penstock, in m; and
e_c	thickness of the concrete lining of the section through a tunnel, in m, as defined below.

A careful appraisal should be made of the different situations where the tunnels form a representative part of the cost estimate, paying special attention to the geological conditions in the region and particularly long tunnels.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.19.34.13)

The **area of foundation to be cleaned**, A_{lf} (m^2), is given by:

$$A_{lf} = A_{ls} + A_{lb} + A_{ln} + A_{la} + A_{lt}$$

where:

$$A_{lb} = N_t \times 4.2 \times D_b \times (D_b + 1) \times \text{sen}(\alpha_3 - \alpha_2)$$

$$A_{ln} = N_t \times \left[(4.5 \times D_b + 1.5) \times \tan\frac{\alpha_3}{2} + d_2 \right] \times (D_b + 2)$$

$$A_{la} = N_t \times N_a \times D_b^2$$

$$A_{lt} = N_t \times \pi \times (D_b + 2 \times e_c) \times L_t$$

$$d_2 = \frac{NA_{xcf} - El.d + \frac{D_b}{2} + 2.5}{\tan\alpha_3}$$

for $L_{h1} \leq 1{,}7 \times D_b$ e $L_t = 0$: $A_{ls} = N_t \times \left(2.1 \times D_b^2 + 3 \times D_b + 0.9\right)$

for $L_{h1} > 1{,}7 \times D_b$ e $L_t = 0$: $A_{ls} = N_t \times 4.2 \times D_b \times (D_b + 1) \times \mathrm{sen}(\alpha_2 - \alpha_1)$

where:

A_{ls}	the area of foundation to be cleaned for the upper anchor block, in m²;
A_{lb}	the area of foundation to be cleaned for the middle anchor block, in m²;
A_{ln}	the area of foundation to be cleaned for the lower anchor block, in m²;
A_{la}	the area of foundation to be cleaned for the saddle block, in m²;
A_{lt}	the area of the tunnel foundation to be cleaned, in m², when applicable;
N_t	number of pressure penstocks;
D_b	internal diameter of the pressure penstock, in m;
α_i	slope of section i, in degrees;
N_a	number of saddle blocks per pressure penstock, as defined below;
e_c	thickness of the concrete lining of the section through a tunnel, in m, as defined below;
L_t	length of the section through a tunnel, in m;
L_{h1}	horizontal projection of section 1, in m; and
d_2	effective distance, in m.

The **length of contact grouting and consolidation grouting holes**, L_{tf}(m), is given by:

$$L_{tf} = 1.0 \times A_{lt}$$

A_{lt}	the area of the tunnel rock to be cleaned, in m², when applicable.

The **total length of the rock anchors** for the anchor blocks, L_{tfp} (m), when applicable, is given by:

$$L_{tfp} = 1.0 \times (A_{ls} + A_{lb} + A_{ln})$$

where:

A_{ls}	the area of foundation to be cleaned for the upper anchor block, in m²;
A_{lb}	the area of foundation to be cleaned for the middle anchor block, in m²; and
A_{ln}	the area of foundation to be cleaned for the lower anchor block, in m².

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m²
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.19.34.14)

The **volume of concrete** for the pressure penstocks, V_{ctf} (m^3), is given by:

$$V_{ctf} = V_{cs} + V_{cb} + V_{cf} + V_{ca} + V_{ct} + V_{cc} + V_{cv} + V_{ce}$$

where:

$$V_{cb} = N_t \times (0.0072 \times D_b^3 + 0.105 \times D_b^2 + 0.08 \times D_b) \times (\alpha_3 - \alpha_2)$$

$$V_{cf} = N_t \times (0.228 \times D_b^2 + 4.77 \times D_b + 5.3) \times (L_{h4} + d_3)$$

$$V_{ca} = N_t \times N_a \times 0.5 \times D_b^3$$

$$V_{ct} = N_t \times \pi \times (D_b + e_c) \times e_c \times L_{ct}$$

$$V_{cc} = \pi \times (D_b + 3 \times e_c) \times e_c \times 5 \times D_b$$

$$V_{cv} = N_t \times 12 \times D_b^3$$

$$V_{ce} = N_g \times 12 \times A^3$$

$$N_a = \text{int}\left[\frac{d_4}{1.6 \times D_b} + 0.5\right] - 2$$

$$e_c = 0.091 \times D_b^{0.62}$$

$$d_3 = \frac{NA_{xfu} + 1.0 - EI_d}{\text{sen}\alpha_3}$$

$$d_4 = L_b - 4 \times D_b \times \left(tg\frac{\alpha_2 - \alpha_1}{2} + tg\frac{\alpha_3 - \alpha_2}{2} \right) - 2 \times D_b - d_3$$

for $L_{h1} \leq 1{,}7 \times D_b$ e $L_t = 0$:

$$V_{cs} = N_t \times [0.565 \times D_b^3 + 10.50 \times D_b^2 + 8.4 \times D_b + \\ + (0.029 \times D_b^3 + 0{,}42 \times D_b^2 + 0.34 \times D_b) \times \alpha]$$

$$\alpha = \alpha_2 - \alpha_1 - 25 \geq 0$$

for $L_{h1} > 1{,}7 \times D_b$ e $L_t = 0$:

$$V_{cs} = N_t \times (0.0072 \times D_b^3 + 0.105 \times D_b^2 + 0.08 \times D_b) \times (\alpha_2 - \alpha_1)$$

where:

V_{cs}	volume of concrete for the upper anchor block, in m^3, when applicable;
V_{cb}	volume of concrete for the middle anchor block, in m^3;
V_{cf}	volume of concrete for the lower anchor block, in m^3;
V_{ca}	volume of concrete for a saddle block, in m^3;
V_{ct}	volume of concrete for the lining of the section through a tunnel, in m^3;
V_{cc}	volume of extra concrete for bifurcations, in m^3, when required;
V_{cv}	volume of concrete for the valve housings at the beginning of the penstock, in m^3, when required;
V_{ce}	volume of concrete for the valve housings at the end of the penstock, in m^3, when required;
N_t	number of pressure penstocks;

D_b	internal diameter of the pressure penstock, in m;
α_i	slope of section i, in degrees;
L_{hi}	horizontal projection of section i, in m;
N_a	number of saddle blocks per penstock;
e_c	thickness of the concrete lining of the section through a tunnel, in m;
L_t	length of the section through a tunnel, in m;
L_{ct}	length of the section through a tunnel lined with concrete, in m;
N	number of generating units;
A	diameter of the inlet to the spiral casing, in m;
NA_{xfu}	maximum water level in the tailrace canal, in m;
El_d	elevation of the center line of the turbine distributor, in m;
L_b	length of the pressure penstocks, in m;
α	secondary angle, in degrees;
d_i	effective distance, in m; and
int(x)	function that returns the integer part of x.

The amounts of cement and reinforcement steel are:

	cement (kg/m^3)	reinforcement steel (kg/m^3)
lining	250	50
bifurcations and anchor blocks; saddle blocks and valve housings	270	70

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- lining and saddle blocks:128.00/m^3
- anchor blocks and valve housings:174.00/m^3

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Steel Plates (account .12.19.34.23.23)

The total weight of the steel plates can be calculated from its thickness and the length of the penstock.

All the thicknesses calculated below should be rounded up to whole centimeters, using the expressions below:

$$x = \frac{e}{2.54}$$

if $x \leq 1$: $e = 2.54 \times \frac{\text{int}(32 \times x + 0.9999)}{32}$

if $1 < x \leq 1{,}5$: $e = 2.54 \times \frac{\text{int}(16 \times x + 0.9999)}{16}$

if $x > 1{,}5$: $e = 2.54 \times \frac{\text{int}(8 \times x + 0.9999)}{8}$

The **minimum thickness for construction purposes**, e_{min} (cm), is given by:

$$e_{min} = \frac{D_b}{4} + 0{,}127 \geq 0.635 \text{ cm}$$

where:

D_b	internal diameter of the pressure penstock, in m.

The **working pressure**, p_s (kgf/cm^2), at points 0 to 4, is given by:

for projects with **no** surge tank:

$p_{s0} = 0.1 \times (NA_{max} - El_0)$

$p_{s1} = 0.1 \times (NA_{max} + i_s \times L_{h1} - El_1)$

$p_{s2} = 0.1 \times (NA_{max} + i_s \times (L_{h1} + L_{h2}) - El_2)$

$p_{s3} = 0.1 \times [NA_{max} + i_s \times (L_{h1} + L_{h2} + L_{h3}) - El_3]$

$p_{s4} = 0.1 \times (NA_{max} + h_{sx} - El_4)$

for projects **with** a surge tank:

$p_{s0} = 0.1 \times (NA_{xch} - El_0)$

$p_{s1} = 0.1 \times (NA_{xch} + i_s \times L_{h1} - El_1)$

$p_{s2} = 0.1 \times (NA_{xch} + i_s \times (L_{h1} + L_{h2}) - El_2)$

$p_{s3} = 0.1 \times [NA_{xch} + i_s \times (L_{h1} + L_{h2} + L_{h3}) - El_3]$

$p_{s4} = 0.1 \times (NA_{xch} + h_{sx} - El_4)$

where:

NA_{max}	maximum normal water level in the reservoir;
El_i	elevation of point i;
h_{sx}	maximum surge pressure due to water hammer, in m;
NA_{xch}	maximum normal water level in the surge tank;
L_{hi}	horizontal projection of section i, in m; and
i_s	energy gradient.

The **thickness of the steel plate required at point i**, e_i (cm), can be calculated by:

$$e_i = \frac{100 \times p_{si} \times D_b}{2 \times \sigma_a} + 0.3$$

where:

D_b	internal diameter of the pressure penstock, in m;
p_{si}	working pressure at point i, in kgf/cm^2;
σ_a	1200 kgf/cm^2, permissible stress in steel; and
0.3	extra thickness to compensate for corrosion, in cm.

The **working pressure withstood by the steel plate of minimum thickness**, p_{sn} (kgf/cm²), is given by:

$$p_{sn} = 2 \times \sigma_a \times \frac{e_{min} - 0.3}{100 \times D_b}$$

where:

σ_a	1200 kgf/cm², permissible stress in steel;
e_{min}	minimum thickness of the steel plate for construction purposes, in cm; and
D_b	internal diameter of the pressure penstock, in m.

The **weight of the steel plates**, P_c (t), can be obtained from one of the cases:

- Case 1: If $p_{s0} \geq p_{sn}$:

$$P_{c1} = k_c \times \frac{e_0 + e_1}{2} \times L_1$$

$$P_{c2} = k_c \times \frac{e_1 + e_2}{2} \times L_2$$

$$P_{c3} = k_c \times \frac{e_2 + e_3}{2} \times L_3$$

$$P_{c4} = k_c \times \frac{e_3 + e_4}{2} \times L_4$$

- Case 2: If $p_{s1} \geq p_{sn} > p_{s0}$:

$$L_{min} = \frac{p_{sn} - p_{so}}{0.1 \times (i_s + tg\alpha_1) \times \cos\alpha_1}$$

$$P_{c1} = k_c \times [e_{min} \times L_{min} + \frac{e_{min} + e_1}{2} \times (L_1 - L_{min})]$$

P_{c2}, P_{c3}, P_{c4} same as case 1.

$e_0 = e_{min}$

- Case 3: If $p_{s2} > p_{sn} > p_{s1}$:

$$L_{min} = \frac{p_{sn} - p_{s1}}{0.1 \times (i_s + tg\alpha_2) \times \cos\alpha_2}$$

$$P_{c1} = k_c \times e_{min} \times L_1$$

$$P_{c2} = k_c \times [e_{min} \times L_{min} + \frac{e_{min} + e_2}{2} \times (L_2 - L_{min})]$$

P_{c3}, P_{c4} same as case 1

e_0, $e_1 = e_{min}$

- Case 4: If $p_{s3} \geq p_{sn} > p_{s2}$:

$$L_{min} = \frac{p_{sn} - p_{s2}}{0.1 \times (i_s + tg\alpha_3) \times \cos\alpha_3}$$

P_{c1} same as case 3

$$P_{c2} = k_c \times e_{min} \times L_2$$

$$P_{c3} = k_c \times [e_{min} \times L_{min} + \frac{e_{min} + e_3}{2} \times (L_3 - L_{min})]$$

P_{c4} same as case 1

e_0, e_1, e_2 = e_{min}

- Case 5: If $p_{s4} \geq p_{sn} > p_{s3}$:

$$L_{min} = \frac{p_{sn} - p_{s3}}{0.1 \times i_s}$$

P_{c1}, P_{c2} same as case 1

$$P_{c3} = k_c \times e_{min} \times L_3$$

$$P_{c4} = k_c \times [e_{min} \times L_{min} + \frac{e_{min} + e_4}{2} \times (L_4 - L_{min})]$$

e_0, e_1, e_2, e_3 = e_{min}

- Case 6: If $p_{sn} > p_{s4}$

P_{c1}, P_{c2}, Pc_3 same as case 5

$$P_{c4} = k_c \times e_{min} \times L_4$$

e_0, e_1, e_2, e_3, e_4 = e_{min}

where:

$$k_c = \frac{7.842}{100} \times \pi \times D_b$$

where:

p_{si}	working pressure at point i, in kgf/cm^2;
k_c	coefficient;
e_i	required thickness of the steel plate at point i, in cm;
L_i	length of section i, in m;
L_{min}	length of section with minimum thickness, in m;
p_{sn}	working pressure withstood by the steel plate of minimum thickness, in kgf/cm^2;
i_s	energy gradient.
α_i	slope of section i, in degrees;
e_{min}	minimum thickness of the steel plate for construction purposes, in cm; and
7.842	specific mass of steel, in kg/cm^3; and
D_b	internal diameter of the pressure penstock, in m.

The **total weight of the steel plates**, P_c (t), including a 10% provision for fastening parts, is given by:

$$P_c = 1{,}10 \times N_t \times (P_{c1} + P_{c2} + P_{c3} + P_{c4})$$

where:

N_t	number of pressure tunnels; and
P_{ci}	weight of the steel lining in section i, in t.

The **acquisition cost of the steel plate** for the pressure penstock is R$ 4,235.00/t – FOB cost excluding tranportation and insurance, assembly and testing and provisions for taxes payable, depending on the current tax regime. The figures are valid for December 2006 and for projects anywhere in Brazil.

The following percentages must be added to the FOB price:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for taxes and charges payable on the equipment.

Butterfly Valve (account .12.19.34.23.24)

The **acquisition cost of each butterfly valve**, C_{vb} (R$), for the pressure penstock – FOB cost – is given by the expression below (or obtained from Graph B.29 from annex B, as a function of its diameter and maximum working pressure). The figures are valid for December 2006 and for projects anywhere in Brazil:

$$C_{vb} = 2.528 \times H_x^{0.35} \times K_B$$

where:

$K_B = 1000 \times \left(9.6 \times D_B^2 + 8.6 \times D_B - 1.85\right)$, for $0.75 \leq D_B \leq 2.0$ m, and $10 \leq H_x \leq 300$

$K_B = 1000 \times \left(10.2 \times D_B^2 + 9.2 \times D_B - 1.97\right)$, for $2.5 \leq D_B \leq 8.0$ m, and $10 \leq H_x \leq 300$

case **a**, with the valve at the beginning of the penstock, just after the surge tank:

$D_B = D_b$ $\qquad\qquad$ $H_x = NA_{xch} - El_0$

case **b**, with the valve at the end of the penstock:

$D_B = A$ $\qquad\qquad$ $H_x = NA_{max} - El_4 + h_s$

where:

H_x	maximum working pressure of the valve, in m;
K_B	coefficient;
D_B	diameter of the butterfly valve, in m;
D_b	internal diameter of the pressure penstock, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
NA_{xch}	maximum water level in the surge tank, in m;
El_i	elevation of the butterfly valve axis, in m;
h_s	maximum surge pressure due to water hammer, in m; and
A	diameter of the inlet to the spiral casing, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

Spherical Valve (account .12.19.34.23.24)

The **acquisition cost of a spherical valve**, C_{ve} (R$), for the pressure penstock – FOB cost – is given by the expression below (or obtained from Graph B.30 from annex B, as a function of its diameter and maximum working pressure). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

$$C_{ve} = 2.528 \times H_x^{0.40} \times K_E$$

where:

$K_E = 1000 \times \left(24.4 \times D_E^2 + 4.4 \times D_E + 12.37\right)$, for $1.0 \leq D_E \leq 4{,}0$ m, and $200 \leq H_x \leq 1500$ m.

$$D_E = A$$

$$H_x = NA_{max} - El_4 + h_s$$

where:

H_x	maximum working pressure of the valve, in m;
K_E	coefficient;
D_E	diameter of the spherical valve, in m;
A	diameter of the inlet to the spiral casing, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
El_4	elevation of the spherical valve axis, in m; and
h_s	maximum surge pressure due to water hammer, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost.

PRESSURE TUNNELS (ACCOUNT .12.19.34)

Basic Data

The main **information required for dimensioning** this item can be obtained from the overall layout of the project and the energy studies, as follows:

- horizontal projection of each section, L_{h1}, L_{h2} and L_{h3}, in m;
- horizontal projection of the steel-lined part of the pressure tunnel, L_{hb}, in m;
- length of the headrace tunnel, L_{ad}, in m, when applicable;
- capacity of one generating unit, P_1, in MW, from item 5.7.2.;
- coefficient k, product of the yield of the generating unit by acceleration due to gravity, from item 2.1.;
- number of generating units, N_g, from item 5.7.2.;
- number of generating units per pressure tunnel, N_f;
- maximum normal water level in the reservoir, NA_{max}, from item 4.4.;
- minimum water level in the reservoir, NA_{min}, from item 5.3.;
- lowest elevation of the land in the intake area, El_{ten};
- mean thickness of the layer of soil in the intake area, e_{te} in m;
- normal water level in the tailrace canal, NA_{fu}, from item 4.4.;
- elevation of the center line of the turbine distributor, El_d, from item 5.7.2.;
- maximum net head, H_1, in m, from item 5.3.;
- head loss in the headrace canal, h_c, in m, from item 5.7.6., when applicable;
- head loss in the headrace tunnel, h_a, in m, from item 5.7.6., when applicable; and
- diameter of the inlet to the spiral casing, A, in m, from item 5.7.2., when applicable.

The main **information required for quantification** purposes is as follows:

- mean hydrostatic load on the tunnel, H in m;
- geological conditions of the area crossed by the tunnels;
- maximum water level in the surge tank, NA_{xch};

- volume of common excavation upstream from the surface powerhouse, V_{ttf}, in m^3; and
- volume of surface rock excavation upstream from the surface powerhouse, V_{rtf}, in m^3.

Considerations and Recommendations

A **section** is understood as being the stretch between two deflections in the vertical plane, counting from the outlet of the intake, point 0, and the inlet of the powerhouse, point 4, as shown in Figure 5.7.6.08.

For methodological reasons, the **slope of section 2**, α_2, is obtained from the other data.

In this text, ther term **bifurcation** is also used for sections that split into three and for separators.

The **horizontal projection** of section 1, L_{h1}, must be long enough to match the curve.

$$L_{h1} \geq 4 \times D_b \times tg\frac{\alpha_2}{2}$$

where:

D_b	internal diameter of the steel-lined part of the pressure tunnel, in m; and
α_2	slope of section 2, in degrees.

Internal diameter and mean velocity

The **number of pressure tunnels**, N_t, is given by:

$$N_t = \frac{N_g}{N_f}$$

where:

N_g	number of generating units; and
N_f	number of generating units per pressure tunnel.

The **internal diameter of the steel-lined part of the pressure tunnel**, D_b (m), can be determined by:

$$D_b = 14.2 \times \frac{(N_f \times P_1)^{0.43}}{H_{b1}^{0.65}}$$

where: $H_{b1} = NA_{max} - NA_{fu}$

where:

N_f	number of generating units per pressure penstock;
P_1	capacity of one generating unit, in MW;
H_{b1}	maximum gross head, in m;
NA_{max}	maximum normal water level in the reservoir, in m; and
NA_{fu}	normal water level in the tailrace canal, in m.

The **internal diameter of the unlined part of the pressure tunnel**, D_c (m), is given by:

$$D_c = 1.1 \times D_b$$

where:

D_b	internal diameter of the steel-lined part of the pressure tunnel, in m.

The **maximum turbine flow of each turbine**, Q_1 (m^3/s), is given by:

$$Q_1 = \frac{10^6 \times P_1}{k \times H_1}$$

where:

$$k = \rho \times g \times \eta_{t1} \times \eta_{g1} \qquad P_2 = \frac{P_1}{f_p}$$

for Francis turbines:

$$\eta_{t1} = 0.856 \times Q_1^{0.013} \qquad \eta_{g1} = 0.92 \times P_2^{0.01}$$

for Pelton and Kaplan turbines:

$$\eta_{t1} = 0.96 \qquad \eta_{g1} = 0.98$$

where:

P_1	capacity of one generating unit, in MW;
k	coefficient;
H_1	maximum net head, in m;
ρ	1000 kg/m³ – specific mass of water;
η_{t1}	turbine output for maximum net head;
η_{g1}	generator output for maximum net head;
g	9.81 m/s² – acceleration due to gravity;
P_2	capacity of one generator, in MVA; and
f_p	capacity factor.

The **maximum flow in each pressure tunnel**, Q_{1f} (m³/s), is given by:

$$Q_{1f} = N_f \times Q_1$$

where:

N_f	number of generating units per pressure tunnel; and
Q_1	maximum turbine flow of each turbine, in m³/s.

The **mean velocity of flow in the steel-lined part** of the tunnel, v_b (m/s), is given by:

$$v_b = \frac{4 \times Q_{1f}}{\pi \times D_b^2} \leq 7 \text{ m/s}$$

where:

Q_{1f}	maximum flow in each pressure tunnel, in m³/s; and
D_b	internal diameter of the steel-lined part of the pressure tunnel, in m.

If this restriction is not fulfilled, the velocity limit should be adopted and the diameter should be recalculated using:

$$D_b = \sqrt{\frac{4}{\pi} \times \frac{Q_{1f}}{7}} \qquad D_c = 1.1 \times D_b$$

where:

Q_{1f}	maximum flow in each pressure tunnel, in m³/s.

The **mean velocity of flow** in the unlined part of the tunnel, v_c (m/s), is given by:

$$v_c = \frac{4}{\pi} \times \frac{Q_{1f}}{D_c^2}$$

where:

Q_{1f}	maximum flow in each pressure tunnel, in m³/s; and
D_c	internal diameter of the unlined part of the pressure tunnel, in m.

The **internal diameter of the pressure tunnel after a bifurcation** or the **lateral diameter after a separator**, D_{1b} (m), can be determined by:

$$D_{1b} = \frac{D_b}{N_f^{3/8}}$$

where:

D_b	internal diameter of the pressure tunnel, in m; and
N_f	number of generating units per pressure tunnel.

Profile of the pressure tunnel

The **elevation of points 0 to 3**, El_0 a El_3, is given by:

$$El_0 = El_1 = El_{sol} - i \times L_{ad} + \frac{D_{ab}}{2}$$

$$El_2 = El_3 = El_d$$

where:

$$El_{sol} = 0.5 \times \mathrm{int}\left(\frac{NA_{min} - h_c - h_s - H_{cp}}{0.5}\right)$$

$$El_{sol} \le 0.5 \times \mathrm{int}\left(\frac{El_{ten} - e_{te}}{0.5}\right) + 1.0$$

$$h_s = 0.8 \times v_{cp} \times \sqrt{H_{cp}}$$

$$B_{cp} = D_{ab} \qquad H_{cp} = D_{ab}$$

for projects with no headrace tunnel and a totally steel-lined pressure tunnel:

$$v_{cp} = \frac{Q_{1f}}{B_{cp} \times H_{cp}} \qquad D_{ab} = D_b$$

for projects with no headrace tunnel and a partially steel-lined pressure tunnel:

$$v_{cp} = \frac{Q_{1f}}{B_{cp} \times H_{cp}} \qquad D_{ab} = D_c$$

for projects with a headrace tunnel:

$$v_{cp} = \frac{Q_t}{B_{cp} \times H_{cp}} \qquad D_{ab} = D_{ad}$$

where:

El_{sol}	elevation of the intake sill, in m;
i	slope of the headrace tunnel, in m/m, defined in the headrace tunnel studies;
L_{ad}	length of the headrace tunnel, in m;
D_{ab}	internal diameter at the start of the pressure tunnel, in m;
El_d	elevation of the center line of the turbine distributor, in m;
NA_{min}	normal minimum water level in the reservoir;

h_{pc}	head loss in the headrace canal, in m, when applicable;
h_s	minimum submergence of the intake (Gordon, 1970), in m;
H_{cp}	height of the intake gate, in m;
El_{ten}	lowest elevation of the land in the intake area;
e_{te}	mean thickness of the layer of soil in the intake area, in m;
v_{cp}	velocity of flow at the intake gate, in m/s;
B_{cp}	height of the intake gate, in m;
Q_{1f}	maximum flow in each pressure tunnel, in m³/s;
D_b	internal diameter of the steel-lined part of the pressure tunnel, in m;
D_c	internal diameter of the unlined part of the pressure tunnel, in m;
Q_t	total maximum turbine flow, in m³/s; and
D_{ad}	internal diameter of the headrace tunnel, in m.

The **slope of section 2**, α_2 (degrees), is given by:

$$\alpha_2 = \arctan\left(\frac{E_1 - E_2}{L_{h2}}\right)$$

where:

El_i	elevation of point i;
L_{h2}	horizontal projection of section 2, in m; and
arctan	arctangent function.

The length of the tunnel is determined from its profile.

The **length of section 2**, L_2 (m), of the tunnel is given by:

$$L_2 = \frac{L_{h2}}{\cos\alpha_2}$$

where:

L_{h2}	horizontal projection of section 2, in m; and
α_2	slope of section 2, in degrees.

The **total length of the tunnel**, L_t (m), is given by:

$$L_t = L_{h1} + L_2 + L_{h3}$$

where:

L_{hi}	horizontal projection of section i, in m; and
L_2	length of section 2, in m.

The **horizontal projection** of the tunnel, L_{ht} (m), is given by:

$$L_{ht} = L_{h1} + L_{h2} + L_{h3}$$

where:

L_{hi}	length of the horizontal projection of section i, in m.

It is not advisable to have a very short unlined part. If $L_{ht} - L_{hb} < 5 \times D_b$

then: $L_{hb} = L_{ht}$

where:

L_{ht}	horizontal projection of the tunnel, in m;
L_{hb}	horizontal projection of the steel-lined part of the tunnel, in m; and
D_b	internal diameter of the steel-lined part of the pressure tunnel, in m.

The **length of the steel-lined part,** L_b (m), and **unlined part** of the pressure tunnel, L_c (m), is given by one of the expressions below:

- **case a** – when steel lining begins in section 1:

$$L_{b1} = L_{hb} - L_{h2} - L_{h3} \qquad L_b = L_{b1} + L_2 + L_{h3}$$

$$L_c = L_t - L_b$$

- **case b** – when steel lining begins in section 2:

$$L_{b2} = \frac{L_{hb} - L_{h3}}{\cos \alpha_2} \qquad L_b = L_{b2} + L_{h3}$$

$$L_c = L_t - L_b$$

- **case c** – when steel lining begins in section 3:

$$L_b = L_{hb} \qquad L_c = L_t - L_b$$

where:

L_{b1}	length of the steel-lined part in section 1, in m;
L_{hb}	horizontal projection of the steel=lined part of the pressure tunnel, in m;
L_{h2}	horizontal projection of section 2, in m;
L_{h3}	horizontal projection of section 3, in m;
L_2	length of section 2, in m;
L_t	total length of the tunnel, in m;
L_{b2}	length of the steel-lined part in section 2, in m; and
α_2	slope of section 2, in degrees.

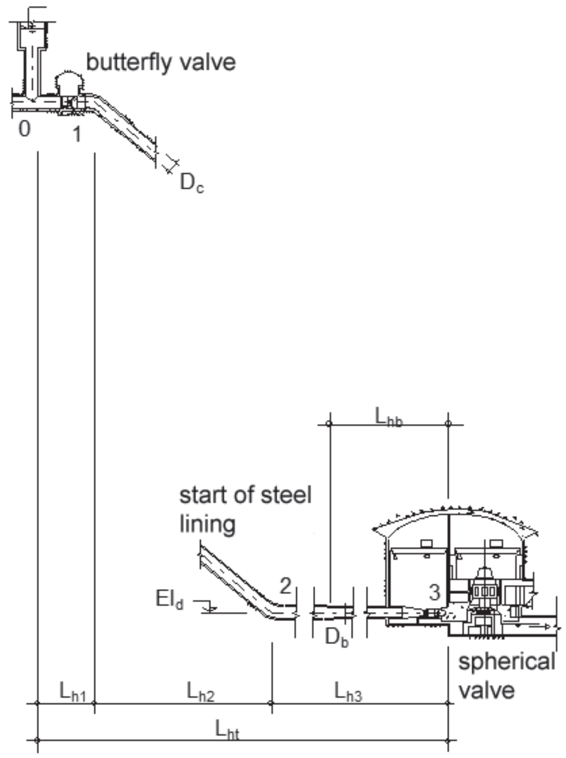


Fig. 5.7.6.10 Cross-section of a pressure tunnel.

Surge Pressure

The **maximum surge pressure** due to water hammer, h_{sx} (m), is given by:

$$h_{sx} = \frac{2 \times (L_b \times v_b + L_c \times v_c)}{g \times T_c} \leq 0.30 \times H_d$$

where: $H_d = NA_{max} - El_d$

T_c	For
6 s	short tunnels ($L_t \leq 3 \times H_{b1}$)
10 s	long tunnels ($L_t > 3 \times H_{b1}$)

where:

L_c	length of the unlined part of the pressure tunnel, in m;
v_c	mean velocity of flow in the unlined part of the pressure tunnel, in m/s;
L_b	length of the steel-lined part of the pressure tunnel, in m;
v_b	mean velocity of flow in the steel-lined part of the pressure tunnel, in m/s;
g	9.81 m/s^2 – acceleration due to gravity;
T_c	time taken to close the valve or distributor, in s;
L_t	total length of the pressure tunnel, in m;
H_d	static head, in m;
NA_{max}	maximum normal water level in the reservoir, in m; and
El_d	elevation of the center line of the turbine distributor, in m.

When maximum surge pressure exceeds the limit, the diameter must be redimensioned using:

$$D_b = \sqrt{\frac{4}{\pi} \times \frac{Q_{1f}}{v_b}}$$

where: $v_b = \frac{0.30 \times H_{b1} \times g \times T_c}{2 \times (L_b + 0.826 \times L_c)}$

where:

Q_{1f}	maximum flow in each pressure tunnel, in m^3/s;
v_b	mean velocity of flow in the steel-lined part of the pressure tunnel, in m/s;
H_{b1}	maximum gross head, in m.
g	9.81 m/s^2 – acceleration due to gravity;
T_c	time taken to close the valve or distributor, in s;
L_b	length of the steel-lined part of the pressure tunnel, in m; and
L_c	length of the unlined part of the pressure tunnel, in m.

The **energy gradient** for maximum surge pressure, i_s, is given by:

$$i_s = \frac{h_{sx}}{L_{ht}}$$

where:

h_{sx}	maximum surge pressure due to water hammer, in m; and
L_{ht}	horizontal projection of the pressure penstock, in m.

Head loss

The **total head loss**, h_p (m), from the intake to the pressure tunnel, is given by:

$$h_p = h_e + h_c + h_a + h_o + h_b + h_r + h_v + h_f$$

where:

for the unlined part:

$$h_o = \sum k_{oi} \times \frac{v_c^2}{2 \times g} \qquad h_r = 0{,}10 \times \frac{(v_b - v_c)^2}{2 \times g}$$

For $(r_i/D_{ad}) < 5$

$$k_{oi} = 0.2147 \times \left(\frac{r_i}{D_c}\right)^{-0.5718} \times \left(0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i\right)$$

For $(r_i/D_{ad}) \geq 5$

$$k_{oi} = \left(0.08 - 0.002 \times \left(\frac{r_i}{D_c} - 5\right)\right) \times \left(0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i\right)$$

and with a butterfly valve at the beginning of the pressure tunnel:

$$h_v = 0.20 \times \frac{v_b^2}{2 \times g}$$

for the steel-lined part:

$$h_o = \sum k_{oi} \times \frac{v_b^2}{2 \times g} \qquad h_r = 0.10 \times \frac{(v_a - v_b)^2}{2 \times g}$$

$$h_b = 0.10 \times \frac{v_b^2}{2 \times g} \qquad h_f = 6.35 \times L_t \times \frac{n^2 \times v_b^2}{D_b^{4/3}}$$

$$v_a = \frac{4}{\pi} \times \frac{Q_1}{A^2}$$

For $(r_i/D_{ad}) < 5$

$$k_{oi} = 0.2147 \times \left(\frac{r_i}{D_b}\right)^{-0.5718} \times \left(0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i\right)$$

For $(r_i/D_{ad}) \geq 5$

$$k_{oi} = \left(0.08 - 0.002 \times \left(\frac{r_i}{D_b} - 5\right)\right) \times \left(0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i\right)$$

with a butterfly valve at the beginning of the pressure tunnel: $h_v = 0.20 \times \frac{v_b^2}{2 \times g}$

with a butterfly valve at the end of the pressure tunnel: $h_v = 0.20 \times \frac{v_a^2}{2 \times g}$

with a spherical valve: $h_v = 0.03 \times \frac{v_a^2}{2 \times g}$

for projects with no headrace tunnel and a totally steel-lined pressure tunnel: $h_e = 0.20 \times \frac{v_b^2}{2 \times g}$

for projects with no headrace tunnel and a partially steel-lined pressure tunnel: $h_e = 0.20 \times \frac{v_c^2}{2 \times g}$

for projects with a headrace tunnel: $h_e = 0.20 \times \frac{v_{ad}^2}{2 \times g}$

where:

h_e	head loss at the inlet, in m;
h_a	head loss in the headrace tunnel, in m, when applicable;
h_o	head loss at the bends, in m;
h_b	head loss at the bifurcation, in m, when applicable;
h_c	head loss in the headrace canal, in m, when applicable;
h_r	head loss at diameter reductions, in m;
h_v	head loss at the valves, in m, when applicable;
h_f	continuous head loss in the pressure tunnel, in m;
k_{oi}	coefficient for head loss at bends i;
r_i	radius of curvature of section i, in meters;
v_c	velocity in the unlined part of the pressure tunnel, in m/s;
g	9.81 m/s² – acceleration due to gravity;
v_b	velocity in the steel-lined part of the pressure tunnel, in m/s;
v_a	velocity at the inlet to the spiral casing, in m/s;
L_t	total length of the tunnel, in m;
n	0.010 – Manning's coefficient for steel;
D_b	internal diameter of the pressure tunnel in the steel-lined part, in m.
Q_1	maximum turbine flow of each turbine, in m³/s;
A	diameter of the inlet to the spiral casing, in m;
v_{ad}	mean velocity of flow in the headrace tunnel, in m/s; and
θ	deflection of the tunnel axis, in radians.

Common Excavation (account .12.19.34.12.10)

The **volume of common excavation**, V_{ttf} (m³), upstream from the surface powerhouse should be determined from the project design.

The unit price of **common excavation** is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the tunnel. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (account .12.19.34.12.11)

The **volume of excavation in rock**, V_{rtf} (m³), upstream from the surface powerhouse should be determined from the project design.

The unit price of **excavation in rock** is R$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the tunnel. The price includes clearing the

vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Underground Excavation of Rock (account .12.19.34.12.12)

The **volume of rock excavated underground** for pressure tunnels, V_{stf} (m^3), is given by:

$$V_{stf} = V_{st} + V_{sb} + V_{se}$$

where:

$$V_{st} = N_t \times \frac{\pi}{4} \times [(D_c + 2 \times e_{cc})^2 \times L_c + (D_b + 2 \times e_c)^2 \times L_b]$$

$$V_{sb} = N_t \times 12 \times (D_b + 2 \times e_c)^3$$

$$V_{se} = N_t \times 12 \times (A + 2 \times e_{ca})^3$$

$$e_{ca} = 0.091 \times A^{0,62}$$

where:

V_{st}	volume of rock excavated underground for pressure tunnels, in m^3;
V_{sb}	volume of rock excavated underground for the butterfly valve housing, in m^3, when installed in a separate cavity;
V_{se}	volume of rock excavated underground for the spherical valve housing, in m^3, when installed in a separate cavity;
N_t	number of tunnels;
D_c	internal diameter of the unlined part of the pressure tunnel, in m;
D_b	internal diameter of the steel-lined part of the pressure tunnel, in m;
e_{cc}	thickness of the concrete lining in the unlined part of the pressure tunnel, in m;
e_c	thickness of the concrete lining in the steel-lined part of the pressure tunnel, in m;
L_c	length of the unlined part of the pressure tunnel, in m;
L_b	length of the steel-lined part of the pressure tunnel, in m;
A	diameter of the inlet to the spiral casing, in m; and
e_{ca}	thickness of the concrete lining at the inlet to the spiral casing, in m.

The unit price of **underground excavation**, P_{us} (R\$/m^3), (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil, can be obtained from the expression below (or from Graph B33, annex B, as a function of the excavated area). This is the price per cubic meter measured using the excavation line of the project design and includes excavating, loading, transportation up to 1.5 km and unloading:

valid for $4 \leq A_{se} \leq 300$: $P_{us} = 474.08 \times A_{se}^{-0.4629}$

where: $A_{se} = \frac{\pi}{4} \times (D_b + 2 \times e_c)^2$

where:

A_{se}	area of the excavated area, in m^2;
D_b	internal diameter of the steel-lined part of the pressure tunnel, in m; and
e_c	thickness of the concrete lining steel-lined part of the pressure tunnel, in m.

A careful appraisal should be made of the different situations where the tunnels form a representative part of the cost estimate, paying special attention to the geological conditions in the region and particularly long tunnels.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Foundation Cleaning and Treatment (account .12.19.34.13)

The **area of foundation to be cleaned**, A_{lf} (m^2), is given by:

$$A_{lf} = N_t \times \pi \times (D_c + 2 \times e_{cc}) \times L_c + N_t \times \pi \times (D_b + 2 \times e_c) \times L_b$$

where:

N_t	number of tunnels;
D_c	internal diameter of the unlined part of the pressure tunnel, in m;
D_b	internal diameter of the steel-lined part of the pressure tunnel, in m;
e_{cc}	thickness of the concrete lining in the unlined part of the pressure tunnel, in m;
e_c	thickness of the concrete lining in the steel-lined part of the pressure tunnel, in m;
L_c	length of the unlined part of the pressure tunnel, in m; and
L_b	length of the steel-lined part of the pressure tunnel, in m.

The **length of contact grouting and consolidation grouting holes**, L_{tf} (m), is given by:

$$L_{tf} = N_t \times 1.0 \times \pi \times (D_c + 2 \times e_{cc}) \times L_c + N_t \times 1.0 \times \pi \times (D_b + 2 \times e_c) \times L_b$$

N_t	number of tunnels;
D_c	internal diameter of the unlined part of the pressure tunnel, in m;
D_b	internal diameter of the steel-lined part of the pressure tunnel, in m;
e_{cc}	thickness of the concrete lining in the unlined part of the pressure tunnel, in m;
e_c	thickness of the concrete lining in the steel-lined part of the pressure tunnel, in m;
L_c	length of the unlined part of the pressure tunnel, in m; and
L_b	length of the steel-lined part of the pressure tunnel, in m.

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m^2
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Concrete (account .12.19.34.14)

The **volume of concrete** for pressure tunnels, V_{ctf} (m^3), is given by:

$$V_{ctf} = V_{cr} + V_{ct} + V_{cf} + V_{cb} + V_{ce}$$

where:

$$V_{cr} = N_t \times \pi \times (D_c + e_{cc}) \times e_{cc} \times L_c + N_t \times \pi \times (D_b + e_c) \times e_c \times L_b$$

$$V_{cf} = \pi \times (D_b + 3 \times e_c) \times e_c \times 5 \times D_b$$

$$V_{cb} = N_t \times 12 \times D_b^3 \qquad V_{ce} = N_g \times 12 \times A^3$$

$$e_{cc} = k_g \times \left[0.091 \times D_c^{0.62} + 0.0034 \times (H\text{-}30)\right]$$

$$e_c = 0.091 \times D_b^{0.62}$$

k_g	Geological conditions
1.0	good
1.4	average
2.0	poor or no information

for $L_c > 0$: $V_{ct} = N_t \times \frac{\pi}{2} \times \left[(D_c + 3)^2 - (D_c + 2 \times e_{cc})^2\right] \times D_c$

for $L_c = 0$: $V_{ct} = N_t \times \frac{\pi}{2} \times \left[(D_b + 3)^2 - (D_b + 2 \times e_c)^2\right] \times D_b$

where:

V_{cr}	volume of concrete for the lining of the pressure tunnels, in m^3;
V_{ct}	volume of concrete for the transition from the square to circular section after the intake, in m^3;
V_{cf}	volume of extra concrete for the bifurcations, in m^3, when required;
V_{cb}	volume of concrete for the valve housing at the beginning of the penstock, in m^3, when required;
V_{ce}	volume of concrete for the valve housing at the end of the penstock, in m^3, when required;
N_t	number of pressure tunnels;
D_c	internal diameter of the unlined part of the pressure tunnel, in m;
e_{cc}	thickness of the concrete lining in the unlined part of the pressure tunnel, in m;
L_c	length of the unlined part of the pressure tunnel, in m;
D_b	internal diameter of the steel-lined part of the pressure tunnel, in m;
e_c	thickness of the concrete lining along the steel-lined part of the pressure tunnel, in m;
L_b	length of the steel-lined part of the pressure tunnel, in m;
N_g	number of generating units;
A	diameter of the inlet to the spiral casing, in m;
k_g	coefficient to represent geological conditions; and
H	mean hydrostatic load in the pressure tunnel, in m.

The amounts of cement and reinforcement steel are:

	cement (kg/m^3)	reinforcement steel (kg/m^3)
transitions and lining	250	50
bifurcations and valve housings	270	70

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- transitions, lining and bifurcations: 129.00/m^3
- valve housings: 174.00/m^3

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Steel Lining

The total weight of the steel lining is obtained from its thickness of the steel plate and the length of the lined part.

All the thicknesses calculated below should be rounded up to whole centimeters, using the expressions below:

$$x = \frac{e}{2.54}$$

if $x \leq 1$: $e = 2.54 \times \frac{\text{int}(32 \times x + 0.9999)}{32}$

if $1 < x \leq 1.5$: $e = 2.54 \times \frac{\text{int}(16 \times x + 0.9999)}{16}$

if $x > 1.5$: $e = 2.54 \times \frac{\text{int}(8 \times x + 0.9999)}{8}$

The **minimum thickness for construction purposes**, e_{min} (cm), is given by:

$$e_{min} = \frac{D_b}{4} + 0.127 \geq 0.635 \text{ cm}$$

where:

D_b	internal diameter of the steel-lined part of the pressure tunnel, in m.

The **working pressure**, p_s (kgf/cm^2), at points 0 to 3 is given by the expressions below, assuming that the steel lining will withstand half the dynamic pressure:

for projects with no surge tank:

$$p_{so} = \frac{0.1 \times (NA_{max} - El_0)}{2}$$

$$p_{s1} = \frac{0.1 \times (NA_{max} + i_s \times L_{h1} - El_1)}{2}$$

$$p_{s2} = \frac{0.1 \times (NA_{max} + i_s \times (L_{h1} + L_{h2}) - El_2)}{2}$$

$$p_{s3} = \frac{0.1 \times (NA_{max} + h_{sx} - El_3)}{2}$$

for projects with a surge tank:

$$p_{so} = \frac{0.1\times(NA_{xch} - El_0)}{2}$$

$$p_{s1} = \frac{0.1\times(NA_{xch} + i_s \times L_{h1} - El_1)}{2}$$

$$p_{s2} = \frac{0.1\times(NA_{xch} + i_s \times (L_{h1} + L_{h2}) - El_2)}{2}$$

$$p_{s3} = \frac{0.1\times(NA_{xch} + h_{sx} - El_3)}{2}$$

where:

NA_{max}	maximum normal water level in the reservoir, in m;
NA_{xch}	maximum water level in the surge tank, in m;
El_i	elevation of point i, in m;
L_{hi}	horizontal projection of section i, in m;
h_{sx}	maximum surge pressure in the tunnel due to water hammer, in m; and
i_s	energy gradient for maximum surge pressure.

The **thickness of steel lining required at point i**, e_i (cm), can be calculated by:

$$e_i = \frac{100\times p_{si}\times D_b}{2\times\tau_a} + 0.3$$

where:

D_b	internal diameter of the steel-lined part of the pressure tunnel, in m;
p_{si}	working pressure at point i, in kgf/cm^2;
τ_a	1200 kgf/cm^2, permissible stress in steel; and
0.3	extra thickness to compensate for corrosion, in cm.

The **working pressure withstood by the minimum-thickness metal plate**, p_{sn} (kgf/cm^2), is given by:

$$p_{sn} = 2\times\tau_a\times\frac{e_{min} - 0.3}{100\times D_b}$$

where:

τ_a	1200 kgf/cm^2, permissible stress in steel;
e_{min}	minimum thickness of the steel plate for construction purposes, in cm; and
D_b	internal diameter of the pressure penstock, in m.

The expressions for determining the **weight of the steel lining**, P_b (t), will depend on the section where the lining begins.

The **working pressure at point B**, beginning of the steel-lined part of the pressure tunnel, p_{sB} (kgf/cm^2), is given by:

$$p_{sB} = \frac{0.1\times(NA_{max} + i_s\times L_c - El_0)}{2}$$

where:

NA_{max}	normal maximum water level in the reservoir, in m;
i_s	energy gradient for maximum surge pressure;
L_c	length of the unlined part of the pressure tunnel, in m; and
El_0	elevation of point 0, in m.

- **case a** – when steel lining begins in section 1:
- **case a1** – if $p_{sB} \geq p_{sn}$:

$$P_{b1} = k_b \times \frac{e_B + e_1}{2} \times L_{b1}$$

$$P_{b2} = k_b \times \frac{e_1 + e_2}{2} \times L_2$$

$$P_{b3} = k_b \times \frac{e_2 + e_3}{2} \times L_{h3}$$

- **case a2** – if $p_{s1} \geq p_{sn} \geq p_{sB}$:

$$L_{min} = 2 \times \frac{p_{sn} - p_{sB}}{0.1 \times i_s}$$

$$P_{b1} = k_b \times e_{min} \times L_{min} + k_b \times \frac{e_{min} + e_1}{2} \times (L_{b1} - L_{min})$$

P_{b2}, P_{b3} same as case a1.

$e_B = e_{min}$

- **case a3** – if $p_{s2} \geq p_{sn} > p_{s1}$:

$$L_{min} = 2 \times \frac{p_{sn} - p_{s1}}{0.1 \times \left(\frac{i_s}{2} + tg\alpha_2\right) \times \cos\alpha_2}$$

$$P_{b1} = k_b \times e_{min} \times L_{b1}$$

$$P_{b2} = k_b \times e_{min} \times L_{min} + k_b \times \frac{e_{min} + e_2}{2} \times (L_2 - L_{min})$$

P_{b3} same as case a1.

$e_B = e_1 = e_{min}$

- **case a4** – if $p_{s3} \geq p_{sn} > p_{s2}$:

$$L_{min} = 2 \times \frac{p_{sn} - p_{s2}}{0.1 \times i_s}$$

P_{b1} same as case a3.

$$P_{b2} = k_b \times e_{min} \times L_2$$

$$P_{b3} = k_b \times e_{min} \times L_{min} + k_b \times \frac{e_{min} + e_3}{2} \times (L_{h3} - L_{min})$$

$e_B = e_1 = e_2 = e_{min}$

- **case a5** – if $p_{sn} > p_{s3}$:

P_{b1}, P_{b2} same as case a4.

$$P_{b3} = k_b \times e_{min} \times L_{h3}$$

$e_B = e_1 = e_2 = e_3 = e_{min}$

- **case b** – when steel lining begins in section 2:

- **case b1** – if $p_{sB} \geq p_{sn}$:

$$P_{b1} = 0$$

$$P_{b2} = k_b \times \frac{e_B + e_2}{2} \times L_{b2}$$

$$P_{b3} = k_b \times \frac{e_2 + e_3}{2} \times L_{h3}$$

- **case b2** – if $p_{s2} \geq p_{sn} \geq p_{sB}$:

$$L_{min} = 2 \times \frac{p_{sn} - p_{sB}}{0.1 \times \left(\frac{i_s}{2} + \tan\alpha_2\right) \times \cos\alpha_2}$$

$$P_{b1} = 0$$

$$P_{b2} = k_b \times e_{min} \times L_{min} + k_b \times \frac{e_{min} + e_2}{2} \times (L_2 - L_{min})$$

P_{b3} same as case b1.

$e_B = e_{min}$

- **case b3** – if $p_{s3} \geq p_{sn} > p_{s2}$:

$$L_{min} = 2 \times \frac{p_{sn} - p_{s2}}{0.1 \times i_s}$$

$$P_{b1} = 0$$

$$P_{b2} = k_b \times e_{min} \times L_{b2}$$

$$P_{b3} = k_b \times e_{min} \times L_{min} + k_b \times \frac{e_{min} + e_3}{2} \times (L_{h3} - L_{min})$$

$e_B = e_2 = e_{min}$

- **case b4** – if $p_{sn} > p_{s3}$:

$$P_{b1} = 0$$

P_{b2} same as case b3.

$$P_{b3} = k \times e_{min} \times L_{h3}$$

$e_B = e_2 = e_3 = e_{min}$

- **case c** – when steel lining begins in section 3:

- **case c1** – if $p_{sB} \geq p_{sn}$:

$$P_{b1} = P_{b2} = 0$$

$$P_{b3} = k_b \times \frac{e_B + e_3}{2} \times L_b$$

- **case c2** – if $p_{s3} \geq p_{sn} > p_{sB}$:

$$L_{min} = 2 \times \frac{p_{sn} - p_{sB}}{0.1 \times i_s}$$

$$P_{b1} = P_{b2} = 0$$

$$P_{b3} = k_b \times e_{min} \times L_{min} + k_b \times \frac{e_{min} + e_3}{2} \times (L_b - L_{min})$$

$$e_B = e_{min}$$

- **case c3** – if $p_{sn} > p_{s3}$:

$$P_{b1} = P_{b2} = 0$$

$$P_{b3} = k_b \times e_{min} \times L_b$$

$$e_B = e_3 = e_{min}$$

where: $k_b = \frac{7.842}{100} \times \pi \times D_b$

where:

p_{si}	working pressure at point i, in kgf/cm²;
k_b	coefficient;
e_i	thickness of the steel plate at point i, in cm;
L_{bi}	length of the steel-lined part of section i where steel lining begins, in m;
L_2	length of section 2, in m;
L_{h3}	length of section 3, in m;
p_{sn}	working pressure withstood by a steel plate of minimum thickness, in kgf/cm²;
i_s	energy gradient for maximum surge pressure;
e_{min}	minimum thickness of steel lining for construction purposes, in cm;
L_{min}	length with minimum thickness, in m;
α_2	slope of section 2, in degrees;
7.842	specific mass of steel, in t/m³; and
D_b	internal diameter of the steel-lined part of the pressure tunnel, in m.

The **total weight of the steel lining**, P_b (t), including a 10% provision for fastening parts, is given by:

$$P_b = 1.10 \times N_t \times (P_{b1} + P_{b2} + P_{b3})$$

where:

N_t	number of pressure tunnels; and
P_{bi}	weight of the steel lining in section i, in t.

The **acquisition cost of the steel plate** for the pressure penstock is R$ 4.235.00/t – FOB cost excluding tranportation and insurance, assembly and testing and provisions for taxes payable, depending on the current tax regime. The figures are valid for December 2006 and for projects anywhere in Brazil.

The following percentages must be added to the FOB price:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for taxes and charges payable on the equipment.

Butterfly Valve (account .12.19.34.23.24)

The **acquisition cost of each butterfly valve**, C_{vb} (R$), for the pressure penstock – FOB cost – is given by the expression below (or obtained from Graph B.29, annex B, as a function of its diameter and maximum working pressure). The figures are valid for December 2006 and for projects anywhere in Brazil:

$$C_{vb} = 2.528 \times H_x^{0.35} \times K_B$$

where:

$K_B = 1000 \times \left(9.6 \times D_B^2 + 8.6 \times D_B - 1.85\right)$, for $0.75 \leq D_B \leq 2.0$ m, and $10 \leq H_x \leq 300$

$K_B = 1000 \times \left(10.2 \times D_B^2 + 9.2 \times D_B - 1.97\right)$, for $2.5 \leq D_B \leq 8.0$ m, and $10 \leq H_x \leq 300$

- case **a**, valve at the beginning of the penstock, just after the surge tank:

$$D_B = D_b \qquad H_x = NA_{xch} - El_0$$

- case **b**, valve at the end of the penstock:

$$D_B = A \qquad H_x = NA_{max} - El_4 + h_s$$

where:

H_x	maximum working pressure of the valve, in m;
K_B	coefficient;
D_B	diameter of the butterfly valve, in m;
D_b	internal diameter of the pressure penstock, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
NA_{xch}	maximum water level in the surge tank, in m;
El_i	elevation of the butterfly valve axis, in m;
h_s	maximum surge pressure due to water hammer, in m; and
A	diameter of the inlet to the spiral casing, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

Spherical Valve (account .12.19.34.23.24)

The **acquisition cost of each spherical valve**, C_{ve} (R$), for the pressure penstock – FOB cost – is given by the expression below (or obtained from Graph B.30, annex B, as a function of its diameter and maximum working pressure). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

$$C_{ve} = 2.528 \times H_x^{0.40} \times K_E$$

where:

$K_E = 1000 \times \left(24.4 \times D_E^2 + 4.4 \times D_E + 12.37\right)$, for $1.0 \leq D_E \leq 4{,}0$ m, and $200 \leq H_x \leq 1500$ m.

$$D_E = A$$

$$H_x = NA_{max} - El_4 + h_s$$

where:

H_x	maximum working pressure of the valve, in m;
K_E	coefficient;
D_E	diameter of the spherical valve, in m;

A	diameter of the inlet to the spiral casing, in m;
NA_{max}	maximum normal water level in the reservoir, in m;
El_4	elevation of the axis of the spherical valve, in m; and
h_s	maximum surge pressure due to water hammer, in m.

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost.

TAILRACE CANAL

The main **information required for dimensioning** this item can be obtained from the overall layout and item 5.1.2. (Hydrometeorological Data) and from the dimensioning underway, as follows:

- minimum water level in the tailrace canal, NA_{fu}, in m;
- width of the powerhouse, with the exception of ones equipped with Pelton turbines, B_{cf}, in m;
- mean length of the tailrace canal, L_{fu}, in m;
- total maximum turbine flow, Q_t, in m³/s;
- mean velocity of flow in the tailrace canal, ideally lower than 1.5 m/s, v_{fu}, in m/s;
- mean elevation of the land in section 0 perpendicular to the tailrace canal, next to the powerhouse, El_{tf0};
- thickness of the layer of soil in section 0, e_{te0}, in m;
- mean elevation of the land in section 1 perpendicular to the tailrace canal in the first third, El_{tf1};
- thickness of the layer of soil in section 1, e_{te1}, in m;
- mean elevation of the land in section 2 perpendicular to the tailrace canal in the second third, El_{tf2};
- thickness of the layer of soil in section 2, e_{te2}, in m; and
- mean lateral slope, horizontal distance for a 1.0 m difference in level, in m.

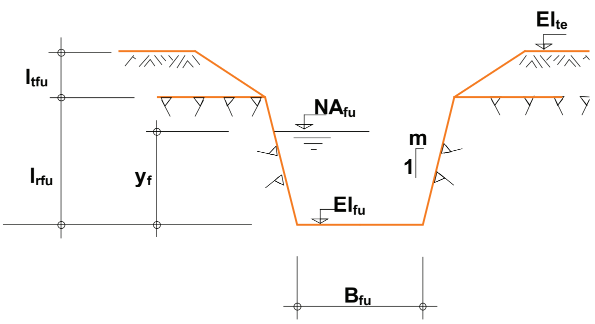


Fig. 5.7.6.11 – Typical cross-section of a tailrace canal.

The methodology expounded below is for short canals with negligible head loss. A canal can be deemed short if its length is no more than three times the width of the canal bottom.

For projects with powerhouses equipped with Pelton turbines, it is better to use the spreadsheet for long intake cannals (576CN.xls), with adaptations.

The mean lateral slope, "m", will depend on the geological conditions and the height of excavation:

m	Height of excavation
0.3	≤ 16 m
0.6	> 16 m

For good geological conditions, the lateral slope can be assumed, provided it is possible to excavate without the need for intermediate berms.

Dimensioning the canal

The **cross-sectional area of flow** in the canal, A_{fu} (m^2), is given by:

$$A_{fu} = \frac{Q_t}{v_{fu}}$$

where:

Q_t	total maximum turbine flow, in m^3/s; and
v_{fu}	mean velocity of discharge.

The **width of the canal bottom**, B_{fu} (m), is given by:

$$B_{fu} = B_{cf} - 2.0$$

where:

B_{cf}	width of the powerhouse, with the exception of ones equipped with Pelton turbines, in m.

The **depth of flow in the canal**, y_f (m), is given by:

$$y_f = \frac{-B_{fu} + \sqrt{B_{fu}^2 + 4 \times m \times A_{fu}}}{2 \times m}$$

where:

B_{fu}	width of the canal bottom, in m;
A_{fu}	area of the cross-section of discharge in the canal, in m^2; and
m	lateral slope, horizontal distance for a 1.0 m difference in level.

The **elevation of the canal bottom**, El_{fu}, is given by:

$$El_{fu} = NA_{nfu} - y_f$$

where:

NA_{nfu}	minimum water level in the tailrace canal, in m; and
y_f	depth of flow of the canal, in m.

The **head loss in the canal** is negligible.

Common Excavation (Account .12.19.35.12.10)

The **volume of common excavation**, V_{tfu} (m^3), is given by:

$$V_{tfu} = \left(\frac{V_{tf0}}{2} + V_{tf1} + V_{tf2}\right) \times \frac{L_{fu}}{3}$$

where:

$$V_{tfi} = \left[B_{fu} - 10 \times m + 2 \times (m \times h_{rfi} + e_{tei})\right] \times e_{tei}$$

$$h_{rfi} = (El_{tei} - El_{fu}) - e_{tei}$$

where:

V_{tfi}	volume of common excavation per meter in section i of the canal, in m^3/m;
L_{fu}	length of the tailrace canal, in m;
B_{fu}	width of the canal bottom, in m;
m	lateral slope, horizontal distance for a 1.0 m difference in level, in m;
h_{ri}	depth of excavation in rock in section i of the canal, in m;
e_{tei}	mean thickness of the layer of soil in section i of the canal, in m;
El_{tei}	elevation of the land along the canal axis in section i of the canal, in m; and
El_{fu}	elevation of the canal bottom, in m.

The unit price of **common excavation** is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the tailrace canal. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

Surface Rock Excavation (Account .12.19.35.11)

The volume of surface rock excavation, V_{rfu} (m³), is given by:

$$V_{rfu} = \left(\frac{V_{rf0}}{2} + V_{rf1} + V_{rf2} \right) \times \frac{L_{fu}}{3}$$

where:

$$V_{rfi} = \left(B_{fu} - 10 \times m + m \times h_{rfi} \right) \times h_{rfi}$$

where:

V_{rfi}	volume of surface rock excavation per meter in section i of the canal, in m³/m;
L_{fu}	length of the tailrace canal, in m;
B_{fu}	width of the canal bottom, in m;
m	lateral slope, horizontal distance for a 1.0 m difference in level, in m; and
h_{ri}	depth of excavation in rock in section i of the canal, in m.

The unit price of **excavation in rock** is R$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the tailrace canal. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

5.7.7 Roads, Railroads and Bridges (account .16)

Road maps should be consulted and field trips made, as and when necessary, to identify the need to build bridges, roads or railroads, their length and the construction categories involved.

The cost of this account should be calculated using the unit prices from tables 4.10.10.01 to 4.10.10.03. When an earth landing strip (1km) is deemed necessary, the cost should be assumed to be R$ 500,000.00 per runway. For larger airport facilities, a survey should be undertaken based on the Manual.

5.7.8 Indirect Costs (account .17)

Indirect costs include:

- building and maintaining the construction site and workers' camp;
- engineering and socioenvironmental studies;
- owner's administration costs.

The costs can be obtained from graphs or as percentages of the total direct cost, as set out below.

Construction Site and Workers' Camp (Account .17.21)

The basic data are:

- notify whether or not there is a workers' camp;
- total volume of concrete for construction, V_{ct}, in m^3;
- total volume of surface rock excavation, V_{et}, in m^3;
- volume of rock excavated underground, V_{es}, in m^3;
- total volume of earthfill or rockfill, V_a, in m^3.

Use spreadsheet 57ope.xls to obtain the building and maintenance cost of the construction site and workers' camp.

In inventory studies, the cost of buildings, and the maintenance and operation of the construction site and workers' camp are a function of the volume of construction work translated by factor F:

for $V_a \leq 30{,}000{,}000\ m^3$

$$F = 30 \times V_{ct} + V_{et} + 5 \times V_{es} + 0.25 \times V_a$$

for $V_a > 30{,}000{,}000\ m^3$

$$F = 30 \times V_{ct} + V_{et} + 5 \times V_{es} + 0.15 \times V_a$$

where:

V_{ct}	total volume of concrete for construction, in m^3;
V_{et}	total volume of surface rock excavation, comprising the sum of the volumes of common excavation, excavation in rock and excavation in borrow areas, in m^3,
V_{es}	volume of underground excavation, in m^3; and
V_a	total volume of earthfill or rockfill, in m^3.

Engineering (Account. 17.22.40)

For this account, which represents the engineering costs, the overall value is taken as a percentage of the total direct cost, as shown below:

- 3.5%: .17.22.40.36 – Basic Project Engineering
- 1.0%: .17.22.40.37 – Special Engineering Services
- 0.5%: .17.22.40.54 – Socioenvironmental Studies and Plans

Owner's Administration Costs (Account. 17.22.41)

The owner's administration costs are estimated as being 10% of the total direct costs.

5.7.9 Interest During Construction (account .18)

The procedure adopted for estimating interest during construction is to distribute the costs according to the construction schedule and apply an interest rate to the entire costs accrued in each year.

Tables 5.7.9.01 and 5.7.9.02 show examples of annual interest rates used to calculate interest during construction (i) at 10% and 12%. Annual interest rates can be obtained from the concession-granting authority.

Based on the estimated construction time, from preparation to generation, the disbursement schedule and respective interest rates should be used during construction.

Table 5.7.9.01 – Interest during construction (for i = 10% p.a.)

Construction time (Years)	Year 1	2	3	4	5	6	7	Interest calculation	% interest
2	40	60						109.55	10
3	25	40	35					115.08	15
4	15	30	35	20				120.89	21
5	10	20	25	30	15			126.11	26
6	5	15	20	21	25	14		130.14	30
7	3	7	15	20	20	21	14	133.22	33

Table 5.7.9.02 – Interest during construction(for i = 12% p.a.)

Construction time (Years)	Year 1	2	3	4	5	6	7	Interest calculation	% interest
2	40	60						111.49	11
3	25	40	35					118.26	18
4	15	30	35	20				125.44	25
5	10	20	25	30	15			131.97	32
6	5	15	20	21	25	14		137.09	37
7	3	7	15	20	20	21	14	141.03	41

5.8 COMPARISON AND SELECTION OF CASCADE OPTIONS

In the analysis of the different cascade options considered in the Final Studies, the energy benefits that would arise from developing the hydroelectric potential of the river basin according to each cascade should be compared with their building costs and the negative socioenvironmental impacts they would cause, as well as the positive socioenvironmental impacts of each cascade.

The energy-economic implications are assessed by the cost/energy benefit index, while the negative socioenvironmental impacts are expressed by the negative socioenvironmental impact index. The positive socioenvironmental impacts on the river basin are assessed separately by the positive socio-environmental impact index. Items 5.8.1, 5.8.2 and 5.8.3 describe how these three indexes are obtained for each cascade in the Final Studies, while item 5.8.4 sets out how these indexes can be used in a multi-objective approach to rank the cascades and choose the most advantageous one.

5.8.1 Cost/Energy Benefit Index

The method used in the Final Studies to obtain the cost/energy benefit index of the different cascades under study is the same as that used in the Preliminary Studies (see item 4.11.1). Clearly, the energy benefits and cost estimates used in the Final Studies must be determined using criteria and procedures compatible with this stage of the studies, which are set out in items 5.3 and 5.7.

5.8.2 Negative Socioenvironmental Impact Index (IAn)

The negative socioenvironmental impact index of a cascade option should express the degree of negative impact the set of projects comprising it would have on the study area. In the Final Studies, the index is used to rank the cascades as a function of the objective of **minimizing negative socio-environmental impacts**, thereby providing a valuable input for their comparison and the selection of the best alternative.

As in the Preliminary Studies, the negative socioenvironmental impact index of the cascade option is built up in two steps:

- an impact index for the cascade option on each synthesis component (representing the impact of the group of projects on the component);
- an impact index for the cascade option on the environmental system (representing its aggregate impact index for all the synthesis components).

Formulating the negative socioenvironmental impact index of cascades on synthesis components

The negative socioenvironmental impact index of each cascade option on each synthesis component should represent the impact of the group of projects that make up the cascade option on the synthesis components in the study area. The impact processes inherent to the group of projects that affect a given sub-area must therefore be considered, including the cumulative and synergistic effects amongst the projects.

The negative socioenvironmental **impact index of each cascade on each synthesis component** in the study area (**IAC**) is obtained from the weighted sum of the impact indexes relative to the sub-areas, which are attributed using the procedures described in item 5.4.:

$$IAC = \Sigma\, I_{SAi} \times P_{SAi} \qquad (5.8.2.01)$$

where:

I_{SAi}	negative socioenvironmental impact index on a component relative to the group of projects affecting sub-area i; and
P_{SAi}	weighting factor relative to each sub-area i, for a given synthesis component.

In order to keep the **IAC** between zero and one, the weights $\mathbf{P_{SAi}}$ must also be attributed on a continuous scale from zero to one, with the sum of the weights relative to the sub-areas equalling one, per synthesis component.

The weights are used to ensure the relativization of the impact indexes calculated for the sub-areas within the context of the study area. The weighting factors are the same as those established per synthesis component in the Preliminary Studies, taking into account the significance of the socioenvironmental processes existing in each sub-area. Note that the consolidation of the diagnosis (item 5.2) may lead to a review of the weighting factors used.

As a function of the specificities of each synthesis component and each study area, different procedures can be adopted to systematize this weighting amongst sub-areas.

A table can be prepared to calculate the index, as shown below.

Table 5.8.2.01 – Negative socioenvironmental impact index of a cascade on a synthesis component

Sub-area (Weight) / Projects	I (0.07)	II (0.08)	III (0.18)	IV (0.12)	V (0.25)	VI (0.30)	
A		x					
B		x	x	x			
C			x		x		
F		x					
G	x						
H	x				x		
I	x	x	x	x	x		
M	x					x	
N	x		x				
Q_2						x	
I_{SAi}	0.65	0.55	0.95	0.20	0.40	1.0	IAC
$I_{SAi} \times P_{SAi}$	0.045	0.044	0.171	0.024	0.10	0.30	0.684

When it comes to the Indigenous Peoples synthesis component, since the spatial unit of analysis is the whole study area, there are no factors used to weight sub-areas. The **IAC** should, then, represent the impact of the group of projects from the cascade in question on this component.

Formulation of a negative impact index for a cascade on the environmental system

A cascade's negative socioenvironmental impact index on the environmental system (IA) should express its total negative socioenvironmental impact on the study area. In other words, it should consider the negative socioenvironmental impacts caused by the cascade on all the synthesis components.

This index is the weighted sum of the impact indexes for the cascade on the synthesis components (IAC) as calculated previously.

$$IA = \Sigma\, IAC_i \times P_{ci} \qquad (5.8.2.02)$$

where:

P_{ci}	weighting factor for each synthesis component.

In order to keep the **IA** values between zero and one, the weights $\mathbf{P_{ci}}$ must also be attributed on a continuous scale from zero to one, with the sum of the weights relative to the components equalling one.

The weights are used to ensure the relativization of the impact indexes calculated for the cascade on the synthesis component within the environmental context of the study area. The weighting factors should represent the relative importance of the impact processes of each syntehsis component on the environmental system, which can be measured by the repercussion of these processes on the other components.

The factors are the same as those established in the Preliminary Studies, and may, if necessary, be reviewed as a function of the systemic analysis of the impact processes of the different components undertaken during the Final Studies.

5.8.3 Positive Socioenvironmental Impact Index (IAp)

The positive socioenvironmental impact index of a cascade option (IAp) should express the intensity of a positive impact on the study area associated with the building of the group of projects in that cascade. The purpose of obtaining this index is to take account of positive socioenvironmental impacts in the final selection of the best cascade.

The positive socioenvironmental impact index is formulated in two steps:

- a positive socioenvironmental impact index of a cascade option relative to each element in the environmental system selected for evaluation;
- a positive socioenvironmental impact index of a cascade option for the study area (the aggregate of the indexes for each element).

Formulating the positive socioenvironmental impact index of a cascade on each element selected

The positive socioenvironmental impact index of each cascade option relating to each element (IAE) should represent the impact of the group of projects that make up the cascade on the element in question in the study area.

Formulating the positive socioenvironmental impact index of a cascade on the environmental system

The positive socioenvironmental impact index of a cascade on the environmental system (IAp) should express its total positive socioenvironmental impact on the study area. In other words, it should consider the positive socioenvironmental impacts caused by the cascade on all the elements.

This index is the weighted sum of the positive socioenvironmental impact indexes for the cascade on each element (IAE) as calculated previously.

$$IAp = \Sigma\ IAEi \times Pei \qquad (5.8.3.01)$$

where:

Pei	weighting factor for each element.

In order to keep the IAp values between zero and one, the weights P_{ei} must also be attributed on a continuous scale from **zero to one**, with the sum of the weights relative to the components equalling one.

The weights are used to ensure the relativization of the positive socioenvironmental impact indexes calculated for the cascades relating to each element in the study area. These weights should represent the relative importance of the impact processes of each element on the environmental system, which can be measured by the repercussion of these processes on the other elements.

5.8.4 Selection of One Cascade

First, the cascade options should be compared by means of a graphic representation, where, as in the Preliminary Studies (item 4.11.3), one of the axes represents the cost/energy benefit index and the other the negative socioenvironmental impact index. The best cascade will be chosen from the points near the lower left-hand corner of the graph.

One criterion for ranking the cascades is to use a **preference index**, ***I*** , which is the weighted sum of the cost/energy benefit indexes and negative socioenvironmental impact indexes, taking care first to standardize the cost/energy benefit index by dividing it by the reference unit cost, CUR (item 2.6):

$$I = p_{cb} \times \frac{ICB}{CUR} + p_{an} \times IAn \qquad (5.8.4.01)$$

where:

$p_{cb} + p_{an} = 1$

$p_{cb} \geq 0$ $\qquad$ $p_{an} \geq 0$ $\qquad$ $p_{ben} \geq 0$

where:

p_{cb}	weight to reflect importance relative to the objective of "minimizing the cost/energy benefit index";
ICB	cost/energy benefit index, in R$/MWh;
CUR	reference unit cost, in R$/MWh;
p_{an}	weight to reflect importance relative to the objective of "minimizing the negative socioenvironmental impact index"; and
IAn	negative socioenvironmental impact index.

In order to rank the cascades as part of a multi-objective approach, the weights $\mathbf{p_{cb}}$ and $\mathbf{p_{an}}$ should be designated taking into account not only the opinion of the experts directly involved in the studies, but also the input given and opinions expressed at the technical meeting held at the end of the Preliminary Studies (item 2.9), so as to reflect the broader context of the analysis and the period during which the studies are undertaken.

Before the final selection of one cascade is made, an additional analysis is recommended, incorporating the positive socioenvironmental impacts on the study area, as represented by the positive socioenvironmental impact indexes, IAp, to the ranking already undertaken.

The closer this index is to **one**, the better the circumstances of the cascade in question with respect to this requirement. In the case of the negative socioenvironmental impact index and cost/energy benefit index, the opposite applies, i.e. the closer the index is to **zero**, the better the cascade in question. In order to aggregate the **IAp** with the preference index ***I***, the IAp complement is introduced to bring the scale of the positive socioenvironmental impact index into line with the other indexes, i.e. **(1 – IAp)**. The **modified preference index *I´*** is given by:

$$I' = (1 - p_{ap}).I + p_{ap}\left(1 - IA_p\right) \qquad (5.8.4.02)$$

where: $0 \leq p_{ap} \leq 1$

p_{ap}	weight to reflect the relative importance of the positive socioenvironmenal impacts; and
IAp	positive socioenvironmenal impact index.

The weight $\mathbf{p}_{ap}$ should be defined in the same way as described for weights $\mathbf{p}_{cb}$ and $\mathbf{p}_{an}$. Given that the selection of the best cascade must take account of three objectives, and that the objective of maximizing positive impacts is supplementary to the other two, it is suggested that this latter's weight ($\mathbf{p}_{ap}$) should not exceed 0.25.

Sensitivity analyses should be undertaken and presented for the values given to the three weights ($\mathbf{p}_{cb}$, $\mathbf{p}_{an}$, $\mathbf{p}_{ap}$).

The SINV system can be used to calculate the cost/energy benefit index by using the "Economic-Energy Assessment" function, while the negative and positive socioenvironmental impact indexes can be calculated using the "Calculate Socioenvironmental Impact" function, and the preference and modified preference indexes can be obtained using "Final Multiobject Analysis". This last function is also used to rank the cascades by the two preference indexes, allowing the most advantageous cascade to be identified and sensitivity analyses to be undertaken on the values given to weights $\mathbf{p}_{cb}$, $\mathbf{p}_{an}$, $\mathbf{p}_{ap}$, in order to ensure the robustness of the cascades identified.

5.9 SEQUENCE OF CONSTRUCTION OF THE PROJECTS IN THE FINAL SELECTED CASCADE

The studies undertaken at the Inventory stage for defining the sequence of construction of the projects in a cascade option from an exclusively economic perspective are based on the criterion of incremental costs (item 5.9.1). According to this criterion, when considering two projects, the one with the lower incremental cost should be constructed first.

5.9.1 Incremental Cost

The incremental cost of a project or group of projects is calculated in much the same way as the cost/energy benefit index is (item 4.11.1). The only difference is the way the firm energy contribution is calculated, which must now assume that the projects already built are those that actually do exist plus those projects that, according to the sequence of construction for the cascade under analysis (item 5.9.2.), are set to be built before the project under analysis.

The "Eliminate" function of the SINV system uses the incremental cost to determine which projects from a cascade option have a ICB that is higher than the reference unit cost (CUR) and which should therefore be eliminated from the cascade.

5.9.2 Sequence of Construction from an Economic Perspective

The sequence of construction of the projects in a cascade option, taking into account only economic factors, is obtained by putting the projects from the cascade in increasing order of incremental cost. As calculating these incremental costs depends on knowing the sequence of construction, the sequence must be defined by an iterative process.

Starting out with a blank sequence, the cost/energy benefit indexes of each project and group of projects yet to be included in the sequence are calculated at each stage, assuming the pre-existence only of those projects that already exist and those that have already been included in the sequence, then choosing the project or group of projects with the lowest index to be added next. The process ends when all the projects from the cascade have been included in the sequence of construction.

One way to reduce the time taken to do this is to carry out the iterative process described above, first checking the outcome of adding the new projects one by one. If the resulting list of incremental costs is monotonically increasing, the sequence of construction can be considered complete. If not, the sequence of the series projects with decreasing marginal costs must be redone, testing the simultaneous addition of two or more projects until a complete list of monotonically increasing incremental costs is obtained.

The "Sequencing" function from the SINV system can be used to sequence the projects from the final selected cascade based on the incremental cost of each project.

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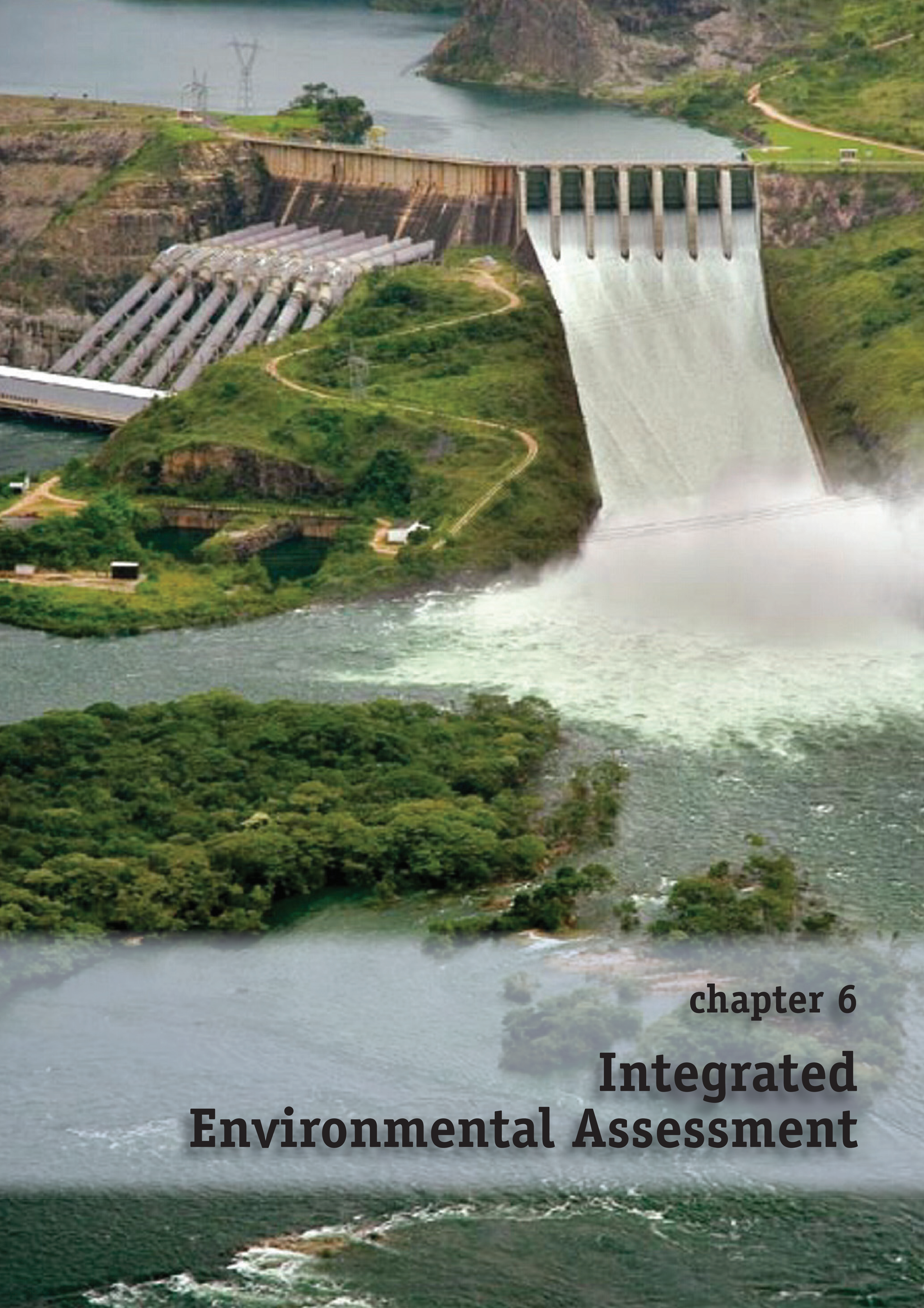


chapter 6

Integrated Environmental Assessment

CHAPTER 6

Inventory Studies and Integrated Environmental Assessments (IEA) both have the objective of analyzing river basins, with different yet complementary objectives. While in Inventory Studies the focus is on comparing and selecting the best cascade option for harnessing the hydroelectric potential of the basin, in an Integrated Environmental Assessment the aim is to evaluate the state of the natural and human environments in terms of their capacity to receive the group of hydropower projects that would make up the final cascade selected.

The main focus of the IEA is to assess the status of the environment in the river basin as a consequence of building the group of existing or planned projects. The cumulative and synergistic effects relative to this group of projects are assessed, considering different development scenarios for the river basin and taking into account the period over which the projects are to be built. Guidelines and recommendations are also put forward as inputs for the design and building of the projects and the environmental licensing process, with a view to assuring the socioenvironmental sustainability of the region as measured by sustainability indicators formulated within the ambit of the study.

In order to integrate the IEA procedures with the methodology used in the socioenvironmental studies undertaken as part of the process to select the best cascade option in the Inventory Studies, a number of additions were made to this edition of the Manual, in both the Preliminary and the Final Studies.

In order to meet the requirements of the IEA and make it possible for the socioenvironmental studies undertaken within the ambit of this Manual to be transformed into a stand-alone document to provide information for environmental entities for the future licensing of the projects, the studies relating to the final cascade are consolidated at this stage, highlighting the following:

- the most significant socioenvironmental aspects for the river basin;
- the results of the assessment of cumulative and synergistic effects brought about by the group of projects related to the different synthesis components and to the aspects selected for assessing positive impacts;
- the areas of environmental fragility and socioeconomic potentiality resulting from the building of the projects for the final cascade selected, considering the scenario of future development formulated for the river basin and assuming all the projects are built.

Guidelines and recommendations should also be prepared for the design and building of the future projects as a function of the issues highlighted in the analyses, with a view to assuring the socioenvironmental sustainability of the river basin and reducing the risks and uncertainties inherent to the process of harnessing its hydroelectric potential.

6.1 OBJECTIVE

The aim of this stage is to supplement and consolidate the socioenvironmental studies undertaken, supplying a broad picture of the future socioenvironmental status of the river basin once the projects in the final cascade have been built, considering:

- their cumulative and synergistic effects on the natural resources and human groupings;
- the current and potential uses of the water resources for current and future planning purposes, aiming to ensure that the generation of electricity is compatible with the conservation of biodiversity; and
- social diversity and socioeconomic development trends in the river basin.

In situations where the inventory studies for a river basin are being reviewed, those projects already in service must also be taken into account, as must those for which a concession or authorization has already been granted by ANEEL.

The other objectives to be met are:

- to develop sustainability indicators for the river basin, focusing on the water resources and their use for energy generation;
- to demarcate areas of fragility and identify the socioeconomic potentialities that could be leveraged by the building of the hydropower projects;
- to indicate conflicts over the different uses of the land and water resources in the river basin;
- to set down socioenvironmental guidelines and recommendations for the Feasibility Studies of the projects in the final cascade; and
- in the future, the guidelines and recommendations should serve as inputs for: (i) environmental studies in the river basin; (ii) the environmental licensing processes for the projects; (iii) any adjustments to projects or programs; (iv) procedures associated with the expansion of electricity supply; and (v) the building of the hydropower projects in the river basin, such that the risks and uncertainties associated with socioenvironmental development and the harnessing of energy in the basin are kept to a minimum.

6.2 STAGES OF THE IEA

The original methodology for IEAs[1] was structured into a number of steps, as shown below:

- Socioenvironmental diagnosis and potential conflicts – this aims to build up an overall picture of the river basin so that the most significant socioenvironmental elements in the current situation and their likely trends can be identified and located. Any existing or potential conflicts that would be exacerbated by building the hydropower projects must also be identified, as well as any conflicts or synergies with policies, plans or programs for the region.
- Distributed Environmental Assessment – this aims to subdivide the river basin into areas with features in common or which stand out from the others, so that the impacts from one or more projects in their vicinity can be identified and assessed, from which a picture of the combined effects on each of them and the effects that extrapolate their boundaries can be built up. In order to assess the impacts, indicators should be used that allow them to be quantified or qualified for the scenarios for different periods in time. For each subdivision, the indicators must be weighted to ensure that the local impacts are ranked by importance. Next, they should be mapped out so that the most sensitive areas can be identified. The local effects capable of leveraging cumulative or synergistic effects with other subdivisions must also be identified.
- Building up scenarios of socioeconomic development, taking into account the state of conservation of the natural resources, with mid- and long-term time frames.
- Integrated Environmental Assessment – at this stage, the interaction between the effects of the different hydropower projects, and between the different variables that characterize the socioenvironmental impacts deemed most significant, is assessed using indicators and simulation models based on the scenarios already built up. The indicators should be such that they permit an analysis to be made of the cumulative and synergistic impacts on the future scenarios. Guidelines and recommendations should be drawn up as a result of this process.
- Public Consultation – with a view to ensuring the involvement of the public throughout the studies, allowing stakeholders to participate and give their feedback on the results, and also to collect inputs and information for the studies themselves, meetings must be held to present, discuss and make contributions to the partial and final results of the IEA. The meetings must be held at venues in all the different states that the river basin occupies.

When this Manual was published, IEAs were a relatively recent addition to the electricity sector planning process, the first having been undertaken in 2005. Between 2006 and 2007, six IEAs were carried out. The procedures were still being developed, with the goal of future consolidation. Further information on these procedures can be obtained from Annex F, which provides a summary of the methodology used in three of the IEAs[2].

The methodology used in the IEAs is consistent with that used in the socioenvironmental studies undertaken in the Preliminary and Final Studies, described in chapters 4 and 5, not only in terms of the procedures used but also the content and scope, despite the difference in the focus of the studies. In order to ensure that the Inventory Studies and IEA are mutually compatible, throughout the description of the procedures for the socioenvironmental studies required for the selection of cascade options, procedures have also been introduced to meet the requirements of the IEA. The procedures for consolidating the studies are set out below.

1 Termo de referência para a Avaliação Ambiental Integrada para a bacia do Rio Uruguai, MMA, 2005.

2 Methodology developed by EPE/Sondotécnica (2007) for the IEAs of the Paranaíba, Doce and Paraíba do Sul river basins. Other examples of methodologies can be found on the EPE website (http://epe.gov.br/Lists/MeioAmbiente/MeioAmbiente.aspx).

6.3 INTEGRATION OF THE SOCIOENVIRONMENTAL STUDIES WITH THE IEA

Some of the activities required to meet the objectives of the IEA are already included in the environmental studies at the Planning stage, the Preliminary Studies and the Final Studies, as shown below:

- Planning (chapter 3): establishing a communication channel for the presentation of information about the studies to be undertaken in the river basin with the environmental and river basin entities;
- Preliminary Studies (chapter 4): Data Gathering and Studies (item 4.1), Socioenvironmental Diagnosis (item 4.3) and Assessment of Negative Socioenvironmental Impacts per Project (item 4.8). At the end of the Preliminary Studies, a technical meeting is held by the Ministry of Mines and Energy to present the findings of the studies (item 2.9);
- Final Studies (chapter 5): Consolidation of the Socioenvironmental Diagnosis (item 5.2), Assessment of the Socioenvironmental Impacts of the Projects (item 5.4) Comparison and Selection of Cascades (item 5.8).

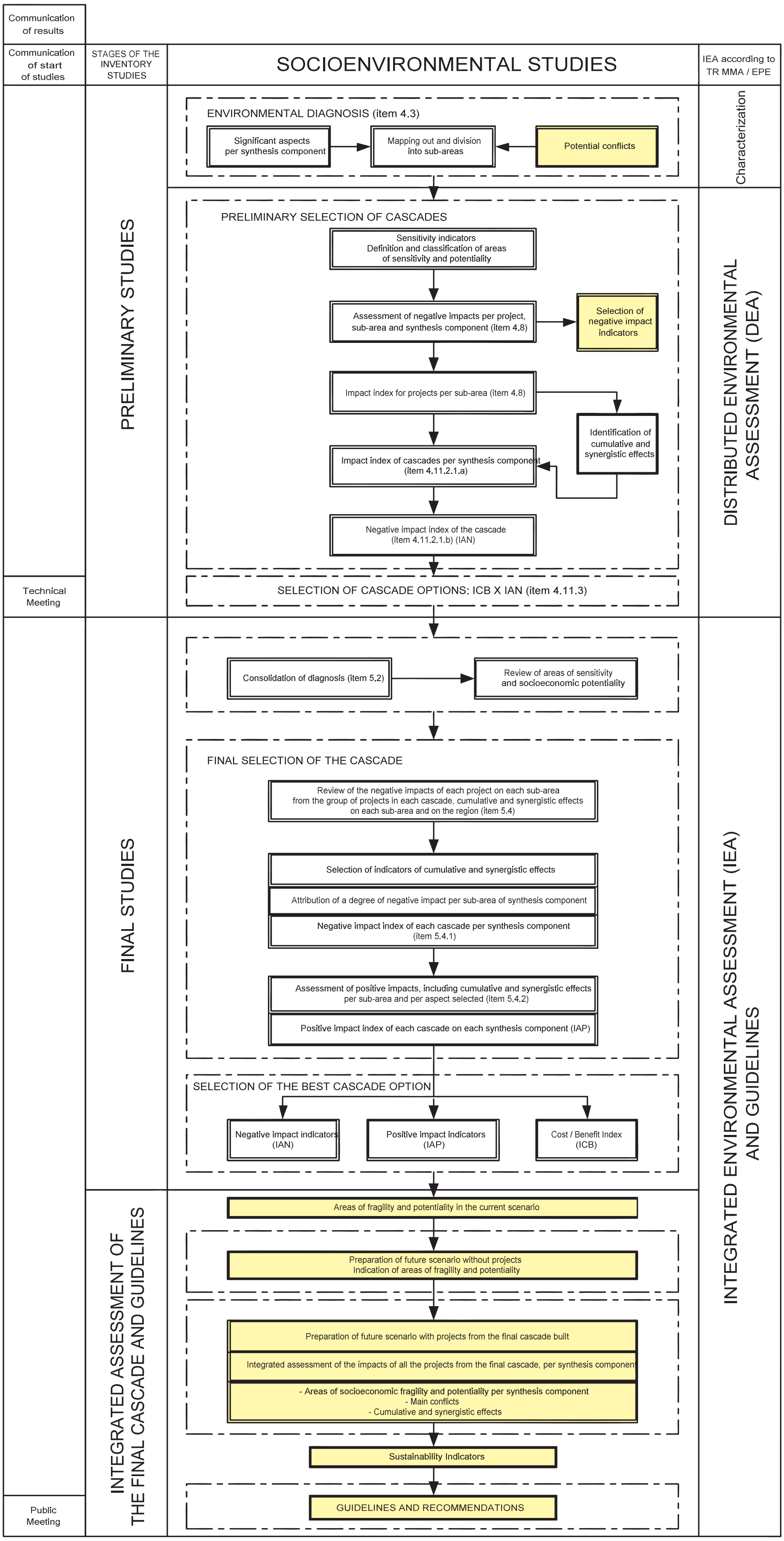
However, in addition to these, further studies are also required to supplement the integrated environmental assessment of the final cascade selected.

The diagram in Figure 6.3.01 shows how the socioenvironmental studies from the previous stages are integrated with the supplementary studies for the IEA. The activities required for the main items in the IEA are highlighted.

As already mentioned, the IEA of the final cascade can be used as a stand-alone document, separate from the Inventory Studies report. Below are all the activities needed to do this, which are split into two sections:

- organization and presentation of the information concerning the final cascade contained in the Preliminary and Final Studies, following the structure set forth for the IEA;
- supplementary activities to fulfill the scope of the IEA.

Figure 6.3.01 – Diagram of integration between the socioenvironmental studies from the Inventory Studies and the IEA



6.4 ORGANIZATION OF INFORMATION FROM PREVIOUS STUDIES

6.4.1 Environmental diagnosis and potential conflicts

Taking the data and information consolidated in the Socioenvironmental Diagnosis (items 4.3 and 5.2), a brief description should be prepared of the main characteristics of the river basin and the most representative socioenvironmental indicators, giving precedence to the most significant socioenvironmental issues for each synthesis component, the areas of environmental management and the main existing and potential conflicts relating to the harnessing of the hydroelectric potential of the river basin. This description should be illustrated with maps of each synthesis component and their respective sub-areas, with details given of the criteria upon which these subdivisions were based, as well as their main characteristics.

The local and regional conflicts identified in the course of the study must be identified and located on the maps, and associated with the projects and the cascade, while identifying the main parties involved.

6.4.2 Main characteristics of the final cascade

A brief description should be provided of the final cascade, accompanied by a map showing where each of its projects is located. The main energy-related features of each project must be included: installed capacity, water levels, regulating volume, and information on the other uses of the waters.

6.4.3 Distributed Environmental Assessment (DEA)

Areas of environmental sensitivity and socioeconomic potentiality

The environmental sensitivity and socioeconomic potentiality indicators must be included, as well as the variables that comprise them and their respective values. The areas of sensitivity and potentiality in the sub-areas for each synthesis component should also be mapped out and classified.

Main impacts of the projects and the cascade

The negative socioenvironmental impacts of each project on each synthesis component should be presented per sub-area, as identified and assessed in the Preliminary Studies (item 4.8) and reviewed in the Final Studies (item 5.4.1). The impact indicators used and impact indexes attributed to the projects must also be presented.

The main negative and positive impact processes arising from the cascade and affecting each synthesis component and sub-area, as identified and assessed in the Final Studies (items 5.4.1 and 5.4.2), should be presented, as should their cumulative and synergistic effects, highlighting the respective indicators, assessment criteria, negative and positive impact indexes per cascade, and the weights used to formulate the indexes.

6.5 SUPPLEMENTARY ACTIVITIES FOR THE IEA

In this section, the procedures for conducting the supplementary activities are described.

6.5.1 Areas of fragility and potentiality in the current scenario

The areas of fragility[3] in each synthesis component must be identified and mapped out. These are the areas of sensitivity where the main impact processes arising from the projects in the cascade are located. The areas of fragility are identified by crossing the maps of the areas of sensitivity for each synthesis component with the spatial distribution of the impacts arising from the group of projects per synthesis component[4]. The areas of socioeconomic potentiality must also be identified.

6.5.2 Preparation of reference scenario

In order to analyze the final cascade, a long-term scenario of socioeconomic development (20 years) must be built up for the region, to be used as a reference for the analyses. It should take into account the state of conservation of the natural resources, but not the harnessing of the hydroelectric potential proposed for the final cascade. In situations where the inventory studies for a river basin are being reviewed, only those projects already in service and those for which a concession or authorization has already been granted by ANEEL should be considered.

In order to build up the reference scenario, projections should be made of how the group of indicators of economic, social and environmental conditions in the study area and the institutional organization of this area will change over the time frame established. The scenario must be compatible with the scenario created to forecast multiple water uses (item 5.1.3).

The most significant socioenvironmental issues and the topics deemed of priority identified in the previous studies, their future trends and their spatial distribution should be used in the identification and location of the main pressures from socioeconomic development on the river basin: on the water and other natural resources; on land use, especially in the areas of sensitivity and potentiality, in the areas with restrictions on use and areas of environmental management; and on the local population's ways of life and the land organization and dynamics.

The following must be considered:

- the economic, social, environmental and cultural indicators that best represent the issues deemed of relevance in the study area;
- the scenarios of water uses in the river basin considered in item 5.1.3;
- development policies, plans and programs for the study area that cover the same time frame, and their main interactions;
- existing and potential conflicts;

3 The definition of "fragility" used here is as follows: "fragility of the environment is understood as meaning the degree of susceptibility to damage as a result of given actions, and can also be defined as the inversion of the capacity to absorb potential alterations without any loss of quality," (Angel Ramos, cited in Iara Verocai, FEEMA/PETROBRÁS, 1990).

4 The procedure for identifying areas of fragility is based on the methodology used in the IEAs for Doce and Paranaíba river basins. (Sondotécnica, 2007).

- the government's policies for the environment, water resources, social development, and agricultural and industrial development, and all policies relating to international agreements, existing environmental quality standards, and standards established by other instruments, such as zoning;
- the risks and trends for future deterioration in socioeconomic and environmental conditions.

A synthesis map should be prepared that represents the socioenvironmental status in the river basin over the time frame established, indicating the areas of environmental sensitivity and socioeconomic potentiality in the absence of the projects.

6.5.3 Integrated environmental assessment of the effects of building all the projects in the final cascade

Future scenario with all the projects from the cascade built

Based on the reference scenario, a scenario should be developed that assumes that all the projects included in the final cascade are built in 20 years.

Integrated analysis of the final cascade

The aim of this stage is to present the results of the assessment of the effects of building the projects that make up the final cascade, including the cumulative and synergistic effects, considering the future scenario for development in the region. The main impacts of the cascade on the synthesis components should be identified, as should those resulting from the interactions between the components. The impact indicators used to represent these processes should be listed. The most significant socioenvironmental processes in the river basin under this future scenario should be highlighted and mapped out, emphasizing those that should be addressed in the design and building of the future projects.

The most sensitive aspects of the environmental system and for each synthesis component should be highlighted and the extent of their influence identified. All areas of fragility for each synthesis component should be identified and mapped out, which are obtained by crossing the areas of sensitivity identified for each synthesis component with the area to be impacted by the group of projects per synthesis component. The areas of socioeconomic potentiality should also be identified.

A synthesis map representing the status of the river basin under the forecast scenario should be prepared. The integrated analysis of the final cascade should highlight:

- the areas of fragility relating to the most significant impacts arising from the group of hydropower projects;
- the areas of socioeconomic potentiality;
- the areas where the most significant cumulative and synergistic effects are identified;
- existing and potential conflicts, such as:
- conflicts over the way the urban and rural populations are resettled;
- changes in land use, breakdown of social relations and production base;
- property speculation;
- interference in archaeological, historical and cultural heritage;
- areas with conflicts over land use;
- interference in natural resources for development;
- loss of tourism potential;
- loss of natural resources (minerals, biodiversity);

- conflicts over the multiple uses of water resources (navigation, energy generation, withdrawals for human and livestock water supply, dilution of wastewaters, irrigation and flood control); and
- interference in indigenous lands and federal, state and municipal conservation areas.

6.5.4 Formulation of socioenvironmental sustainability indicators for the region

Sustainability indicators are formulated for the river basin with the purpose of setting reference parameters for the guidelines and recommendations to be proposed in the IEA.

The formulation of the environmental sustainability indicators must draw on the socioenvironmental conditions set out in the current and prospective scenarios, and are represented by the fragility and potentiality indicators relating to the synthesis components.

These conditions are crossed with the data and information contained in federal, state and municipal legal provisions and standards concerning environmental preservation/conservation, such as ecological/economic zoning, legislation on land use and occupation, regulations on conservation areas and the use of water resources, river basin plans, and policies or plans for the social and economic development of the region. Aside from these references, national and international scientific data on socioenvironmental sustainability, and perceptions of the regional community's aspirations for their living conditions and environmental conservation in the study area should be included amongst the reference elements used to formulate the socioenvironmental sustainability indicators for the river basin.

6.5.5 Guidelines and recommendations

The analyses undertaken should provide the data needed to set down guidelines, which should have the following goals:

- to serve as inputs for the socioenvironmental assessments of projects to be analyzed within the ambit of Ten-Year Electricity Sector Expansion Plans;
- to contribute towards the consolidation of a georeferenced information system which, once expanded and made operational, could become a key planning and environmental management tool for the river basin, serving not only the needs of environmental entities, but also, and more importantly, those of the communities living there;
- to contribute towards the design and building of hydropower plants, taking into account the main issues identified, land use and occupation, regional development, critical areas and potentialities; and
- serve as a guide for future environmental studies for hydropower projects and environmental licensing processes for projects at their planning stage or being licensed by the competent environmental authorities.

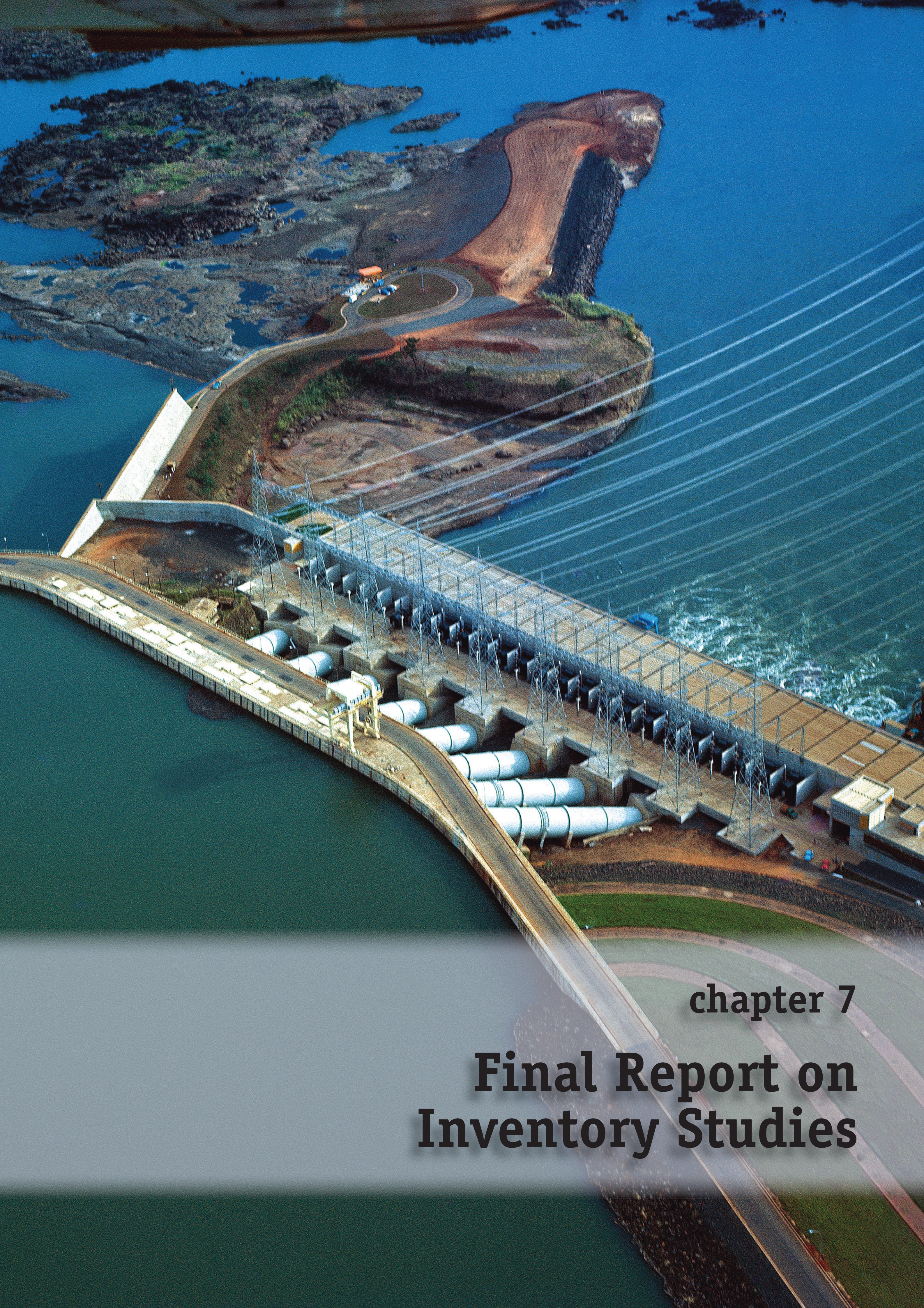
Recommendations should also be made to:

- provide more in-depth or supplementary information for the IEAs when their results contain a high level of uncertainty as to the reliability or suitability of the data and information used, and for undertaking future environmental studies of interest to the electricity industry;
- foster and develop activities to promote integration in the river basin, taking into account the multiple stakeholders in the use of the waters and lands, and the different public and private entities operating in it;

- supplement Feasibility Studies for future projects; and
- assist in the building of the projects.

6.5.6 Final Communication of the Studies

At the end of the studies, a public consultation meeting is held by the Ministry of Mines and Energy to present the results of the final cascade selected, as well as the IEA, its guidelines and recommendations.



chapter 7

Final Report on Inventory Studies

CHAPTER 7

The Final Report is the document that presents the Final Studies of the Hydropower Inventory Studies of a river basin, the results obtained and the recommendations made for the Feasibility stage. It should include the energy potential of the river basin and the scheme proposed to harness this potential, while also containing a characterization of the projects, the main socioenvironmental issues associated with them, the key points highlighted in the Integrated Environmental Assessment, and the socioenvironmental guidelines and recommendations for the building of the projects, which are also to be used in the Feasibility Studies and licensing processes. Below, basic recommendations are set out to ensure the standardized presentation of the contents and results of Hydropower Inventory Studies.

7.1 ORGANIZATION OF THE REPORT

The Final Report on Hydropower Inventory Studies should comprise a General Report and its Appendices, which should be printed in as few volumes as possible.

Ideally, the information from the General Report should be organized into the sections listed at the end of this chapter. The appendices should contain a description of the studies undertaken, in order to characterize them in greater depth and breadth. Depending on their size, the appendices can be compiled and printed in separate volumes.

The volumes should be printed in A4 format (ABNT). Any figures that cannot be adapted to this format can use A3.

The text of the General Report and its appendices should be organized in line with the guidelines set out in Brazilian standard (ABNT) NB-69. It should also meet the requirements of the regulation entity and should be submitted in a hard copy and in digital form.

7.2 GENERAL REPORT

7.2.1 General information

The General Report should be bound with a letter of presentation, by which the executive entity submits the Final Report for the appreciation of the relevant authority.

This letter should be followed by the title page, which should contain the following minimum information: title of the report, name of the executive entity, and issue date.

Next should come the contents and the lists of appendices, tables, and figures. The contents should contain first- and second-level section headings. The lists of figures and tables should be numbered according to the first-level sections they are part of.

The foreword should provide a presentation of the work. As a rule, it should be brief and should identify or highlight the main characteristics of the studies, the results obtained and the means employed to obtain them. Normally, the foreword will include the references and credits to individuals and organizations that did not participate directly in the work, but still made significant contributions.

After the foreword comes an abstract, containing excerpts from the most important parts of each chapter, providing an overview of the studies undertaken and conclusions reached. The abstract may only contain information from the main text of the report; it cannot include any information from other sources. Some schematic diagrams or condensed tables can be included in the abstract, provided they are pertinent. A compilation of the summarized table of data from the Final Studies must be included in the abstract.

It is common practice to begin the abstract with a brief description of the aims of the study, the location of the river basin, the amount of energy to be harnessed, the number of projects identified in the Final Studies, the total estimated cost of the final cascade selected, and the corresponding unit costs, as well as the information about the socioenvironmental studies and other pertinent information.

The data that has been gathered, analyzed and processed should be described in the General Report or in specific Appendices. In order to make this information available for future use, it should also be submitted in digital form. The formats of the digital files containing the data used and results obtained are available from the concession-granting authority. The use of the correct format ensures that the data can be transferred directly to the database, where all information from Inventory Studies is stored by this authority. This information should include the maps and georeferenced data used or created in the Inventory Studies.

7.2.2 Introduction

The introduction should contain the following sections:

- aim of the studies;
- characterization of the area studied;
- previous studies; and
- basic criteria.

In defining the aim of the studies, the reasons behind the execution of the studies must be presented, and the scope of the studies of each of the main rivers and their tributaries must be justified. The recommendations made in the Preliminary Studies must also be included, as should the means by which they were addressed in the Final Studies.

The study area should be characterized by its geographical location, physiography, political and administrative organization, human occupation and economic activity, both current and potential, as well as its main socioenvironmental issues. Maps and graphics should be included in this section to illustrate the points made.

Any prior studies made of the river basin or contiguous areas should be described critically, noting the results obtained, the basic criteria and the methodologies used.

Finally, the basic criteria selected for use in the Inventory Studies should be listed and justified. These criteria correspond to the energy, economic, multiple water use and socioenvironmental parameters employed to define, compare and select the different cascade options. Any variations between the methodology used and that set forth in this manual must be justified and described.

7.2.3 Planning

This section should contain a summary of the Planning report, using the sub-items suggested in the Format of the Final Report provided at the end of this chapter.

The basic data gathered, analyzed and processed throughout this and the successive stages of the Inventory Studies should be presented once only, in item 4.2 of the Final Report. This should include a summary of communications with environmental entities and water authorities to notify them about the studies planned.

7.2.4 Preliminary Studies

A brief description should be provided of the Preliminary Studies undertaken, based on the list of items proposed. It should also include a summary of the issues highlighted at the technical meeting held at the end of this stage.

As suggested in the previous item, in order to present the information in a more reader-friendly fashion and avoid repeating the same information, all the data gathered, analyzed and processed at this stage and studied in greater depth in the Final Studies should be presented in detail or in summary form, and included in the Appendices in question, and in Section 4, Final Studies.

7.2.5 Final Studies

This section contains the consolidated results of the Final Studies, which should be organized into the items suggested below, highlighting the key aspects noted from each section:

Cartographic Studies

Description of the aerial and land reconnaissance activities, the topographic and aerial photogrammetric surveys, and the other surveys undertaken to determine the cartographic parameters, following the procedures set forth in this manual. The cartographic knowledge of the study area achieved by the end of the studies should be illustrated in map form. If necessary, details of the studies may be presented in a corresponding Appendix.

Geological and Geotechnical Studies

Description of the studies undertaken to identify the general geological features of the river basin, and the geology of the reservoirs and dam sites. A general geological map of the river basin should be included, as should a schematic cross-sectional diagram of each site, indicating the main lithological and stratigraphic features identified.

Hydrometeorological Studies

Description of the analyses undertaken to test the consistency of the hydrometeorological data, and other studies to determine the hydrological parameters to be used in the Inventory Studies, as well as the climatology, sedimentology and water quality investigations and studies undertaken. Figures giving a concise representation of the basic hydrological and climatological information should be included in this section, such as isohyetal maps, maps of drainage areas, hydrographs from the main gauging stations, maps showing the regional extent of hydrological features, including long-term minimum and mean flows, minimum and mean flows for the critical period of the reference system, flood flows for diversion, design flood flows for spillways, sediment transport rates, and others. The basic gauging station network and the equations used to transfer the fluviometric information in terms of time series of flows, as well as the other hydrological data of relevance from the gauging stations used for the project sites should all be presented clearly. The details of the studies should be included in the corresponding Appendix.

Socioenvironmental Studies

Description of the investigations and studies undertaken to build up an understanding of the environmental system, represented by the aquatic and terrestrial ecosystems, ways of life, territorial organization, regional economy and indigenous peoples; preparation of the socioenvironmental diagnosis, and division of the study area into sub-areas; assessment of the negative and positive socioenvironmental impacts and indexes of the cascades, and the weights used. This section should also contain maps of the synthesis components with the sub-areas marked on them, and a presentation of the areas of sensitivity and potentiality. The environmental assessment of the cascades studied and the projects from the final selected cascade should be presented clearly, with the corresponding Appendix containing the details of these and the other studies undertaken.

Studies of Multiple Water Uses

The diagnosis of the multiple water uses in the river basin should be presented, as undertaken in the Preliminary Studies and consolidated in the Final Studies, which provided the foundation for the scenarios of multiple water uses formulated for use in the Final Studies. The sources of the information used to formulate these scenarios should be cited and justified. The methodologies used to formulate or adapt the scenarios of multiple water uses (when they are based on sector plans) should also be described in detail.

Studies of Cascade Options

This section should describe the cascades selected in the Preliminary Studies, and the Final Studies of these cascades and the variations derived from them. The cascades investigated in the Preliminary Studies should be described in brief. The variants resulting from new data and corresponding adjustments should also be described, as should the analysis of the conclusions reached, a list of the cascades selected to go through to the Final Studies, their reformulations and all additional studies undertaken.

For the energy studies, all the simplifications adopted should be listed and justified. When the SINV system is not used, the mathematical model used to simulate the system of power plants must be justified. A table of summarized information should be included for each cascade, indicating its respective firm energy contribution to the reference system and corresponding installed capacity. These tables should also contain a description of the projects making up each cascade: their energy characteristics (live capacity, reference head, installed capacity, etc.) and their cost/energy benefit index.

The socioenvironmental studies should contain a brief description (details to be included in the corresponding Appendix) of the assessment of the negative and positive socioenvironmental impacts of the cascades (item 5.4), highlighting their cumulative and synergistic effects.

For the final layouts, dimensioning and cost estimates, a description should be given of the methodology used to dimension the civil construction work and equipment whenever procedures are used that differ from those set out in this manual. The guiding principles used for designing the layout of the structures for the projects and for determining the quantities and costs should also be provided, making specific reference to any deviation from the criteria and instructions set out in this manual.

Finally, a general, summarized table should be included for each cascade and its respective projects, indicating their location, maximum gross head, live storage of the reservoir, reference capacity and cost estimates, using the OPE format (items 5.5 to 5.7), and including their main socioenvironmental impacts.

Comparison and selection of cascades

This section should contain a description of the comparative analyses of the cascades from an energy-economic and socioenvironmental perspective, as shown in item 5.8 of this manual. A summarized table should be included with the cost/energy benefit index, negative socioenvironmental impact index, preference index, positive socioenvironmental impact index and modified preference index of each cascade. This table should also set out the weights used for calculating the preference and modified preference indexes.

A sensitivity analysis of the weights used to calculate the preference and modified preference indexes should also be provided.

7.2.6 Characterization of the final selected cascade

This section should contain the studies undertaken of the final selected cascade, including not only the adjustments and refinements made to its projects, but also a characterization of special cases, such as multi-purpose projects, pumped storage facilities and any others identified in the studies. The final results of the Inventory Studies should be presented, consolidated in the final selected cascade, after adjustments, which should represent the most advantageous scheme for the river basin.

A general description should be provided of the cascade proposed, covering energy, geographical and socioenvironmental data. The main negative and positive impact processes associated with the cascade should also be included, highlighting all cumulative and synergistic effects.

At the end of the section, a summarized map should be presented showing the location of the projects and the longitudinal sections of the rivers indicating the maximum normal water levels in the reservoirs, and others showing the areas of resettlement and main interferences brought about by the cascade, highlighting areas of fragility and potentiality in the current scenario.

Characterization of the projects

Each project from the final selected cascade should be characterized in this section, including a brief description, a map showing the project's location, a general plan, longitudinal sections of the water intake, powerhouse, spillway and diversion scheme, and cross-sections of the dam axis and intakes, as well as the elevation/area and elevation/volume curves of the reservoir and the rating curve of the tailrace canal. These three curves should be accompanied by lists of the measurements that served as the basis for the plotting of the curves. A table of the mean monthly flows for each site should also be included.

The Technical Form for each project should also be included, using the template from Annex E, so that the information can be transferred to the database held by the concession-granting authority.

7.2.7 Integrated environmental assessment of the final selected cascade

Initially, a summary of the long-term scenario for socioeconomic development in the region should be presented, taking account of the state of conservation of the natural resources in the river basin, but not the harnessing of its hydroelectric potential by the projects in the cascade.

Next, a long-term scenario should be presented, assuming that all the projects that make up the final selected cascade have been built and highlighting their main impacts, including cumulative and synergistic effects. The most significant socioenvironmental processes should be mapped out and highlighted, as well as the most significant conflicts in the river basin for this future scenario, emphasizing those that should be addressed in the design and building of the projects in the future. All areas of fragility and socioeconomic potentiality should also be identified and mapped out.

The socioenvironmental sustainability indicators chosen for the river basin that served as a basis for formulating the guidelines and recommendations should be presented.

7.2.8 Conclusions and recommendations

This section should not only provide a general overview of the results of the studies, but should also make recommendations for the Feasibility Studies, including specific suggestions for additional studies and surveys. The guidelines and recommendations should be presented for the purposes of electricity sector planning, the design, building and socioenvironmental management of the projects, and the environmental licensing of future projects, established in the IEA.

7.2.9 Supplementary Information

At the end of the main sections of the General Report, the following should be added:

- list of the main terms used and their definitions; and
- list of abbreviations and symbols used in the texts and figures.

7.2.10 Database

All the information gathered, used and produced in the Inventory Studies that is cited in the previous items and included in the tables and maps must be submitted in digital form, using the format designated by the concession-granting authority. As for the maps and georeferenced information, these should be submitted in files that are compatible with the geoprocessing systems. Details of the format of the files can be obtained from the concession-granting authority.

7.3 APPENDICES

The appendices to the General Report should contain the details of the surveys and studies undertaken, and should present the basic data collected in an organized way so that the document can be understood clearly and fully.

The number of appendices will depend on the nature of the work undertaken, but the following will certainly have to be prepared:

- Appendix A – Topographic Studies
- Appendix B – Geological and Geotechnical Studies
- Appendix C – Hydrometeorological Studies
- Appendix D – Socioenvironmental Studies
- Appendix E – Studies into Multiple Water Uses
- Appendix F – Studies of Cascade Options
- Appendix G – Report on the Integrated Environmental Assessment
- Appendix H – Organization and Background on the Studies

7.3.1 Appendix A – Topographic Studies

This appendix should list the topographic data and describe the topographic surveys of the river basin, the reservoirs and the dam sites undertaken as part of the Inventory Studies.

The information and data obtained should be described critically, including its technical characteristics and the survey methodology and survey dates. Maps should be prepared indicating the areas covered by aerial photogrammetric surveys and triangulation and leveling networks. The main topographic marks should be listed, with their altitude, geographical coordinates, if available, and instructions for their future location.

7.3.2 Appendix B – Geological and Geotechnical Studies

This appendix should list the data and describe the geological and geotechnical investigations undertaken in the river basin, the reservoirs and the dam sites.

The geological photo-interpretations and surface and sub-surface prospecting should be described, as well as the local tests and laboratory tests carried out.

7.3.3 Appendix C – Hydrometeorological Studies

This appendix should list the primary hydrological and climatological data on the river basin and the analyses and studies undertaken.

7.3.4 Appendix D – Socioenvironmental Studies

This appendix should list the data gathered and describe the surveys and studies undertaken for the socioenvironmental diagnosis of the river basin, the impact assessments and the findings obtained, as well as the extent to which they would interfere with the final selected cascade.

7.3.5 Appendix E – Studies of Multiple Water Uses

This appendix should list and critically describe the studies into the different uses of the waters in the river basin, both those undertaken by third parties and those conducted during the Inventory Studies, so that a diagnosis can be prepared and scenarios can be built up for the multiple water uses.

7.3.6 Appendix F – Studies of Cascades

This appendix should contain a detailed description of the studies and analyses involved in formulating, assessing, comparing and selecting the cascade options.

In order to broaden the scope of the Final Report on Inventory Studies, brief descriptions should be provided of the Preliminary Studies undertaken for the first selection of cascade options. The cascades selected to go through to the Final Studies should be described and the choice of their location should be justified. The energy dimensioning process should be discussed in great enough detail for the results to be understood. In the assessment of quantities and costs, the main characteristics of the structures taken into account should be indicated for each cascade option. The selection of the cascades to go through to the Final Studies should be illustrated using tables that permit a clear understanding of the reasons for their choice.

When it comes to the Final Studies, in all situations where the models recommended in this manual are not adopted, the mathematical models used should be presented, and their parameters and criteria should be given. In the assessment of quantities and costs, the main features of the structures taken into account should be given, alongside a summary of the cost estimates. Details should be given of the comparison of the cascades, clearly indicating the values of the cost/energy benefit index and corresponding negative and positive socioenvironmental indexes, as well as the weights used when calculating the preference and modified preference indexes.

7.3.7 Appendix G – Report on the Integrated Environmental Assessment

This appendix should provide a consolidation of the socioenvironmental studies of the final cascade selected, so that the studies undertaken following the procedures set out in this manual can be transformed into a stand-alone document with the objective of providing information for environmental entities as part of the licensing process for the projects in the future.

As such, this appendix should have the following structure:

1. Summary of the studies undertaken for the selection of the cascade options

Here, details should be given of the comparisons between the cascades in the Final Studies, clearly indicating the values of the cost/energy benefit index and corresponding negative and positive socioenvironmental indexes, as well as the weights used when calculating the preference and modified preference indexes.

2. Description of the main features of the final cascade selected

3. Socioenvironmental diagnosis and potential conflicts

3.1. Characterization elements

3.2. Identification of existing and potential conflicts

3.3. Aspects of relevance

3.4. Sensitivity indicators

3.5. Indicators of socioeconomic potentiality

4. Distributed Environmental Assessment

4.1. Mapping of sensitivity and potentiality indicators

4.2. Selection and scope of indicators of cumulative and synergistic impacts

4.3. Assessment of the cumulative and synergistic effects of the cascade per synthesis component and per sub-area

5. Integrated Environmental Analysis of the final selected cascade

5.1. Reference scenario

5.1.1. Basic assumptions and models

5.1.2. Reference scenario without the projects being built – areas of sensitivity and potentiality

5.1.3. Reference scenario with the projects built

5.1.3.1. Areas of fragility and potentiality per synthesis component and per project

5.1.3.2. Areas of socioeconomic potentiality per project

5.2. Formulation of socioenvironmental sustainability indicators for the region

5.3. Guidelines and recommendations

6. Final communication of the studies

7.3.8 Appendix H – Organization and summary of the work undertaken

This appendix should include the data on the operational organization and execution of the Inventory Studies.

The organizational chart of the operational team should be included, as should details of any additional agreements signed with third parties, and a brief summary of the main parts of the work undertaken.

The work schedule should also be included, citing the maximum number of people allocated to the studies and the number of man-hours involved in undertaking them.

The equipment and materials should be described, as should the logistic support and any special services provided by outsourced companies and/or consultants.

Finally, an economic and financial statement of the Inventory Studies should be presented.

Format of the Final Report on Inventory Studies

LETTER OF PRESENTATION
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LIST OF APPENDICES, TABLES AND FIGURES
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    1.1. OBJECTIVE OF THE STUDIES
    1.2. CHARACTERIZATION OF THE STUDIED AREA
    1.3. PRIOR STUDIES
    1.4. BASIC CRITERIA
2. PLANNING
    2.1 SUMMARY OF THIS STAGE
        2.1.1 Preliminary identification of the cascade options and estimate of energy potential
        2.1.2 Program of work to be undertaken and cost estimate
    2.2 BASIC DATA GATHERING AND ANALYSIS
    2.3 IDENTIFICATION OF DAM SITES
    2.4 IDENTIFICATION OF RESTRICTIONS
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3. PRELIMINARY STUDIES
    3.1 SUMMARY
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        3.2.1 Cartography
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    3.3 MULTIPLE WATER USES
        3.3.1 Diagnosis of multiple water uses
        3.3.2 Scenario of multiple water uses in the river basin
    3.4 SOCIOENVIRONMENTAL DIAGNOSIS
    3.5 FORMULATION OF CASCADE OPTIONS
    3.6 TECHNICAL FORM FOR PROJECTS
    3.7 ENERGY STUDIES
    3.8 PROJECT LAYOUTS
    3.9 ASSESSMENT OF SOCIOENVIRONMENTAL IMPACTS PER PROJECT
    3.10 STANDARD ELETROBRAS COST ESTIMATE (ORÇAMENTO PADRÃO ELETROBRÁS)
    3.11 DIMENSIONING AND COST ESTIMATE
    3.12 COMPARISON AND SELECTION OF CASCADES
        3.11.1 Cost/energy benefit index
        3.11.2 Negative socioenvironmental index
        3.11.3 Selection of cascades
        3.11.4 Characterization of cascades selected

chapter 1

Appendix I

APPENDIX I

APPENDIX I OVERVIEW OF HYDROELECTRIC POWER IN BRAZIL AND AROUND THE WORLD AND THE INSTITUTIONAL CONTEXT FOR HYDROPOWER INVENTORY STUDIES IN BRAZIL

1 Introduction

Water is a resource that is essential for human life. It is the responsibility of the Union to manage its water resources, ensuring the people of the nation the benefits deriving from human and animal consumption, food and energy production and other uses.

Using hydraulic energy for generating electricity is one potential use of water. Other uses, such as human consumption and food production, can in some situations provide greater benefits to society. Therefore, whenever the use of water resources for generating energy is being considered, the multiple water uses should be taken into account.

Over the years, Brazil has harnessed its hydroelectric potential, making it self-sufficient in electricity by making use of this low-cost renewable source of energy and developing national technology.

As the electricity industry is a major user of water, it has the responsibility and duty to plan the use of this resource as an input for the cost-effective, optimized production of electricity, alongside the other users of water.

This is a topic that is currently legislated and regulated by several government entities and involves several different agents.

This Appendix to Chapter 1 of the Manual for Hydropower Inventory Studies presents an overview of the issues concerning hydroelectric power and its potential from a Brazilian and international perspective, as well as the institutional context in which Inventory Studies are undertaken in Brazil. It also explains certain key issues involved in the relationship between institutions and the legal processes and procedures needed for hydropower generation at this stage of planning.

2 Hydroelectric Power Around the World

2.1 Introduction

This Manual provides guidelines for studies that are undertaken in a context where long-term strategies for the country's development are still being examined. Evidently, there are other energy sources that compete with hydropower. It is therefore inevitable and even helpful to analyze hydropower generation from a geopolitical, strategic and global standpoint, in a context where social and environmental issues are gaining in importance and decisiveness.

Expanding energy generation means addressing an insoluble paradox in society: people want more energy for development and comfort, but they also question how it is produced, and particularly how it affects the environment and impinges on the other potential uses of natural resources. Good planning, transparency and democratic decision-making processes are the only way to resolve such conflicts. In this sense, Hydropower Inventory Studies are especially important, because it is at this stage and with this foresight that one can get a real picture of the potentialities and the impacts of hydropower projects on a river or river basin.

The world has reached a point when two major challenges must be faced:

- doubts as to how long it will be before the world's oil reserves are depleted; and
- changes to the environment on a global scale.

In this context, renewable energy has started to warrant greater attention worldwide. Indeed, the state of affairs is such that even nuclear energy, which has been severely criticized in recent years, is starting to be included in the energy mix, as it does not contribute to global warming. Conservation policies and the increased efficiency of equipment and energy users will certainly be important in shaping public policy for the future. So now more than ever the qualities of different energy sources should be analyzed from the perspective of promoting energy efficiency and preserving natural resources.

Obviously, every kind of energy production affects the environment one way or another and to a greater or lesser extent, since all involve transforming natural resources. Even so-called renewable energy sources can cause problems.[1]

Evidently, choices cannot be made purely by going for the option with the least impact, but rather by weighing up the positive and negative impacts in each case. In making this choice, the impact on regional development brought about by an energy project is an important factor, since the use of certain energy sources may affect one particular area of land, but its benefit is felt in other areas which are often distant from the place where the energy is generated.

In this sense, a power plant could be seen as the overlapping of one economic reality with another, normally exacerbating pre-existing differences.

In 2004, the International Hydropower Association published its *Sustainability Guidelines* with the aim of giving recommendations on how to take account of social and environmental issues in conjunction with the economic aspects of hydropower projects, the latter of which have been covered formally in this Manual since its 1997 edition. It is therefore an important tool towards ecological efficiency, which is based on three pillars:

- reducing the consumption of natural resources;
- reducing the impact on nature;
- enhancing the benefits of projects, taking into account multiple uses of the resources.

Focusing on the decision-making process and the criteria for making comparisons between different energy options, the IHA presents key criteria in this assessment:

1 For instance, wind energy, which is a clean form of electricity generation, occupies huge tracts of land, causes noise pollution and can pose a threat to birdlife. Solar energy does not cause pollution at the operating stage, but it uses solar panels which are manufactured using hazardous materials such as arsenic, cadmium and inert silicone. The burning of non-edible feedstocks (biomass), while it does absorb CO_2 at the crop growing stage, pollutes the air with particulate matter. The growing of non-edible feedstocks also occupies large areas of land that could otherwise be used to grow crops for human consumption.

- to promote energy efficiency on the demand side, viewing this as the equivalent of increasing energy production;
- to analyze the options for expending energy generation in view of the following aspects:
  - the availability of the resource, in view of the depletion of some primary sources;
  - energy payback ratio;
  - projective life of facilities;
  - technology efficiency and state-of-the-art;
  - multiple use benefits;
  - job creation and benefits for local communities;
  - impacts of carbon emissions;
  - area affected;
  - waste produced.

With these criteria in mind, hydropower schemes should avoid affecting vulnerable groups in society and focus on:

- upgrading existing projects;
- projects with multiple-use benefits;
- river basins that have already been developed;
- projects that minimize the area flooded per unit of energy produced;
- projects that prevent or minimize population displacement;
- projects that cause the least impacts on threatened species;
- projects that bring benefits to local communities, including in downstream areas.

2.2 The Issue of Dams Around the World

2.2.1 Size and Number of Dams

Table 2.2.1.01 shows the places where dams have been built around the world, independent of their function or size. Of course, simply counting them gives a distorted picture, because although Brazil has just 1% of the total number of dams, it produces almost 12% of the world's hydroelectric power. Even so, it is interesting to see that over 75% of all dams are in just four countries, showing that not all the problems involving reservoirs are connected to energy production.

The World Commission on Dams issued a report in 2000 (WCD 2000) in which they set out some recommendations based on a broad-based analysis of the experience of using dams of all kinds around the world. One of their recommendations is reproduced below:

"Debate and controversy initially focused on specific dams and their local impacts. Gradually these locally driven conflicts evolved into a global debate about the costs and benefits of dams."

The International Commission on Large Dams (ICOLD) defines a large dam as being one that is at least 15 meters high (counting from the lowest point of the foundations). Dams that are between 5 m and 15 m high and have a reservoir with the capacity to store over 3 million m^3 are also classified as large-scale. Based on this definition, there are today over 45,000 large dams around the world.

Half of the world's large dams were built exclusively for irrigation purposes, and it is estimated that dams boost the world's food production by 12%-16%. Also, in at least 75 countries, large dams have been built for flood control, and in many countries dams are the largest individual projects, measured in terms of their investments.

In the past, the provision of drinking water, energy generation, irrigation or flood control were normally seen as enough to justify the massive investments needed to build dams. Other benefits often went hand in hand, such as greater economic prosperity for the region thanks to multiple harvests, electrification of rural areas, and the expansion of basic and social infrastructure, such as roads and schools. These benefits were taken for granted. When they were compared with the construction and operating costs – from both an economic and financial perspective – the benefits seemed to make dam building easily the most competitive option.

However, projects of this kind have come under increasing scrutiny in recent years, and greater demands have started to be imposed on planned projects.

Table 2.2.1.01 – Number of dams around the world, per country (1994)

China	46%
USA	14%
India	9%
Japan	6%
Spain	3%
Other countries	23%
Others	16%
Canada	2%
South Korea	2%
Turkey	1%
Brazil	1%
France	1%
Total	100%

Source: World Resources Institute – Eathtrends Environmental information
http://earthtrends.wri.org/maps_spatial/index.php?theme=2

The distribution of the world's 306 major dams[2] is shown in the map in figure 2.2.1.01, based on data from the World Research Institute (www.eathtrends.org). The same study shows that there are rivers that are fragmented by hundreds of dams measuring 15 m or more and thousands of small dams (<15m high). According to the study, there are 40,000 dams measuring 15 m or more in height and up to 800,000 of a smaller scale.

Dams are one way of analyzing how much rivers have been modified, which happens when they are impounded. Of the 106 river basins from around the world analyzed in the report, 46% have been modified by at least one major dam (there are 14 large dams in the Paraná river basin alone). In 1994, 56 new major dams were being planned or under construction.

The list of 56 projects for new dams is incomplete as few countries will divulge information of this nature. The dams in question are built in just five river basins: 11 in the Yangtze, seven each in the Tigris and Euphrates river basins, six in the Ganges river basin, and four each in the Hwang He and Paraná river basins.

From this information, it would appear that the only river basins with more than ten large dams are in Brazil, north-west America and west Canada.

2 In this case, a "major dam" is defined as being at least 150 m high or having a volume greater than 15 million m^3, or a generating capacity greater than 1000 MW. This is different from the ICOLD definition.

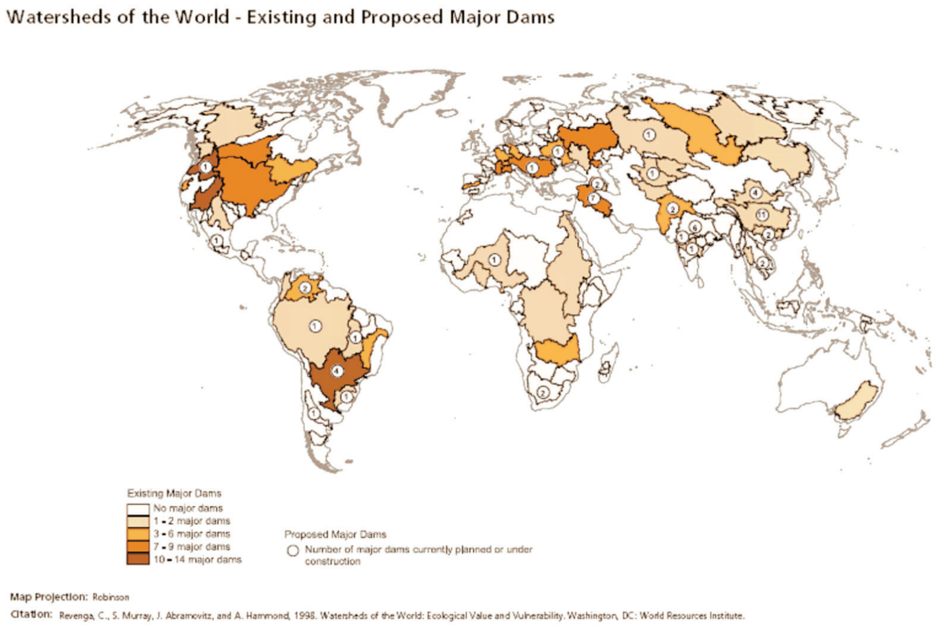

Figure 2.2.1.01 – Distribution of 306 major dams[3] around the world

2.2.2 The Debate over Impacts and Benefits

The International Rivers Association, an NGO devoted to "*defending the rights of communities that depend on rivers*", published Twelve Reasons to Exclude Large Hydro from Renewables Initiatives, which clearly exemplifies the strength of the lobby against large-scale dam projects. According to this organization:

1) Large hydro does not have the poverty reduction benefits of decentralized renewables do not reduce poverty.
2) Including large hydro in renewables initiatives would crowd out funds for new renewables.
3) Promoters of large hydro regularly underestimate costs and exaggerate benefits.
4) Large hydro will increase vulnerability to climate change.
5) There is no technology transfer benefit from large hydro.
6) Large hydro projects have major negative social and ecological impacts.
7) Efforts to mitigate the impacts of large hydro typically fail.
8) Most large hydro developers and funders oppose measures to prevent the construction of destructive projects.
9) Large reservoirs can emit significant amounts of greenhouse gases.
10) Large hydro is slow, lumpy, inflexible and getting more expensive.
11) Many countries are already overdependent on hydropower.
12) Large hydro reservoirs are often rendered non-renewable by sedimentation.

In an analysis of the subject, the World Bank presents counter-examples in a bid to show that this is a false dichotomy. Table 2.2.2.01 shows the data from a study that correlates reservoir area to installed capacity.

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3 In this case, a "major dam" is defined as being at least 150 m high or having a volume greater than 15 million m^3, or a generating capacity greater than 1000 MW. This is different from the ICOLD definition.

Table 2.2.2.01 – Average Size of Reservoir per Capacity (1995)

Size of the Plants (MW)	Number of Plants	Size of the Reservoir(ha/MW)
3,000 to 18,200	19	32
2,000 to 2,999	16	40
1,000 to 1,999	36	36
500 to 999	25	80
250 to 499	37	69
100 to 249	33	96
2 to 99	33	249

Source: Goodland, Robert (1995), How to Distinguish Better Hydros from Worse: the Environmental Sustainability Challenge for the Hydro Industry, The World Bank.

Both positions look at the issue from a broad perspective. Evidently, each case has its differences and size is not the only issue at stake. Indeed, the "big vs. small" discussion is far from drawing to an end, with respected experts taking a stand on both sides of the divide and increasing attention being given to the subject in important documents from the energy industry.

Whatever the merit of the question, there is clearly a strong lobby against large-scale projects. Below we reproduce a short section of the report that illustrates the concern and the conflicts identified by the World Commission on Dams.

"The enormous investments and widespread impacts of large dams have seen conflicts flare up over the siting and impacts of large dams - both those in place and those on the drawing board, making large dams one of the most hotly contested issues in sustainable development today.

Proponents point to the social and economic development demands that dams are intended to meet, such as irrigation, electricity, flood control and water supply. Opponents point to the adverse impacts of dams, such as debt burden, cost overruns, displacement and impoverishment of people, destruction of important ecosystems and fishery resources, and the inequitable sharing of costs and benefits." (WCD, 2000:6)[4]

The report recommends a number of stances organized as shown below:

National governments can:

- Require a review of existing procedures and regulations concerning large dam projects;
- Adopt the practice of time-bound licenses for all dams, whether public or privately owned;
- Establish an independent, multi-stakeholder committee to address the unresolved legacy of past dams.

National and international NGOs can:

- Monitor compliance with agreements and assist any aggrieved party to seek resolution of outstanding disagreements or to seek recourse;
- Actively assist in identifying the relevant stakeholders for water and energy projects using the rights-and-risks approach.

Affected people's organizations can:

- Identify unresolved social and environmental impacts and convince the relevant authorities to take effective steps to address them;
- Develop support networks and partnerships to strengthen technical and legal capacity for needs and options assessment processes.

4 WCD, 2000. Dams and Development: A New Framework for Decision-Making – The Report of the World Commission on Dams – An Overview. Available in http://www.rivernet.org/general/wcd/wcd_overview_english.pdf.

Professional associations and agencies can:

- Develop processes for certifying compliance with WCD guidelines;
- Extend national and international databases, such as the ICOLD World Register of Dams, to include social and environmental parameters.

The private sector can:

- Develop and adopt voluntary codes of conduct, management systems and certification procedures for best ensuring and demonstrating compliance with the Commission's guidelines, including, for example, through the ISO 14001[5] management system standard;
- Abide by the provisions of the antibribery convention of the Organisation for Economic Co-operation and Development;
- Adopt integrity pacts for all contracts and procurement, as developed by Transparency International.

Bilateral aid agencies and multilateral development banks can:

- Ensure that any dam options for which financing is approved emerge from an agreed process of ranking alternatives and respect the Commission's guidelines;
- Accelerate the shift from project- to sector-based finance, especially through increasing financial and technical support for effective, transparent, and participatory needs and options assessment, and the financing of non-structural alternatives;
- Review the portfolio of past projects to identify those that may have underperformed or present unresolved issues.

However, despite this huge effort and however complex the procedures that need to be analyzed, when it is a matter of comparing different energy sources, the methods used do not seem entirely satisfactory when it comes to hydropower generation. This sense of non-adaptation, which is so controversial, arises from the fact that unlike most of the other energy sources, the generation of electricity by hydropower plants could even be seen as a byproduct of a series of other benefits provided by a project that could be classified as being of an entirely different nature in the region in question. While a thermoelectric power station is no more nor less than a facility to generate power, a hydroelectric power plant can have many other vocations, some of which could even be more important. For instance, how can one compare a dam built for flood control and which has also been designed to produce energy with the energy generated by a thermoelectric plant?

2.3 Some International Data on Hydropower

With these issues in mind, some international data are presented below to contribute to a global geopolitical understanding of the problem. The idea is to give an overview of the situation, the part played by hydroelectricity and Brazil's role in all this. In 2004, if we consider all the primary sources of energy generation, the world produced the equivalent of 10.2 billion tons of oil (Energy Information Administration – US Department of Energy – 2005). The current mix of primary sources is shown in table 2.3.01.

5 ISO 14001 is the standard used for the certification of organizations' environmental management. Certification is not granted by the ISO, which is an international standards body, but by an accredited third-party entity. In Brazil, the National Council for Metrology, Standards and Industrial Quality, CONMETRO, established the Brazilian Compliance Evaluation System, and designated Inmetro (the Brazilian weights and measures institute) to be Brazil's official accreditation agency. Any accreditation given within the ambit of CONMETRO's Sistema Brasileiro de Avaliação da Conformidade must be carried out by an entity accredited by Inmetro. However, as the ISO 14001 is voluntary in nature, it can be certified outside the Brazilian system by entities with or without Inmetro accreditation. Whether or not the certification is given within the Brazilian system, when it is carried out by an Inmetro accredited entity, it is based on the same requirements and methods.

Table 2.3.01 – Primary Sources of Energy in the World (2003)

Source	Proportion (%)
Crude Oil and Manufactured Gas	38
Coal	24
Natural Gas	24
Hydropower	7
Nuclear	6
Other	1

Source: Table 11.1 World Primary Energy Production by Source, 1970-2003
Energy Information Agency – US Dept. of Energy

However, it is important to consider just electricity generation if the weight of the different forms of production is to be correctly evaluated.

Table 2.3.02 – Sources of Electricity in the World (2003)

Source	Proportion (%)
Oil	6.9
Coal	39.9
Natural Gas	19.3
Hydropower	16.3
Nuclear	15.7
Other	1.9

Source: Electricity in World in 2003 – International Energy Agency Statistics
http://www.iea.org/Textbase/stats/

It is important to bear in mind that hydroelectric power accounts for around 16% of all energy produced in a world where oil, natural gas and coal still rule supreme. In this sense, the recent hike in oil prices, partly because of the depletion of known reserves, allied with the precarious state of the global environment, can now be seen as increasingly important variables in any strategic investigation of energy in the world.

Of all renewable sources of energy, water power is still the most promising, as it can generate large quantities of electricity with good economies of scale. The ten largest producers of electricity in the world are listed in table 2.3.03.

Table 2.3.03 – Ten Largest Producers of Electricity in the World

Country	TWh	% of total
USA	4,150	23.8
China	2,187	12.5
Japan	1,110	6.4
Russia	931	5.3
India	651	3.7
Germany	607	3.5
France	572	3.3
Canada	568	3.3
UK	400	2.3
Brazil	386	2.2
Other countries	11,561	33.8

The ten largest producers of hydroelectric power are shown in table 2.3.04, with Canada, China, Brazil and the USA occupying the top of the table.

Table 2.3.04 – Largest Producers of Hydroelectric Power (and proportion of hydropower to the energy mix) (2001)

Country	TWh	% of total
Canada	344	12.0
China	334	11.7
Brazil	326	11.4
USA	269	9.4
Russia	180	6.3
Norway	111	3.9
Japan	102	3.6
India	86	3.0
Venezuela	72	2.5
France	67	2.3
Others	1,890	35.1%

Source: WEC Member Committees, 2000/2001; Hydropower & Dams World Atlas 2001, supplement to The International Journal on Hydropower & Dams, Aqua Media International; Energy Statistics Yearbook 1997, United Nations; national and international

Another aspect that is worth observing is the 'productivity' of hydroelectric systems around the world. As Table 2.3.05 shows, it is not every system that can obtain capacity factors greater than 50%. In many countries, hydro plants are only used to meet peak demand or do not have enough reserve capacity to regulate their production. This last aspect is of the utmost importance and sets Brazil's generation system apart from its peers. Indeed, of all the large-scale systems around the world, only the Canadian one has a similar reserve capacity to Brazil's. This is an important factor when analyzing the feasibility of new projects. If viewed from the perspective of the growing need to strike a balance between social and environmental impacts and energy benefits, Brazil's system is particularly efficient.

Table 2.3.05 – Leading Countries and the Capacity Factors of the Hydroelectricity Systems (1999)

Country	Capacity in Operation (MW)	Generation in 1999 (TWh)	Capacity Factor(%)
Canada	66,954	341	58
Brazil	57,517	286	57
Venezuela	13,165	61	53
Norway	27,528	122	51
Sweden	16,192	71	50
USA	79,511	319	46
India	22,083	82	43
Russia	44,000	161	42
Austria	11,647	42	41
Mexico	9,390	32	39
Turkey	10,820	35	37
China	65,000	204	36
Japan	27,229	84	35
France	25,335	77	35
Italy	16,546	47	32
Switzerland	13,230	37	32
Spain	15,580	28	21

Source: WEC Member Committees, 2000/2001; Hydropower & Dams World Atlas 2001, supplement to The International Journal on Hydropower & Dams, Aqua Media International;

When it comes to different countries' potential for expanding their hydropower generation, Brazil is one of the countries with the greatest potential as it has by far the largest amount of water resources, as we can see in table 2.3.06.

Table 2.3.06 – Total Water Resources per Country (2003)

Country	Water resources entirely in national territory (km3/year)	Water resources that originate outside national territory (km3/year)	Total Resources	% of total
Brazil	5,418.0	2,815.0	8,233.0	19
Russia	4,312.7	194.6	4,507.3	10
Canada	2,850.0	52.0	2,902.0	7
Indonesia	2,838.0	0.0	2,838.0	6
Mainland China	2,812.4	17.2	2,829.6	6
USA	2,000.0	71.0	2,071.0	5
Peru	1,616.0	297.0	1,913.0	4
India	1,260.5	636.1	1,896.6	4
Congo	900.0	383.0	1,283.0	3
Venezuela	722.5	510.7	1,233.2	3
Top 10	24,730.1	4,976.6	29,706.7	57
World	43,764.0		43,764.0	100

Source: FOOD AND AGRICULTURE ORGANIZATION OF THE UNITED NATIONS – Review of World Water Resources by Country, Rome, 2003
Internal renewable water resources is that part of the water resources (surface water and groundwater) generated from endogenous precipitation.
External water resources as the part of a country's renewable water resources that enter from upstream countries through rivers (external surface water) or aquifers (external groundwater resources).

Table 2.3.07 sets out an international appraisal of the potential expansion of hydropower generation capacity. However, it should be noted that there is much room for error in the estimates not only for Brazil but for other countries in view of the increasing restrictions on large-scale and even medium-scale projects. Even so, Brazil is still an important player, even in this scenario of growing constraints on the construction of new hydropower projects.

Table 2.3.07 – Estimate of Hydroelectric Potential around the World (2000/2001)

Country	Theoretical Capacity (TWh/year)	Technically Available (TWh/ year)	Economically Feasible (TWh/ year)	% of world total
China	5,920	1,920	1,260	13
Russia	2,800	1,670	852	12
Brazil	3,040	1,488	811	10
Canada	1,289	951	523	7
Congo	1,397	774	419	5
USA	4,485	529	376	4
Tajikistan	527	264	264	2
Ethiopia	650	260	260	2
Peru	1,578	260	260	2
Norway	600	200	180	1
Nepal	727	158	147	1

Source: WEC Member Committees, 2000/2001; Hydropower & Dams World Atlas 2001, supplement to The International Journal on Hydropower & Dams, Aqua Media International.

2.4 Competitive Aspects of Hydropower

Despite its many problems, hydropower has several advantages which are often understated. These include:

- replacement or postponement of the need for generating electricity from fossil fuels, with corresponding benefits for air quality;
- provision of a very reliable service using technology which has been tried and tested for over a century, with low operating costs, high energy efficiency and a long working life;

- totally renewable and the only source in this category that can produce energy on a large scale;
- low greenhouse gas emissions[6] if compared to fossil fuels;
- any negative impacts are only felt in its area of influence;
- building a hydropower plant can, given its size, present the opportunity to meet regional development needs, such as flood control, river transportation, irrigation, etc.;
- although investments are intensive for a limited period, they can be made using domestic technology, skills and materials, contributing to a country's strategic independence;
- part of the investments are catalysts for other sectors of the economy, creating direct and indirect jobs;
- as an operationally flexible energy source, it can support large energy distribution networks;
- particularly good for providing ancillary services for the electricity grid, such as spinning and non-spinning reserves, frequency regulation and response, voltage control and stability.

The issue of future energy supply is more than ever of international concern. Increasingly, countries are becoming aware of the limitations inherent to exploiting the planet's natural resources, especially given the inevitable depletion of reserves of oil, which is still the main primary source of energy, and the environmental impacts associated with this and other forms of energy production.

Some international initiatives are starting to be taken in response to this scenario. The Clean Development Mechanism, established as part of the Kyoto Protocol, provides an incentive for companies from developed nations to invest in optional emission reduction projects in developing nations. The Kyoto Protocol requires all CDM projects to be validated and inspected/certified by a "designated operational entity"; i.e. subject to an independent evaluation.

This all indicates that the world is turning its attention to energy projects, especially in developing countries. It also means that energy options will be scrutinized according to far more complex criteria, without the "linearity" traditionally seen in financial and economic assessments.

In this context, inventory studies of the hydroelectric potential of Brazil's river basins gain greater importance, as they address future scenarios and can pinpoint actions that can be taken in advance to minimize impacts or even make new hydropower projects feasible.

Around the world, great concern is being expressed about the impacts of dams and reservoirs, and there is a clear dividing line between projects according to their scale.

While hydropower is clearly not a panacea for the world's energy problems, it is certainly part of the solution to the dual issue of energy production and socioenvironmental impacts. As Brazil still has such great and as yet untapped water resources, it can plan out its future energy needs and the expansion of its electricity system on the back of hydropower. Hydropower inventory studies are a key part of this process.

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6 Some authors propose that large reservoirs that flood forests in tropical ecosystems are responsible for high CO_2 emissions. Recent studies of the Tucuri reservoir show that in the worst case scenario, the lake emits 213 g CO_2 per kWh generated. This is five times lower than CO_2 emissions by coal-fired power stations. See Hydropower and the World's Energy Future – International Hydropower Association – International Commission on Large Dams - International Energy Agency – Nov 2005.

3 Hydropower in Brazil

3.1 Brazil's Hydroelectic Potential and the Feasibilty of its Being Harnessed

The drafting and future use of this Manual for Hydropower Inventory Studies is part of a broad review of criteria, methods and even legislation pertaining to hydropower generation in Brazil. Indeed, the scenario of potential future projects is very likely to change substantially after this procedure. Therefore, what is set out here is merely a portrayal of what is available from previous studies and will probably be subject to review.

The information available in the SIPOT[7] database maintained by Eletrobrás gives a full assessment of Brazilian territory. The data set out below can be found on the Eletrobrás website and may not represent the latest assessment of Brazil's hydroelectric potential[8].

The sites earmarked for hydropower projects and registered in the SIPOT system are classified into three groups according to the stage their studies have reached, as shown below.

- Estimated Potential (sections of river, SR, and individual dam sites, In).

The hydroelectric potential of sections of river is estimated from office-based studies of sections of river (head available at multiple dam sites). The potential for individual dam sites is also estimated from office-based studies undertaken when deciding on specific sites to be harnessed. The existence of this potential in Brazil's different regions (based on data from the SIPOT system) is shown in table 3.1.01 below.

Table 3.1.01 – Estimated Potential per Region (MW)

Region	SR	In	SR + In
North	16,034.76	37,288.03	53,322.79
Northeast	267.6	874.78	1,142.38
Southeast	2,373.30	2,858.10	5,231.40
Central-West	7,545.61	8,607.53	16,153.14
South	2,020.72	2,602.69	4,623.41
Total	28,241.99	52,231.13	80,473.12

- Potential under Study (Inventory (I), Feasibility (F) and Basic Design (BD)).

Potential classified under "Inventory" is the outcome of Inventory Studies of different river basins. The sites classified under "Feasibility" are the ones whose overall conception is being examined with a view to verifying their technical and economic feasibility. The sites at the "Basic Design" stage are already being detailed for public tender. The existence of hydroelectric potential at these different stages in the different regions of Brazil is shown in table 3.1.02 below.

Table 3.1.02 – Potential under Study per Region (MW)

Region	I	F	BD	Deactivated (D)	I+F+BD-D
North	17,275.59	28,744.60	1,327.23	2.34	47,345.08
Northeast	6,593.64	7,050.50	406.16	0.8	14,049.50
Southeast	10,236.03	3,974.45	1,753.02	2.67	15,960.83
Central-West	9,535.40	1,501.75	2,286.72	2.33	13,321.54
South	9,758.32	4,676.58	2,826.36	0	17,261.26
Total	53,398.98	45,947.88	8,599.49	8.14	107,938.21

7 SIPOT – Information System on Brazil's Hydroelectric Potential, Eletrobrás. sipot@eletrobras.com.

8 There is actually some difference between the SIPOT data and the data on installed capacity provided by ANEEL, as Eletrobrás is no longer responsible for storing and updating these data and keeping them consistent with other sources. Despite this, it is still the best source for revealing the percentages of projects at different stages of development.

- Plants Under Construction (C) and In Operation (O)

Hydroelectric potential classified as being "under construction" relates to plants that have already started being constructed, while "in operation" relates to those with at least one unit already in service. The existence of projects in the different regions of Brazil is shown in table 3.1.03.

Table 3.1.03 – Energy Potential under Construction and In Operation per Region (MW)

Region	C	O	Total
North	3,109.50	7,229.85	10,339.35
Northeast	25	10,783.25	10,808.25
Southeast	1,313.38	22,109.10	23,422.48
Central-West	642.8	9,006.89	9,649.69
South	2,725.77	18,631.10	21,356.87
Total	7,816.45	67,760.19	75,576.64

The sum of these three categories is shown in table 3.1.04.

Table 3.1.04 – Total Potential per Region (MW)

Region	Total
North	111,011.90
Northeast	26,001.73
Southeast	44,620.05
Central-West	39,129.03
South	43,241.54
Total	264,004.25

It is possible to use data from Eletrobrás to further break down some of these categories according to the size of the projects. The projects whose basic design is being prepared are detailed in table 3.1.05.[9]

Table 3.1.05 – Capacity of projects at Basic Design (BD) stage

Capacity Range	Total Capacity (MW)	%	No. of Plants	%
P > 1000	2,820	32.0	2	0.8
500< P < 1000	700	7.9	1	0.4
200< P < 500	1,072	12.2	4	1.6
100< P <200	549	6.2	4	1.6
30< P <100	1,051	11.9	27	10.6
0< P <30	2,625	29.8	217	85.1
	8,817		255	

A breakdown of the power available from plants at the Feasibility stage[10] is shown in table 3.1.06.

Table 3.1.06 – Capacity of Projects at Feasibility (F) stage

Capacity Range	Total Capacity (MW)	%	No. of Plants	%
P > 1000	30,903	80.3	11	15.9
500< P < 1000	1,352	3.5	2	2.9
200< P < 500	2,956	7.7	9	13.0
100< P <200	2,011	5.2	14	20.3
30< P <100	1,150	3.0	21	30.4
0< P <30	91	0.2	12	17.4
	38,462		69	

9 There is a slight difference between the total for this table and the total for table 3.2, as the data from different versions of SIPOT are not consistent.

10 There is a slight difference between the total for this table and the total for table 3.2, as the data from different versions of SIPOT are not consistent.

A breakdown of the power available from projects that are at the Inventory Studies stage is shown in table 3.1.07.

Table 3.1.07 – Capacity of Projects at Inventory Studies (I) stage

Capacity Range	Total Capacity (MW)	%	No. of Plants	%
P > 1000	20,270	33	8	1
500< P < 1000	9,268	15	13	1
200< P < 500	6,521	11	22	2
100< P <200	7,566	12	54	4
30< P <100	8,850	14	161	12
0< P <30	9,537	15	1,136	81
	62,012		1,394	

It is also interesting to look at how the hydroelectric potential being investigated at the Inventory Studies, Basic Design and Feasibility stages is distributed across the country's river basins. This is in keeping with CNRH Resolution 32 of October 15th 2009, by which a new National Hydrography Division was established.

Table 3.1.08 – Breakdown of projects at Inventory Studies, Basic Design and Feasibility stages per river basin

	Amazon	Tocantins	East Atlantic	São Francisco	Southeast Atlantic	Paraná	Uruguay	South Atlantic
S	30%	10%	2%	17%	14%	16%	8%	4%
F	49%	16%	1%	13%	3%	7%	7%	4%
BP	21%	3%	0%	1%	18%	35%	13%	8%

These percentages reveal some important facts:

- Around 30% of the potential evaluated from the SIPOT (264 GW) is in operation or under construction (~ 75 GW).
- Around 30% is just estimated (~ 80GW).
- Around 40% is under study (107 GW).
- Of the proportion under study, 8% is at the Basic Design stage, 43% is having Feasibility Studies done and the remaining 49% is the object of Inventory Studies.
- The regional breakdown of the projects at the Basic Design stage is as follows: 27% in the central-west region; 33% in the south, 20% in the southeast, 15% in the north and 5% in the northeast.
- Of all the projects at the Basic Design stage, just two of the total of 255 projects account for 30% of the power (~ 9 GW). Around 85% of all the projects are for plants with an installed capacity of less than 30 MW.
- Of the projects at the Feasibility stage (~39 GW), 63% are in the north, 10% are in the south, 9% are in the southeast, 3% are in the central-west and 15% are in the northeast. Around 80% are for large-scale projects whose installed capacity would be over 1 GW. Just 0.2% of the total energy would come from plants with an installed capacity of less than 30 MW.
- The majority of the projects at the Feasibility stage are in the Amazon and Tocantins river basins.
- 33% of the projects at the Inventory Studies stage are in the north, 19% are in the southeast, 18% in the south, 12% in the northeast and 18% are in the central-west. 96% of these projects are for plants with less than 200 MW, while 81% would be under 30 MW.

It seems clear that geographically speaking, the most important untapped resources are in the north and central-west regions of the country. The longer-term trend in terms of scale seems to be towards medium-sized plants.

3.2 Background on Hydropower Generation

The experiences that marked the beginning of Brazil's electricity industry were in the areas of lighting and public transportation. It was in 1879 that indoor electric lighting was inaugurated at Dom Pedro II railroad station (Central do Brasil) in Rio de Janeiro. Two years later the first street lighting in the city was installed along a section of gardens in Campo da Aclamação, now Praça da República, using dynamos run by locomotives. In the same year, at the opening of the Industrial Exhibition, electricity was used to light the Transportation Ministry building at Largo do Paço (now Praça XV), also in Rio de Janeiro. In 1883, the first electricity generation plant came into operation, with 52 kW installed capacity, in Campos (Rio de Janeiro state). It was a thermoelectric unit fired by steam produced from timber and provided energy for 39 light bulbs. This was the first ever public lighting service in South America. Meanwhile, the first use of electricity as traction for public transportation came in 1883 in Niterói, with Brazil's first ever electric streetcars[11].

However, it was probably the opening of Marmelos power plant[12] in 1889 in Juiz de Fora, Minas Gerais state, which really marked the beginning of Brazil's electricity industry. This was followed by several other important projects, including Monjolinho and Piracicaba in 1893, Corumbataí in 1900, Fontes over Ribeirão das Lajes in 1908, and the famous Delmiro Gouveia plant on São Francisco river in 1913.

However, Brazil's slow-growing system of almost continental proportions only started to gain more structure after the creation of CHESF in 1945 and CEMIG in 1946. With these companies came the beginning of a period of large-scale, long-term, consistent state intervention in the electricity sector. The following decades spawned a wealth of major projects, including Três Marias in 1962 and Furnas in 1963. It was also in 1963 that the renowned project for Canambra was begun. The scale and depth of the studies undertaken in the 1960s were a turning point in Brazil's electricity industry.

- They were the first comprehensive hydropower inventories, leading in some cases to feasibility studies;
- they were decisive in the option for large dams; and
- they contributed to the training of hundreds of skilled workers, who went on to join the ranks of the planning entities of the country's leading electricity companies and private consultancies.

All this was decisive in the development of large dams, which started to be included in Brazil's power generation expansion plans, including Plan 90, Plan 95, and Plans 2000, 2010 and 2015, published in 1974, 1979, 1982, 1988 and 1994, respectively.

Centrais Elétricas Brasileiras S.A. (Eletrobrás) was officially incorporated on June 11th 1962 at a ceremony held by Conselho Nacional de Águas e Energia Elétrica (Cnaee) at Palácio Laranjeiras in Rio de Janeiro, with the presence of the Brazilian President João Goulart (1961-1964). The investment portfolio and the administration of the Federal Electrification Fund was taken from the Brazilian Development Bank (Banco Nacional de Desenvolvimento Econômico, BNDE) and put under Eletrobrás's control[13].

As part of the reorganization of the sector, hundreds of small companies were grouped together or taken over by state-owned utilities. For instance in 1966 the São Paulo government merged 11 state-owned companies into one: Centrais Elétricas de São Paulo (CESP).

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11 O SETOR ELÉTRICO Faria, Gomes, Abarca and Fernandes. http://www.bndes.gov.br/conhecimento/livro_setorial/setorial14.pdf

12 Built in the Paraibuna sub-basin, Juiz de Fora, Minas Gerais state, and owned by Companhia Mineira de Eletricidade, this was the first hydroelectric power plant to supply energy for street lighting. Marmelos, with 250 kW installed capacity, was developed by Max Nothman & Co. and used equipment supplied by Westinghouse.

13 Memória da Eletricidade (www.eletrobras.gov.br)

More important yet, the interdependence of the systems means that entities representing the industry had to be created, such as the Coordinating Committee for Interconnected Operations (Comitê Coordenador para Operação Interligada, CCOI), formed in 1969 by generation and distribution companies from the southeast under the technical guidance of Eletrobrás and the supervision of the National Department of Water and Electricity (Departamento Nacional de Águas e Energia Elétrica, DNAEE). Two years later, another committee was formed by companies from the south of Brazil.

The system grew with the development of new projects in the southeast, starting with the Tietê and Paraíba do Sul river basins. Until 1950, most power plants were near the coast in São Paulo, Rio de Janeiro and Minas Gerais states. Little by little, this picture changed, with new facilities being built in São Paulo, Minas Gerais, Mato Grosso do Sul and Goiás. This growing distance from load centers was resolved by introducing cascades of hydropower projects to maximize the water resources of a given river, and thanks to the hydrological diversity that emerged as different river basins were harnessed.

The biggest step in the development of Brazil's electricity system was the building of large-scale plants in the 1960s and 70s, especially in the Paraná river basin. With their large reservoirs, these projects brought long-lasting advantages not previously seen in hydro-dependent electricity systems. Using a carefully dimensioned transmission system, it was possible to provide a reasonable level of assurance, since by interconnecting the reserves, it became feasible to share them throughout the system, effectively "regulating" all the power plants.

3.3 Hydropower System Today, Prospects for Growth, and the Role of the Transmission System in Interconnecting River Basins

In 2006 Brazil had 1,588 projects in operation totaling 96,340,783 kW installed capacity.

Table 3.3.01 – Brazil's Installed Capacity

Type of Project	Quantity	Contracted Capacity (kW)	Actual Capacity (kW)	%
Hydro (< 1 MW)	196	104,655	104,208	0.11
Wind Power	14	189,250	186,850	0.19
Small Hydro	269	1,457,551	1,415,863	1.47
Solar	1	20	20	0
Hydroelectric	156	73,348,695	71,820,411	74.55
Thermoelectric	950	23,950,514	20,806,431	21.60
Thermonuclear	2	2,007,000	2,007,000	2.08
Total	1,588	101,057,685	96,340,783	100

Source: ANEEL

In other words, around three quarters of Brazil's installed capacity comes from hydroelectric power. However, what makes this system even more unique is the fact that all the plants that are linked up to the grid share the same assurance thanks to the fact that Brazil has a higher reserve capacity than any other hydro-based system in the world.

This is one of the factors that justified the expansion of the transmission system, not just to meet demand but also to provide a wider range of dispatch options by improving the system's interconnectivity. This architecture required the adoption of an operationally centralized system, where several agents sell not their own production but a portion of energy from the whole system as if they were shares in a company. This flexibility combined with centralized operations means that large chunks of energy can be "transferred" from one basin to another, providing a gain of almost 25% over local energy sources.

Although this reserve capacity is declining as a result of the fact that no new large reservoirs have been added, it will still be the largest capacity in the world for a long time and is certainly a unique feature of Brazil's system. The importance of this to Hydropower Inventory Studies is that it affects how the feasibility of new projects is analyzed.

Even if social and environmental demands, which are likely to grow in importance as time goes by, are such that they limit the reserve capacity of new hydro projects, these may still not be made unfeasible, as even with limited reserve capacity, they will be able to draw on the "virtual" reserve of the interconnected system.

3.4 Effect Of Social and Environmental Issues on Hydro Generation In Brazil

Many of the hydropower plants in operation today were planned and built under a quite different political, institutional and developmental context than what prevails today. The decision-making process that is inherent to the introduction of new hydropower projects has developed greatly, not only by allowing greater participation and transparency, but also by factoring in issues concerning the distribution of costs and benefits. There is also increasing awareness on the part of Brazilian society about social and environmental issues, which is reflected in a rigorous, comprehensive, new legal framework aimed at assuring social and environmental sustainability and the conservation of water resources. It covers a broad range of aspects, including inspection activities and the defense of the environment and minorities, for which it provides a special public prosecution service, as the public prosecution service (Ministério Público Federal) plays a major part in the process of developing new hydropower projects.

Alongside the establishment of this legal framework, a critical analysis of social and environmental experiences has led Brazil's electricity sector to expend its efforts not only to adapt to the new legal context, but above all to adopt a new attitude from the study and design stage through to the building and operation of projects. Since the early 1990s clear guidelines organized into different levels of regulation have been developed to incorporate social and environmental variables at all stages of the decision-making process.

With the introduction of the principles of socioenvironmental feasibility, regional involvement and a more broad-based decision-making process in the second Master Plan for the Environment (II Plano Diretor de Meio Ambiente, II PDMA), published in 1991, new guidelines were drafted for the introduction of hydropower plants and manuals were drafted that set out the methods and procedures to be used in conjunction with engineering and environmental considerations.

Until the mid 1980s, generation projects had been prioritized in sector expansion plans almost exclusively as a function of the unit cost of the energy they would produce (in US$/MWh), without any systematic method for taking into account the measurable environmental costs, not to mention the non-quantifiable environmental variables. Based on the principles set out in the II PDMA, new efforts were made to incorporate socioenvironmental costs into the overall costs of hydropower projects, including estimates of such costs in Hydropower Inventory Studies. At the subsequent stages of developing these projects – the Feasibility Studies, Basic Design and Executive Design – these costs are included in greater detail so the costs of environmental compensation and mitigation can be accounted for with greater accuracy.

At the end of the 1990s, ANEEL and Eletrobrás republished their set of manuals providing guidelines for the studies required for Inventory Studies, Feasibility Studies, Basic Designs, transmission systems and projects for small hydros. All these documents provide guidelines for contracting and undertaking the studies from the perspective of integrated planning and including in the decision-making process all institutional, legal, energy, economic and socioenvironmental aspects of relevance.[14]

Alongside the development of the legislation and regulatory framework for the electricity sector, the system for planning and implementing projects is determined by the actions of private agents, inspection agents and government entities. Figure 3.4.01 shows the main stages of their interaction in this process.

14 The manuals produced between 1995 and 1998 are available at www.aneel.gov.br and www.eletrobras.gov.br.

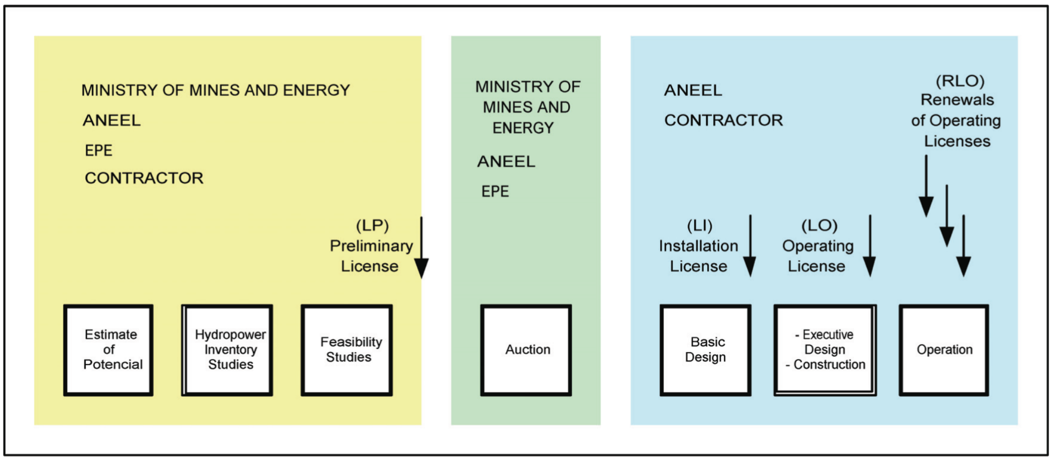


Figure 3.4.01 – Stages in the development of new hydropower projects

In the early years of the 21st century, hydroelectricity entities around the world and a new international commission published new guidelines for the development of hydropower projects that adopted the principles of sustainable development.[15] Methodologies and tools were also developed for socioenvironmental studies, such as georeferenced information systems and environmental zoning, which are fundamental for the integrated planning of river basins. It would not be wrong to say that Brazil's electricity sector entered the 21st century with the instruments required for it to address socioenvironmental issues adequately during the planning, building and operating of its projects.

However, even though hydropower projects use a clean, renewable source of energy, improving the quality of life of the population by the production of electricity, it would be wrong to ignore the major impacts caused by some hydroelectric power plants when it comes to the sustainability of ecosystems and local communities.

One way of getting a general idea of how these impacts are changing is to calculate the area flooded and the number of people affected as a ratio to the power generated, providing two broad indicators of the performance a hydropower plant.

In December 1989 there were 60 hydro plants operating in Brazil with an installed capacity of over 30 MW, summing 52,225 MW. Their reservoirs occupied 23,847 km^2, or 0.28% of national territory, and their ratio was 0.46 km^2/MW installed (assuming their final capacity). The 60 reservoirs for energy generation represented 11% of the total of 516 dams in the country (IIPDMA, vol. I, p.75).

In 2005, there were 116 hydroelectric power plants with over 30 MW in operation in Brazil, providing a total installed capacity of 71,000 MW and occupying 36,847.64 km^2. The areas of the reservoirs in use represented around 0.4% of national territory. Jointly, their ratio of area flooded to installed capacity was 0.52 km^2/MW.

According to the forecasts in the Ten-Year Plan for 2006-2015, which analyzed 46 out of 83 new reservoirs, land covering 5,862.21 km^2 would be flooded for these plants, raising the total amount of land flooded by 79% in comparison the 1989 figure.

The ratio between the flooded area and the final installed capacity remained practically unchanged between 1989 and 2005, staying at around 0.46 km^2/MW.

15 The aims of the report by the World Commission on Dams were: to examine what contribution the construction of large dams makes to development and assess alternatives for water use and energy generation; to develop internationally acceptable criteria, guidelines and standards for the planning, design, assessment, construction, operation, supervision and deactivation of dams. WORLD COMMISSION ON DAMS, 2000 – *Dams and Development: A New Framework for Decision-Making*. Earthscan Publication Ltd, London.

It is harder to gather data on the number of people affected, so in order to gain an idea of how this has changed over the years the information on the number of people affected by power plants with over 100 MW between 1992 and 2002 was taken. In this ten-year period, 17 hydropower plants came into service mostly in the southeast, northeast, central-west and south. Their combined installed capacity was 15,647 MW and they flooded 6,990 km² in total, affecting a total of 20,912 households (83,650 people). It is expected that the power plants included in the ten-year plan for 2005-2016 will affect some 73,300 people.

If we bear in mind the extra 30,963 MW generated in the period, the favorable hydrographic network and the low population density in Brazil, it is reasonable to state that the expansion of the country's hydroelectric power system has taken place in conjunction with improved indicators, especially if one bears in mind that the present rates already encompass the large reservoirs that were built many years ago.

4 Hydropower Projects in The Context of the Electricity Sector's Institutional Framework

Brazil's electricity sector was remodeled on March 15th 2004 by Act 10.848, which establishes how electricity is to be sold between the different industry agents and by these agents to end users within the context of the National Interconnected System (Sistema Interligado Nacional, SIN). By the new law, energy could now be sold on the regulated market (Ambiente de Contratação Regulada, ACR) or the free market (Ambiente de Contratação Livre, ACL).

The regulated market (ACR) is the market segment where electricity is bought and sold by brokers and distribution agents, preceded by tenders, while the free market (ACL) is the market where electricity can be bought and sold under the terms of freely-negotiated contracts.

The purchase and sale of energy in the regulated market takes place using bilateral contracts called Contracts for the Sale of Energy in the Regulated Market (Contratos de Comercialização de Energia no Ambiente Regulado, CCEAR), which must be signed by the generation companies and each of the distribution companies.

The tenders are different depending on whether the energy comes from an existing or new generation facility (something akin to the concepts of "old" and "new" energy). The law defines new generation plants as those that have no concession, permit or authorization, or are part of an existing venture that is being expanded and will result in an increase in installed capacity, by the beginning of the bidding process. There was one circumstance in which existing generation projects or expansion projects could take part in "new" energy bids by 2007. They had to fulfill all the following requirements: a) have obtained a concession or authorization by the date on which the act was passed; b) not have entered commercial service before January 1st 2000; and c) not have their energy contracted by the data the act was passed.

Customers of Independent Energy Producers (Produtor Independente de Energia Elétrica, PIE) may be electricity utilities operating in the regulated market. They may also be free-choice consumers, consumers of electricity and steam from independent cogeneration plants, groups of electricity consumers, independent of the voltage or load, according to the conditions previously agreed with the local distribution utility. Finally, they are also any consumer which demonstrates to the concession-granting authority that its local utility has not provided energy by 180 days from submission of the request. In the first and last two cases, the price of the electricity sold is subject to the general criteria set by the concession-granting authority.

One of the ways of selling energy to small hydros is to take part in PROINFA, the Alternative Energy Program, which was created on April 26th 2002 by Act 10.438. Originally, the starting date for the supply of energy under PROINFA contracts was December 30th 2006, but Act 11.075 of 2004 extended this date to December 30th 2008.

Act 9.427 (1996) provides for another way of buying and selling energy. It states that all hydropower plants with 30,000 kW or less, characterized as small hydros, can sell their electricity to a consumer or set of consumers that form a market, whether formally constituted as such or not, whose load is 500 kW or greater, independent of the grace periods defined in article 15 of Act 9.074 of 1995 (which defines a free consumer as one that consumes 10,000 kW or more and receives its energy supply at 69 kV or higher), with the possibility of the supply being supplemented by generation projects using the sources quoted in the act, with a view to assuring the supply of energy but limiting this proportion to 49% of the mean energy they produce. When the consumer or group of consumers are in isolated systems, the limit for the load drops to 50 kW.

Another way of selling energy is by distributed generation, which in the case of a hydropower project, and according to Decree 5.163 of 2004, is the output of electricity from concession-holders, permit-holders or authorization-holders connected directly to the buyer's electricity distribution system, whose plant has an installed capacity of 30 MW or less.

Before electricity can be bought from distributed generation projects, it must be made public by the distribution agent in order to assure publicity, transparency and equity of access to all stakeholders.

Additionally, there are a number of items that apply to the issue of benefits and charges.

Decree 2003 of 1996 gives independent generators and autogenerators free access to the transmission and distribution systems run by electricity utilities, for which they reimburse the transmission costs involved, which are calculated using the formula set by the regulatory agency of the concession-granting authority.

Act 9.427 of 1996 provides for and ANEEL stipulates (see regulation 77, 2004) a 50% reduction to be applied to tariffs for the use of electricity transmission and distribution systems on energy produced and consumed from hydroelectric power plants of an installed capacity of 30,000 kW or less.

Acts 10.438 (2002) and 10.848 (2004) gave power stations built in isolated electricity systems that replace energy generated by thermal plants fired by oil products the right to benefit from the Conta de Consumo de Combustível (CCC), an account to cover the fuel expenses of thermal plants, until 2002. This applies to independent generation or autogeneration hydropower plants between 1,000 kW and 30,000 kW, provided they are classified as small hydros, and the associated transmission and/or distribution system.

The following also have the right to benefit from the CCC in isolated systems:

1) autogenerators – for the portion of energy sold to distribution utilities;
2) independent generators – in the portion of energy sold by energy utilities to electricity consumers, independent of voltage or load, by the terms previously agreed with the local distribution utility or with any consumer that proves to ANEEL that its local utility cannot assure it a supply within 180 days of the date the request was made. Additionally, when the plants are classified as small hydros, it relates to the portion of energy sold to a consumer of group of consumers that form a market, whether or not it is formally constituted as such, whose load is 50 kW or over, provided that serving their requirements implies in reducing CCC expenses.

Act 10.848 of 2004 also states that the subrogation of hydropower projects with over 30 MW installed capacity and that have already received a concession, to be built entirely in an isolated system to replace thermal generation fired by oil products, is limited to 75% of the value of the project and until the quantity of the subrogated project reaches a total of 120 MW mean power, and they will be able to sell the energy generated to electricity utilities.

Additionally, independent generators and autogenerators are subject to the following financial charges: financial compensation for the states and municipalities and any federal entities that directly administrate water resources for the water used for electricity generation; electricity service inspection charge; and monthly charges for the Fuel Consumption Account (Conta de Consumo de Combustíveis, CCC).

5 Institutional Organization

The National Council for Energy Policy (Conselho Nacional de Política Energética, CNPE) was established on August 6th 1997 by Act 9.478. Its task is to propose national policies and measures related to energy to the President of the Republic. Its members are: the Minister of Mines and Energy (chair), Minister of Science and Technology, Minister of Planning, Budget and Management, Minister of Finance, Minister of the Environment, Minister of Development, Industry and Foreign Trade, President's Chief of Staff, one representative from each state and the federal district, one Brazilian citizen who is a specialist in energy, and one representative from a Brazilian university who is a specialist in energy.

The Ministry of Mines and Energy (Ministério de Minas e Energia, MME) was established on July 22nd 1960 by Act 3.782, extinguished in 1990 by Act 8.028 and re-established in 1992 by Act 8.422. It covers the following subjects: a) geology, mineral resources and energy resources; b) generation of hydroelectric energy; c) mining and metallurgy; and d) oil, fuels and electricity, including nuclear energy. The MME is also responsible for: a) rural energy projects, agricultural energy products, including countryside electrification, when this is covered by funds from the National Electricity Grid; and b) oversee the balance of electricity supply and demand in Brazil from a structural and industry perspective.

The Electricity Industry Monitoring Committee (Comitê de Monitoramento do Setor Elétrico, CMSE) was established in 2004 by Act 10.848 and constituted by Decree 5.175 of 2004 with the purpose of overseeing and assessing the continuity and security of electricity supplies across Brazil. It is chaired by the Minister for Mines and Energy and has four representatives from the Ministry of Mines and Energy and representatives from ANEEL, ANP, CCEE, EPE and ONS.

The National Electricity Agency (Agência Nacional de Energia Elétrica, ANEEL) is an autonomous state entity linked to the Ministry of Mines and Energy that was created on December 26th 1996 by Act 9.427. It is the electricity sector's regulatory agency and has the following powers: a) to regulate electricity services and provide ongoing inspections of the rendering of said services; b) to arbitrate conflicts of interest between agents from the electricity sector and between these and consumers; c) to implement federal government policies and guidelines for the exploitation of electricity and harnessing of water resources for energy generation; and d) to hold public tenders to contract utilities for the generation, transmission and distribution of electricity and to grant contracts for hydroelectric power plants.

The Energy Research Company (Empresa de Pesquisa Energética, EPE) was created on March 15th 2004 by Act 10.847 and operates under the auspices of the Ministry of Mines and Energy. It undertakes studies and research required for the planning of the energy sector in the areas of electricity, fossil fuels, renewable energy sources and energy efficiency.

The National Operator of the Electricity System (Operador Nacional do Sistema Elétrico, ONS) was created on August 26th 1998 by Act 9.648. It is a private, not-for-profit entity which is responsible for coordinating and controlling the operation of electricity generation and transmission facilities that make up the National Interconnected System (Sistema Interligado Nacional, SIN). It is inspected and regulated by ANEEL.

The North Region Technical Group (Grupo Técnico Operacional da Região Norte, GTON) was created on November 29th 1990 by Ministry of Infrastructure directive 895. It is responsible for planning isolated systems in the north region and overseeing their operation. Among other tasks, it is responsible for preparing the Operating Plan and Monthly Program for the Operation of Isolated Systems.

Câmara de Comercialização de Energia Elétrica (CCEE) was created on March 15th 2004 by Act 10.848 to replace the wholesale energy market (Mercado Atacadista de Energia Elétrica, MAE). It is a private, not-for-profit entity that is regulated and inspected by ANEEL. Its task is to enable the sale of electricity in the National Interconnected System (SIN). The tasks undertaken by CCEE include holding auctions for the purchase and sale of electricity, when this is delegated by ANEEL; keeping a record of all energy purchase and sale contracts signed within the regulated market; keeping a record of the amounts of energy sold under contracts signed in the free market; measuring and recording data on electricity purchase and sale operations; and calculating the Preço de Liquidação de Diferenças (PLD) for the different sub-markets.

Eletrobrás (Centrais Elétricas Brasileiras) was created in 1962 to carry out studies and projects for the construction and operation of power plants, power lines and substations, and to provide support for the government's strategic programs. Its main functions are to provide supplementary funding for the expansion of the electricity industry, to function as a holding of state-owned companies, to administrate industry charges and funds, to sell energy from Itaipu and from alternative sources of energy covered by Proinfa, and to coordinate GTON.

Generation Agents:

- Electricity Autogenerator – individual, company or joint venture of companies that are granted a concession or authorization to generate electricity for their own use.
- Public Service Utility (concession- or permit-holder) – company or joint venture of companies that holds a contract to render public electricity services on the federal ambit, granted by the concession-granting authority by means of public tender, regulated by Act 8.987 of February 13th 1995.
- Independent Electricity Generator (Produtor Independente de Energia Elétrica, PIE) – Company or joint venture of companies which are granted a concession or authorization from the competent authority to generate electricity, all or part of which they sell on their own account and at their own risk.

Transmission Agents:

- Transmission Companies – companies responsible for providing the equipment and facilities needed to convey electricity from power plants to centers of consumption.

Distribution Agents:

- Distribution Utilities – The electricity distribution market comprises 64 state-owned and private utilities that operate around the country. They serve around 47 million consumers, of which 85% are residential consumers from over 99% of Brazil's municipalities.

Energy Brokers:

- Broker – This is a company established especially to buy and sell electricity to utilities or consumers that have free choice of energy supplier. The activities of brokers are regulated by Decree 5.163 of July 30th 2004.

The National Environment System (Sistema Nacional do Meio Ambiente, SISNAMA) was created on August 31st 1981 by Act 6.938. It is made up of federal and state entities and by foundations established by the government that are responsible for protecting and improving the quality of the environment. It has the following structure:

- supreme body: government board;
- advisory body: National Council for the Environment (Conselho Nacional do Meio Ambiente, CONAMA);
- central body: Ministry for the Environment;
- executive body: Brazilian Institute for the Environment and Natural Renewable Resources (Instituto Brasileiro do Meio Ambiente e dos Recursos Naturais Renováveis, IBAMA);
- section bodies: state entities responsible for conducting programs and projects and for controlling and inspecting activities that may harm the environment; and
- local bodies: municipal entities responsible for controlling and inspecting these activities in their respective jurisdictions.

The National Council for Water Resources (Conselho Nacional de Recursos Hídricos, CNRH) is an entity that draws up rules for mediation between different water users, and is therefore one of the most important entities for implementing water resource management in Brazil. As it coordinates the integration of Brazil's public policies, it is known by society for conducting transparent dialog in decision-making processes in the field of water resource legislation. Its powers include:

- analyzing proposals to change legislation that will impact on water resources;
- setting complementary guidelines for the implementation of the National Policy for Water Resources;
- promoting the coordination of water resource planning with national, regional and state-wide plans and users;
- arbitrating conflicts over water resources;
- deliberating on projects for hydropower plants whose impacts will reach out beyond the states in which they will be built;
- approving proposals for new river basin committees;
- setting general criteria for authorizing the right to use water resources and for charging for their use; and
- approving the National Plan for Water Resources and overseeing its execution.

The Ministry of the Environment (MMA) is a public federal administration entity which has the following powers: a) national policy for the environment and water resources; b) policy for the preservation, conservation and sustainable use of ecosystems, biodiversity and woodland; c) proposals for social and economic strategies, mechanisms and instruments for improving the quality of the environment and the sustainable use of natural resources; d) policies for integrating the environment and production; e) environmental policies and programs for the Amazon region; and f) ecological/environmental zoning.

The National Water Agency (Agência Nacional de Águas, ANA) was created on July 17th 2000 by Act 9.984 as an autonomous state entity connected to the Ministry of the Environment but with administrative and financial autonomy. It is responsible for providing the technical conditions for the Water Bill to be implemented; promoting decentralized, participative management in conjunction with the entities that make up the National Water Resource Management System; implementing the management instruments provided for in Act 9.433 of 1997, including permits to use water resources; charging for water use and inspecting its use; and pursuing solutions to the country's two major problems: prolonged droughts, especially in the northeast, and river pollution.

CONAMA is the advisory body for the national environment system (SISNAMA) and was created on August 31st 1981 by Act 6.938 with the purpose of assisting, studying and proposing to the

Government Board guidelines for government policies for the environment and water resources, and to give opinions on standards compatible with ensuring the ecological balanced of the environment and a healthy quality of life.

IBAMA is an autonomous state entity with administrative and financial autonomy that was created on February 22nd 1989 by Act 7.735 by merging four environmental entities: the Department of the Environment (Secretaria do Meio Ambiente, SEMA), the Rubber Department (Superintendência da Borracha, SUDHEVEA), the Fishing Department (Superintendência da Pesca, SUDEPE) and the Brazilian Forest Institute (Instituto Brasileiro de Desenvolvimento Florestal, IBDF). It is connected to the Ministry of the Environment and has the following tasks: to put into practice national environmental policies for the preservation, conservation and sustainable use of environmental resources and their inspection and oversight; and to carry out further activities on the behalf of the Union in compliance with current legislation and guidelines provided by the Ministry of the Environment.

The state environmental agencies are responsible for issuing environmental licenses for plants on state rivers.

The National Development Bank (Banco Nacional de Desenvolvimento Econômico e Social, BNDES) was created on June 20th 1952 by Act 1.628 as an autonomous state entity, then redefined as a private federal company with its own equity on June 21st 1971 by Act 5.662. It reports to the Ministry of Development, Industry and Foreign Trade and its main purpose is to provide support for projects that contribute to the country's development.

6 Legislation Concerning Hydropower Developments

6.1 General Information

Act 5.655 of May 20th 1971 covers the legal remuneration for investments made by electricity utilities.

The Federal Constitution of 1988 states that all hydroelectric potential and all rivers in more than one state are owned by the Union, which has the right to directly exploit it for generating electricity or do so indirectly by means of authorization, concession or permit, in conjunction with the states where the hydropower potential is located.

Act 8.631 of March 4th 1993 contains provisions on the setting of tariffs for public electricity services and revokes the previous practice of setting fixed rates.

In 1995, Act 8.987 was passed that brought a major change to the way public services were to be contracted. It regulates article 175 of the Federal Constitution of 1988, which states that contracts for public services and works and permits for public services must be the object of public tenders.

There is one provision of particular note, which states that the winning bidder is responsible for reimbursing the expenses incurred in carrying out the studies, investigations, surveys, designs, works and other expenses or investments involved in the contract that were of use for the bid and were undertaken by the Concession-granting authority or on its authorization, as set forth in the notice of tender. This effectively gives parties interested in taking part in public tenders access to these documents.

Act 9.074 (1995) was passed in response to article 1, single paragraph, of Act 8.987, passed in the same year, chapter 2 of which addresses electricity services, and sets out by what terms all concessions, permits and authorizations for exploiting electricity services and facilities and plants to harness water courses are to be given, extended or granted.

This law sets out what projects can only be contracted after a public bidding process, in which case the concession-granting authority must define the "optimal project". This is the overall energy potential of a plant from the cascade scheme selected for a river basin that is provided by the best dam axis, general physical layout, operating water levels, reservoir and installed capacity.

The National Electricity Agency (Agência Nacional de Energia Elétrica, ANEEL) was created on December 26th 1996 by Act 9.427 with the purpose of regulating and overseeing the generation, transmission, distribution and sale of electricity in compliance with government policies and guidelines. One of its powers is to hold public tenders for contracts for electricity generation, transmission or distribution services and to grant contracts for hydropower plants, as well as signing and managing contracts or permits for the rendering of public electricity services, public utility contracts, the issuing of authorizations, and the direct inspection or indirect inspection by means of agreements with state entities of contracts for the rendering of electricity services.

This document was amended by Acts 9.648 (05/27/98) and 10.438 (04/26/02), which establish rules for the granting of authorization for hydropower projects of over 1,000 kW and up to 30,000 kW for independent generation or autogeneration that are classified as small hydros.

ANEEL took over the technical archives, equity, duties, powers and revenues of DNAEE. The network of river gauging stations, technical archives and hydrology activities required for hydroelectric projects continue to be the responsibility of the Ministry of Mines and Energy under the temporary administration of ANEEL, as a member of the National Water Resource Management System.

On July 18th 2000, Act 9.984 was published in the federal publication *Diário Oficial da União* creating the National Water Agency (Agência Nacional de Águas, ANA) as an autonomous state entity with administrative and financial autonomy and linked to the Ministry of the Environment. Its remit is to implement the National Policy for Water Resources, making up part of the National Water Resource Management System. It is also responsible for granting the right to use water resources from bodies of water controlled by the Union by means of authorizations. It also sets and oversees the conditions for the operation of reservoirs with the purpose of assuring the multiple uses of the water resources and, in the case of reservoirs for hydropower generation, in conjunction with the ONS.

There is therefore some common ground in the remits of ANEEL and ANA. They should work together to obtain the declaration of available water resources for bidding processes for concessions or authorizations to harness the energy potential of bodies of water under Union control. However, when the energy potential is in water bodies controlled by states or the Federal District, the declaration of available water resources must be obtained from the state entity water resource management entity.

In 2003 a new bill was proposed for organizing Brazil's electricity sector, whose objective is to ensure that energy demands are fulfilled reliably, profitably and in an economically sustainable manner.

This bill culminated in Act 10.848, passed on March 15th 2004, which pertains to the sale of electricity and amends other laws, including Acts 9.074/95 and 9.427/96.

Act 10.848 states that the sale of electricity among agents that hold concessions, permits or authorizations for electricity services or facilities, and between these agents and their consumers within the National Interconnected System (SIN), should be contracted within the regulated or free markets, and that the distribution agents in this system should make sure they meet the entire demand of their markets by taking out regulated contracts with the winners of bidding processes. These tenders should include procedures for contracting energy from existing generation facilities, new generation facilities and alternative sources.

The new law states that electricity generation concessions granted prior to December 11th 2003 will have up to 35 years to pay off their investments, counting from the date the contract was signed, with the right to extend this term for a further 20 years on the discretion of the concession-granting authority, which is an express amendment of the provisions of Act 9.074 (1995).

By the provisions of this law, distribution agents are prevented from undertaking electricity generation or transmission activities or from selling electricity to free-choice consumers, unless this be under the same conditions as those established for captive consumers. This restriction does not apply to distribution utilities that provide services for isolated electricity systems or whose market is limited to 500 GWh/year or less. It also amends Act 9.427/96, by which ANEEL was created, stripping this agency of its power to coordinate activities with states and the federal district, the harnessing of the hydroelectric potential of water courses and bringing these activities into line with the national policy for water resources, the signing of contracts granting concessions or permits for electricity utilities, the granting of authorizations, the definition of optimal projects, and the termination of contracts, all of which are returned to the Ministry of Mines and Energy (concession-granting authority).

This law was regulated on July 30th 2004 by Decree 5.163, which stipulates which of the agents that will take part in the new sector model are the winning agents, which are understood as being the holders of concessions, permits or authorizations issued by the competent authority for the generation, importation or sale of electricity.

The winning bidders for energy from new plants will be eligible to pay a fee in exchange for concessions for the generation of electricity as a public service, or concessions for the use of public assets if they are autogenerators or independent generators. New generation facilities are those which had no kind of concession, authorization or permit, nor were part of an existing project that was being expanded to increase its installed capacity, by the beginning of the bidding process.

When generation projects win that are classified as small hydros, as per ANEEL Resolution 652 of December 9th 2003 or as thermoelectric generation plants, they are granted an authorization. When electricity is being imported, these authorizations should include the introduction of the associated transmission systems and provide for unrestricted access to these systems within their technical boundaries by means of the payment of a charge to be approved by ANEEL.

Another law, Act 10.847, was passed together with Act 10.848, by which Empresa de Pesquisa Energética (EPE) was created, replacing CCPE as the entity to provide support for the Ministry of Mines and Energy's planning activities. EPE provides the research services and studies required for the planning of the energy sector, and is under the auspices of the Ministry of Mines and Energy. It is responsible for conducting studies to determine the "optimal projects" for hydropower projects, social impact studies, technical, economic and socioenvironmental feasibility studies for electricity projects and projects involving renewable resources, obtaining the preliminary environmental license and the declaration of available water resources needed for public tenders involving electricity generation and transmission projects, and conducting the studies needed for drawing up short-, medium- and long-term electricity generation and transmission expansion plans.

6.2 Contracts for Electricity Sector Projects

All public utility concessions, whether or not they are preceded by public works, must be put to public tender according to the terms of the specific legislation and in accordance with the principles of legality, morality, publicity, equity and objective judgment of criteria, and according to the terms of the notice of tender (art 14 of Act 8.987 of February 13 1995).

The notice of tender must be prepared by the concession-granting authority in compliance with the general criteria and standards contained in the legislation covering public tenders and contracts (art. 18 of Act 8.987 of 1995). In particular, in the case of public services preceded by public works, it must contain the details of the works, including the elements of the basic design needed to fully characterize said work, as well as the guarantees required for this specific part of the contract, according to each case and limited to the value of the work (item 15, art. 18, Act 8.987, 1995, plus the revision in Act 9.648 of May 27th 1999).

According to article 4 of Act 9.074 (July 7th 1995), all concessions, permits and authorizations for the rendering of electricity services and exploitation of electricity facilities and plants that harness water courses must be contracted out, extended or authorized according to the terms of this law and Act 8.987 and any other pertaining to the matter.

All public tenders held for contracts of this nature should be in compliance with the provisions of the following acts: 8.987 (February 13th 1995), 9.074 (July 7th 1995), 9.427 (December 26th 1996) and, as a general standard, 8.666, (June 21st 1993) and article 23 of Act 9.427 (1996).

The public tenders for the exploitation of hydroelectric energy must take the form of a competition or an auction and the concessions will be granted upon payment of a fee (Art. 24, Act 9.427, 1996).

No hydroelectric project may be subject to a bidding process without the "optimal project" having been defined by the concession-granting authority. The winning bidder may be responsible for developing the basic and executive designs (item 2, article 5, Act 9.074, 1995).

An "optimal project" is understood as being all the energy to be generated by a project in the final selected cascade option for a river basin, defined by its general layout, with the best dam axis, general physical layout, operating water levels, reservoir and installed capacity, as set out in article 5 of Act 9074, 1995

ANEEL should be notified for registration purposes whenever feasibility studies[16], pre-projects or projects for hydroelectric power plants are being developed, although this does not grant any preferential treatment in obtaining the concession for rendering a public service or use of a public asset (art. 28, Act 9.427, 1996).

Hydroelectric facilities of 1,000 kW or less and thermoelectric power plants of 5,000 kW or less do not require a concession or authorization, although the concession-granting authority's regulatory agency and inspection unit should be notified of their existence for registration purposes (art. 5, Decree 2.003 of September 10th 1996).

Hydroelectric facilities of over 1,000 kW and up to 30,000 kW for independent generation or autogeneration should first be authorized by ANEEL, provided they are classified as a small hydro (item 1, Art. 26, Act 9.427 of 1996, revised in Art. 4, Act 9.648 of 1998).

ANEEL resolution 394 of December 4, 1998 establishes that small hydros are hydroelectric power plants with an installed capacity of over 1 MW and no more than 30 MW, with a total reservoir area of up to 3.0 km^2 (art. 2).

The area of the reservoir is set by the water elevation associated with the 100-year flood flow (Single Paragraph of Art. 2, ANEEL resolution 394 of 1998).

Projects that exceed the maximum reservoir area may, depending on regional characteristics, also be defined as small hydros upon special consideration by the ANEEL board based on a technical report which sets out the related economic and socioenvironmental aspects, among others (Art 3, ANEEL resolution 394 of 1998).

Authorization for hydroelectric facilities of over 1 MW and no more than 30 MW, as set out in paragraph 8 of this section, is granted after the basic design is approved by ANEEL (art 2, ANEEL Resolution 395 of December 4th, 1998).

16 The overall design of a given plant from the best cascade scheme selected in the survey of hydroelectric potential is defined at the feasibility stage, with a view to achieving optimal technical, economic and environmental performance and the best evaluation of associated costs and benefits.

According to the terms of ANEEL Directive 173 of May 7th 1999, authorization to exploit a hydroelectric power plant with a capacity of over 1 MW and no more than 30 MW for independent generation or autogeneration can only be granted upon the submission of the basic design and proof of receipt of the documents submitted for environmental licensing purposes by the relevant environmental management entity.

When an authorization is issued by ANEEL according to the terms of this directive, the start of construction works are still dependent on the submission of the Installation License and approval of the Basic Design. The Basic Design can be approved before the authorization is given, provided a Preliminary License has been submitted.

Those projects that fall outside the category of small hydros, as defined in ANEEL resolution 394 of 1998, as listed in paragraphs 10 to 20 of this item, are subject to concession via a bidding process (single paragraph, art. 2, ANEEL resolution 395 of 1998).

Those parties interested in applying for a concession to run hydroelectric power plants of a capacity greater than 30 MW or those that fall outside the category of small hydro as set forth in paragraph 14 of this text should submit their Feasibility Studies or Basic Designs to ANEEL, requesting the project's inclusion in the public tender program (art. 3 of ANEEL resolution 395 of 1998).

The general procedures for registering the Feasibility Studies and Basic Designs are set forth in ANEEL resolution 395 of December 4th 1998 (articles 4-12).

The subjects that must be covered in the Feasibility Studies and Basic Designs include work in conjunction with environmental and water resource management entities with a view to defining the optimal project, the assurance of multiple water uses, and the obtainment of the relevant environmental license(s).

The general procedures for the choice of the Feasibility Studies and Basic Design to be put to tender are established in ANEEL resolution 395 of December 4th 1998 (art. 13 and 14).

As noted in paragraph 8 of this section, the final Feasibility Study report may form the technical basis for the bidding process for hydroelectricity generation projects.

The overall design of a given project from the best cascade option selected in the Inventory Studies is defined at the Feasibility stage, with a view to achieving optimal technical, economic and environmental performance and the best evaluation of associated costs and benefits.

The Basic Design should incorporate the main items set forth in the notice of tender for the project and those that are defined in the Feasibility Studies in order to assure the energy generation as originally estimated. These items are:

- maximum upstream water level;
- downstream water level;
- minimum capacity; and
- geographic coordinates.

The building of thermoelectric power plants of over 5,000 kW for autogeneration or independent generation is subject to authorization (see Art. 4, Decree 2.003 of September 10th 1996).

Those hydropower plants with a capacity of 1,000 kW or less and thermoelectric plants of 5,000 kW or less do not require a concession or authorization, but the concession-granting authority's regulatory and inspection entities must be notified of their existence for registration purposes (art. 5, Decree 2.003, September 10th 1996).

All transmission facilities that are part of the Basic Network of Interconnected Electric Systems[17] must be included in a bidding process, and will work as integral facilities of these systems. Their operating rules will be defined by an agent under federal control so as to assure the optimization of existing or future electricity resources (item 1, art. 17, Act 9.074, 1995).

Those transmission facilities within the ambit of a distribution utility may be considered by the concession-granting authority as making up part of the distribution contract already granted (item 2, article 17, Act 9.074, 1995).

Those transmission facilities of interest only to generation plants may be considered an integral part of their respective concessions, permits or authorizations (item 2, article 17, Act 9.074, 1995, and textual review in act 9.648 of 1998).

Imports and exports of electricity require authorization from ANEEL, as does the implementation of the respective transmission systems.

6.3 Resolutions Relating to Hydropower Inventory Studies

Hydropower Inventory Studies of river basins are defined as the planning stage when the hydroelectric potential of a river basin is quantified by studying different cascade options and deciding on an optimal project. An optimal project is understood as being all the energy to be generated by a project in the final selected cascade option for a river basin, defined by its general layout, with the best dam axis, general physical layout, operating water levels, reservoir and installed capacity, as set out in article 5 of Act 9074, 1995.

Inventory Studies may be undertaken by any of the stakeholders upon their registration with ANEEL. The general procedures for registering and approving these studies are set down in ANEEL resolution 393 of December 4th 1998.

ANEEL resolution 393 also recognizes the right to the reimbursement of costs incurred and recognized by ANEEL should any of the projects identified in the Inventory Studies be included in a public tender for electricity generation contracts.

The procedures for presenting expenses incurred in developing studies or designs for hydropower projects and the regulation of the remuneration of these sums are set forth in DNAEE Directive 40 of February 26th 1997.

ANEEL resolution 398 of September 21st 2001 sets out the general requirements for the submission of studies and the specific conditions and criteria for analyzing and comparing Inventory Studies, with a view to selecting amongst competing studies.

6.4 Financial Compensation and Royalties

The Federal Constitution of 1988 states that Brazilian states, the federal district, municipalities, and entities that are administrated directly by the Union have the right to a share of the profits from activities involving the exploration of oil, natural gas and water resources for the purposes of generating electricity, and of other mineral resources within Brazilian territory, the continental shelf, territorial waters or the exclusive economic zone, or to financial compensation for such activities.

Act 7.990 of December 28th 1989 provides financial compensation for the use of water resources for electricity generation. The amount currently paid by contract- or authorization-holders for such activities is 6.75% of the value of the energy produced, which is calculated by multiplying the energy generated by a reference tariff. When this legislation was passed, plants with a nameplate capacity of

17 The Basic Network Interconnected Electric Systems will consist of all the substations and transmission lines at voltages of 230 kV or more that are part of electricity service concessions that have been granted by the competent authority.

10 MW or less were exempted from this payment. This exemption has been extended to plants of a capacity of 30 MW or less, provided they are classified as small hydros.

Of the total of 6.75%, 6% goes to the municipalities in which the dams are built and the states that host the reservoirs. This amount is broken down as follows: 45% for states, 45% for municipalities and 10% for the Union, of which 3% goes to the Ministry of the Environment, 3% to the Ministry of Mines and Energy and 4% goes to the National Fund for Scientific and Technological Development, which is administrated by the Ministry of Science and Technology.

In the case of hydropower plants that benefit from upstream reservoirs, the extra energy is calculated as being generated by these regulation reservoirs, and ANEEL is responsible for making the evaluation necessary to determine the proportion of financial compensation due to the states, federal district and municipalities affected by these reservoirs.

In the case of Itaipu, the percentages owed to the entities directly administrated by the union, the states and the municipalities directly affected by the plant are unchanged, but the other percentages established above are based on 85% of the royalties owed by Itaipu Binacional ao Brasil, as per Annex C, item III of the Itaipu Treaty, signed on March 26th 1973 between Brazil and Paraguay, and its related documents, while 15% goes to the states and municipalities with reservoirs upstream from the power plant, which contribute to the amount of energy it produces.

The remaining 0.75% goes to the Ministry for the Environment for the implementation of the National Policy for Water Resources and the National Water Resource Management System. The portion of the 6% that the Ministry for the Environment receives should be used for implementing the National Policy for Water Resources, the National Water Resource Management System and for managing the country's network of water gauging stations.

6.5 National Policy for Water Resources

The management of water resources should always provide for multiple water uses based on the guiding principles of the National Policy for Water Resources. The harnessing of hydroelectric power is one such use of water resources.

The concern for multiple water uses is stressed in Decree 24.643, better known as the "Water Code", which establishes that in all hydroelectric plants the "requirements of general interests" must be assured, which are:

- consumption and other needs of riverside communities;
- public health;
- shipping;
- irrigation;
- protection against flooding;
- conservation and free circulation of fish;
- flow maintenance.

However, the conditions for exploiting water for other uses are limited by the provisions of the decree that state that water and energy reserves for public services (Union, states or municipalities) should not deprive a hydropower plant of more than 30% of the energy at its disposal. The Water Code further states that utilities lose the rights granted to them in their concessions if they repeatedly use flows greater than those to which they were entitled, whenever this infraction affects the quantity of water reserved for other uses.

According to Federal Constitution 1988, the lakes, rivers and any water courses in land under its control or which flow through or occupy more than one state, which form borders with other countries or extend into foreign territory or come from it, as well as the land along their banks, the river beaches and the hydroelectric potential are all the property of the Union. Meanwhile, the States own all flows, sources or stores of surface or ground water, except those owned by the Union and which arise as the result of works undertaken by the Union.

The Federal Constitution of 1988 states that it is the Union's right to exploit electricity facilities and harness the energy from water courses, either directly or through authorization, concession or permit, working in coordination with the States where these sources of energy are. The harnessing of "low capacity renewable energy potential" is exempted from the requirement to obtain an authorization or concession.

It also states that it is the right of all citizens to live in an ecologically balanced environment, which is to the benefit of all people and is essential for a healthy quality of life, the defense and preservation of which for present and future generations is the duty of the public authorities and society as a whole. As such, the public authorities are bound to carry out a number of actions, including that of "requiring, in the terms of the law, that prior environmental studies be undertaken and made public before any works or activities can be undertaken that could potentially harm the environment."

Another concern is the indigenous issue, which is highlighted in the Constitution. An example of this is the fact that it is the exclusive power of the National Congress to authorize the exploitation and harnessing of water resources and the exploration and extraction of mineral resources on indigenous lands, and grants indigenous peoples the right to occupy their traditional lands, stating that "the harnessing of water resources, including hydropower potential, and the exploration and extraction of mineral resources on indigenous lands can only be carried out upon authorization by the National Congress, after the communities affected have been heard, and assuring them a share of the profits of the mining, in abidance with the law."

In this chapter we set out the main legal instruments passed on a federal level that pertain to the planning and operating of hydropower stations, starting with the issue of financial compensation addressed in the Constitution.

Legislation Pertaining to Water Resources

Act 9.433 of January 8th 1997 created the National Policy for Water Resources and the National Water Resource Management System.

The aim of the National Policy for Water Resources is to assure the availability of water to meet the needs of present and future generations and levels of water quality compatible with its respective uses; the efficient, integrated use of water resources, including water transportation, with a view to sustainable development; and the prevention of and defense against critical hydrological events of a natural origin or arising from the inappropriate use of natural resources.

The following are instruments of the National Policy for Water Resources:

- all plans for water resources;
- the classification of bodies of water according to their most prevalent uses;
- the granting of rights for the use of water resources;
- charging for the use of water resources;
- compensation for local authorities; and
- information database on water resources.

The harnessing of hydroelectric power is one of the water uses that is subject to the granting of rights, as are all other uses that alter the hydrological patterns, and the quantity or quality of water existing in the body of water.

These rights are granted by the competent authority of the federal, state or federal district executive, though the federal executive may also delegate to the states or the federal district the power to grant rights for the use of waters under Union control.

The granting of such rights is subject to the priorities of use established in the individual plans for water resources, and should respect the classification of the body of water in question and assure that the conditions required for water transportation, when such exists, are maintained, adding that the multiple uses of the water should also be preserved. The plans for water resources are master plans that provide guiding principles for implementing the National Policy for Water Resources and water resource management, and are prepared per individual river basin, per state and for the whole country.

The granting of such rights and the use of water resources for the purposes of generating electricity is subordinated to the National Plan for Water Resources. Until the National Plan for Water Resources is approved and regulated, the use of hydraulic potential for generating electricity will continue to be covered by the relevant sector legislation.

It is the responsibility of River basin Committees to approve the Plan for Water Resources for the river basin in question and that of the National Council for Water Resources (Conselho Nacional de Recursos Hídricos, CNRH) to oversee its execution and approve the National Plan for Water Resources and to determine any measures needed to meet its targets.

On January 30th 2006 at the 17th extraordinary meeting of the National Council for Water Resources, the National Plan for Water Resources was approved as per the terms of resolution 58 of the same date, published in *Diário Oficial da União* on March 8th 2006, comprising the following volumes:

I – Overview and State of Water Resources in Brazil;

II – Waters for the Future: Scenarios for 2020;

III - Guidelines; and

IV – National Programs and Targets.

This resolution states that the operational details of the programs and targets contained in item 4 should be coordinated by the Department of Water Resources, within the ambit of the Ministry of the Environment, and submitted for approval by the National Council for Water Resources by December 31st 2007.

One of the major guidelines of subprograms V.2 (Ensuring Compatibility and Integration between Sector Projects and Incorporation of Guidelines of Interest to the Integrated Management of Water Resources) and V (Program for Inter-Sector, Inter-Institutional and Intra-Institutional Coordination in the Management of Water Resources) is to assess ways of implementing article 52 of Act 9.433 of 1997 (pp 20 and 61 of Volume 4 – National Programs and Targets).

It is also the responsibility of the River Basin Committee to hold discussions about the issues concerning the water resources within its area of influence and to coordinate the action of the entities involved, and provide arbitration at a first administrative level for any conflicts over water resources. The National Council for Water Resources is also responsible for ensuring that water resource planning is in line with national and state-wide planning and with plans for different groups of users, and to deliberate on projects for harnessing water resources whose repercussions may extend beyond the state which hosts them.

Act 9.984 of July 17th 2000, which covers the creation of the National Water Agency (ANA), establishes that in its interaction with the electricity sector this agency has the power to:

- issue authorizations that grant the right to the use of water resources in bodies of water controlled by the Union;
- inspect the uses of water resources in bodies of water under Union control;
- define and inspect the operating conditions of reservoirs by public and private entities, with a view to assuring the multiple uses of water resources, as set forth in the water management plans for the respective river basins. The conditions for the operation of reservoirs for hydroelectric facilities will be set in coordination with the ONS;
- foster the coordination of the activities undertaken by the national network of river gauging stations in association with the public or private entities that are part of it or use it;
- organize, implement and manage the National Water Resource Information System (Sistema Nacional de Informações sobre Recursos Hídricos).

The granting of rights to use water resources under Union control, including hydropower facilities, will respect the following deadlines, counting from the date that the respective authorizations are published:

- up to two years to start implementing the project in question;
- up to six years to conclude the implementation of the project;
- up to 35 years for the term of the authorization is granted.

Additionally, in the case of concessions or authorizations for utilities or for the generation of hydroelectric energy, the deadlines are the same as those on their respective concession or authorization contracts. Meanwhile, the deadlines for beginning and concluding the implementation of projects may be extended when the size and the socioeconomic importance of the project so justify, in the view of the National Council for Water Resources, while the duration of the concession or authorization can be extended by the National Water Agency (ANA), in compliance with the priorities set in the Plans for Water Resources.

Act 9.984 of 2000 states that for public tenders for concessions or authorizations to harness the hydroelectric potential of water bodies under Union control, ANEEL should work together with ANA to first obtain a declaration of available water resources. When the water resources are under state or federal district control the declaration of available water resources is obtained from the respective water resource management entity.

The declaration of available water resources is automatically transformed by the competent authority into an authorization to use such water resources for the institution or company that receives the concession or authorization from ANEEL to harness the energy potential of the water resource in question.

According to the provisions of Act 10.847 of March 15 2005, EPE is responsible, among other things mentioned in the law, for obtaining the preliminary environmental license and the declaration of available water resources, which are needed for tenders for hydroelectricity generation and electricity transmission projects selected by the company.

Resolution 16 issued by the National Council for Water Resources (CNRH) on May 8th 2001 sets the basic guidelines for granting the right to use water resources, stating that ANEEL should obtain a declaration of available water resources before it can hold a bid for a concession or authorize the harnessing of hydroelectric potential, and this declaration will be turned into an instrument that grants the right to use said water resources. Further, CNRH Resolution 37 of March 26th 2004 sets more specific guidelines for granting the use of water resources for the construction of dams on bodies of water under state, federal district or union control. It sets the following terms:

- minimum outflow: minimum flow for the satisfactory fulfillment of the needs of multiple uses of the waters, which is instrumental in the operation of the reservoir;
- contingency plan: set of measures and procedures required to ensure that the authorized multiple uses of the water continue to be fulfilled, while observing minimum outflows;
- emergency action plan: document containing the procedures to be taken in emergency situations, as well as the flood maps indicating the maximum range of floodwaters and their respective times of arrival in the event of the rupture of the dam;
- sector declaration: declaration issued by the respective government department.

CNRH Resolution 37 states that at the start of the project planning stage, the main stakeholder should contact the competent authority to request a list of the documents and details of the technical studies required for the corresponding application for the right to exploit the water resources, and that this competent authority will define what information must be included in the technical studies at the planning, design, construction and operation stages of the project, formulating a *Termo de Referência* (official list of technical criteria) with the hydrological characteristics of the river basin, the size of the dam, the purpose of the works and the uses of the water resources. The competent authority will notify the main stakeholder of when it should submit the most important documents, such as environmental licenses, sector plans and emergency action plans for the project. It should be noted that the absence of a sector plan, when justified, should not prevent the application for the right to use water resources from being made or analyzed. In this case, the competent authority will be liable for taking any measures required for the process to advance normally.

The rules for operating the reservoirs, the emergency action plan and the contingency plan can be reassessed by the competent authority and by ANA (as far as the items within its ambit are concerned), taking into account the multiple water uses, risks arising from accidents, and critical hydrological events. The resolution also states that users should establish and maintain reservoir monitoring activities (upstream and downstream), sending the competent authority the data recorded and/or measured in the format determined by said authority.

ANA resolution 131 of March 11th 2003 covers procedures for the issue of the declaration of available water resources and the concession or authorization for the right to use water resources for projects of over 1 MW in bodies of water under Union control. This resolution lists the documents that ANEEL must send ANA to obtain the declaration, as well as setting its validity at up to 3 years, renewable for a further 3 years. It also exempts holders of concessions or authorizations issued by March 11th 2003 from applying for a new concession or authorization.

6.6 National Policy for the Environment

This section sets out the main legal instruments covering the environmental aspects of hydropower facilities, as well as the process for environmental licensing. Rather than a very detailed explanation of the legislation, a summary of the most important aspects is given.

Brazil's Environmental Legislation and Hydropower Projects

Brazil's environmental legislation is formed of instruments for the control, prevention, promotion, fostering of the environment, and has taken great strides even with respect to international legislation. Brazil's legislation aims to encourage voluntary actions, provide for environmental planning and preventive actions, going beyond the limitations of corrective and mitigating actions.

Act 6.938 of 1981 introduced the National Policy for the Environment, its objectives, and the mechanisms by which it was formulated and is to be implemented. By this law, the National

Environment System (Sistema Nacional do Meio Ambiente, SISNAMA) and the Technical Federal Register of Instruments and Activities for the Defense of the Environment (Cadastro Técnico Federal de Atividades e Instrumentos de Defesa Ambiental) were established. The policy's instruments include[18]:

- environmental quality standards;
- environmental zoning;
- assessment of environmental impacts;
- licensing and review of activities that cause or could cause pollution.

By this law, the construction, installation, expansion and operation of facilities and activities that use environmental resources that are considered polluting or potentially polluting, as well as those that could in any way harm the environment, are subject to prior licensing by the relevant state entity. These requirements are within the ambit of the National Environmental System (Sistema Nacional do Meio Ambiente, SISNAMA) and IBAMA, and do not substitute the other licenses required. In the case of activities and works within a national or regional ambit that cause a major environmental impact, IBAMA is the entity responsible for issuing licenses.[19]

Environmental licensing and environmental impact assessments are essential instruments for environmental planning and prevention of damage to the environment. The environmental licensing process for those activities that affect the environment is regulated by CONAMA resolution 001 of January 23rd 1986. This resolution sets the liabilities, basic criteria and general guidelines for the use and implementation of environmental impact assessments within the scope of the National Policy for the Environment. The resolution also states that the licensing of activities that alter the environment, such as electricity generation plants over 10 MW, whatever their primary source of energy, require an Environmental Impact Assessment (EIA) and Environmental Impact Report (RIMA), which must be submitted for approval by the competent authority[20].

CONAMA resolution 006 of September 16th 1987 sets out the general rules for the environmental licensing of large-scale works, especially those in which the Union has a particular interest, with the purpose of bringing into line the concepts and language used by the different stakeholders in the process. In virtue of the importance and specific nature of the cycle of development of electricity projects, specific procedures have been designed for licensing projects in this area. From the analysis of their feasibility during the initial study phase until their construction and operation, hydropower facilities are subject to a set of studies and procedures in order to ensure that the projects are as environmentally friendly as possible.

The environmental licensing process requires the following environmental licenses at the different stages of introducing new hydroelectric power plants[21]:

- Preliminary License (Licença Prévia, LP) – granted at the project planning stage, approving its location and design, attesting to its environmental feasibility, and setting requirements and preconditions to be met at the location, installation and operation phases, in accordance with existing municipal, state and federal plans for land use;
- Installation License (Licença de Instalação, LI) – authorizes the beginning of construction in compliance with the specifications contained in the approved plans, programs and projects, including the environmental control measures and other factors;

18 Act 6938/81 – Art. 9, items 1 to 4.

19 Act 6938/81 – Art. 10.

20 CONAMA resolution 001/86 – Art. 2

21 CONAMA resolution 006/87 – Art. 4

- Operating License (Licença de Operação, LO) – authorizes the operation of the plant, upon verifying the effective compliance with the previous licenses, environmental control measures and other factors required for its operation. The granting of this license depends on compliance with all items examined and approved for the first two licenses (LP and LI).

Environmental licenses can be issued individually or successively, depending on the nature, characteristics and phase of the project or activity.

The EIA and RIMA are required before the preliminary license can be granted, and should be prepared at the preliminary planning phase. They contain the basic or essential requirements, guidelines, recommendations and limitations that must be addressed at the planning, installation and operating phases of the project. The final project should meet all the recommendations set out in the EIA/RIMA.

The Federal Constitution of 1988 makes the public authorities responsible for "demanding, in the terms of the law, that a preliminary environmental study be undertaken and made public before a work or activity that could potentially cause major environmental damage is installed," (art. 225, item &1, VI). This consolidates the role of environmental impact studies as preventive instruments within the ambit of the National Policy for the Environment, and gives the Public Prosecution Service (Ministério Público) a central role in protecting Brazil's environment.

Another concern is the indigenous issue, which is highlighted in article 49 of the Constitution, which states that it is the exclusive power of the National Congress to authorize the exploitation and harnessing of waters on indigenous lands. In article 231, paragraph 3, in acknowledging indigenous peoples' original rights to the lands they traditionally occupy, it states that "the harnessing of water resources, including hydropower potential, and the exploration and extraction of mineral resources on indigenous lands can only be carried out upon authorization by the National Congress, after the communities affected have been heard, and assuring them a share of the profits of the mining, in abidance with the law," thereby guaranteeing these peoples' rights.

CONAMA resolution 237/97 amends resolution 001/86 in aspects pertaining to environmental licensing. The new resolution covers new topics, including a list of projects subject to environmental licensing, confirming that environmental licensing will depend on the prior issue of an EIA and RIMA for projects that could cause environmental damage, and environmental studies of a relevant nature are required for those that are unlikely to cause harm to the environment.

While amending and expanding on resolutions 001/1986 and 006/87, CONAMA resolution 237/1997 also sets out the powers of environmental entities, previously mentioned in Act 6.938/1981. It defines not only the powers of the different environmental bodies from different spheres of government, but also states that projects should be licensed by a single entity.

Article 4 of this resolution sets out the situations in which IBAMA is responsible for the environmental licensing, these being for activities and projects with a major environmental impact of a national or regional scope.

In November 2002, Resolution 15 of the National Council for Energy Policy (Conselho Nacional de Política Energética, CNPE) created a working group to propose procedures and mechanisms designed to ensure that as of 2004, all projects designed to expand electricity supply have a Preliminary Environmental License before they can be authorized or put to tender. Likewise, Act 10487/2004 authorizes the creation of EPE, giving it the power to obtain preliminary licenses and issue a declaration of available water resources needed for tenders of hydroelectric generation facilities selected by EPE.

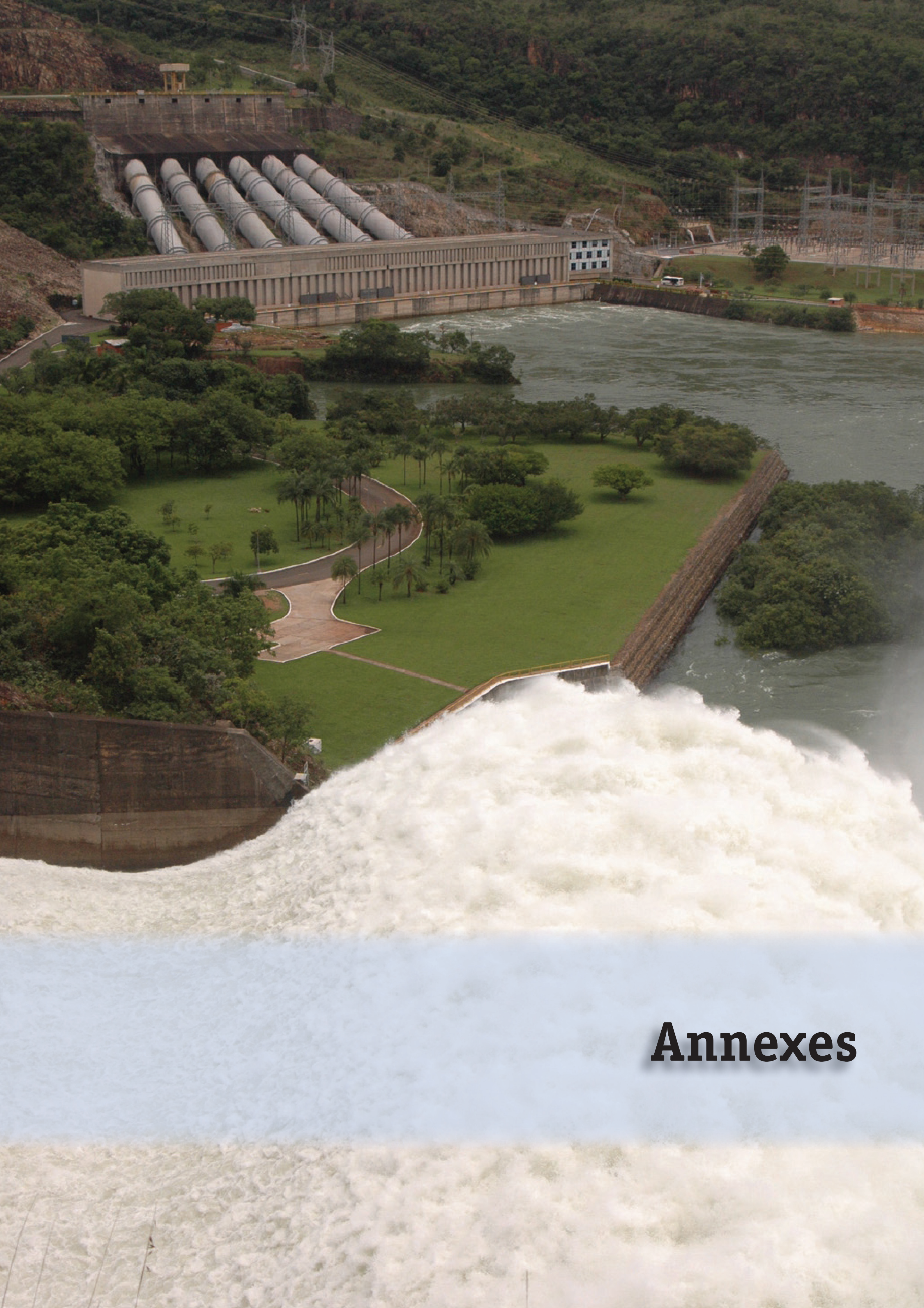
Aside from these general legal and normative instruments, when studies and projects are being developed for the electricity industry, all federal, state and municipal environmental legislation should be fulfilled, not only for environmental licensing purposes, but for the different topics relating to the physical, biota, socioeconomic and cultural factors covered in environmental studies.

Although CONAMA resolutions 006/87 and 237/97 regulate and adjust environmental licensing requirements for large-scale works, especially for electricity projects, they do so focusing on projects in isolation. However, with the passage of time and the experience gained by companies from this sector, it has been found that for hydroelectric projects, socioenvironmental studies have to be made with a broad enough reach to cover the set of projects from a single river basin, which is the scope of the Hydropower Inventory Studies, and which has a direct impact on industry planning. The first initiative in this respect was taken when the Manual for Hydropower Inventory Studies was reviewed in 1996 and 1997. The socioenvironmental aspects addressed in it were expanded and gained new depth, being given equal importance as the other topics covered (engineering and economic) in the decision-making process for selecting the projects to comprise the best cascade.

The integration of socioenvironmental aspects from the earliest stages of planning is an increasing requirement. An example of this is the analysis of the likely interferences brought about by the projects in the final selected cascade, which is now required by environmental entities and the Public Prosecution Service, resulting in the execution of Integrated Environmental Analyses (IEA) for basins with projects that already exist, are under construction or are being planned.

Methodological improvements of this nature are now common practice in the electricity sector and are increasingly being incorporated into the methodology used in Inventory Studies in a quest to refine them. However, these improvements have not yet led to any change in environmental legislation, since Inventory Studies precede the studies and design stages addressed by the legislation that pertains to the licensing process for such projects.

However, studies for hydroelectricity projects in a given river basin – which are of strategic importance to planning the supply of electricity – are gaining in importance for the environmental area. Even if they are not covered by the law, they can enhance the quality of environmental licensing processes, in that Inventory Studies include Integrated Environmental Assessments, which provide an analysis of the synergistic and cumulative effects arising from the environmental impacts caused by the hydropower projects within the basin, providing indicators, parameters and guidelines for the preparation of environmental studies that are required for the environmental licensing of each project.



Annexes

ANNEXES

ANNEX A

Graphs and Tables from the Preliminary Studies

Name of Graph – PRELIMINARY STUDIES	Name of File
HYDRAULIC TURBINES selection of type of turbine	grfA01.pdf
LAND DEVELOPMENTS IN THE PLANT AREA unit cost per installed MW	grfA02.pdf
RIVER DIVERSION AND CONTROL global cost of civil construction work and hydromechanical equipment	grfA03.pdf
EMBANKMENT DAMS (earthfill and rockfill) – COST PER METER OF LENGTH OF CREST cost per meter of crest	grfA04.pdf
ROLLER-COMPACTED CONCRETE DAM cost per meter of crest	grfA05.pdf
CONVENTIONAL CONCRETE DAM cost per meter of crest	grfA06.pdf
TRANSITION AND RETAINING WALLS MADE OF CONCRETE global cost	grfA07.pdf
CHANNEL SPILLWAY (LOW OGEE CREST) CIVIL CONSTRUCTION WORK – global cost	grfA08.pdf
ROLLER-COMPACTED CONCRETE OVERFLOW SPILLWAY (HIGH OGEE CREST) – CIVIL CONSTRUCTION cost per m^3/s of capacity	grfA09.pdf
CONVENTIONAL CONCRETE OVERFLOW SPILLWAY (HIGH OGEE CREST) – CIVIL CONSTRUCTION cost per m^3/s of capacity	grfA10.pdf
SURFACE SPILLWAY HYDROMECHANICAL AND HOISTING EQUIPMENT unit cost	grfA11.pdf
INTAKE – CIVIL CONSTRUCTION cost per block	grfA12.pdf
INTAKE – EQUIPMENT FOR KAPLAN TURBINES WITH A CONCRETE SEMI-SPIRAL CASING global cost of hydromechanical and lifting equipment for the water intake	grfA13.pdf
INTAKE – EQUIPMENT FOR BULB TURBINES global cost of hydromechanical and lifting equipment for the water intake	grfA14doc
INTAKE – EQUIPMENT FOR FRANCIS TURBINES OR KAPLAN TURBINES WITH A STEEL SPIRAL CASING global cost of hydromechanical and lifting equipment for the water intake	grfA15.pdf
HEADRACE CANALS cost per meter	grfA16.pdf
HEADRACE TUNNELS cost per meter	grfA17.pdf
STEEL SURFACE PENSTOCKS cost per meter (without valves)	grfA18.pdf
PRESSURE TUNNELS cost per meter (without valves)	grfA19.pdf
FRANCIS TURBINES unit cost	grfA20.pdf
KAPLAN TURBINES WITH A STEEL SPIRAL CASING unit cost	grfA21.pdf
KAPLAN TURBINES WITH A CONCRETE SEMI-SPIRAL CASING unit cost	grfA22.pdf
BULB TURBINES unit cost	grfA23.pdf

VERTICAL-AXIS GENERATORS unit cost	grfA24.pdf
CONVENTIONAL HORIZONTAL-AXIS GENERATORS unit cost	grfA25.pdf
HORIZONTAL-AXIS BULB GENERATORS unit cost	grfA26.pdf
TOTAL COST OF CIVIL CONSTRUCTION FOR OUTDOOR POWERHOUSE global cost	grfA27.pdf
ROADS unit costs (R$/km)	quadA01.pdf
RAILROADS unit costs	quadA02.pdf
ROAD BRIDGES unit costs (R$/m^2)	quadA03.pdf

ANNEX B

Graphs and Tables from the Final Studies

Name of Graph – FINAL STUDIES	Name of File
HYDRAULIC TURBINES selection of type of turbine	grfB01.pdf
PELTON TURBINES initial specific velocity	grfB02.pdf
PELTON TURBINES coefficient of peripheral velocity	grfB03.pdf
FRANCIS TURBINES initial specific velocity	grfB04.pdf
FRANCIS TURBINES coefficient of peripheral velocity	grfB05.pdf
KAPLAN TURBINES initial specific velocity	grfB06.pdf
KAPLAN TURBINES coefficient of peripheral velocity	grfB07.pdf
BULB TURBINES initial specific velocity	grfB08.pdf
BULB TURBINES coefficient of peripheral velocity	grfB09.pdf
FRANCIS TURBINES unit cost	grfB10.pdf
KAPLAN TURBINES WITH A STEEL SPIRAL CASING unit cost	grfB11.pdf
KAPLAN TURBINES WITH A CONCRETE SEMI-SPIRAL CASING unit cost	grfB12.pdf
BULB TURBINES unit cost	grfB13.pdf
CONVENTIONAL HORIZONTAL-AXIS GENERATORS unit cost	grfB14doc
HORIZONTAL-AXIS BULB GENERATORS unit cost	grfB15.pdf
VERTICAL-AXIS GENERATORS unit cost	grfB16.pdf
MAIN BRIDGE CRANE FOR POWERHOUSE unit cost	grfB17.pdf
MAIN GANTRY CRANE FOR POWERHOUSE unit cost	grfB18.pdf
LAND DEVELOPMENTS IN THE PLANT AREA unit cost	grfB19.pdf
INSTALLATIONS AND FINAL WORKS FOR THE POWERHOUSE unit cost per installed MW	grfB20.pdf
TAINTER GATE FOR SURFACE SPILLWAY unit cost	grfB21.pdf
TAINTER GATE FOR BOTTOM SPILLWAY unit cost	grfB22.pdf
FIXED-WHEEL GATE unit cost	grfB23.pdf
SURFACE STOPLOG unit cost	grfB24.pdf
BOTTOM STOPLOG unit cost	grfB25.pdf

GANTRY CRANE FOR SPILLWAY unit cost	grfB26.pdf
GANTRY CRANE FOR INTAKE unit cost	grfB27.pdf
STEEL TRASH RACKS FOR INTAKE unit cost	grfB28.pdf
BUTTERFLY VALVE unit cost	grfB29.pdf
SPHERICAL VALVE unit cost	grfB30.pdf
BUILDING OF CONSTRUCTION SITES AND WORKERS' CAMPS total cost	grfB31.pdf
MAINTENANCE AND OPERATION OF CONSTRUCTION SITES AND WORKERS' CAMPS total cost	grfB32.pdf
UNDERGROUND EXCAVATION IN ROCK cost per m^3	grfB33.pdf
ROADS unit costs (R$/km)	quadB01.pdf
RAILROADS unit costs	quadB02.pdf
ROAD BRIDGES unit costs (R$/$m^2$)	quadB03.pdf
INTEREST RATES DURING CONSTRUCTION	quadB04.pdf

ANNEX C

Spreadsheets for Dimensioning and Quantification

Spreadsheets – Final Studies	Name of File
Standard Cost Estimate – Preliminary Studies	49ope.xls
Standard Cost Estimate – Final Studies	56ope.xls
Unit Costs	57punit.xls
Powerhouse – Pelton turbines	572p.xls
Powerhouse – vertical-axis Francis turbines	572fv.xls
Powerhouse – horizontal-axis Francis turbines	572fh.xls
Powerhouse – Kaplan turbines with a steel spiral casing	572ka.xls
Powerhouse – Kaplan turbines with a concrete spiral casing	572kc.xls
Powerhouse – Bulb turbines	572b.xls
Cofferdam for river diversion through tunnels or galleries	573ert1.xls
Cofferdam for river diversion in stages	573ert2.xls
Diversion Channel	573c.xls
Diversion Gallery	573ga.xls
Diversion Tunnel	573td.xls
Earthfill Dam	574t.xls
Rockfill dam with vertical clay core	574enav.xls
Rockfill dam with inclined clay core	574enai.xls
Concrete-Faced Rockfill Dam	574efc.xls
Conventional Concrete Gravity Dam	574ccg.xls
Conventional Concrete Gravity Dam with Sluiceways	574ccgad.xls
Roller-Compacted Concrete Dam	574ccr.xls
Roller-Compacted Concrete Dam with Diversion Sluiceways	574crad.xls
Transition and Retaining Walls	574m.xls
Gated Spillway with a High Ogee Crest and Stilling Basin	575cobd.xls
Gated Spillway with a High Ogee Crest and Stilling Basin and Diversion Sluiceways	575cobda.xls
Gated Spillway with a High Ogee Crest and Ski Jump	575cose.xls
Gated Spillway with a High Ogee Crest, Ski Jump and Diversion Sluiceways	575cosea.xls
Gated Abutment Spillway with Stilling Basin	575coenb.xls
Gated Abutment Spillway with Ski Jump	575coens.xls
Ungated Spillway with a High Ogee Crest and Stilling Basin	575lobd.xls
Ungated Spillway with a High Ogee Crest and Stilling Basin and Diversion Sluiceways	575lobda.xls
Ungated Spillway with a High Ogee Crest and Ski Jump	575lose.xls
Ungated Spillway with a High Ogee Crest, Ski Jump and Diversion Sluiceways	575losea.xls
Ungated Abutment Spillway with Ski Jump	575loens.xls
Ungated Abutment Spillway with Stilling Basin	575loenb.xls
Headrace Canal	576cn.xls
Gravity Intake	576tg.xls
Intake Penstock	576ca.xls
Surge Tank	576ch.xls
Pressure Penstock with no Intake Tunnel or Surge Tank	576cf.xls
Pressure Penstock with Intake Tunnel and Surge Tank	576cfch.xls
Pressure Tunnel with no Intake Tunnel or Surge Tank	576tf.xls
Pressure Tunnel with Intake Tunnel and Surge Tank	576tfch.xls

ANNEX D

SINV System

SINV 6.0

FEATURES

SINV version 6.0 has **seven functions** for **Energy Studies**:

- ***Firm Energy***: calculates the firm energy of a given cascade option and the projects it contains.
- ***Optimize Live Storage***: determines the live storage of the projects that make up a given cascade option.
- ***Energy Dimensioning***: determines the live storage, reference head and installed capacity of the projects from a group of cascades.
- ***Live Storage Replenishment***: checks whether the time taken to replenish the live storage of the reservoirs in a cascade option is more or less than 36 months counted as of the end of the critical period.
- ***Eliminate***: calculates the cost/energy benefit index (ICB) of the projects in a given cascade option, and identifies which projects' ICB is greater than the reference unit cost, so that the user can eliminate these from the cascade.
- ***Economic-Energy Assessment***: calculates the ICB of the different cascade options under study and ranks them by their ICB.
- ***Sequencing***: after the final cascade has been selected, based on its cost/energy benefit index (ICB), negative socioenvironmental index (IAn) and positive socioenvironmental index (IAp), ranks the projects from this cascade by their cost/energy benefit indexes (ICB).

The six functions used in the energy studies can be calculated by means of a ***simplified procedure*** during the ***Preliminary Studies***, or by ***simulating operations*** during the ***Final Studies***.

For the Socioenvironmental Studies, SINV version 6.0 has a function for calculating the negative socioenvironmental impacts of the cascade options (***Calculate Negative Socioenvironmental Impact***). This function has two procedures for calculating the socioenvironmental impact indexes of the cascade options, one for use in the Preliminary Studies and the other for the Final Studies. In the Final Studies, there is also a function for calculating the positive socioenvironmental impact indexes.

There are two more functions for selecting the best cascade option. One is used in the Preliminary Studies (***Preliminary Multiobjective Analysis***) and the other in the Final Studies (***Final Multiobjective Analysis***). Both functions are based on multiobjective analysis.

ANNEX E

Technical Specifications of Projects - Inventory Studies

Technical Specifications of Projects – Inventory Studies
1. IDENTIFICATION:
Name of project
River
Distance to mouth
Drainage area of project
Drainage area of river basin
urban infrastructure: [ ] good [ ] average [ ] poor
Basin
Sub-basin
latitude
longitude
State(s)
Municipality(ies)
Codename
Status in cascade option: [ ] included or [ ] excluded
Name of next plant downstream
Type of System
2. BASIC DATA:
2.1. Topography:
2.1.1. Geographical maps available:
Entity
Name
Number
Scale
Date
2.1.2. Remote sensing data available:
2.1.2.1. Aerial photographs:
Entity
Performed by
Scale
Date
Section
Service
Photos
2.1.2.2. Multispectral Images:
Entity
Performed by
Scale
Date
Service

Technical Specifications of Projects – Inventory Studies
2.1.2.3. Radar Images:
Entity
Performed by
Scale
Date
Service
2.1.3. Aerial photogrammetric maps available:
Entity
Performed by
Scale
Date
Contract
2.1.4. Topographic maps available:
Entity
Performed by
Scale
Date
Contract
2.1.5. Other topographic services available (polygonal, sections, leveling, etc.):
Entity
Performed by
Scale
Date
Contract
2.2. Geology:
2.2.1. Reservoir:
Are there any rocks or geological features that could compromise the watertightness of the reservoir? [ ] yes [ ] no
Describe in brief:
Are there slopes or rocks that could compromise the stability of the reservoir's banks? [ ] yes [ ] no
Describe in brief:
Is there any geotectonic evidence that the reservoir could be subject to natural and/or general induced seismic activity? [ ] yes [ ] no
Describe in brief:
2.2.2. Dam Axis:
Mean estimated thickness of soil cover:
On the river bed:
On the right bank of the river:
On the left bank of the river:

Technical Specifications of Projects – Inventory Studies
On the right abutment at the height of the crest:
On the left abutment at the height of the crest:
Type of rock
Are there any geological features that could jeopardise the building of this kind of construction? [ ] yes [ ] no
Describe in brief:
2.2.3. Natural building materials, availability of:
Clay: [ ] yes [ ] no
distance to borrow areas on the right-hand bank
distance to borrow areas on the left-hand bank
Sand and gravel: [ ] yes [ ] no
distance to deposits
Rock: [ ] yes [ ] no
distance to quarries on the right-hand bank
distance to quarries on the left-hand bank
2.3. Hydrometeorology:
Climate classification:
2.3.1. Temperatures:
maximum:
minimum:
mean monthly:
hottest three months:
coldest three months:
2.3.2. Net evaporation:
Net evaporation
2.3.3. Precipitation:
Gauging stations used:
Code
Name
lattitude
longitude
elevation
period of observation
drainage area
operating entity
months with most rainfall
months with least rainfall
Mean monthly precipitation:
2.3.4. Streamflow:
Gauging stations used:
Code
Name
lattitude
longitude
elevation
period of observation
drainage area

Technical Specifications of Projects – Inventory Studies
opeating entity
river
2.3.5. Flows and water levels:
Long-term mean flow
Period of long-term mean flow
Specific long-term streamflow
Max. mean monthly streamflow
Month of max. mean monthly streamflow
Min. mean monthly streamflow
Month of min. mean monthly streamflow
Max. observed daily streamflow
10,000-year flood
Min. outflow: (defined by environmental and/or operating constraints)
Series of natural mean monthly streamflows
Observed water level for max. mean streamflow
Staff gauge zero
Min. mean monthly streamflow
Date of min. observed streamflow
Observed water level for min. mean streamflow
Staff gauge zero
Date of 10,000-year flood
Water level for 10,000-year flood
Staff gauge zero
2.3.6. Reservoir:
Maximum normal water level (NAmxn)
Minimum normal water level (NAmin)
Mean water level (NAm)
Normal downstream water level
Total reservoir volume
Live storage
Sum of live storage capacities upstream
Maximum drawdown
Volume corresponding to crest of the spillway sill
Water level corresponding to half the live storage
Volume at minimum normal water level
Elevation of spillway sill (m)
Area flooded for Namax
Area flooded for Namin
Evaporation losses
Loss due to other uses of the waters
Net regulated flow
Gross regulated flow
Residence time (days)
3. ENERGY PARAMETERS:
Maximum gross head (Hb1)
Maximum net head (H1)

Technical Specifications of Projects – Inventory Studies
Mean net head (H2)
Minimum net head (Hb1)
Mean flow during critical period (Qr)
Firm energy (Ef) [Mwmed]
Reference capacity factor (Fk)
Reference capacity (Pr)
Installed capacity (P)
Reference head (m)
Hydraulic losses (m)
Mean tailrace canal water level (m)
Normal downstream water level (m)
Points and volume/area, elevation/area and streamflow/downstream water level curves (incl. notes about any peculiarities).
4. LAND, RESETTLEMENTS, RELOCATIONS AND OTHER SOCIOENVIRONMENTAL ACTIONS:
4.1. Urban land and land developments affected:
District
Municipality
State
Total population
Population affected
Population affected (%)
Urban infrastructure: [ ] good [ ] average [ ] poor
Average standard of buildings: [ ] good [ ] average [ ] poor
4.2. Rural land and land developments affected:
Municipality
State
Total area
Area affected
Area affected (%)
Total population
Population affected
Population affected (%)
crops
pasture
fields
forests
4.3. Indigenous communities and/or other ethnic groups affected:
Name
Municipality
State
Total population
Population affected
Population affected (%)
Total area
Area affected
Area affected (%)

Technical Specifications of Projects – Inventory Studies
4.4. Conservation Units and permanent preservation areas affected:
Name
Municipality
State
Total area
Area affected
Area affected (%)
4.5. Other developments affected:
4.6. Resettlements and relocations:
4.6.1. Roads:
paved federally-owned roads
unpaved federally-owned roads
paved state-owned roads
unpaved state-owned roads
paved municipally-owned roads
unpaved municipally-owned roads
4.6.2. Railroads:
gauge
length
4.6.3. Bridges:
type
length
4.6.4. Transmission and distribution system:
voltage
type of tower
length
4.6.5. Communications system:
4.6.6. Population:
urban
rural
indigenous communities and/or other ethnic groups affected
4.6.7. Other:
airport
river port
other
4.7. Other socioenvironmental actions:
4.7.1. Reservoir cleaning:
area corresponding to drawdown
total area
area to be cleared
area to be cleared (%)
type of vegetation
4.7.2. Conservation areas and permanent preservation areas created:
name
municipality
state
total area

Technical Specifications of Projects – Inventory Studies
area purchased
area purchased (%)
5. POWERHOUSE:
type
installed capacity (P)
type of turbines
number of units (N)
capacity of each turbine (P1)
capacity of each generator (P1)
synchronous velocity (n)
diameter of rotor (D3)
output of turbine-generator set
maximum turbine discharge (Qt)
reference date for costs
currency (at the time)
payment code
currency used for costs and payment schedule
total cost of construction
payment schedule
6. OPERATORS' VILLAGE:
expected population
location
7. RIVER DIVERSION AND CONTROL:
diversion flow
recurrence time
type of scheme: [ ] through tunnels [ ] through sluiceways [ ] through galleries [ ] through a canal
7.1. Tunnels:
number of tunnels
location
Exclusively for diversion? [ ] yes [ ] no
form of section
diameter
length
maximum flow per tunnel
maximum velocity
7.2. Galleries:
number of galleries
location
height
width
maximum flow per gallery
maximum velocity
7.3. Sluiceways:
number of sluiceways
location
height

Technical Specifications of Projects – Inventory Studies
width
maximum flow per sluiceway
maximum velocity
7.4. Canal:
location
depth
width
length
maximum flow
maximum velocity
8. DAMS AND DIKES:
type
maximum height
length
mean height
volume
9. TRANSITION AND RETAINING WALLS:
type
maximum height
length
mean height
volume
10. SPILLWAY:
type
design flood
recurrence time
maximum height
length
mean height
volume
number of gates
type of gates
width of gates
height of gates
11. INTAKE STRUCTURE:
11.1. Water Intake:
type
maximum height
length
mean height
volume
number of intakes
maximum discharge per intake
number of gates
type of gate
width of gates
height of gates

Technical Specifications of Projects – Inventory Studies
11.2. Low-Pressure Intake Conduit:
type: [ ] canal [ ] tunnel
number of tunnels
length
velocity
cross-section
maximum flow per conduit
11.3. Surge Tank:
type
diameter
height
11.4. Pressure Tunnel:
flow through tunnel
maximum velocity
diameter
length
length of lined section
volume of excavation in rock
length of unlined section
volume of excavation in soil
11.5. Pressure Penstock:
type
number of penstocks
mean unit length
diameter
flow through penstock
maximum velocity

Technical Specifications of Projects – Inventory Studies
11.6. Tailrace Canal:
flow
maximum velocity
volume of common excavation
length
volume of excavation in rock
depth
width
11.7. Tailrace Tunnel:
flow
maximum velocity
volume of common excavation
length
volume of excavation in rock
12. ACCESS TO CONSTRUCTION SITE: 12.1.Roads:
type
length
12.2.Railroads:
type
length
12.3.Bridges:
type
length
12.4.Airport:
type
length

ANNEX F

Integrated environmental assessment: example of methodological procedure

This annex contains a summary of the methodology used for an IEA, as a suggestion. This methodology was used successfully in three IEAs conducted by EPE/Sondotécnica (2007)[1].

Characterization of the study area

The aim of this stage is to build up a current scenario of the river basin, seeking to identify the areas with the greatest environmental sensitivity to human intervention, and attempting to foresee the main impacts associated with the building of the hydropower plants. If it is to meet these goals, the characterization must be designed to:

- identify the most significant social, environmental and economic features of the basin;
- select suitable indicators for each topic to be assessed so as to build up a general, comprehensive socioenvironmental characterization of the study area;
- obtain the necessary information by consulting databases, environmental inventory studies and the scientific literature, as well as carrying out field surveys and interviews, etc.;
- organize the above information in thematic maps, using a geographical information system (GIS), basing its use on the later stages by superimposing maps and undertaking multicriteria analyses, so that the information thus mapped out can be used in the environmental sensitivity analysis;
- analyze the spatial distribution of the information in such a way that subunits of analysis can be identified, especially for wider, more complex regions, which will make it easier to undertake the theme-based analysis or integrate the different topics, and thus to identify the areas of sensitivity;
- describe the socioenvironmental context of the river basin from a macro-regional viewpoint, including an initial identification of the main conflicts encountered and the most significant aspects to be taken into account when selecting the variables and formulating the indicators to represent the environmental sensitivity of the major topics under study in the ambit of the environmental system and its synthesis components.

The environmental characterization should seek to identify the topics that are most pertinent for assessing environmental impacts and for the study of prospective scenarios, taking into account not just the socioenvironmental aspects relative to the area under study, but whenever relevant their ramifications in the macro-regional processes with which the projects under study are most closely related.

Likewise, the conflicts identified at this stage will mainly be focused on the regional scale (the whole river basin), but can also be local or on a smaller scale if they are important in the decision-taking process, in view of national, regional or local interests.

1 Alongside these IEAs, there are others available for consultation on the EPE website which use methodologies that are slightly different from these (http://epe.gov.br/Lists/MeioAmbiente/MeioAmbiente.aspx).

At this stage, then, the following should be identified:

- the most significant socioenvironmental issues in the current situation (existing socioenvironmental processes);
- indications of the future trends of the socioenvironmental issues of relevance and their spatial distributon;
- the social agents involved in each of the socioenvironmental issues and significant conflicts;
- the spatial subunits to be analyzed;
- the socioenvironmental sensitivity indicators; and
- the mapping of these indicators such that the areas of sensitivity in the river basin can be identified.

Technical Support

The GIS (geographical information system) architecture, which provides technical support for the methodology, should be developed as of the start of the characterization process, making it possible to systematize and analyze the whole set of data and information so that the aspects of relevance can be identified, serving as a basis for formulating the sensitivity indicators. The main stages in structuring the GIS are: (i) composition of planimetric and altimetric databases and thematic maps for the characterization; (ii) image-based mapping and adjustments; and (iii) integration of statistical information with the maps.

Distributed Environmental Assessment

Sensitivity Indicators

Configuration of Environmental Sensitivity: based on the key aspects identified in stage 1 (diagnosis/characterization), the aim is to formulate a set of environmental sensitivity indicators (ESIs) based on variables that represent the natural conditions and current state of conservation or degradation of the natural resources in the region.

These sensitivity indicators are then organized into an indicator matrix, which provides the structure for including and ranking the variables. This matrix consists of a set of progressive assessments that result in a table of references for interpreting the information produced in the environmental studies, using a system of comparative assessments that is guided by ranking and weighting criteria. The Indicator Matrix is one of the tools used in the assessment made for the purpose of producing maps and diagrams to represent the environmental conditions in the river basin.

Having obtained these initial definitions, multidisciplinary meetings should be held to weight the variables and then reclassify them on the thematic maps from the GIS database. This means that the resulting environmental sensitivity indicators can be expressed in map form, where the degrees of sensitivity can be visualized, based on the previously established parameters.

The flow chart in Figure 1 shows the main steps in assessing environmental sensitivity indicators (ESIs).

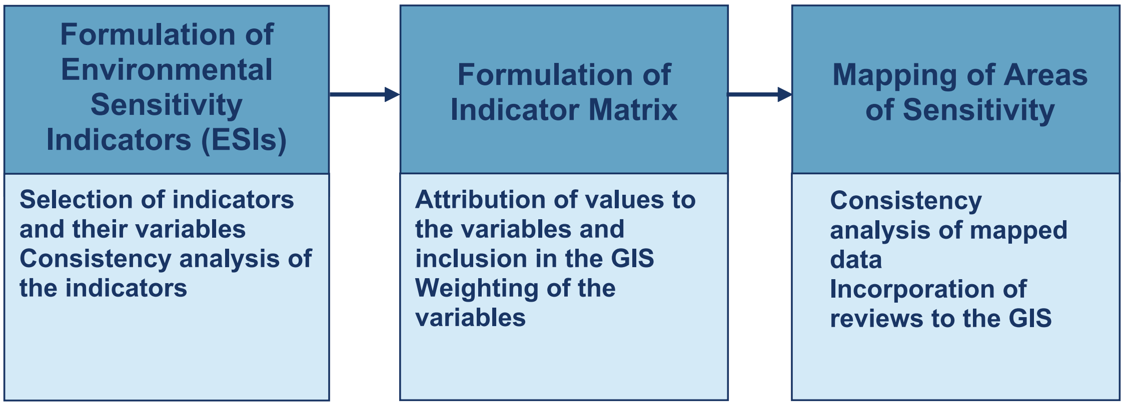


Figure 1 – Flow chart showing the activities involved in assessing environmental sensitivity indicators

Below, a step-by-step description is given of the activities involved in each of the main stages of assessing environmental sensitivity indicators.

Selecting the Environmental Sensitivity Indicators (ESI's)

The aim of the first activity of this phase is to consolidate the set of ESI's included in the "most significant aspects" item of the characterization report for the IEA. However, the concepts of magnitude, scale and scope used in these indicators, which are specific to each topic and sub-topic, present some inconsistencies. As such, they have to be adjusted before they can be included in the system of analysis.

In order to do so, the selection of ESI's is supplemented by a consistency assessment, addressing the following points:

- the need to minimize the amount of overlapping environmental information;
- an assurance of the maximal objectivity possible in the mapping activities;
- observance of the availability of information and assessment of the consistency of indicators in terms of their representativeness;
- observance of the main interfaces with the overall objective of the work (to assess the cumulative impacts of the hydroelectric projects in the river basin);
- assessment of the possibility of representing the information spatially and of exceeding the time frame.

The final consolidation of the ESI's begins by listing them, as they are presented in the characterization, and then reducing them to the smallest possible number. Many can be grouped together and some can be excluded as a result of the assessment undertaken.

Formulating the Indicator Matrix

Not only does the characterization provide an understanding of the general socioenvironmental issues in the river basin, but it is needed for thematic databases to be organized in a GIS environment, which provide more detailed, structured and spatially-defined information about the environmental aspects under study. This set of information serves as a basis for selecting the variables to be used in formulating the ESI's.

The Matrix is first built up by selecting the variables that make up each ESI, taking note of certain aspects, such as what kind of variable they are and what weight they have in the indicator.

Figure 2 shows a broad diagram of the Matrix of ESI's, explicating how the variables should be included in the formulation of each indicator, as well as the main steps for assessing them by attributing different weights and degrees of sensitivity.

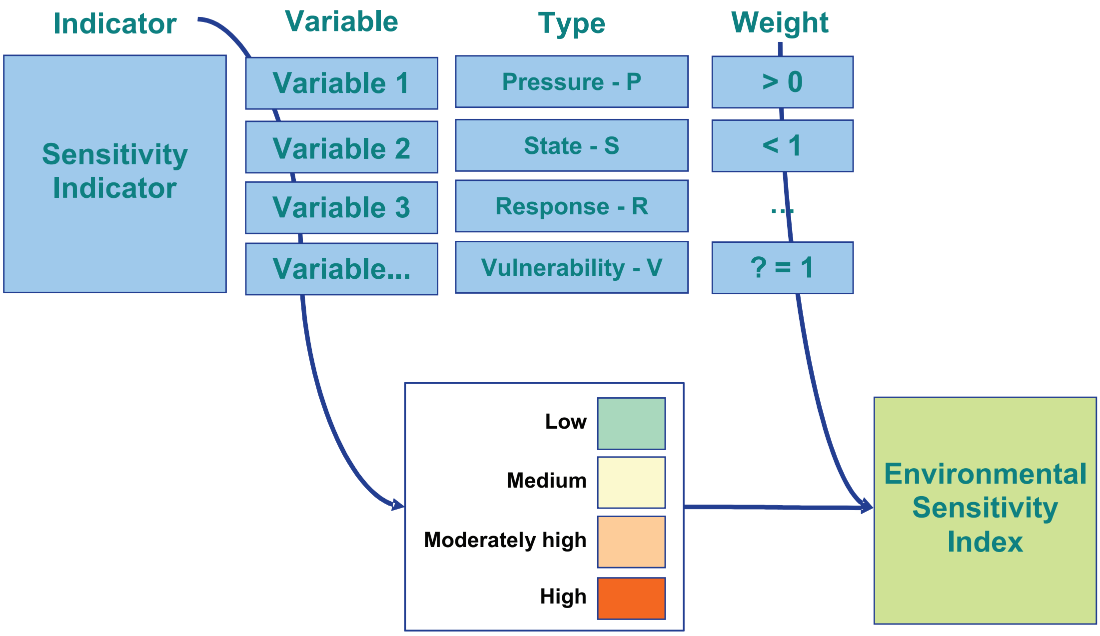


Figure 2 – Components of the Matrix of Environmental Sensitivity Indicators

Defining Types of Variables

The development of the environmental variables should be based on the OECD (Organisation for Economic Cooperation and Development) methodology, providing control and environmental monitoring mechanisms for its member countries. The OECD system is based on environmental indicators that assess the integrity of, pressure on and society's interest in preserving the main natural resources used by man. It is founded on the pressure-state-response (PSR) model (explained below), and is used to choose the most suitable variables to be monitored and assessed when diagnosing the environmental conditions in different environments and ecosystems.

- **Indicators of pressure (P)** serve not only to identify the main natural resources used by society, but also to indicate the state of preservation and degradation of the natural habitats, while also, whenever possible, identifying the carrying capacity of the natural resources.
- **Indicators of state (S)** are associated with the natural state of the resources and identify the quality of the natural habitats. As such, they involve issues such as biological diversity, size of remaining forest patches, stocks, etc.
- **Indicators of response (R)** identify mechanisms created by society for monitoring, controlling and recuperating given resources and also, whenever possible, the efficiency of these mechanisms.

Alongside these indicators, the use of another indicator is also recommended, which is designed to identify special conditions of vulnerability in given habitats. This classification is included so that the variables can be ranked objectively, and is the main element used in classifying the weights for the variables.

- P – Pressure: levels of conservation or degradation;
- S – State: quality of natural habitats;
- R – Social Response: control mechanisms;
- V – Vulnerability: special conditions, such as indigenous areas and conservation units.

Weighting the Variables

The process of weighting and ranking the variables is started by defining and identifying the kinds of variables used to make up the indicators. In order to define the index for each ESI, weights should be attributed numerically, with a view to converting the environmental sensitivity scale to a comparable scale.

The weights are attributed in such a way that they represent the importance of each variable with respect to the construction of the indicators, in the scale of sensitivity of the ES'Is, and ultimately which areas should be considered as being environmentally sensitive.

Defining Degrees of Sensitivity

In order to define the degree of sensitivity of each variable, the variation scale of the information in the database for each variable must be consulted, seeking from this scale which references could determine the ranges to be used in identifying degrees of sensitivity, as shown in Figure below.

Each variable has its own scale of values, so to select the range of values that define what sensitivity parameters to use, it is necessary always to consult the references for these parameters.

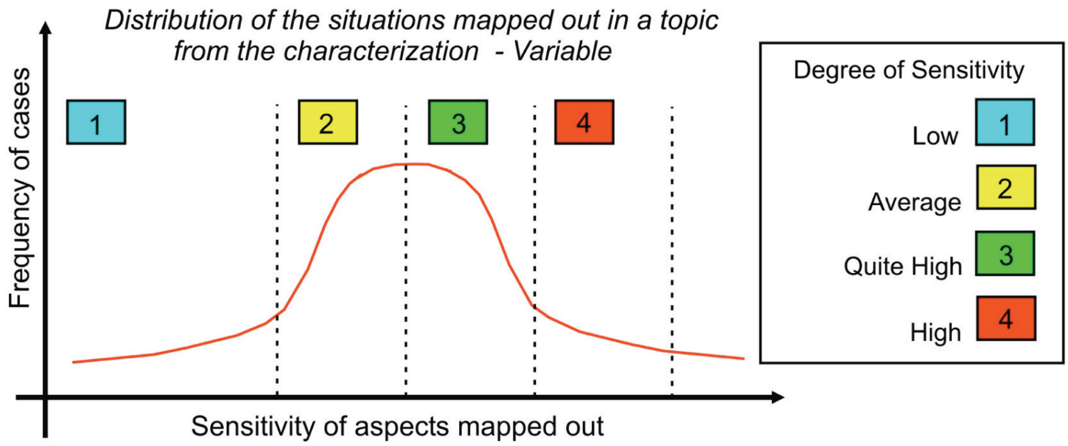


Figure 3 – Schematic representation of process for defining levels of sensitivity

The formulation of environmental sensitivity indicators offers a number of advantages that can assure the fulfillment of some of the main aims of the work. Starting with structuring a set of analysis tools, the ESI's can be used to accomplish the activities required for several of the stages of the IEA's. The main advantages of using ESI's are:

- they represent in map form the different ranges of sensitivity of the environmental aspects, using the working scales proposed, thereby preventing spatial generalizations from being made that could mask local differences and heterogeneities;
- the matrixes and maps ensure that a record is kept of the characterization information, since the map representations of the ESI's involve reclassifying the databases of thematic maps, with the information being kept accessible in the GIS throughout the assessment process;
- the areas of fragility and potentiality inside the river basin and each sub-area are identified.

Mapping out the indicators

A GIS that permits the integration of the different elements that make up the environmental sensitivity indicators should be structured from the maps created during the characterization.

The maps of the different geographical features, alongside the database containing information about the different environmental topics, provide the basis for an integration of the components from the

DEA, especially by means of the restructuring of the databases with information about the weights and degrees, which are determinant in the integration of the attributes with the GIS platform.

Having restructured the database, the thematic maps should be cross-referenced by geographically superimposing the information they contain, where the attributes that have been weighted and with values on the scales of sensitivity can be summed, defining the presentation of the indicators with the areas of sensitivity represented in the maps.

Integrating the indicators into topics

The sensitivity indicators are integrated under the topics proposed in the study (water resources and water ecosystems, physical environment and terrestrial ecosystems, and socioeconomics) by cross-referencing the sensitivity maps as per their identification. Table 1 gives an example from a particular river basin.

Table 1 – List of sensitivity indicators per integration topic

INDICATOR	INTEGRATION TOPIC
Sensitivity of water quality	Water Resources and Aquatic Ecosystems
Sensitivity to conflicts over water use	
Sensitivity of water habitats	
Geological and geotechnical sensitivity	Physical Environment and Terrestrial Ecosystems
Sensitivity to soil erosion	
Sensitivity of terrestrial ecosystems	
Sensitivity to population pressures	Socioeconomic Aspects
Sensitivity of ways of life	
Sensitivity to conflicts over land use (agriculture)	
Sensitivity of natural, archaeological, historical, and cultural heritage	
Sensitivity to alterations or destructuring of communities	
Sensitivity to curtailment of economic activity	
Sensitivity of social and land organization	

The maps of potentialities are not integrated, since they cover different topics.

The integration of the indicators into topics gives rise to three representations, based on the sum of the grouped ESI's. The representation based on the weighted sum of the ESI's can be understood as a spatial superimposition of the sensitivities.

The maps of each of the topics should be integrated according to different criteria, which will depend on the characteristics of the river basin.

The areas where several elements of high sensitivity occur are designated areas of sensitivity.

The socioeconomic indicator must be considered in the integrated representation of potentialities. The ranges of higher sensitivity to socioeconomic potentialities represent local features that could leverage positive impacts.

Impact Indicators

The main information from the environmental impact assessment undertaken as part of the Preliminary Inventory Studies should be about the main characteristics of the projects analyzed, seeking to associate and dimension the potentially impacting elements of each of them. Thus, as well as listing and selecting the set of potential impacts, values should be attributed for measuring each impact for each of the projects under study. Figure 4 shows the main stages, activities and tools used in the process of assessing potential impacts.

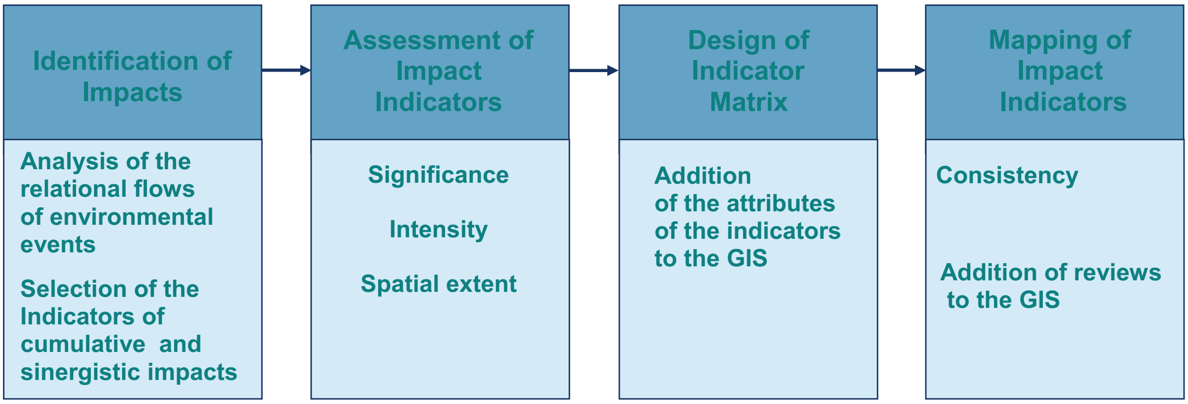


Figure 4 – Flow chart of impact analysis

The impact assessment is designed to identify which aspects from the group of impacts and projects under study could affect the intensity of manifestation of these impacts and the spatial extent of their effects. As such, the following guidelines were established:

- Differentiate the intensity according to each project: since the prospective projects in a river basin will have particular features, such as their size, the capacity of the reservoir, their regulating capacity and other aspects that differentiate them, which must be taken into account throughout the analysis;
- Rank the impacts according to their importance and significance: each impact has a differentiated state as a function of its potential to generate cumulative and synergistic effects and the nature of how it is manifested and whether it is direct or indirect, etc.;
- Establish the spatial distribution of the effects by representing them geographically in line with the spatial distribution of the resources directly associated with the impacts identified, such that the spatial interactions between the impacts can be highlighted;
- Assess the cumulative and synergistic effects between the projects, observing what additional effects could be generated by the projects in question.

Below are presented the main stages in the assessments of environmental impacts that make up the DEA.

Identification of Environmental Impacts

The impacts associated with the hydropower projects should be inferred from an association matrix, called the Environmental Flowchart (FREA – “Fluxo Relacional de Eventos Ambientais”), which represents the relationship between each phase of development and the main impacts associated.

The idea behind formulating the flowchart is to list the potential impacts that could take place while building or operating the hydropower plants.

Interaction networks are used to identify the events responsible for the most significant environmental changes. The flowchart is an inference model that provides a framework for identifying a more comprehensive set of impacts, so that a broad context can be provided to which selection criteria can be applied for determining which impacts are most significant in the process under analysis.

Selection of Environmental Impacts

Based on an initial set of impacts, analyses are carried out with the aim of grouping or eliminating them as a function of the characteristics of the impacts. Thus, the following steps should be taken in the selection process:

- identify the permanent or long-term impacts. Temporary impacts should be disregarded insofar as they are of little significance on the time scale used, which is 10 years at the least;

- identify the impacts with different spatial distribution within the river basin, since those which extrapolate the boundaries of the basin will not give rise to a comparative differentiation;
- identify the impacts that can be objectively distinguished, meaning that they can be measured on the working scale established for the assessment. In this way, impacts with a local reach which are neither cumulative nor synergistic are also discarded.

Figure 5 shows a schematic flow chart of the process for selecting impacts based on the methodology described in this section.

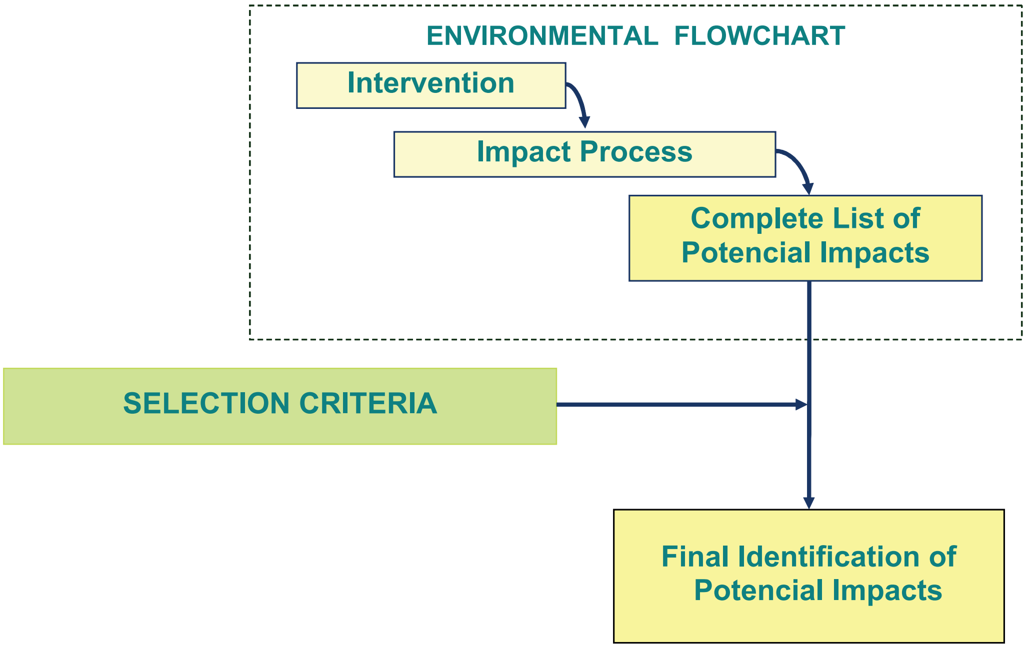


Figure 5 – Flow chart of impact selection process

Assessment of Environmental Impacts

Depending on the information from each impact, the environmental analysis to be undertaken consists of an assessment of the significance, spatial extent and intensity of the environmental impacts related to each existing project.

The significance of a given impact is taken as being the value that expresses the manifestation of this impact on the environment. In order to define the significance of an environmental impact, its magnitude, intensity and spatial extent are taken into account; the assessment is based on the attributes of the impacts and the perceptions and experience of the professionals in the multidisciplinary team.

The magnitude of an impact is given by assessing its attributes that induce large or small and fast or slow changes to the quality of the environment in the area where they are manifested. Thus, the methodology includes an objective analysis of three attributes: whether the impact is direct or indirect; whether it is local or regional; and whether it is medium- or long-term.

The same assessment also seeks to differentiate between the impacts per project, investigating their basic criteria and thereby giving a clearer picture of the relative contribution of each of the projects to the interaction between the impacts. Next, the main characteristics used to assess the impact indicators are described, as well as how the degrees of sensitivity were established based on the set of plants in the river basin.

Operating Regime – differentiates plants with regulation reservoirs from run-of-river plants.

Residence Time – the mean retention time of water in the reservoir.

In-Stream Flow – the mean flow released downstream after regulation.

Regulating Capacity – measures retention efficiency as a percentage.

Capacity – the installed capacity of the turbines.

Flood Control – the capacity to contain the natural cyclical floods in the river.

Reservoir Area – describes the area covered by the water surface.

Section with Reduced Flow – identifies and measures the distance over which flow is reduced because of the use of a diversion channel or other similar scheme.

Dam Height – describes the head at the dam.

Preparation of indicator matrix of synergistic and cumulative effects

The indicator matrix is a set of interrelated assessment tables used to transfer the data from the different phases of assessment to the maps. It is built up from a set of information about and the main characteristics of the existing and planned plants in the river basin, and includes an assessment of significance and spatial extent, and the selection of the kind of intensity indicator to be used for each of the impacts. A flow chart of the components in the matrix is shown in Figure 6.

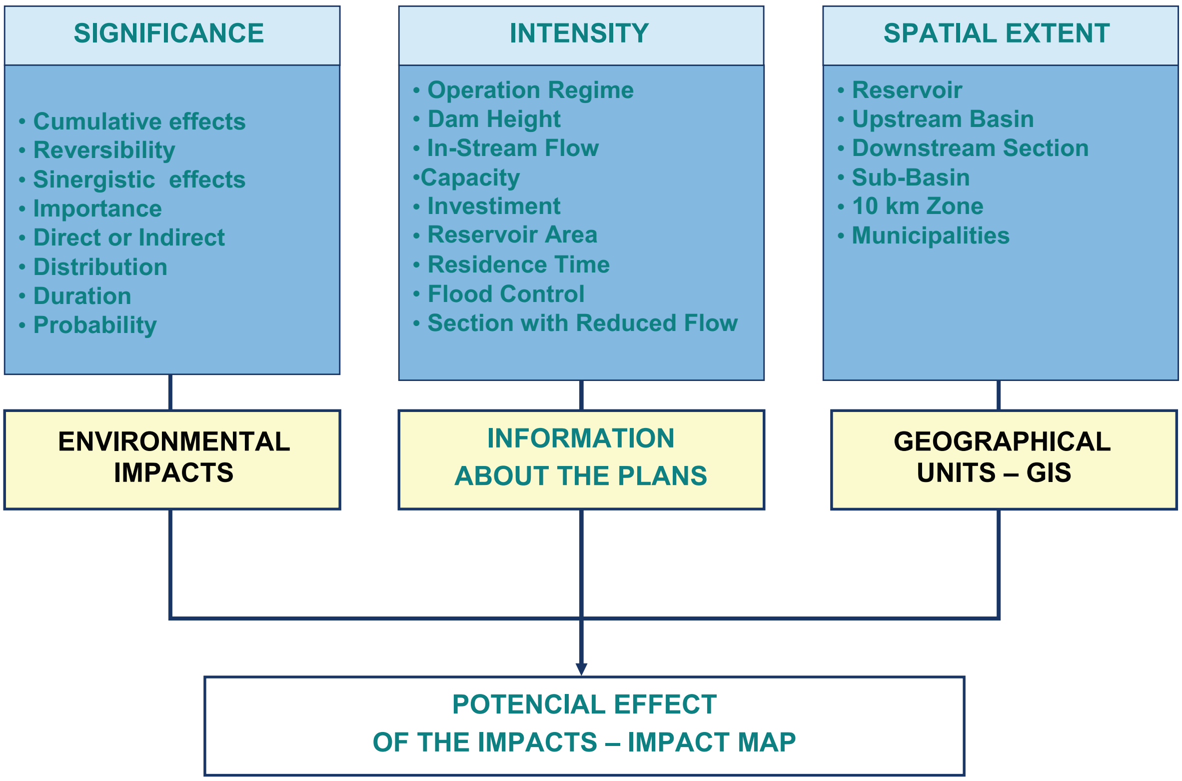


Figure 6 – Diagram of the flow of information in the Impact Matrix

Based on these definitions, the values obtained for each impact are grouped together according to the spatial extent of their effects, so that they can be included in the other maps formulated as part of the DEA.

Mapping Impact Indicators

The impacts are represented spatially by identifying the geographical elements that could best represent the natural resources directly involved in each environmental impact. The concept of “spatial extent” was developed for this representation.

The spatial extent of each impact is a representation of the extent to which the effects of each impact are felt in space, so that the impact indicator can be included in the GIS, as can its analysis integrated with its sensitivity.

First, the possible "spatial extent" of the impacts must be established, albeit conservatively, meaning that the worst-case scenarios should be addressed.

Reservoir: the area of the reservoir or the land to be occupied by it;

Upstream Basin: the whole hydrographic network that contributes to the formation of the reservoir;

Downstream Section: a 10 km section around the section downstream from the dam until the backwaters in the reservoir from the next project downstream;

Sub-Basin: the whole sub-basin where the project is built or is to be built;

10 km buffer around the reservoir: a 10 km area around the reservoir, adopting the same principle that defines the buffer zones around conservation areas as defined in the Brazilian System of Conservation Areas (SNUC);

Municipality: municipalities under the direct influence of each project, meaning those whose land is partially flooded to form the reservoir.

The analysis of the environmental impacts of the existing hydropower plants should be prepared using the methodology described, seeking to assess how these impacts would change from the time they were first manifested until the present.

Analysis of cumulative and synergistic effects

The assessment of cumulative and synergistic effects should be carried out by crossing the data from the environmental sensitivity mapping and the mapping of impact indicators. The geographical distribution of the two indicators will give a clear picture of the main relationships between the different impacts analyzed in the study, especially identifying the areas where there is a greater overlapping of these effects.

Two items should be checked in this spatial representation:

- **concerning cumulative effects**: defined as effects that are combined by overlapping spatially;
- **concerning synergistic effects**: obtained by constructing and assessing the interrelations between the impacts; quantified by the number of synergy correlations between impacts obtained from a matrix whose lines and columns represent the impacts identified from experience of the projects already built. The greater the number of synergies in the matrix, the greater the synergistic effect of each impact.

To undertake these analyses, some methods should be selected, such as:

a) geographical information system (GIS);

b) analysis networks (of diagrams);

c) biological/geographical analysis;

d) analysis of assimilation capacity (limits and constraints).

Integrated Environmental Assessment

Prospective Scenarios

The current scenario taken from the socioenvironmental characterization of the river basin and represented by the areas of sensitivity is the reference scenario against which the prospective scenarios should be built up, with and without the projects from the best cascade. During the IEA, these scenarios project the socioeconomic development trends and prospects for environmental degradation/

conservation, crossing them against the prospects for expansion of energy supply thanks to the building of the planned hydropower plants and those already being built in the river basin.

The prospective scenarios are built up in seven stages:

- modeling for the formulation of reference socioeconomic scenarios using system dynamics models, such as STELLA (High Performance Systems, 1997);
- survey of the main scenarios for electricity generation;
- formulation of system for assessing synergistic and cumulative effects;
- comparison of areas of fragility and potentiality in the reference socioeconomic scenarios with the main synergistic and cumulative effects projected for the groups of hydropower projects planned for the river basin;
- analysis of fragility and potentiality maps created for the prospective scenario with the projects built, resulting from crossing the fragilities and potentialities with the main synergistic and cumulative effects.

Guidelines and Recommendations

Socioenvironmental guidelines and recommendations for each sub-area of the river basin should be drawn up for the electricity industry and other public and private entities operating in the basin, taking into consideration the use of the water resources and the land, the conservation and sustainable use of the natural resources, and the sustainability indicators and criteria.

ANNEX G

Format of the File Showing the Monthly Flows of the Projects

Name of project

YEAR	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec
1931												
1932												

Observation:

A spreadsheet should be prepared for each project using the above format. The number of lines should be the same as the number of years available in the mean monthly flow series of the project.

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p.33: Construction work for Simplício hydropower plant (Marcos Pinto – Furnas archive)
p.61: Tailrace canal area, Simplício hydropower plant, powerhouse (Marcos Pinto – Furnas archive)
p.79: Itaipu power plant (Eletrobrás archive)
p.187: Porto Colômbia power plant (Eletrobrás archive)
p.595: Furnas power plant: aerial view with spillway open (Roberto Rasa – Furnas archive)
p. 607: Marimbondo power plant (Eletrobras archive)
p.619: Tucuruí power plant (Eletrobras archive)
p.659: Luiz Carlos Barreto de Carvalho power plant with spillway open (Aluísio de Souza – Furnas archive)

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