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

Sub-Areas / Projects	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.

$$Indprel = \frac{\sum Emp_{sub}}{\sum PEA_{sub}} \times 100 \qquad (5.4.2.01)$$

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.

$$IPMT=\sum peso_{sub} \times Ind_{sub} \tag{5.4.2.02}$$

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:

$$GT(k)=\sum_{i \in J(k)} g(i,k) \tag{5.4.2.03}$$

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:

$$CFA(k) = 0.06 \times 0.45 \times GT(k) \times TAR \quad (5.4.2.04)$$

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:

$$PMA(m,k) = \frac{AMA(m,k)}{A(k)} \quad (5.4.2.05)$$

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:

$$CFMA\ (m,k) = CFA(k) \times PMA(m,k) \quad (5.4.2.06)$$

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:

$$CFM(m) = \sum_{j \in AP} CFMA\ (m,k) \quad (5.4.2.07)$$

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.

$$CFS(s) = \sum_{i \in MS(s)} CFM(i) \qquad (5.4.2.08)$$

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:

$$ISS(m,i) = CTS(i) \times p(m,i) \times AL(m) \qquad (5.4.2.09)$$

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.

$$ISSV(m,i) = \frac{ISS(m,i) \times \left[(1+a)^t \times a\right]}{(1+a)^t - 1} \qquad (5.4.2.10)$$

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:

$$I(s) = \text{Min}\left\{1, \frac{BT(s)}{R(s)} \times \frac{1}{VS}\right\} \tag{5.4.2.15}$$

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.

$$\sum_j (Ia_j \times p_j) = (Ia_{irr} \times p_{irr}) + (Ia_{cc} \times p_{cc}) + \\ + (Ia_{aq} \times p_{aq}) + (Ia_{nf} \times p_{nf}) + (Ia_{as} \times p_{as}) + (Ia_{tr} \times p_{tr}) \qquad (5.4.2.17)$$

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:

$$Ia_{irr}^{x,y} = \frac{\sum_{n}\left(Airr_{n}^{x,y}\right)}{\sum_{m}\left(Airrplan_{m}^{x}\right)} \tag{5.4.2.18}$$

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.

$$Ia_{as}^{x,y} = \frac{\sum_{n}\left(Nabs_{n}^{x,y}\right)}{\sum_{m}\left(Nabsplan_{m}^{x}\right)} \qquad (5.4.2.21)$$

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.

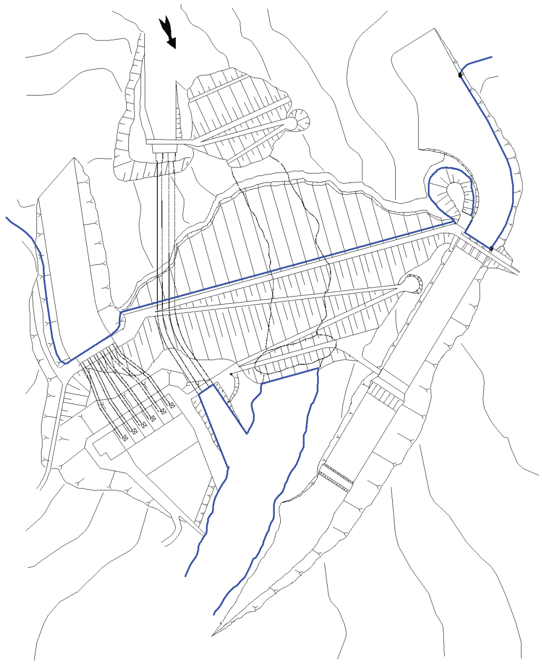


Fig. 5.5.1.01 – Typical layout for a medium-wide river valley (Gov. Bento Munhoz da Rocha Neto hydropower plant – Foz do Areia, Iguaçu River, South of Brazil).

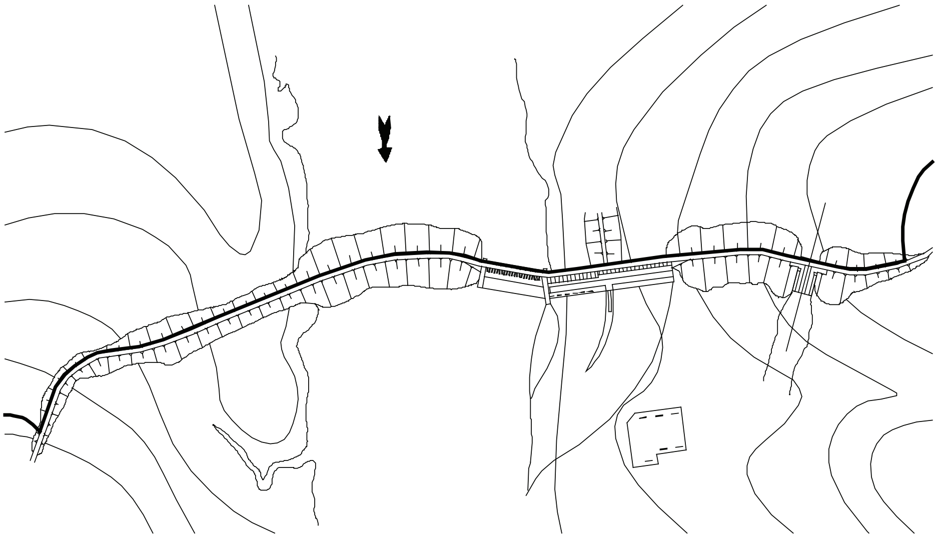


Fig. 5.5.1.02 – Typical layout for a very wide river valley (Tucuruí hydropower plant, Tocantins River, Noth of Brazil).

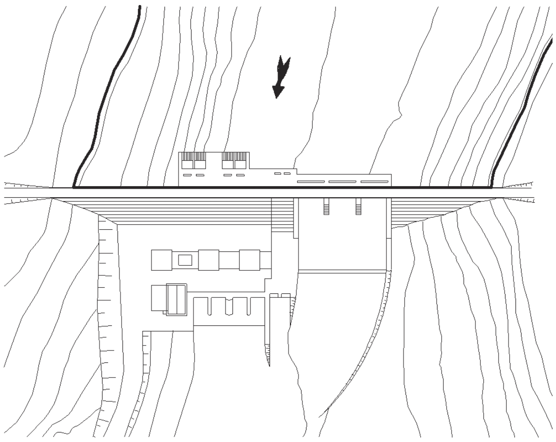


Fig. 5.5.1.03 – Typical layout for a narrow river valley.

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.

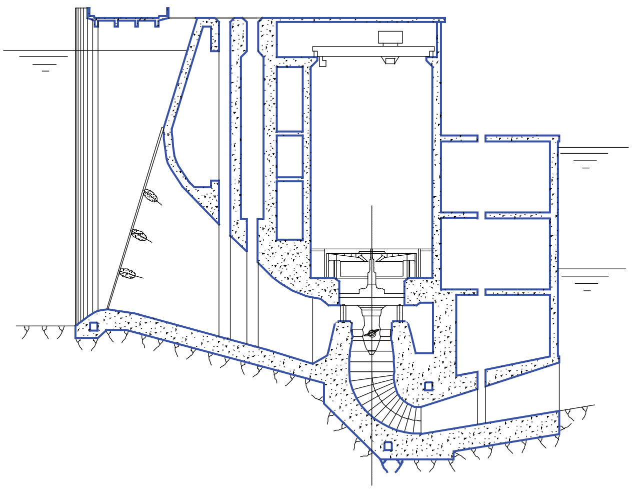


Fig. 5.5.2.01 – Project with integral intake powerhouse (Esperança hydropower plant).

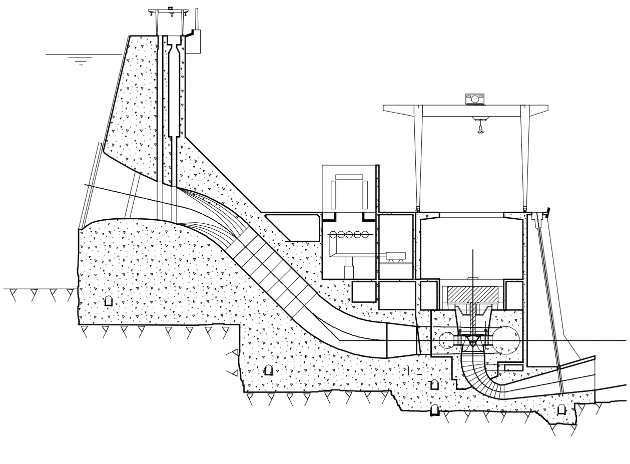


Fig. 5.5.2.02 – Compact project with pressure penstocks (Água Vermelha hydropower plant).

- 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.

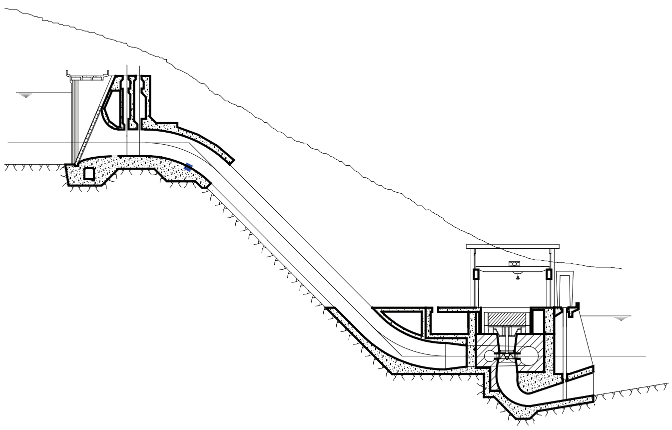


Fig. 5.5.2.03 – Project with river diversion through an open channel (Erveira hydropower plant).

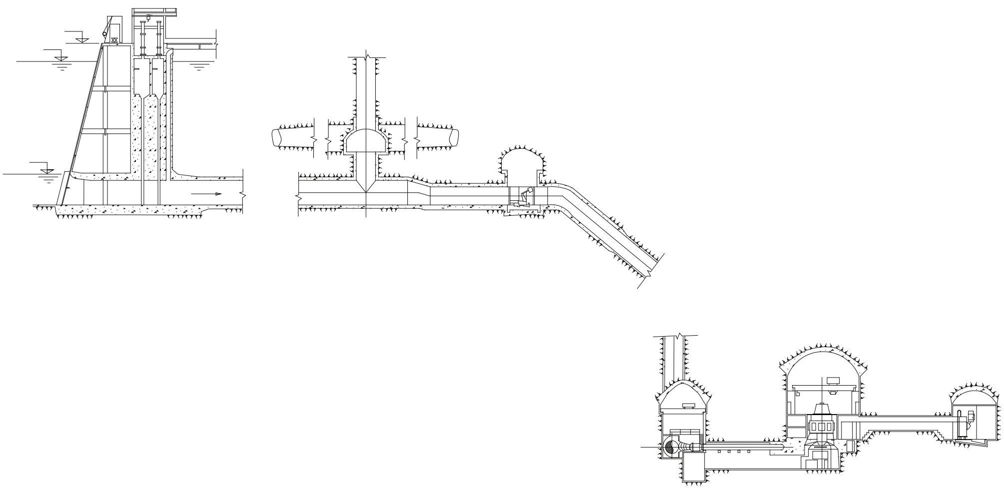


Fig. 5.5.2.04 – Project with diversion through a penstock (Capivari Cachoeira hydropower plant).

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.

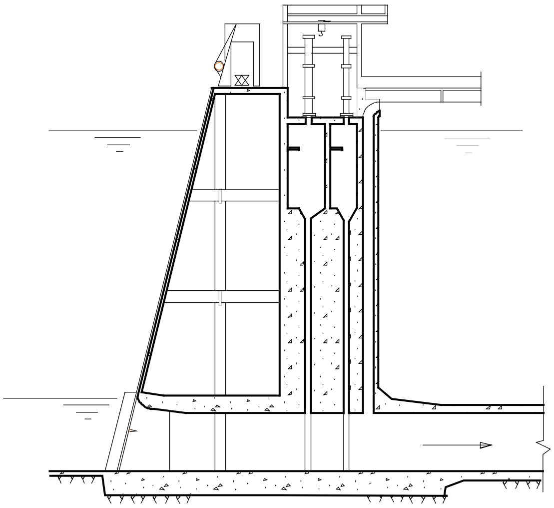


Fig. 5.5.2.05 – Intake Tower.

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.

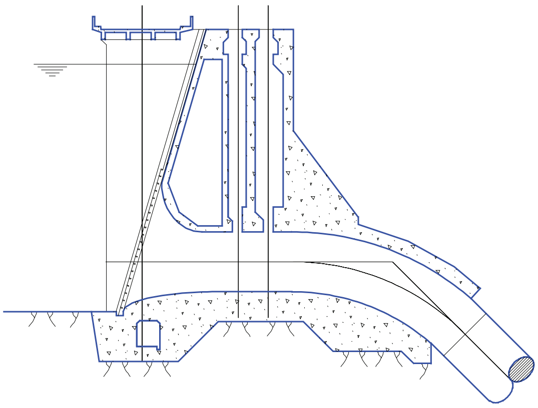


Fig. 5.5.2.06 – Gravity intake with surface penstocks.

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.

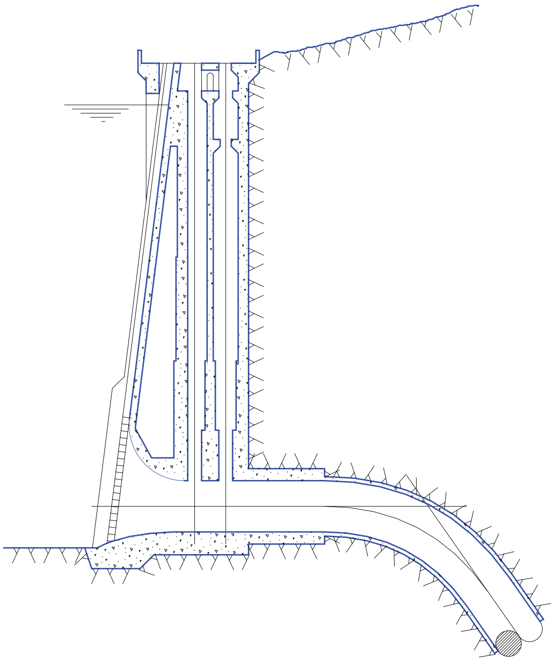


Fig. 5.5.2.07 – Reduced Gravity Intake.

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.

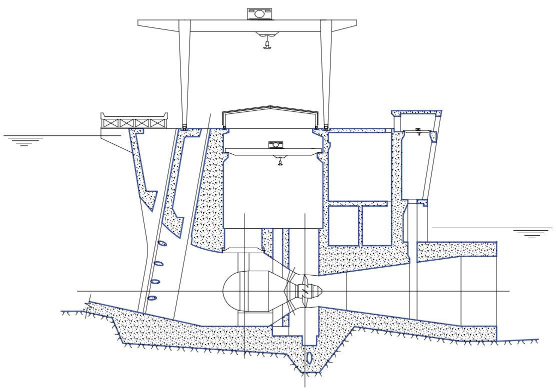


Fig. 5.5.2.08 – Integral Intake Powerhouse.

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.

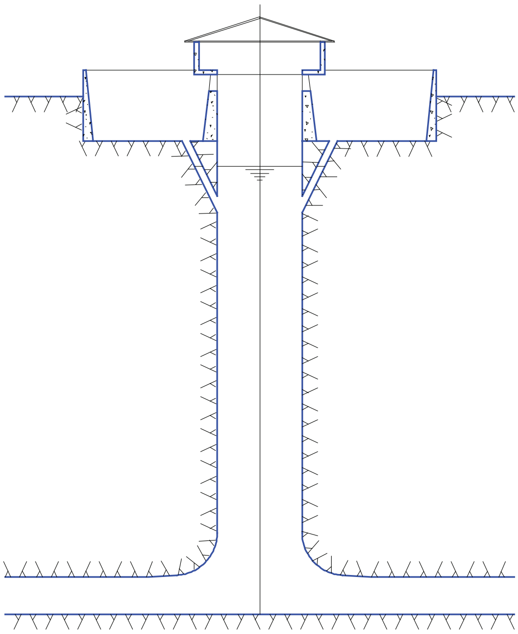


Fig. 5.5.2.09 – Surge Tank.

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).

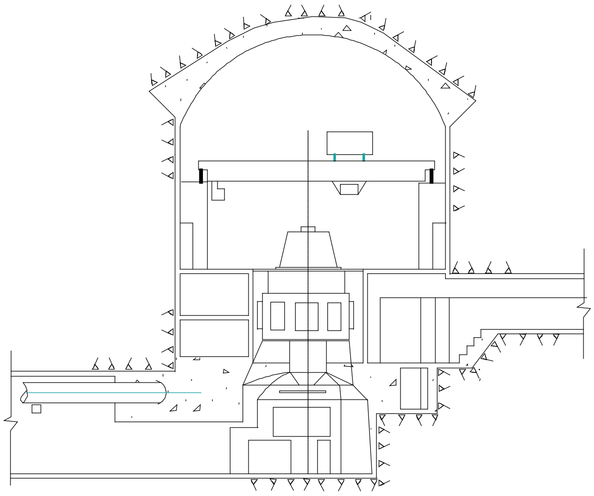


Fig. 5.5.2.10 – Underground Powerhouse – typical cross-section.

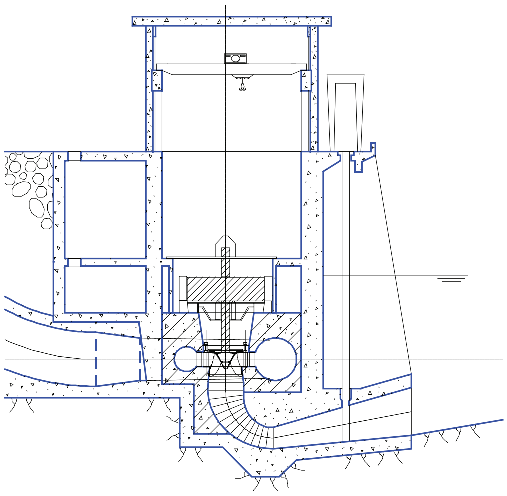


Fig. 5.5.2.11 – Indoor Powerhouse – typical cross-section.

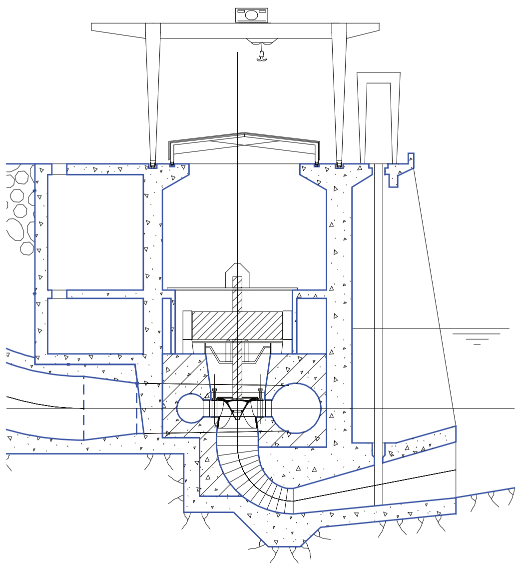


Fig. 5.5.2.12 – Semi-Outdoor Powerhouse – typical cross-section.

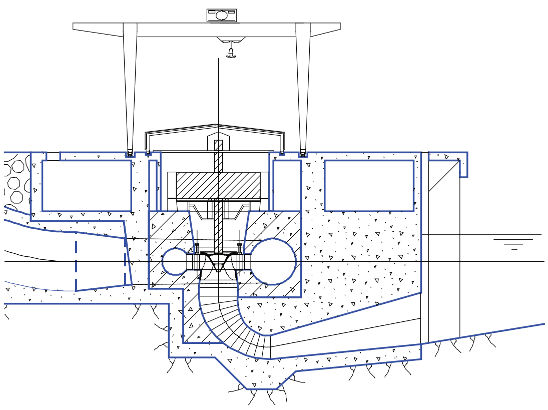


Fig. 5.5.2.13 – Outdoor Powerhouse – typical cross-section.

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).

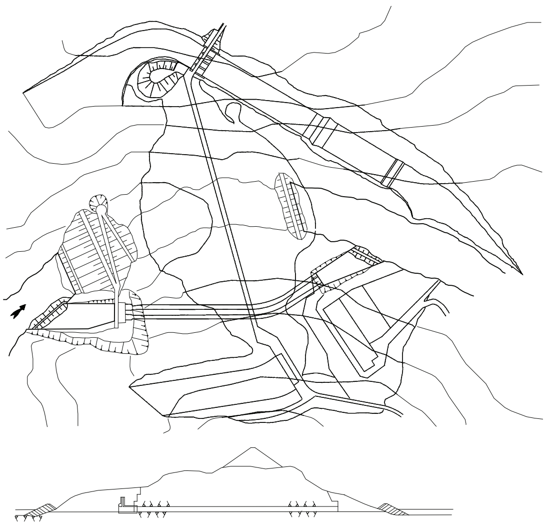


Fig. 5.5.3.01 – River diversion through tunnels in an abutment – plan and cross section.

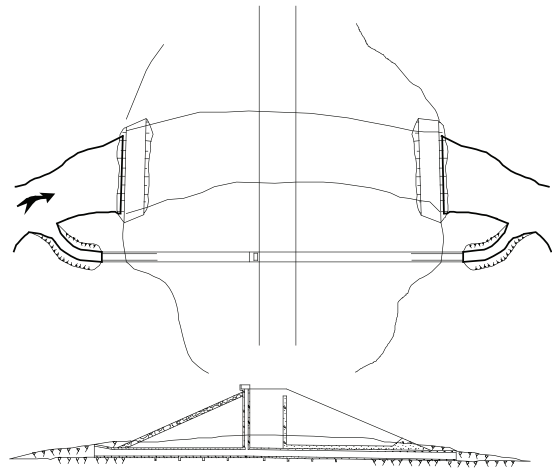


Fig. 5.5.3.02 – River diversion through a gallery under the dam – plan and cross section.

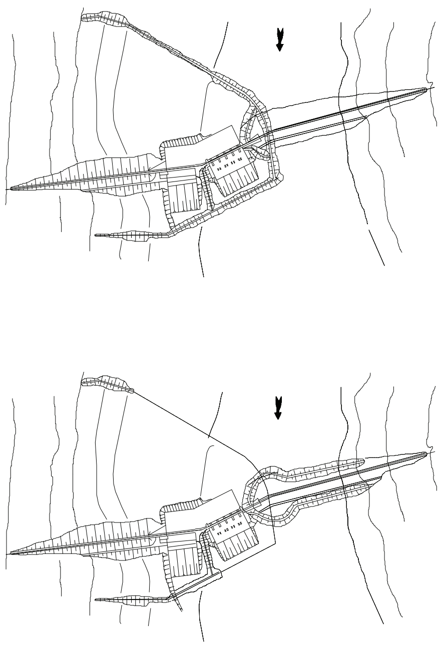


Fig. 5.5.3.03 River diversion through sluiceways.

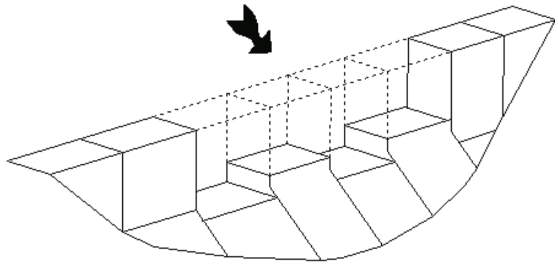


Fig. 5.5.3.04 – River diversion over the top of alternate lowered blocks in a concrete dam.

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.

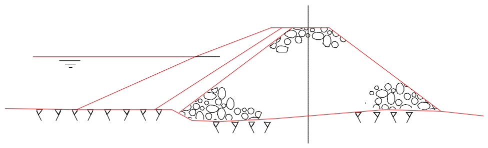


Fig. 5.5.3.05 Cofferdam across a river.

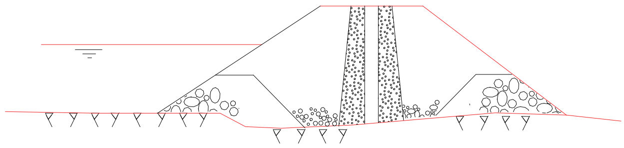


Fig. 5.5.3.06 – Cofferdam along a river.

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.

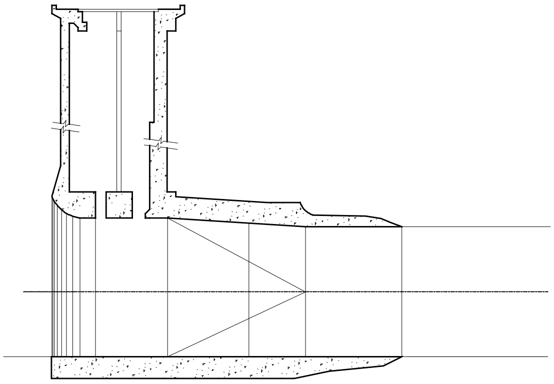


Fig. 5.5.3.07 Typical inlet to a diversion tunnel.

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.

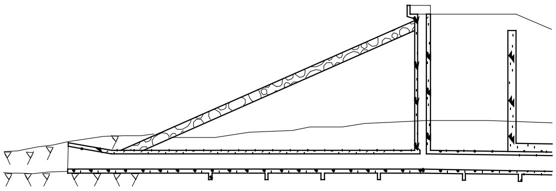


Fig. 5.5.3.08 Diversion Gallery.

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.

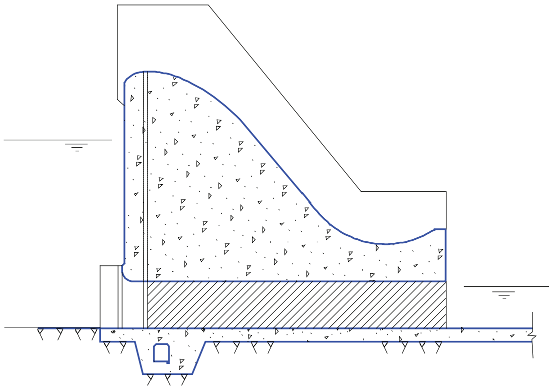


Fig. 5.5.3.09 Longitudinal section of a diversion sluiceway in a high ogee spillway.

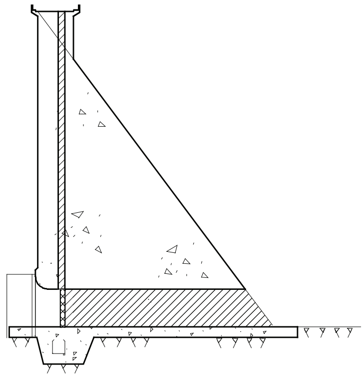


Fig. 5.5.3.10 Longitudinal section of a sluiceway in a concrete gravity dam.

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.

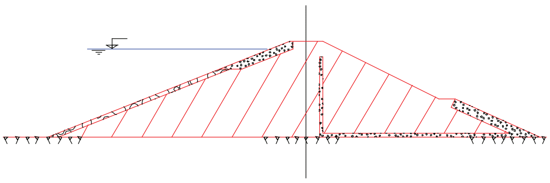


Fig. 5.5.4.01 – Typical cross-section of an earthfill dam.

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.

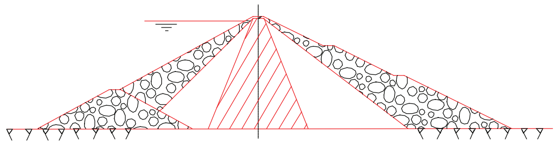


Fig. 5.5.4.02 – Typical cross-section of a rockfill dam with a central clay core.

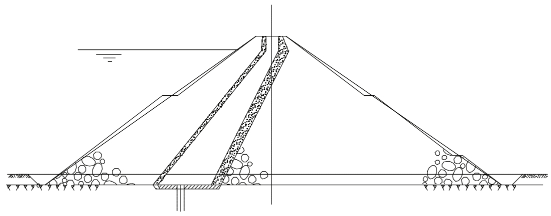


Fig. 5.5.4.03 – Typical cross-section of a rockfill dam with an inclined clay core (Salto Osório hydropower plant).

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.

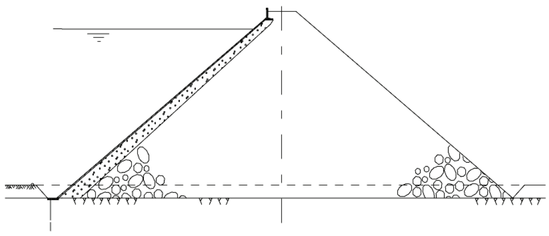


Fig. 5.5.4.04 – Typical cross-section of a concrete faced rockfill dam (Foz do Areia hydropower plant – Gov. Bento Munhoz da Rocha).

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.

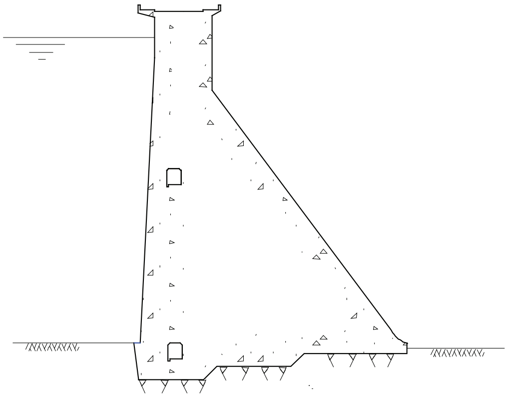


Fig. 5.5.4.05 – Typical cross-section of a conventional concrete dam.

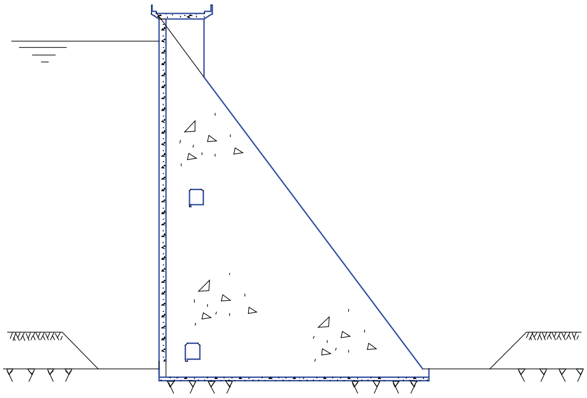


Fig. 5.5.4.06 – Typical cross-section of a roller compacted concrete dam.

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.

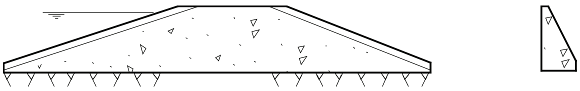


Fig. 5.5.4.07 – Typical cross-section of a retaining wall

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.

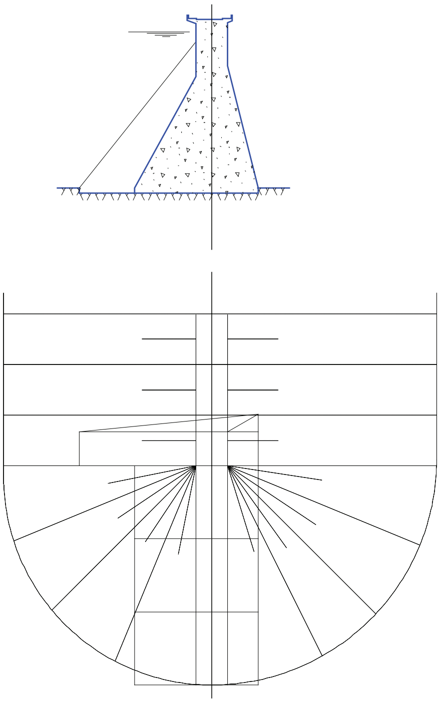


Fig. 5.5.4.08 – Typical cross-section of a transition wall

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.

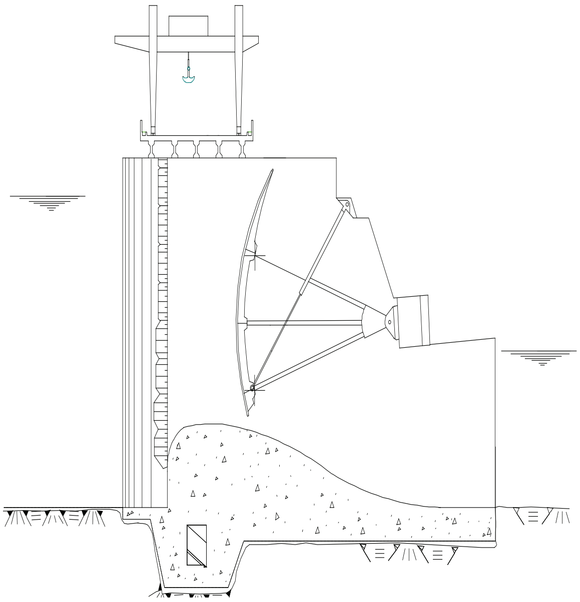


Fig. 5.5.5.01 – Typical cross-section of a gated surface spillway with a low ogee crest.

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.

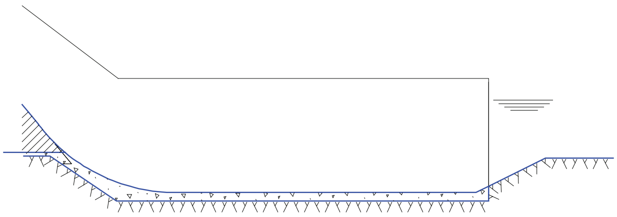


Fig. 5.5.5.02 – Typical cross-section of a stilling basin.

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.

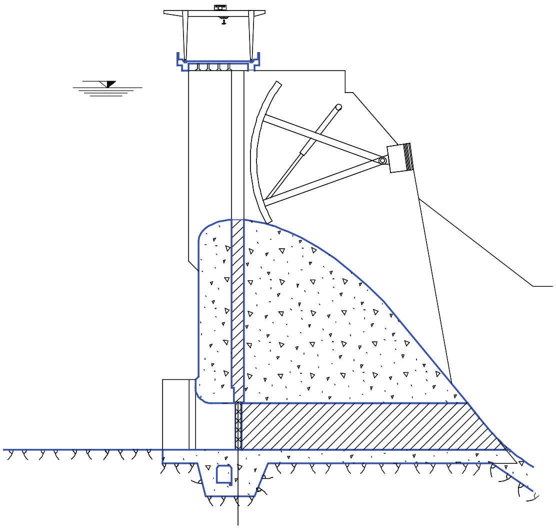


Fig. 5.5.5.03 – Typical cross-section of a gated surface spillway with a high ogee crest.

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.

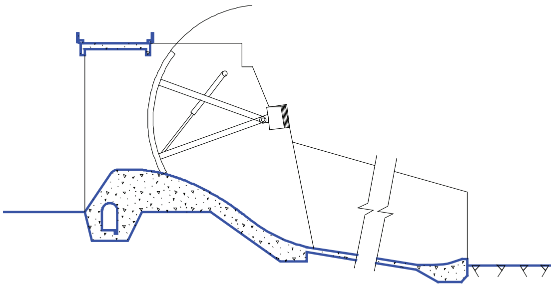


Fig. 5.5.5.04 – Typical cross-section of a gated abutment spillway.

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.

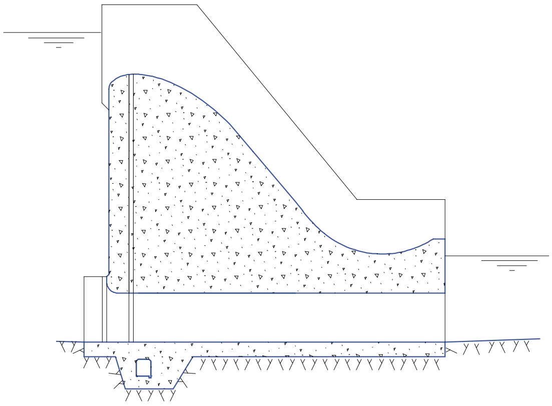


Fig. 5.5.5.05 – Typical cross-section of an ungated surface spillway with a high ogee crest.

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.

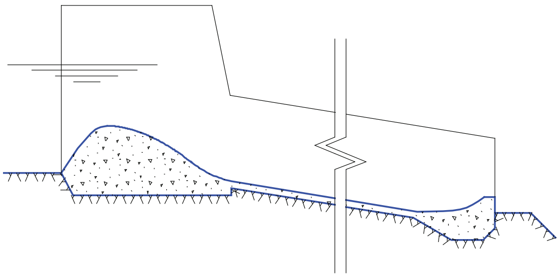


Fig. 5.5.5.06 – Typical cross-section of an ungated surface spillway with a low ogee crest.

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	m³			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	m³			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	m³		7.60	0
.12.16.24.12.11	surface rock	m³		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	m³		7.60	0
.12.17.25.12.10	borrow area	m³			0
.12.17.25.12.11	surface rock	m³		21.00	0
.12.17.25.12.11	quarry	m³			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	m³			0
.12.17.25.14.15	reinforcement steel	t		4,327.00	0
.12.17.25.24	Compacted earthfill	m³		2.69	0
.12.17.25.25	Rockfill	m³			0
.12.17.25.26	Clay core	m³		11.10	0
.12.17.25.29	Transitions / drains	m³		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	m³		7.60	0
.12.17.26.12.11	surface rock	m³		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	m³			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	m³			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	m^3		7.60	0
.12.17.27.12.11	surface rock	m^3		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	m^3			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	m^3		7.60	0
.12.18.28.12.11	surface rock	m^3		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	m^3			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	m^3		7.60	0
.12.18.29.12.11	surface rock	m^3		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	m^3			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	m^3		7.60	0
.12.19.30.12.11	surface rock	m^3		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	m^3			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	m^3		7.60	0
.12.19.31.12.11	surface rock	m^3		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: $150 \le H_1 \le 1500$ m
- for **vertical-axis Francis** turbines: $27 \le H_1 \le 600$ m
- for **horizontal-axis Francis** turbines: $27 \le H_1 \le 350$ m
- for **Kaplan** turbines: $8 \le H_1 \le 70$ m
- for **Bulb** turbines: $4 \le H_1 \le 23$ 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:

$$N_g = \text{int}\left(\frac{P'_t}{1000 \times P_{1xt}} + 0.999\right) \ge 2$$

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:

  $27 \leq H_1 \leq 200$ m: $P_{1nt} = 5$

  $200 \leq H_1 \leq 350$ m: $P_{1nt} = 0.0071 \times H_1^{1.2386}$

  $350 \leq H_1 \leq 600$ m: $P_{1nt} = 8.36 \times 10^{-6} \times H_1^{2.5312}$
- for **horizontal-axis Francis** turbines:

  $27 \leq H_1 \leq 200$ m: $P_{1nt} = 5$

  $200 \leq H_1 \leq 350$ m: $P_{1nt} = 0.0071 \times H_1^{1.2386}$
- for **Kaplan** turbines:

  $8 \leq H_1 \leq 50$ m: $P_{1nt} = 5$

  $50 \leq H_1 \leq 70$ m: $P_{1nt} = 0.0016 \times H_1^{2.06}$
- for **Bulb** turbines:

  $4 \leq H_1 \leq 23$ 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	$P_1' \leq 10$MW
0.5	$10 < P_1' \leq 80$MW
1.0	$P_1' > 80$MW

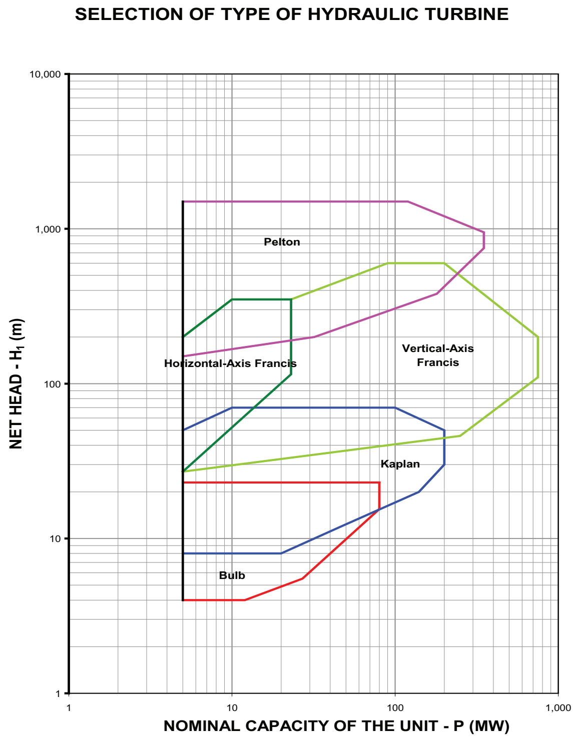
where:

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 (m^3/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	$Q_1 \geq 20.0$

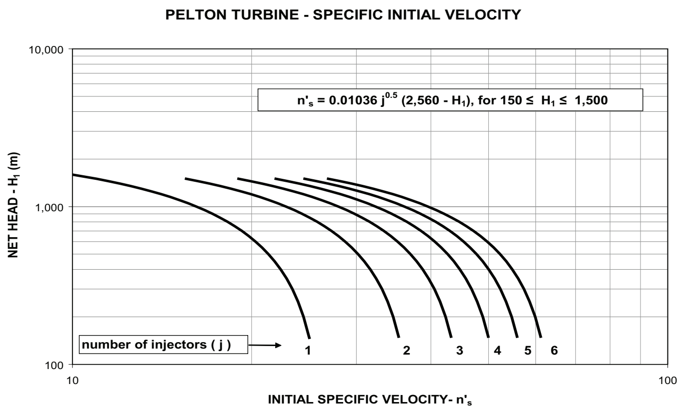
$$Q_t = \frac{10^6 \times P_1}{k \times H_1}$$

where:

$k = \rho \times g \times \eta_{t1} \times \eta_{g1}$ $\eta_{t1} = 0.89$ 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 m^3/s;
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 maximum net head;
η_{g1}	generator output at maximum net head; and
g	9.81 m/s^2 – 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 (m^3/s)
horizontal	$Q_1 < 7.0$
vertical	$Q_1 \geq 7.0$

where:

Q_1	maximum flow for each turbine, in m^3/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 50 Hz	Synchronous Velocity 60 Hz	No. of generator poles	Synchronous Velocity 50 Hz	Synchronous Velocity 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.

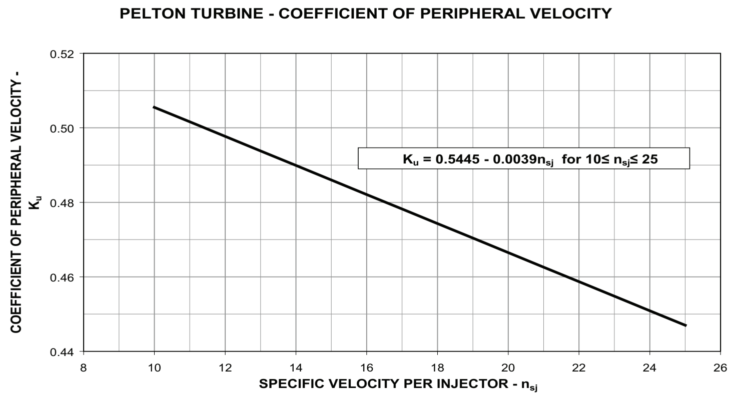
Diameter and position of the turbine rotor

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.

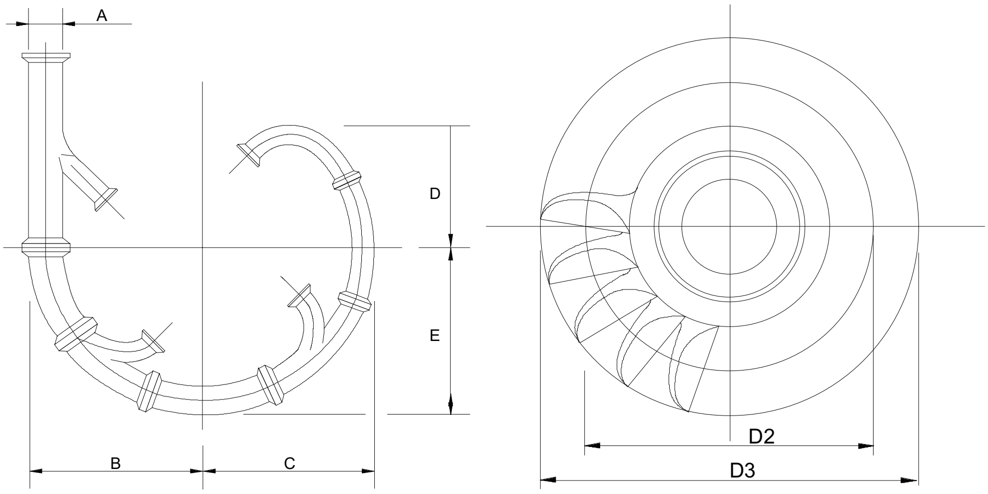


Fig. 5.7.2.01 – Plan of nozzles and rotor – Pelton Turbine.

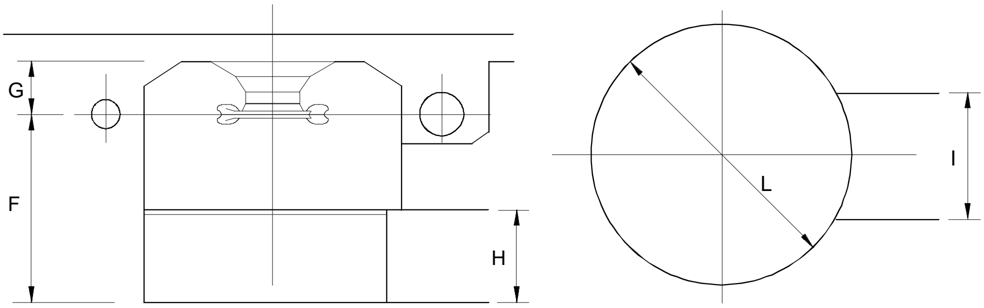


Fig. 5.7.2.02 – Plan and cross-section of the steel-lined chamber – Pelton Turbine.

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.

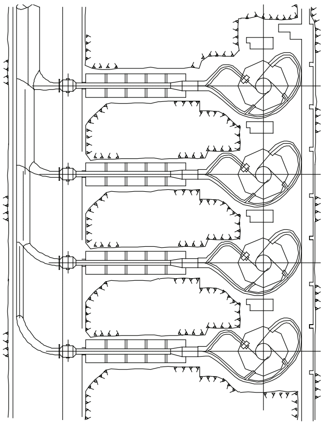


Fig. 5.7.2.03 – Plan of the valve house, powerhouse and assembly area – Pelton turbine.

Common excavation (account .11.13.00.12.10)

The **common excavation** volume, V_{tcf} (m^3), 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} (m^3), for **underground powerhouses** is given by:

$V_{tcf} = 0$

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 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: $A_{se} = L^2_{cs}$

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} (m²), 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/m²
- 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/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.

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 $30 \leq P \leq 1{,}450$ 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 $30 \leq P \leq 1{,}450$ 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 $0.0004 \leq \lambda \leq 0.0483$: $C_{gh} = 29580(\lambda)^{0.6323}$

- for vertical-axis generators:

  valid for $0.0329 \leq \lambda \leq 1.9834$: $C_{gv} = 42280(\lambda)^{0.6298}$

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: $z = 1000 \times \frac{P_2}{n}$

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 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:

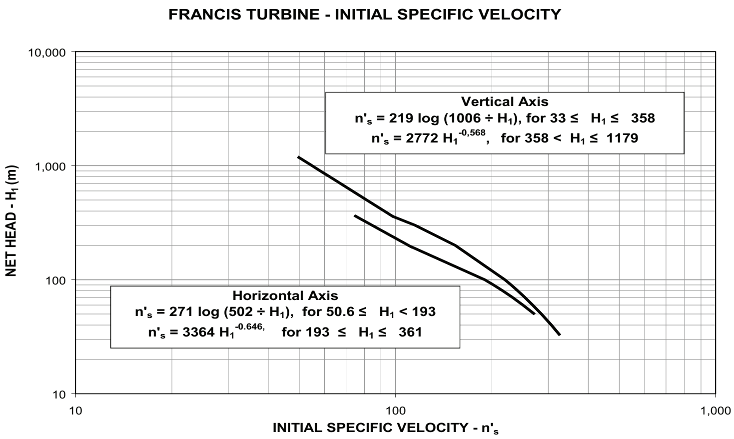
$$Q_1 = \frac{10^6 \times P_1}{k \times H_1} \qquad k = \rho \times g \times \eta_{t1} \times \eta_{g1}$$

$$\eta_{t1} = 0.856 \times Q_1^{0.013} \qquad \eta_{g1} = 0.92 \times P_2^{0.01}$$

$$P_2 = \frac{P_1}{f_p}$$

where:

Q_1	maximum turbine flow of each turbine, in m³/s;
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.



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: $P_{1t} = \frac{10^3 \times P_1}{\eta_g}$

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 $n' \geq 5 \times f$: $p = 2 \times \mathrm{int}\left(120 \times \frac{f}{n'} \times \frac{1}{2} + 0.778\right)$

for $1.2 \times f \leq n' < 5 \times f$: $p = 2 \times \mathrm{int}\left(120 \times \frac{f}{n'} \times \frac{1}{2}\right)$

without using 54, 74 and 94

for $n' < 1.2 \times f$: $p = 4 \times \mathrm{int}\left(120 \times \frac{f}{n'} \times \frac{1}{4}\right)$

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.

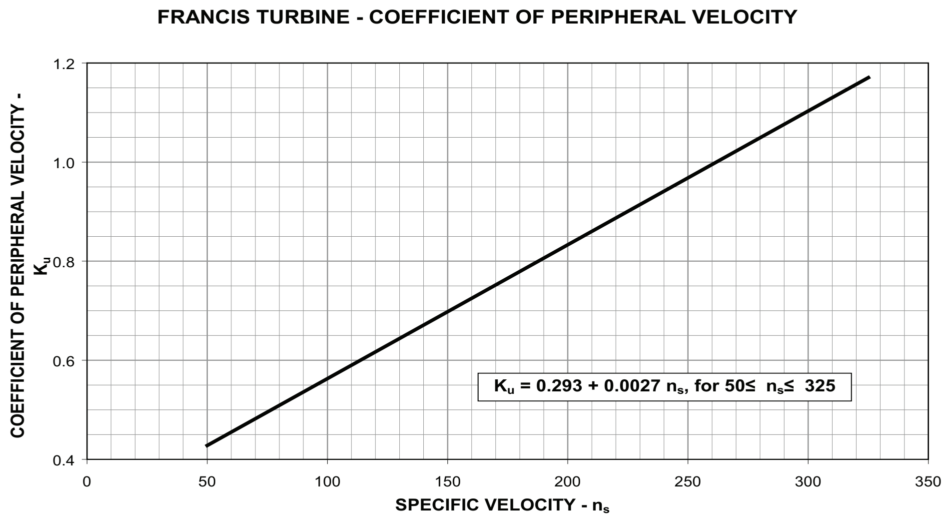
Diameter and position of the turbine rotor

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: $K = 10.33\text{-}0.0012 \times NA_{fu} - 0.013 \times T$

$\sigma = 7.54 \times 10^{-5} \times n_s^{1.41}$

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.

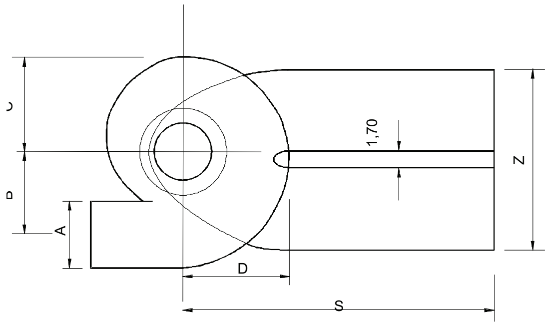


Fig. 5.7.2.04 – Plan of the spiral casing and draft tube – vertical-axis Francis turbine.

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.

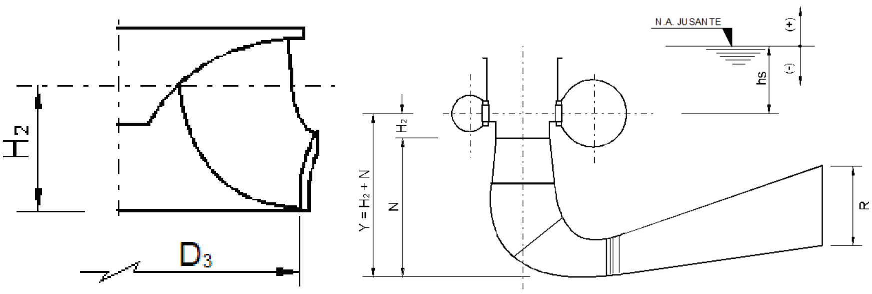


Fig. 5.7.2.05 – Cross-section of the spiral casing and draft tube – vertical-axis Francis turbine.

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

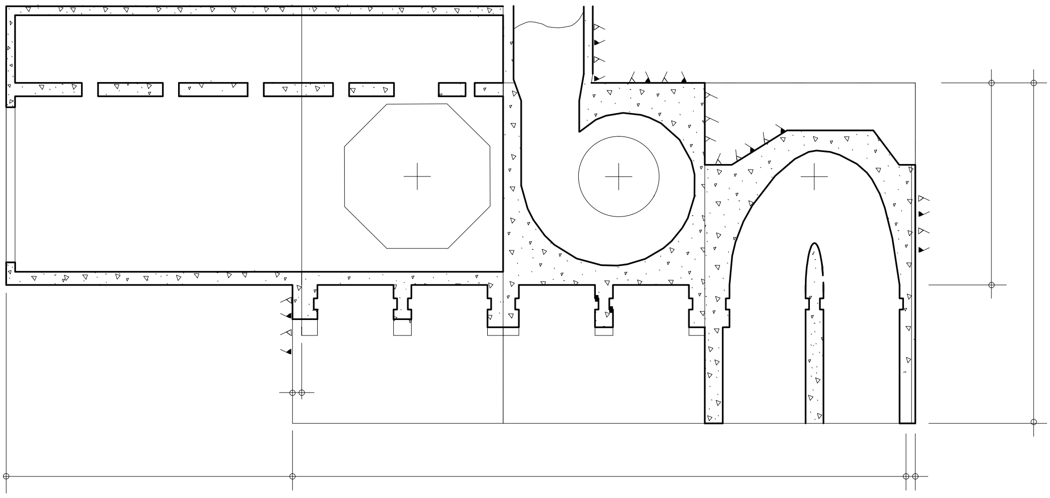


Fig. 5.7.2.06 – Plan of the powerhouse and assembly area for vertical-axis Francis turbines.

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} (m^3), for a **surface powerhouse** is given by.

$$V_{rcf} = V_{re} + V_{rp} + V_{rd}$$

and is valid for $1.5 \le D_3 \le 8.0$ m:

$$V_{re} = \left(B_{cf} + B_{am} + 2 \times B_{1cf} + 0.6 \times h_r\right) \times L_{cf} \times h_r$$

$$V_{rp} = B_{cf} \times L_{cf} \times \left(NA_{xfu} + 1.5 - El_d\right)$$

$$V_{rd} = N_g \times 700 \times e^{0.54 \times D_3}$$

where:

V_{re}	volume of excavation in rock above the elevation of the assembly area, in 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 (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} (m^3), for an **underground powerhouse** is given by:

$$V_{rcf} = 0$$

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 .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$/m³) (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:

$$V_{ccf} = N_g \times (V_{cf} + 1.5 \times V_{cs}) + V_{ce} + V_{cd} + V_{cn} + V_{ca}$$

for:

valid for $1.5 \le D_3 \le 8.0$ 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 $N_g \le 3$: $V_{ca} = V_{cs}$

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 $0.16 \leq z \leq 54.5$: $C_{sl} = 72.9 \text{ x } z^{0.716}$

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 $0.0329 \leq \lambda \leq 1.9834$: $C_{gv} = 42280(\lambda)^{0.6298}$

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 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 m³; and
- volume of concrete, V_{ccf} in m³.

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):

$K_u = 0.293 + 0.0027 \text{ x } n_s$

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:

$h_s = K – \sigma \text{ x } H_1 – D_3$

for:

$K = 10.33 – 0.0012 \text{ x } NA_{fu} – 0.013 \text{ x } T$

$\sigma = 7.54 \text{ x } 10^{–5} \text{ x } n_s^{1.41}$

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.

$A = 1.15 \times D_3$ | $B = 1.50 \times D_3$

$C = 3.80 \times D_3$ | $D = 1.90 \times D_3$

$E = 2.0 \times D_3$ | $R = 2.0 \times D_3$

$S = 5.2 \times D_3$ | $Y = 2.60 \times D_3$

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.

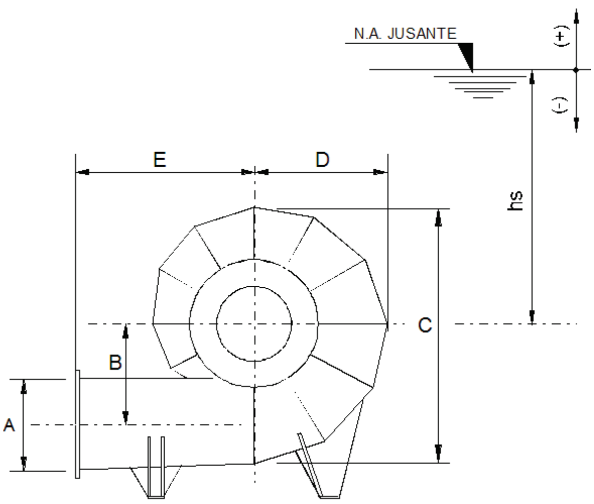


Fig. 5.7.2.07 – View of spiral casing – horizontal-axis Francis turbine.

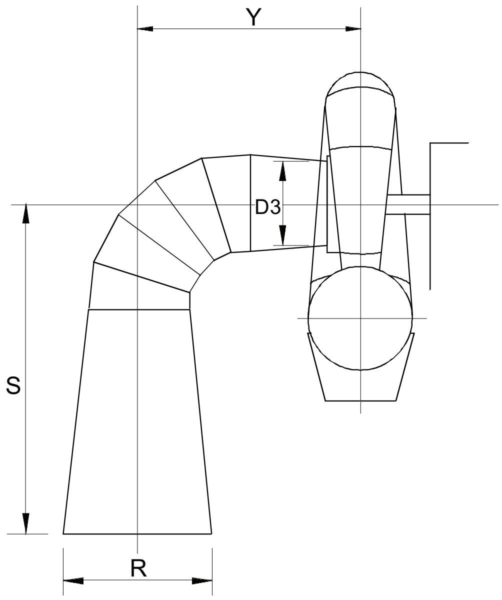


Fig. 5.7.2.08 – View of draft tube – horizontal-axis Francis turbine.

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.

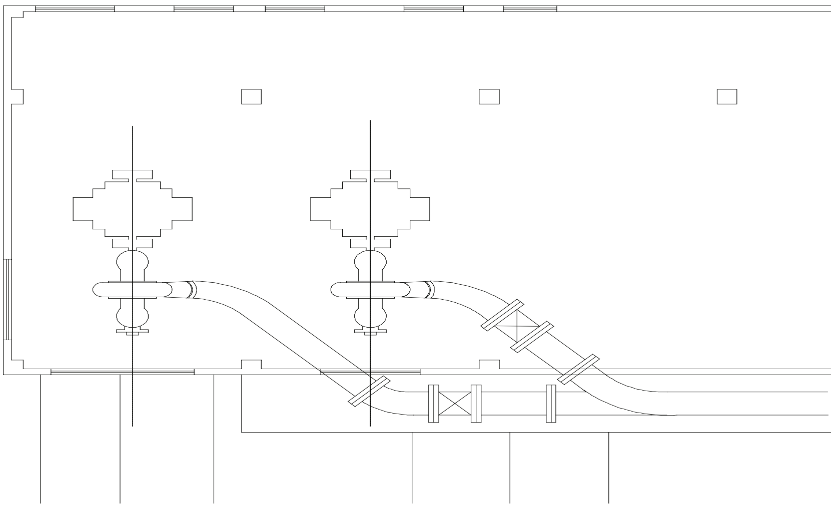


Figure 5.7.2.09 – Powerhouse with horizontal-axis Francis turbines.

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} (m^2), 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/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 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: $\lambda = \dfrac{P_2}{n}$ 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 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

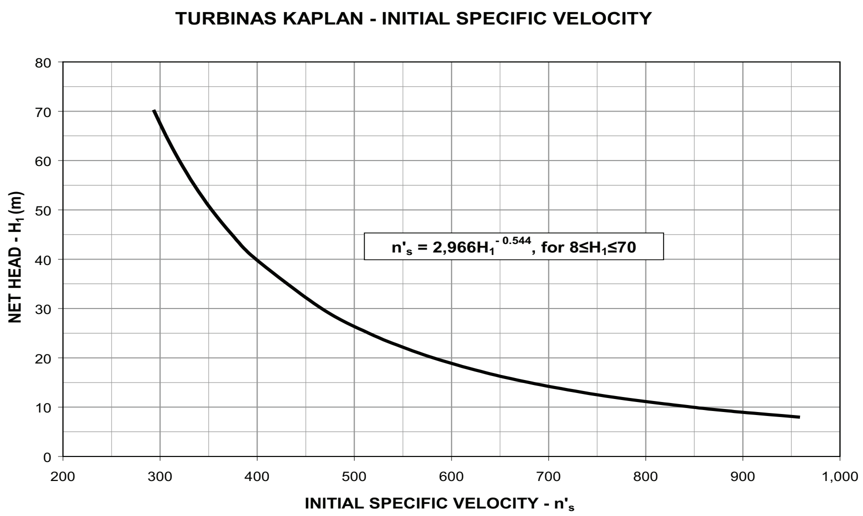
Velocities

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: $$P_{1t} = \frac{10^3 \times P_1}{\eta_g}$$

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 $n' \geq 1.2 \times f$: $$p = 2 \times \text{int}\left(120 \times \frac{f}{n'} \times \frac{1}{2} + 0.999\right)$$

without using 54, 74 and 94

for $n' < 1.2 \times f$: $$p = 4 \times \text{int}\left(120 \times \frac{f}{n'} \times \frac{1}{4} + 0.999\right)$$

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.

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.

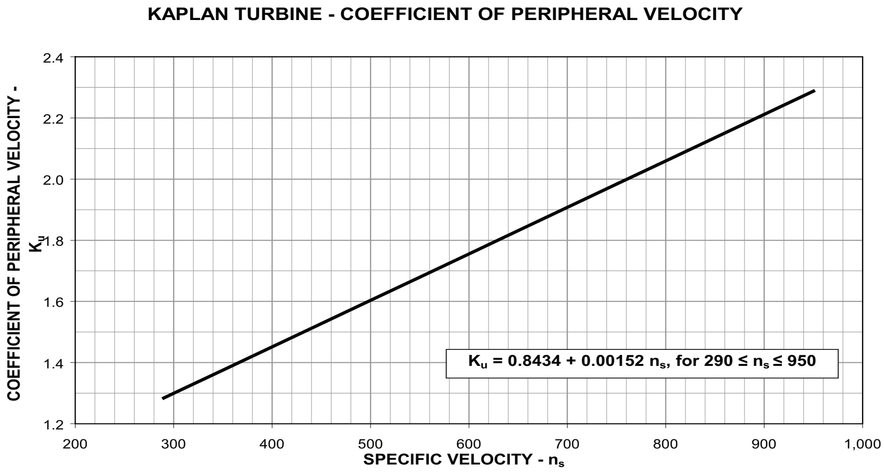
Diameter and position of the turbine rotor

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:

$$D_K = 0.01 \times \mathrm{int}\left(84.5 \times K_u \times \frac{H_1^{0.5}}{n} \times \frac{1}{0.01} + 0.5\right)$$

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.

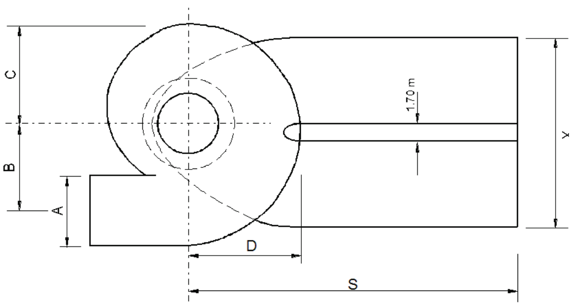


Fig. 5.7.2.10 – Plan of the spiral casing and draft tube – Kaplan turbine with steel spiral casing.

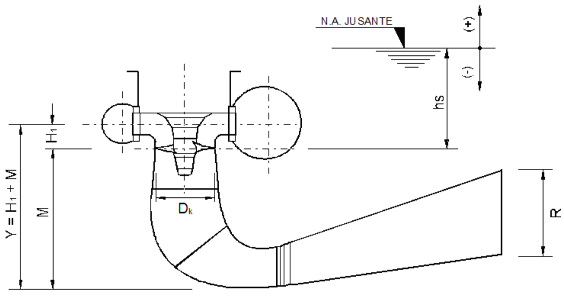


Fig. 5.7.2.11 – Cross-section of spiral casing and draft tube – Kaplan turbine with steel spiral casing.

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.

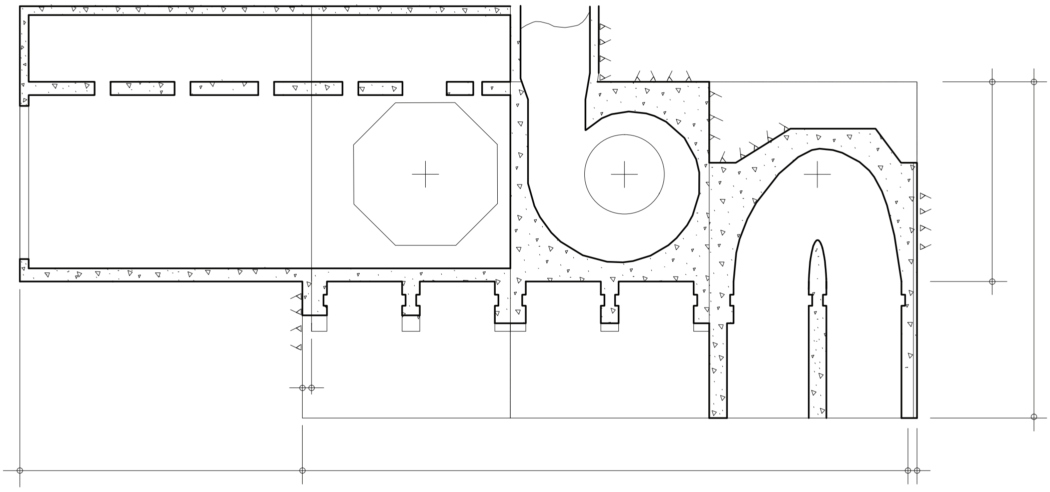


Fig. 5.7.2.12 – Plan of the powerhouse and assembly area for Kaplan turbines with a steel spiral casing.

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 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	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
- concrete dental: 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 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 ≤ 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 $0.0004 \le \lambda \le 0.0483$: $C_{gv} = 42280 \times \lambda^{0.6298}$

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
- 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: $z = 1000 \times \dfrac{P_2}{n}$

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 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: $z = 1000 \times \dfrac{P_2}{n}$

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 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 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, 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.

$B = 1.8 \times D_k$

$C = 1.2 \times D_k$

$D = 1.5 \times D_k$

$F = 1.65 \times D_k$

$G = 1.3 \times D_k$

$S = 4.6 \times D_k$

$Y = 2.65 \times D_k$

$U = 1.7m$

$R = 1.2 \times D_k$

$X = 3.0 \times D_k$

$H'_1 = 0.4 \times D_k$

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.

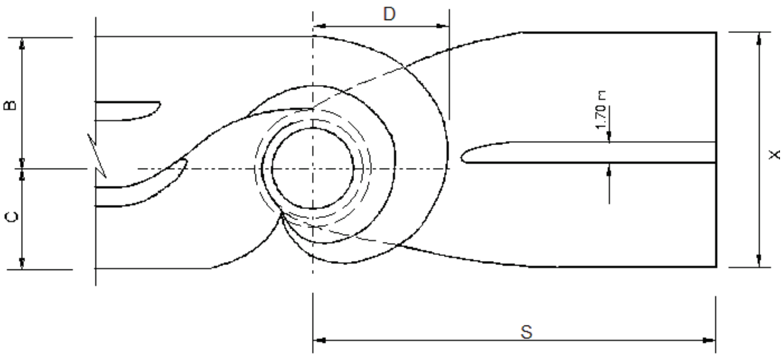


Fig. 5.7.2.13 – Plan of semi-spiral casing and draft tube – Kaplan turbine with semi-spiral casing made of concrete.

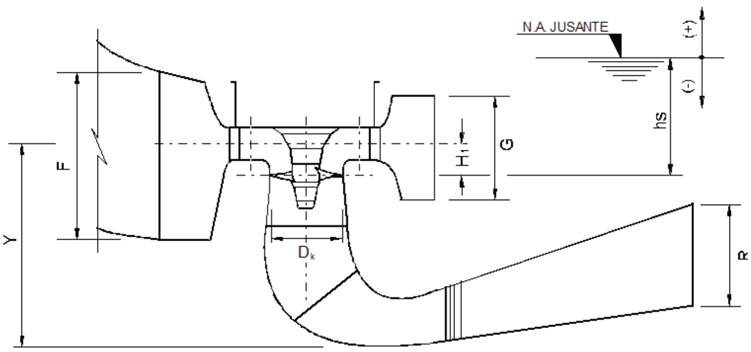


Fig. 5.7.2.14 – Cross-section of semi-spiral casing and draft tube – Kaplan turbine with semi-spiral casing made of concrete.

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.

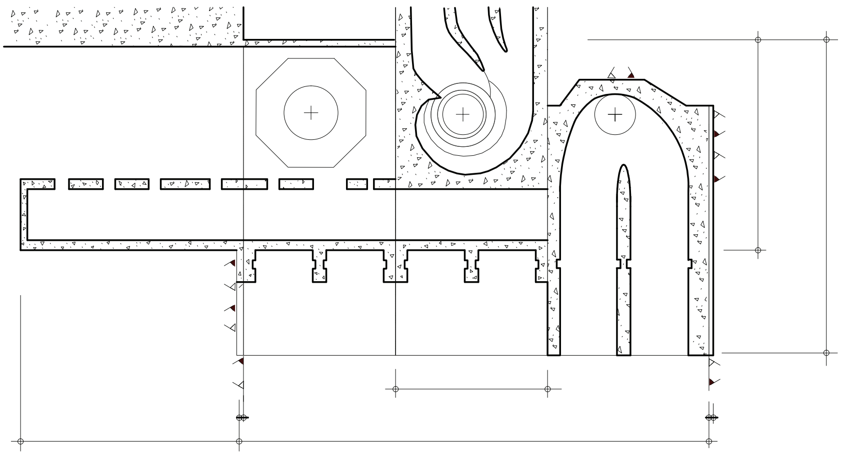


Fig. 5.7.2.15 – Plan of the powerhouse and assembly area – Kaplan turbine with semi-spiral casing made of concrete.

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} (m³), 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/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 the **powerhouse** is given by:

$$V_{rcf} = V_{re} + V_{rp} + V_{rd}$$

valid for $1.5 \le D_K \le 8.0\,m$

$$V_{re} = (B_{cf} + B_{am} + 2 \times B_{1cf} + 0.6 \times h_r) \times L_{cf} \times h_r$$

$$V_{rp} = B_{cf} \times L_{cf} \times (NA_{xfu} + 1.5 - El_d)$$

$$V_{rd} = N_g \times 700 \times e^{0.54 \times D_K}$$

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/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/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 .11.13.00.14)

The volume of **concrete**, V_{ccf} (m^3), 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 $1.5 \leq D_K \leq 8.0$ 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 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 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/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 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 $30 \leq P \leq 1450$ 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 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: $z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$

$H_{cp} = R$ $\quad$ $H_x = NA_{xfu} - El_d + Y$

$B_{cp} = \frac{Z - U}{N_{vs}}$

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: $\lambda = \frac{P_2}{n}$ 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.

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.

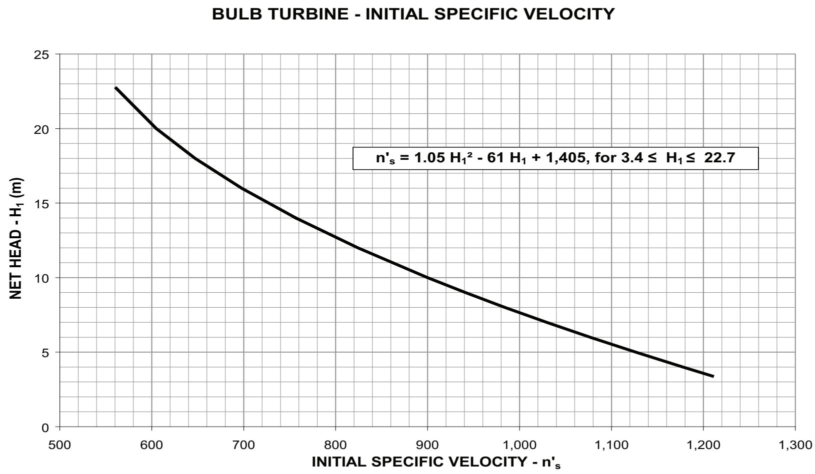
Velocities

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 $3.4 \le H_1 \le 22.7$ 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 $n' \ge 1.2 \times f$: $p = 2 \times \text{int}\left(120 \times \dfrac{f}{n'} \times \dfrac{1}{2} + 0.999\right)$

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.

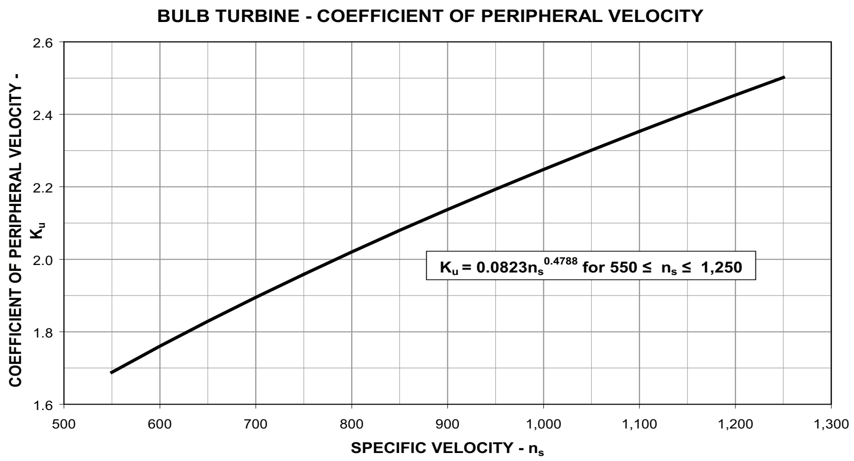
Diameter and position of the turbine rotor

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:

$$K = 10.33\text{-}0.0012 \times NA_{fu} - 0.013 \times T$$

$$\sigma = 0.0035 \times n_s - 1.12$$

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.

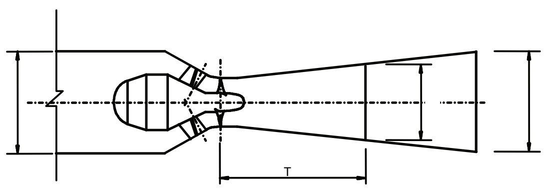


Fig. 5.7.2.16 – Plant of a unit with a Bulb turbine.

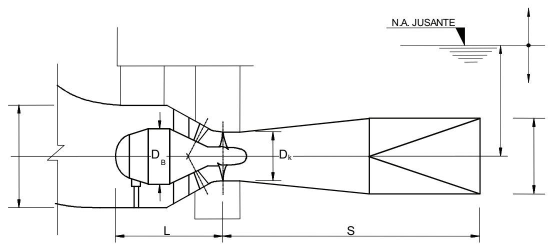


Fig. 5.7.2.17 – Cross-section of a unit with a Bulb turbine.

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.

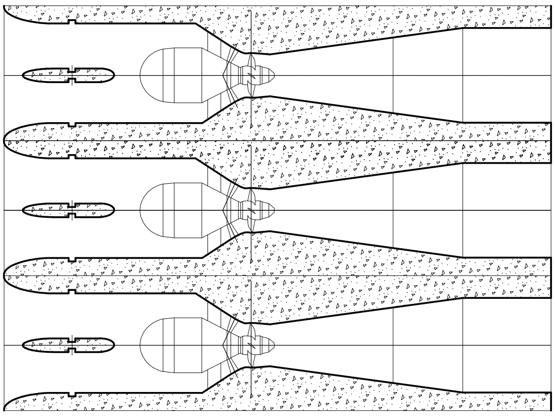


Fig. 5.7.2.18 – Plan of the powerhouse – Bulb turbine.

Common excavation (account .11.13.00.12.10)

In the absence of more accurate information, the common excavation volume, V_{tcf} (m³), 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/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)

In the absence of more accurate information, the excavation in rock, V_{tcf} (m³), for the **powerhouse**, can be determined by:

$$V_{rcf} = V_{re} + V_{rp} + V_{rd}$$

for:

$$V_{re} = (B_{cf} + B_{am} + 2 \times B_{1cf} + 0.6 \times h_r) \times L_{cf} \times h_r$$

$$V_{rp} = B_{cf} \times L_{cf} \times \left(NA_{xfu} + 1.5 - El_d + \frac{A}{2} \right)$$

where:

V_{re}	volume of excavation in rock above the elevation of the assembly area, in m³;
V_{rp}	volume of excavation in rock below the elevation of the assembly area, 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; and
A	height of the opening at the inlet, 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 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 ≤ P ≤ 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 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: $z = \frac{P_{1t}}{n}$

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 $N_g \le 10$: $N_{sl} = 2$

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 $0.0396 \leq \lambda \leq 0.9289$: $C_{gb} = 34120 \times \lambda^{0.7091}$

for: $\lambda = \frac{P_2}{n}$ and $P_2 = \frac{P_1}{f_p}$

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 $68.9 \le z \le 4582$: $C_{prv} = 25.12 \times z^{0.6961}$

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 $68.9 \le z \le 4582$: $C_{pcr} = 59.506 \times z^{0.6621}$

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:

$$T_r = 5 \times \mathrm{int}\left(\frac{1}{1-\sqrt[t]{1-R}} \times \frac{1}{5} + 0.999 \right)$$

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 (m³), 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.

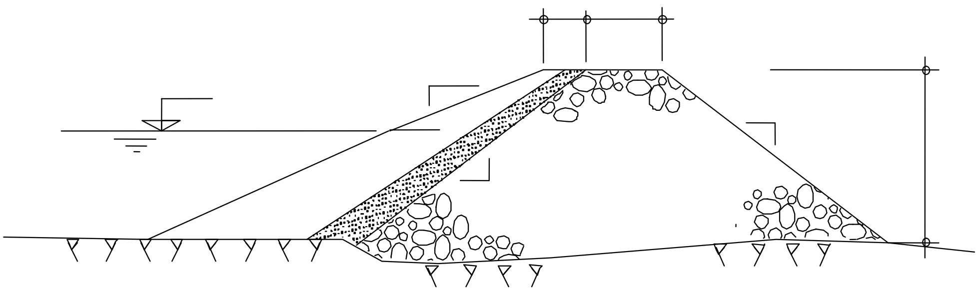


Fig 5.7.3.01 – Cross-section of a cofferdam across the river.

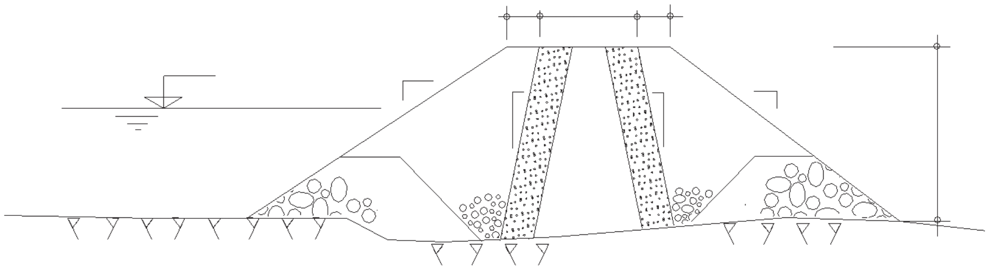


Fig 5.7.3.02 – Cross-section of a longitudinal cofferdam.

Earth-rock cofferdam (account .12.16.22.19)

The **volume of the cofferdam**, V_d (m^3), is given by:

$$V_d = \sum_k V_{dk} + V_{dl}$$

for:

$$V_{dk} = V_{dek} + V_{dak} + V_{dtk} \qquad V_{dek} = \left(1.5 \times H_{dk}^2 + 7 \times H_{dk}\right) \times L_{dk}$$

$$V_{dak} = \left(0.75 \times H_{dk}^2 + 3 \times H_{dk}\right) \times L_{dk} \qquad V_{dtk} = 0.15 \text{ x } V_{dek}$$

$$H_{dk} = \sqrt{\frac{1}{n_k} \sum_i H_{dki}^2} \qquad H_{dki} = NA_{dk} + 2 - El_{teki}$$

$$V_{dl} = V_{del} + V_{dal} \qquad V_{del} = \left(1.3 \times H_{dl}^2 + 4 \times H_{dl}\right) \times L_{dl}$$

$$V_{dal} = \left(0.2 \times H_{dl}^2 + 6 \times H_{dl}\right) \times L_{dl} \qquad H_{dl} = \sqrt{\frac{1}{n_l} \sum_i H_{dli}^2}$$

$$H_{dli} = NA_{dli} + 2 - El_{teli}$$

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 m^3;
V_{dek}	volume of rockfill for cofferdam k, in m^3;
V_{dak}	volume of earthfill for cofferdam k, in m^3;
V_{dtk}	volume of transition for cofferdam k, in m^3;
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 m^3;
V_{del}	volume of rockfill for the longitudinal cofferdam, in m^3;
V_{dal}	volume of earthfill for longitudinal cofferdam, in m^3;
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.

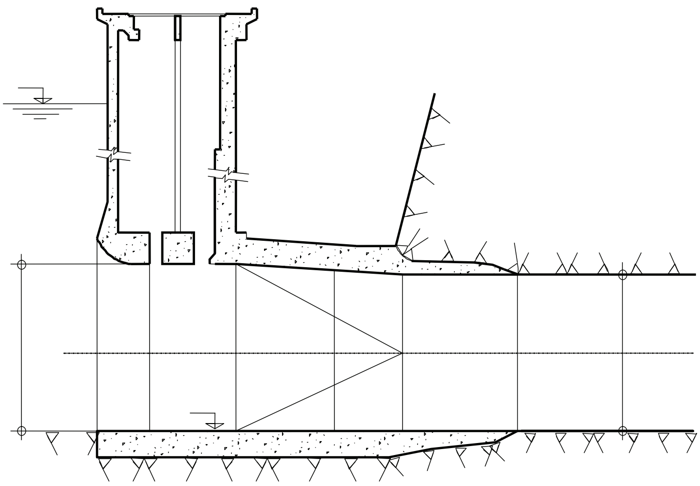


Fig. 5.7.3.03 – Typical longitudinal section of a diversion channel.

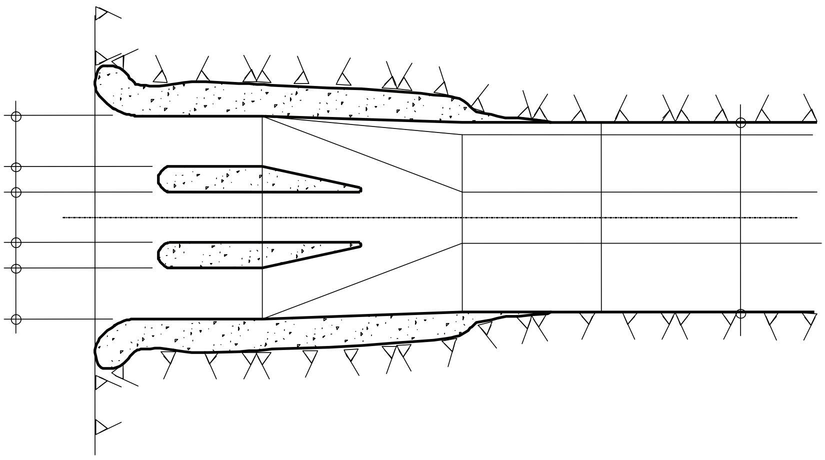


Fig. 5.7.3.04 – Plan of a diversion channel.

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:

$$D_{td} = \sqrt{\frac{Q_k}{0.8927 \times N_{td} \times v_{td}}} \geq 2.0\,m$$

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:

$$El_{ca} = El_{de} \qquad El_{cr} = El_{ds}$$

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:

$$H = k_H \times H_{cp}$$

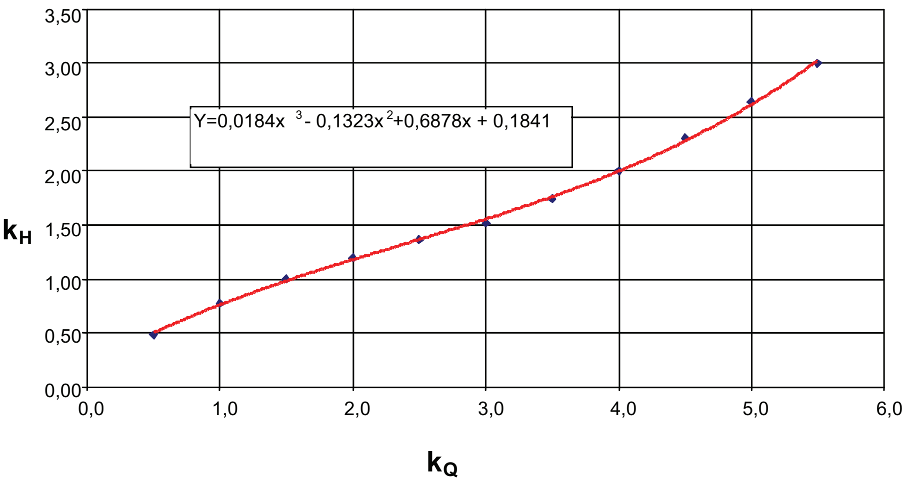
$$k_H = 0.0184 \times k_Q^3 - 0.1323 \times k_Q^2 + 0.688 \times k_Q + 0.18$$

$$k_Q = \frac{Q_k}{N_{td} \times N_v \times B_{cp} \times H_{cp}^{3/2}} \qquad N_v = \mathrm{int}\left(\frac{D_{td}}{4.5} + 0.9\right)$$

$$B_{cp} = \frac{0.88 \times D_{td}}{N_v} \qquad H_{cp} = 1.13 \times D_{td}$$

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} (m^3), in the absence of more accurate information, can be determined by:

$$V_{ttd} = V_{tca} + V_{tcr}$$

for:

$$V_{tca} = \left(\frac{V_{ta0}}{2} + V_{ta1} + V_{ta2} \right) \times \frac{L_{ca}}{3}$$

$$V_{tai} = \left[B_{ca} - 6 + 2 \times \left(0.6 \times h_{rai} + e_{te} \right) \right] \times e_{te}$$

$$h_{rai} = El_{tai} - El_{ca} - e_{te}, \; i = 0,\ 1,\ 2$$

$$B_{ca} = \left(N_{td} - 1 \right) \times 2 \times D_{td} + 2 \times D_{td}$$

$$V_{tcr} = \left(\frac{V_{tr0}}{2} + V_{tr1} + V_{tr2} \right) \times \frac{L_{cr}}{3}$$

$$V_{tri} = \left[B_{cr} - 6 + 2 \times \left(0.6 \times h_{rri} + e_{te} \right) \right] \times e_{te}$$

$$h_{rri} = El_{tri} - El_{cr} - e_{te}, \; i = 0,\ 1,\ 2$$

$$B_{cr} = \left(N_{td} - 1 \right) \times 2 \times D_{td} + 1.5 \times D_{td}$$

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;
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 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/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 .12.16.23.12.11)

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 $0.16 \le z \le 54.43$: $C_{sl} = 72.896 \times z^{0.716}$

for:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000} \qquad H_x = NA_{max} - El_{td}$$

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 m³/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.

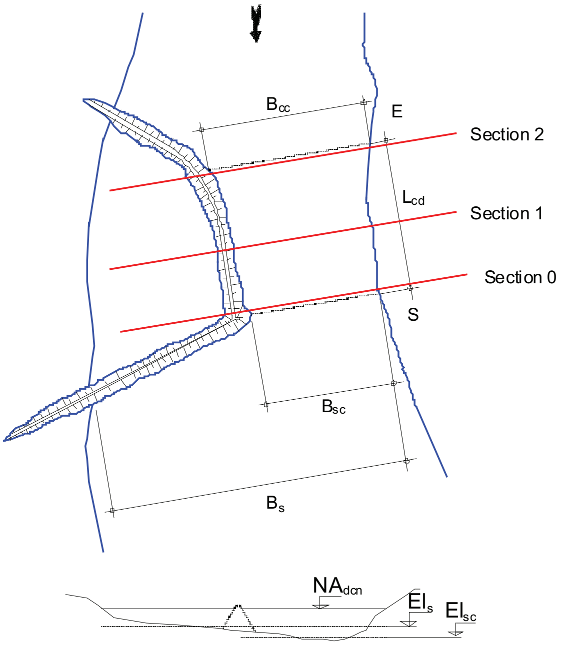


Fig 5.7.3.05 – Typical plan and cross-section of a diversion channel.

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:

$$V_{tca} = \left(\frac{V_{ta0}}{2} + V_{ta1} + V_{ta2}\right) \times \frac{L_{ca}}{3}$$

$$V_{tai} = \left[B_{mc} - 6 + 2 \times (0.6 \times h_{rai} + e_{te})\right] \times e_{te}$$

$$h_{rai} = El_{tai} - El_{ec} - e_{te}\ ,\ i = 0,\ 1,\ 2$$

$$V_{tcr} = \left(\frac{V_{tr0}}{2} + V_{tr1} + V_{tr2} \right) \times \frac{L_{cr}}{3}$$

$$V_{tri} = \left[B_{mc} - 6 + 2 \times \left(0.6 \times h_{rri} + e_{te} \right) \right] \times e_{te}$$

$$L_{cr} = L_{cd} - L_{ca}$$

$$h_{rri} = El_{tri} - El_{sc} - e_{te}\ ,\ i = 0,\ 1,\ 2$$

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:

$$V_{rca} = \left(\frac{V_{ra0}}{2} + V_{ra1} + V_{ra2} \right) \times \frac{L_{ca}}{3}$$

$$V_{rai} = [B_{mc} - 6 + 0.6 \times h_{rai}] \times h_{rai}$$

$$V_{rcr} = \left(\frac{V_{rr0}}{2} + V_{rr1} + V_{rr2} \right) \times \frac{L_{cr}}{3}$$

$$V_{rri} = [B_{mc} - 6 + 0.6 \times h_{rri}] \times h_{rri}$$

where:

V_{rca}	volume of excavation in rock in the upstream half of the canal, in m^3 (COPEL, 1996);
V_{rcr}	volume of excavation in rock in the downstream half of the canal, in m^3 (COPEL, 1996);
V_{rai}	volume of excavation in rock 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;
V_{rri}	volume of excavation in rock 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; 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/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)

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/s² – acceleration due to gravity;
Q_k	diversion flow for a recurrence time of k years, in m³/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/s² – 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 m³/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/s² – acceleration due to gravity;
Q_k	diversion flow for a recurrence time of k years, in m³/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 m³/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.

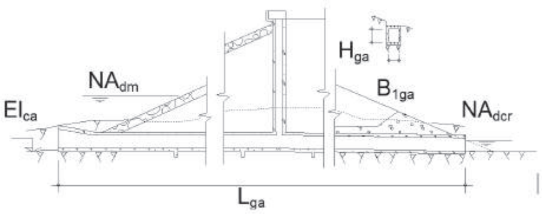


Fig. 5.7.3.06 – Cross-section and longitudinal section of a diversion gallery in a dam.

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 m³/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/s^2 – 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 m^3/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:

$$NA_{dm} = El_{de} + H$$

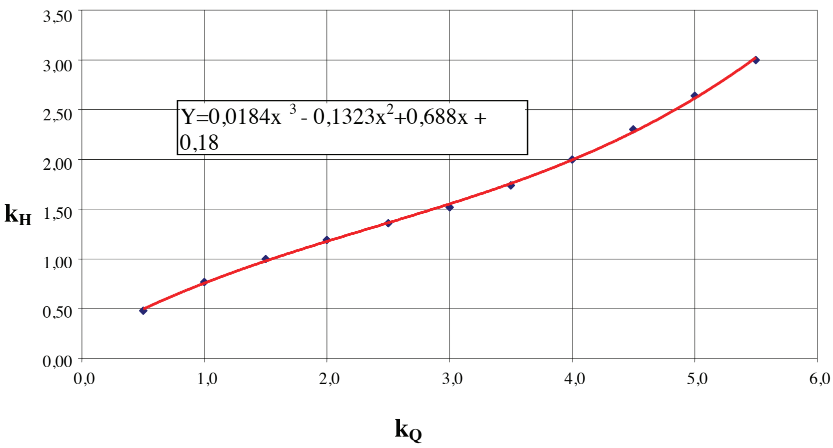
for:

$$H = k_H \times H_{ga}$$

$$k_H = 0.0184 \times k_Q^3 - 0.1323 \times k_Q^2 + 0.688 \times k_Q + 0.18$$

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:

$$V_{tca} = \left(\frac{V_{ta0}}{2} + V_{ta1} + V_{ta2} \right) \times \frac{L_{ca}}{3}$$

$$V_{tai} = \left[B_{ca} - 6 + 2 \times (0.6 \times h_{rai} + e_{te}) \right] \times e_{te}$$

$$B_{ca} = B_{ga}$$

$$h_{rai} = El_{tai} - El_{ca} - e_{te}, \ i = 0,\ 1,\ 2$$

$$V_{tes} = B_{ga} \times L_{ga} \times e_{te}$$

$$V_{tcr} = \left(\frac{V_{tr0}}{2} + V_{tr1} + V_{tr2} \right) \times \frac{L_{cr}}{3}$$

$$V_{tri} = \left[B_{cr} - 6 + 2 \times (0.6 \times h_{rri} + e_{te}) \right] \times e_{te}$$

$$B_{cr} = B_{ga}$$

$$h_{rri} = El_{tri} - El_{cr} - e_{te}, \ i = 0,\ 1,\ 2$$

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 m³ (COPEL, 1996);
V_{res}	volume of excavation in rock in the gallery area, in m³;
V_{rcr}	volume of excavation in rock for the downstream channel, in m³ (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 m³/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/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)

The **area of foundations** to be cleaned for the galleries, A_{lf} (m²), 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/m²

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} (m³), is given by:

$$V_{cga} = V_{cpl} + V_{cto} + V_{ctt}$$

for:

$$V_{cpl} = [\, B_{ga} \times (H_{ga} + 2 \times e_{pl}) - N_{ga} \times B_{1ga} \times H_{ga} \,] \times L_{ga}$$

$$V_{cto} = \left(B_{ga} \times 2.3 - 4 \times B_{1ga} \times 0.4\right) \times \left(NA_{max} - El_{ca} - H_{ga}\right)$$

$$V_{ctt} = N_{ga} \times 3 \times e_{pl} \times B_{1ga} \times H_{ga}$$

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/m³)	reinforcement steel (kg/m³)
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.

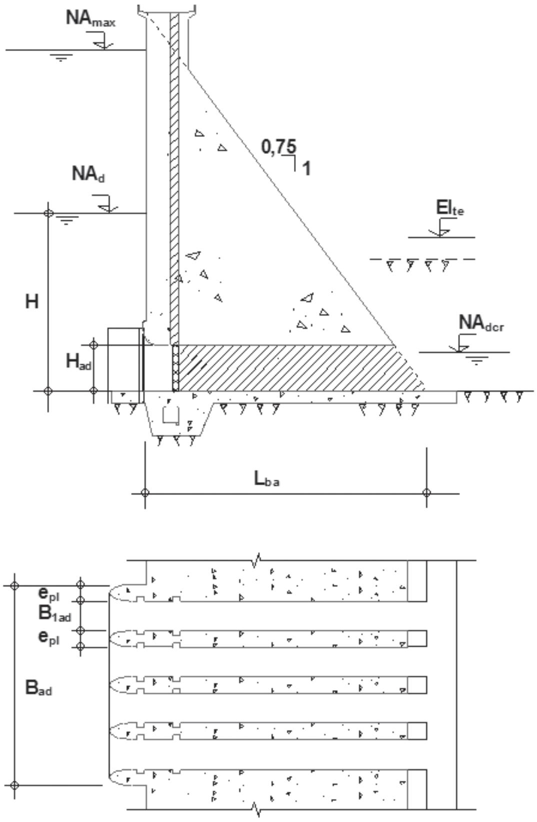


Fig. 5.7.3.07 – Typical cross-section and plan of sluiceways in a concrete gravity dam.

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:

$$B_{1ad} = \left(\frac{Q_k}{4 \times k_Q \times N_{ad}}\right)^{0.4} \geq 1.5\,m$$

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:

$$H_{ad} = \left(\frac{Q_k}{k_Q \times N_{ad} \times B_{1ad}}\right)^{2/3} \geq 1.9\,m$$

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:

$$v_a = \frac{Q_k}{N_{ad} \times B_{1ad} \times H_{ad}} \leq 15\,m/s$$

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} (m³), is given by:

$$V_{tad} = V_{tca} + V_{tcr}$$

for:

$$V_{tca} = \left(\frac{V_{ta0}}{2} + V_{ta1} + V_{ta2} \right) \times \frac{L_{ca}}{3}$$

$$V_{tai} = \left[B_{ca} - 6 + 2 \times \left(0.6 \times h_{rai} + e_{te} \right) \right] \times e_{te}$$

$$B_{ca} = B_{ad}$$

$$h_{rai} = E_{tai} - E_{a} - e_{te} \text{, } i = 0, 1, 2$$

$$V_{tcr} = \left(\frac{V_{tr0}}{2} + V_{tr1} + V_{tr2} \right) \times \frac{L_{cr}}{3}$$

$$V_{tri} = \left[B_{cr} - 6 + 2 \times \left(0.6 \times h_{rri} + e_{te} \right) \right] \times e_{te}$$

$$B_{cr} = B_{ad}$$

$$h_{rri} = El_{tri} - El_{cr} - e_{te} \text{, } i = 0, 1, 2$$

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 $0.13 \le z \le 9.17$: $C_{cp} = -4.399 \times z^2 + 124.8 \times z + 110$

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 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 $0.16 \le z \le 54.5$: $C_{sl} = 72.9 \times z^{0.716}$

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 m^3/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.

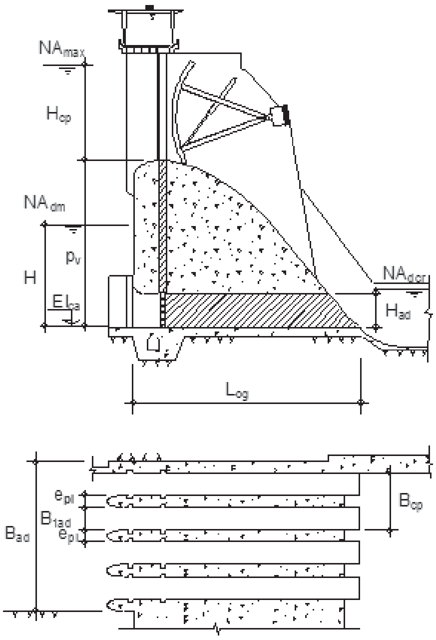


Fig. 5.7.3.08 – Typical cross-section and plan for sluiceways through a spillway with a high ogee crest and a stilling basin.

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 m³/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:

$$N_{ad} = 2 \times \text{int}(0.75 \times N_{cp}) + 1$$

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:

$$H_{ad} \leq NA_{MAX} \text{ x } H_d - El_{ca}$$

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 $0.13 \le z \le 9.17$: $C_{cp} = -4.399 \times z^2 + 124.8 \times z + 110$

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 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 $0.16 \le z \le 54.43$: $C_{sl} = 72.896 \times z^{0.716}$

for:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000} \qquad H_x = NA_{max} - El_{td}$$

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.

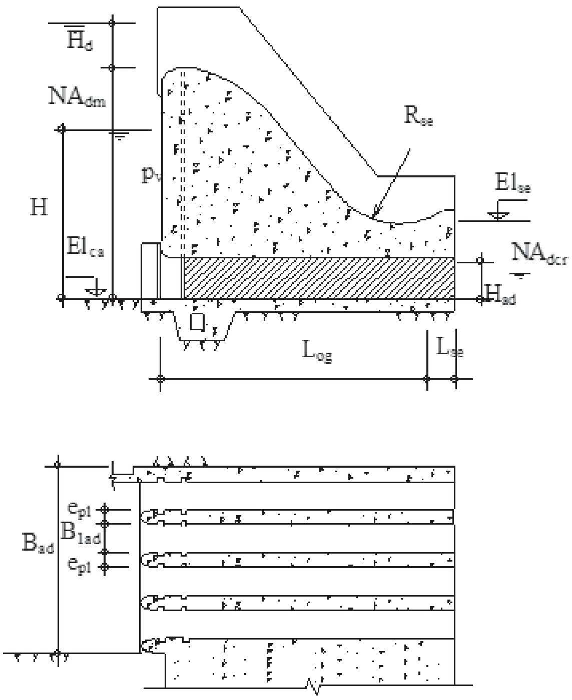


Fig. 5.7.3.09 – Typical cross-section and plan of sluiceways through spillways with a high ogee crest and a ski jump.

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 m^3/s.

The width of sluiceway, B_{1ad} (m), is given by:

$$B_{1ad} = \left(\frac{Q_k}{4 \times k_Q \times N_{ad}}\right)^{0.4} \quad \geq 1.5\,\text{m}$$

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:

$$H_{ad} = \left(\frac{Q_k}{k_Q \times N_{ad} \times B_{1ad}}\right)^{2/3} \quad \geq \; 1.9\,\text{m}$$

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:

$$v_a = \frac{Q_k}{N_{ad} \times B_{1ad} \times H_{ad}} \quad \leq 15\,\text{m/s}$$

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 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/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:

$$NA_{dm} = E_{dcr} + h_p$$

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/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 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 $0.13 \leq z \leq 9.17$: $C_{cp} = -4.399 \times z^2 + 124.8 \times z + 110$

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 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 $0.16 \leq z \leq 54.43$: $C_{sl} = 72.896 \times z^{0.716}$

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 **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.