## 5.7.4 Dams and Dikes (account .12.17)

### EARTHFILL DAMS (ACCOUNT .12.17.25)

#### Basic Data

The main information required for dimensioning and quantification purposes can be obtained from the overall layout, as follows:

- maximum normal water level in the reservoir, $NA_{max}$, from item 4.6, in m;
- mean elevation of the land in section i perpendicular to the longitudinal dam axis, $El_{tei}$, in m;
- distance between sections i and i-1, $\Delta L_i$ in m;
- minimum water level in the reservoir, $NA_{min}$, from item 5.3, in m;
- depth of the cutoff trench, $H_{tr}$ in m, when applicable.

#### Considerations and recommendations

**This text** relates to dams with a typical cross-section, as shown in Fig. 5.7.4.01.

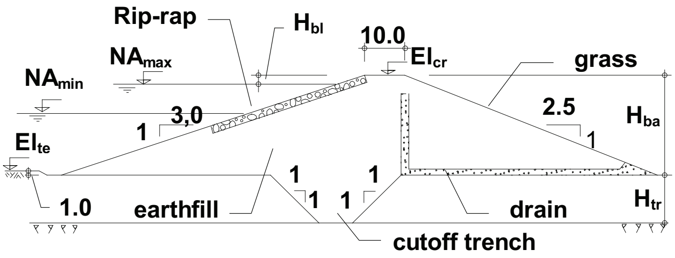



Fig. 5.7.4.01 – Typical cross-section of a homogeneous earthfill dam.

**Sections should be selected** over contour lines and at important features, like the bottom of saddles, tops of hills, river banks and at any points of contact, such as at intakes or spillways.

#### Quantities per type of service

The **quantities per type of service**, Qtd, are given by the general expression:

$$Qtd = \sum_i \frac{Sec_i + Sec_{i-1}}{2} \times \Delta L_i$$

where:

| | |
|---|---|
| $Sec_i$ | length, area or volume per meter of dam at section i, in m/m, $m^2$/m or $m^3$/m; and |
| $\Delta L_i$ | distance between sections i and i-1, in m. |

The **height of the dam in section i**, $H_{bai}$ (m), is given by:

$$H_{bai} = El_{cr} - (El_{tei} - 1.0)$$

where: $El_{cr} = NA_{max} + H_{bl}$

where:

| | |
|---|---|
| $El_{cr}$ | elevation of the dam crest, in m; |
| $El_{tei}$ | mean elevation of the land in section i perpendicular to the longitudinal dam axis, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $H_{bl}$ | 4.0m, height of the dam freeboard. |

The **maximum reservoir drawdown**, d (m), is given by:

$$d = NA_{max} - NA_{min}$$

where:

| | |
|---|---|
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $NA_{min}$ | minimum water level in the reservoir, in m. |

The **width of the base of the cutoff trench** in section i, $B_{tri}$ (m), required for permeable foundations, is given by:

$$B_{tri} = 0.3 \times (H_{bai} - H_{bl} + H_{tr}) \geq 6.0 \text{ m}$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m; |
| $H_{bl}$ | 4.0m, height of the dam freeboard; and |
| $H_{tr}$ | depth of the cutoff trench at section i, in m. |

When an impervious blanket has to be used, the following dimensions should be assumed:

The **width of the impervious blanket at section i**, $B_{tpi}$ (m), is given by:

for: $H_{tr} \geq 15.0$ m: $B_{tpi} = 10 \times (H_{bai} - H_{bl})$

for: $H_{tr} < 15.0$ m: $B_{tpi} = 0$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m; |
| $H_{bl}$ | 4.0m, height of the dam freeboard; and |
| $H_{tr}$ | depth of the cutoff trench at section i, in m. |

The **thickness of the impervious blanket at section i**, $e_{tpi}$ (m), is given by:

$$e_{tpi} = 0.1 \times (H_{bai} - H_{bl})$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m; and |
| $H_{bl}$ | 4.0m, height of the dam freeboard. |

### Common excavation in borrow areas (account .12.17.25.12.10)

**The volume of common excavation per meter of dam at section i**, $V_{ti}$ ($m^3/m$), is given by:

$$V_{ti} = V_{tbi} + V_{tri} + V_{tpi}$$

where:

$$V_{tbi} = 5.5 \times H_{bai} + 30$$

$$V_{tri} = (B_{tri} + H_{tr}) \times H_{tr}$$

$$V_{tpi} = B_{tpi} \times 1.0$$

where:

| | |
|---|---|
| $V_{tbi}$ | volume of common excavation for the dam at section i, in $m^3/m$; |
| $V_{tri}$ | volume of common excavation for the cutoff trench at section i, in $m^3/m$; |
| $V_{tpi}$ | volume of common excavation for the impervious blanket at section, in $m^3/m$; |
| $H_{bai}$ | dam height at section i, in m; |
| $B_{tri}$ | width of the base of the cutoff trench at section i, in m; |
| $H_{tr}$ | depth of the cutoff trench at section i, in m; and |
| $B_{tpi}$ | width of the impervious blanket at section i, in m. |

The **volume of common excavation in borrow areas**, $V_{tp}$ ($m^3$), is given by:

$$V_{tp} = \sum V_{aj} - 0.9 \times \sum V_{tj} \geq 0$$

where:

| | |
|---|---|
| $V_{aj}$ | volume of landfill for structures j, in $m^3$; and |
| $V_{tj}$ | volume of common excavation for structures j, in $m^3$. |

The unit price of common excavation is R$ 7.60/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.17.25.13)

The area of foundation to be cleaned **per meter of dam at section i**, $A_{lfi}$ ($m^2$/m), is given by:

$$A_{lfi} = 5.5 \times H_{bai} + 10$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m. |

The **volume of excavation and concrete leveling** (without reinforcement steel but with 200 kg cement per $m^3$) at the base of the cutoff trench, **per meter of dam at section i**, $V_{TFi}$ ($m^3$/m), is given by:

$$V_{TFI} = 0.3 \text{ x } (H_{bai} - H_{bl} + H_{tr}) \geq 6.0 \text{ m}$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m; |
| $H_{bl}$ | 4.0m, height of the dam freeboard; and |
| $H_{tr}$ | depth of the cutoff trench at section i, in m. |

The **length of the grout curtain per meter of dam at section i**, $L_{tf}$ (m/m), is given by:

$$L_{tf} = \frac{1}{3.0} \times L_{1tf}$$

$$L_{1tf} = H_{bai} - H_{bl} \leq 40 \text{ m}$$

where:

| | |
|---|---|
| $L_{1tf}$ | length of one grout hole, in m; |
| $H_{bai}$ | dam height at section i, in m; and |
| $H_{bl}$ | height of the dam freeboard, in m. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- leveling (dental) concrete: 113.00/$m^3$
- cleaning of rock surface: 39.70/$m^2$

- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

When they are needed, the **length of relief holes per meter of dam at section i**, $L_{tf}$ (m/m), is given by:

$$L_{tf} = \frac{1}{10.0} \times (H_{bai} - H_{bl})$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m; and |
| $H_{bl}$ | 4.0m, height of the dam freeboard. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of earth surface: 4.96/m$^2$
- cleaning of rock surface: 39.70/m$^2$
- rotary percussive drilling: 168.00/m
- dental concrete: 113.00/m$^3$

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Compacted Earthfill (account .12.17.25.24)

The **volume of compacted earthfill per meter of dam at section i**, $V_{ai}$ (m$^3$/m), is given by:

$$V_{ai} = V_{abi} + V_{ari} + V_{api}$$

where:

$$V_{abi} = 2.75 \times H_{bai}^2 + 4.25 \times H_{bai} + 10 - 4.74 \times d_p$$

$$V_{ari} = (B_{tri} + H_{tr}) \times H_{tr} \qquad V_{api} = B_{tpi} \times e_{tpi}$$

$$d_p = H_{bl} + d + 4 \leq H_{bai}$$

where:

| | |
|---|---|
| $V_{abi}$ | volume of earthfill for the dam at section i, in m$^3$/m; |
| $V_{ari}$ | volume of earthfill for the cutoff trench at section i, in m$^3$/m; |
| $V_{api}$ | volume of earthfill for the impervious blanket at section i, in m$^3$/m; |
| $H_{bai}$ | dam height at section i, in m; |
| $d_p$ | effective distance, in m; |
| $B_{tri}$ | width of the base of the cutoff trench at section i, in m; |
| $H_{tr}$ | depth of the cutoff trench at section i, in m; |
| $B_{tpi}$ | width of the impervious blanket at section i, in m; |
| $e_{imi}$ | thickness of the impervious blanket at section i, in m; |
| $H_{bl}$ | 4.0m, height of the dam freeboard; and |
| d | maximum reservoir drawdown, in m. |

The unit price of **compacted earthfill** is R$ 2.69/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the earthfill as defined by the design lines of the dam, and includes only spreading and compacting services and basically depends on the source of the material.

Below are the unit prices for earthfill-related services (in Reais, from December 2006 database) which can be used for projects in the south, southeast, central west and northeast regions of Brazil:

- stock handling: 6.98/m³
- excavation of soil in borrow areas: 8.66/m³
- extra for earth transportation: 2.29/m³.km

Whenever an intermediate stockpile has to be used, the cost of the volume in question will relate to the replenishment and transportation services. If there should be a shortage of material, soil should be excavated from a borrow area, to which the cost of clearing the vegetation from the area, excavation, loading, transportation up to 1.5 km and unloading will be added. When the mandatory excavation sites are further away than 1.5 km, the cost of the transportation will be added to the volume cost.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Transitions, filters and drains (account .12.17.25.29)

The **volume of vertical filter per meter of dam at section i**, $V_{vi}$ (m³/m), is given by:

$$V_{vi} = 2 \times (H_{bai} - 5)$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m. |

The **volume of horizontal drain per meter of dam at section i**, $V_{hi}$ (m³/m), is given by:

$$V_{hi} = 3.75 \times H_{bai}$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m. |

The unit price of **drains and filters** is R$ 26.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the earth or rockfill as defined by the design lines of the dam, and includes producing artificial sand and gravel by processing the excavated rock, as well as transportation, spreading and compacting. Should natural sand have to be used, a specific market survey should be done or unit prices already charged in the region should be used, adding R$ 9.86/m³ to the cost for the loading, local transportation, unloading and compacting services, based on the same database.

In order to obtain the final price of the compacted material and in view of the fact that the proportion of its component parts and services will considerably affect the unit price of the earthfill, an estimate should be made on a case-by-case basis of the proportion of volumes per type of service and origin of materials, which will give the mean weighted unit price.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Rip-Rap for the Upstream Face (account .12.17.25.32.18)

The **volume of rip-rap per meter of dam at section i**, $V_{pi}$ ($m^3/m$), is given by:

$$V_{pi} = 4.74 \times d_p$$

where:

$$d_p = H_{bl} + d + 4 \leq H_{bai}$$

where:

| | |
|---|---|
| $d_p$ | effective distance, in m; |
| $H_{bl}$ | 4.0m, height of the dam freeboard; |
| d | maximum reservoir drawdown, in m; and |
| $H_{bai}$ | dam height at section i, in m. |

The unit price of **rip-rap** is R$ 10.40/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the rip-rap as defined by the design lines of the dam, and includes only spreading and compacting services and basically depends on the source of the material.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Protection of the downstream face (account .12.17.25.32.19)

The **area of grass per meter of dam at section i**, $A_{gi}$ ($m^2/m$), is given by:

$$A_{gi} = 2.69 \times H_{bai} - 4$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m. |

The unit price for **grass** is R$ 5.90/m² (December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per square meter measured using the cross-section as defined by the design lines of the dam, and includes supplying and laying the grass.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Other Costs (account .12.17.25.17)

**Other costs** are estimated as accounting for 2% of the total cost of the corresponding structure.

## ROCKFILL DAMS WITH A CENTRAL CLAY CORE (ACCOUNT .12.17.25)

### Basic Data

The main information required for dimensioning and quantification purposes can be obtained from the overall layout, as follows:

- maximum normal water level in the reservoir, $NA_{max}$, from item 4.6;
- mean elevation of the land in section i perpendicular to the longitudinal dam axis, $El_{tei}$, in m;
- mean thickness of the layer of soil in the dam area, $e_{tei}$ in m; slope of the upstream face, horizontal distance for a 1.0 m difference in level, $m_m$, in m;
- slope of the downstream face, horizontal distance for a 1.0 m difference in level, $m_j$, in m; and
- distance between sections i and i-1, $\Delta L_i$, in m.

## Considerations and recommendations

**This text** relates to dams with a typical cross-section, as shown in Fig. 5.7.4.02.

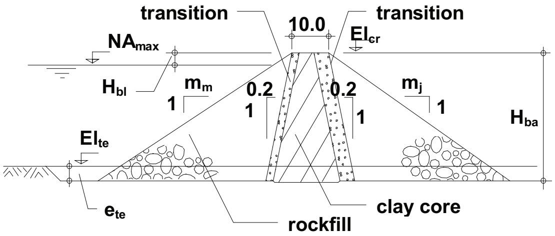



Fig. 5.7.4.02 – Typical cross-section of a rockfill dam with a central clay core.

**Sections should be selected** over contour lines and at important features, like the bottom of saddles, tops of hills, river banks and at any points of contact, such as at intakes or spillways.

The thickness of the **layer of soil** on the river bed may be different from the mean thickness on the abutments, or there may be no soil at all.

Quantities per type of service

The **quantities per type of service**, Qtd, are given by the general expression:

$$Qtd = \sum_i \frac{Sec_i + Sec_{i-1}}{2} \times \Delta L_i$$

where:

| | |
|---|---|
| $Sec_i$ | length, area or volume per meter of dam at section i, in m/m, $m^2$/m or $m^3$/m; and |
| $\Delta L_i$ | distance between sections i and i-1, in m. |

The **height of the dam in section i**, $H_{bai}$ (m), is given by:

$$H_{bai} = El_{cr} - (El_{tei} - e_{tei})$$

where:

$$El_{cr} = NA_{max} + H_{bl}$$

| $H_{bl}$ | For |
|---|---|
| 3.0 | dams with a maximum height of less than 20 m and a reservoir of less than 50 $km^2$ |
| 4.0 | all other cases |

where:

| | |
|---|---|
| $El_{cr}$ | elevation of the dam crest; |
| $El_{tei}$ | mean elevation of the land in section i perpendicular to the longitudinal dam axis; |
| $e_{tei}$ | thickness of the layer of soil at section i of the dam, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $H_{bl}$ | height of the dam freeboard, in m. |

## Common excavation in borrow areas (account .12.17.25.12.10)

The **volume of common excavation per meter of dam at section i**, $V_{ti}$ ($m^3$/m), is given by:

$$V_{ti} = \left[ (m_m + m_j) \times H_{bai} + 30 + e_{tei} \right] \times e_{tei}$$

where:

| m | for |
|---|---|
| 1.30 | low dam at a site with good quality foundations and with no intermediate berms |
| 1.75 | very high dam at a site with poor quality foundations and with intermediate berms |

where:

| | |
|---|---|
| $m_m$ | slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $m_j$ | slope of the downstream face, in m; |
| $H_{bai}$ | dam height at section i, in m; and |
| $e_{tei}$ | thickness of the layer of soil at section i of the dam, in m. |

The **volume of common excavation in borrow areas**, $V_{tp}$ ($m^3$), is given by:

$$V_{tp} = V_n - 0.9 \times \sum V_{tnj} \geq 0$$

where:

| | |
|---|---|
| $V_n$ | volume of clay core, in $m^3$; and |
| $V_{tnj}$ | volume of common excavation for structures j that is suitable for the core, in $m^3$. |

The unit price of common excavation is R$ 7.60/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface rock excavation (account .12.17.25.12.11)

The volume necessary for leveling is calculated as part of foundation treatment.

If there is any need to excavate below the shells or if cutoff trenches are required, this volume should be estimated and added to this account.

The volume of rock excavated from quarries, $V_{rp}$ ($m^3$), is given by:

$$V_{rp} = \frac{\sum V_{ej}}{1.3} + \sum V_{cj} - 0.9 \times \left( \sum V_{rj} + \sum V_{sj} \right) \geq 0$$

where:

| | |
|---|---|
| $V_{ej}$ | volume of rockfill for structures j, in $m^3$; |
| 1.3 | coefficient assuming 30% increase in volume of rock after excavation; |
| $V_{cj}$ | volume of concrete for structures j, in $m^3$; |
| 0.9 | coefficient considering a 10% loss; |
| $V_{rj}$ | volume of excavated rock for structures j, in $m^3$; and |
| $V_{sj}$ | volume of rock excavated for underground structures j, in $m^3$. |

The unit price of **excavation in rock** is R$ 21.00/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.17.25.13)

The area of foundation to be cleaned **per meter of dam at section i**, $A_{lfi}$ ($m^2$/m), is given by:

$$A_{lfi} = (m_m + m_j) \times H_{bai} + 10$$

where:

| | |
|---|---|
| $m_m$ | slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $m_j$ | slope of the downstream face, in m; and |
| $H_{bai}$ | dam height at section i, in m. |

The **volume of exavation and concrete leveling** (without reinforcement steel but with 200 kg cement per $m^3$) **under the core, per meter of dam at section i**, $V_{TFi}$ ($m^3$/m), is given by:

$$V_{TFi} = 0.2 \times H_{bai} + 2$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m. |

The **length of the grout curtain per meter of dam at section i**, $L_{tf}$ (m/m), is given by:

$$L_{tf} = \frac{1}{3.0} \times L_{1tf}$$

$$L_{1tf} = H_{bai} - H_{bl} \leq 40\text{m}$$

where:

| | |
|---|---|
| $L_{1tf}$ | length of one grout hole, in m; |
| $H_{bai}$ | dam height at section i, in m; and |
| $H_{bl}$ | height of the dam freeboard, in m. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- surface rock excavation: 21.00/ $m^3$
- leveling (dental) concrete: 113.00/$m^3$
- cleaning of rock surface: 39.70/$m^2$
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Rockfill (account .12.17.25.25)

The **volume of rockfill per meter of dam at section i**, $V_{ei}$ ($m^3/m$), is given by:

$$V_{ei} = \left( \frac{m_m + m_j - 0.4}{2} \right) \times H_{bai}^2$$

where:

| | |
|---|---|
| $m_m$ | slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $m_j$ | slope of the downstream face, in m; and |
| $H_{bai}$ | dam height at section i, in m. |

The unit price of **rockfill** is R$ 10.40/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the rip-rap as defined by the design lines of the dam, and includes only spreading and compacting services and basically depends on the source of the material.

Below are the unit prices for rockfill-related services (in Reais, from December 2006 database) which can be used for projects in the south, southeast, central west and northeast regions of Brazil:

- stock handling: 6.98/$m^3$
- excavation in quarries: 14.43/$m^3$
- rockfill transportation: 2.21/$m^3$.km

Whenever an intermediate stockpile has to be used, the cost of the volume in question will relate to the replenishment and transportation services. If there should be a shortage of material, rockfill should be excavated in a quarry, to which the cost of clearing the vegetation from the area, excavation, loading, transportation up to 1.5 km and unloading will be added. When the mandatory excavation sites are further away than 1.5 km, the cost of the transportation will be added to the volume cost.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Clay Core (account .12.17.25.26)

The **volume of clay core per meter of dam at section i**, $V_{ni}$ ($m^3/m$), is given by:

$$V_{ni} = 0.2 \times H_{bai}^2 + 4 \times H_{bai}$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m. |

The unit price of the **clay core** is R$ 11.10/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the earthfill as defined by the design lines of the dam, and includes only spreading and compacting services and basically depends on the source of the material.

Below are the unit prices for earthfill-related services (in Reais, from December 2006 database) which can be used for projects in the south, southeast, central west and northeast regions of Brazil:

- stock handling: 6.98/$m^3$
- excavation of soil in borrow areas: 8.66/$m^3$
- transportation of soil: 2.29/$m^3$.km

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Transitions, Drains and Filters (account .12.17.25.29)

The **volume of transitions, drains and filters per meter of dam at section i**, $V_{vi}$ ($m^3/m$), is given by:

$$V_{vi} = 6 \times H_{bai}$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m. |

The unit price of **transitions, drains and filters** is R$ 26.00/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the earth or rockfill as defined by the design lines of the dam, and includes producing artificial sand and gravel by processing the excavated rock, as well as transportation, spreading and compacting. Should natural sand have to be used, a specific market survey should be done or unit prices already charged in the region should be used, adding R$ 9.86/$m^3$ to the cost for the loading, local transportation, unloading and compacting services, based on the same database.

In order to obtain the final price of the compacted material and in view of the fact that the proportion of its component parts and services will considerably affect the unit price of the earthfill, an estimate should be made on a case-by-case basis of the proportion of volumes per type of service and origin of materials, which will give the mean weighted unit price.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Other Costs (account .12.17.25.17)

**Other costs** are estimated as accounting for 2% of the total cost of the corresponding structure.

## ROCKFILL DAMS WITH AN INCLINED CLAY CORE (ACCOUNT .12.17.25)

### Basic Data

The main information required for dimensioning and quantification purposes can be obtained from the overall layout, as follows:

- maximum normal water level in the reservoir, $NA_{max}$, from item 4.6, in m;
- mean elevation of the land in section i perpendicular to the longitudinal dam axis, $El_{tei}$, in m;
- thickness of the layer of soil at section i of the dam, $e_{tei}$, in m;
- slope of the upstream face, horizontal distance for a 1.0 m difference in level, $m_m$, in m;
- slope of the downstream face, horizontal distance for a 1.0 m difference in level, $m_j$, in m; and
- distance between sections i and i-1, $\Delta L_i$, in m.

### Considerations and recommendations

**This text** relates to dams with a typical cross-section, as shown in Fig. 5.7.4.03.

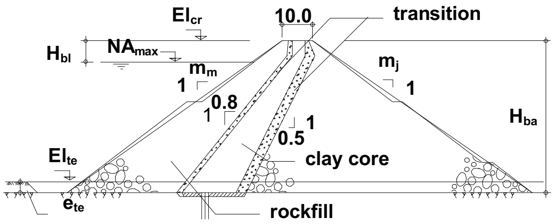



Fig. 5.7.4.03 – Typical cross-section of a rockfill dam with inclined clay core.

**Sections should be selected** over contour lines and at important features, like the bottom of saddles, tops of hills, river banks and at any points of contact, such as at intakes or spillways.

The thickness of the layer of soil on the river bed may be different from the mean thickness on the abutments, or there may be no soil at all.

## Quantities per type of service

The **quantities per type of service**, Qtd, are given by the general expression:

$$Qtd = \sum_i \frac{Sec_i + Sec_{i-1}}{2} \times \Delta L_i$$

where:

| | |
|---|---|
| $Sec_i$ | length, area or volume per meter of dam at section i, in m/m, m$^2$/m or m$^3$/m; and |
| $\Delta L_i$ | distance between sections i and i-1, in m. |

The height of the dam in section i, $H_{bai}$ (m), is given by:

$$H_{bai} = El_{cr} - (El_{tei} - e_{tei})$$

where: $El_{cr} = NA_{max} + H_{bl}$

| $H_{bl}$ | For |
|---|---|
| 3.0 | dams with a maximum height of less than 20 m and a reservoir of less than 50 km$^2$ |
| 4.0 | all other cases |

where:

| | |
|---|---|
| $El_{cr}$ | elevation of the dam crest, in m; |
| $El_{tei}$ | mean elevation of the land in section i perpendicular to the longitudinal dam axis, in m; |
| $e_{tei}$ | thickness of the layer of soil at section i of the dam, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir; and |
| $H_{bl}$ | height of the dam freeboard, in m. |

## Common excavation in borrow areas (account .12.17.25.12.10)

The **volume of common excavation per meter of dam at section i**, $V_{ti}$ (m$^3$/m), is given by:

$$V_{ti} = \left[(m_m + m_j) \times H_{bai} + 30 + e_{tei}\right] \times e_{tei}$$

where:

| m | For |
|---|---|
| 1.30 | low dam at a site with good quality foundations and with no intermediate berms |
| 1.75 | very high dam at a site with poor quality foundations and with intermediate berms |

where:

| | |
|---|---|
| $m_m$ | slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $m_j$ | slope of the downstream face, in m; |
| $H_{bai}$ | dam height at section i, in m; and |
| $e_{tei}$ | thickness of the layer of soil at section i of the dam, in m. |

The **volume of common excavation in borrow areas**, $V_{tp}$ ($m^3$), is given by:

$$V_{tp} = V_n - 0.9 \times \sum V_{tnj} \geq 0$$

where:

| | |
|---|---|
| $V_n$ | volume of clay core, in $m^3$; and |
| $V_{tnj}$ | volume of common excavation for structures j with material that is suitable for the core, in $m^3$. |

The unit price of common excavation is R\$ 7.60/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation (account .12.17.25.12.11)

The volume necessary for leveling is calculated as part of the foundation treatment.

If there is any need to excavate below the shells or if cutoff trenches are required, this volume should be estimated and added to this account.

The **volume of rock excavated from quarries**, $V_{rp}$ ($m^3$), is given by:

$$V_{rp} = \frac{\sum V_{ej}}{1.3} + \sum V_{cj} - 0.9 \times \left(\sum V_{rj} + \sum V_{sj}\right) \geq 0$$

where:

| | |
|---|---|
| $V_{ej}$ | volume of rockfill for structures j, in $m^3$; |
| 1.3 | coefficient assuming 30% increase in volume of rock after excavation; |
| $V_{cj}$ | volume of concrete for structures j, in $m^3$; |
| 0.9 | coefficient considering a 10% loss; |
| $V_{rj}$ | volume of excavated rock for structures j, in $m^3$; and |
| $V_{sj}$ | volume of rock excavated for underground structures j, in $m^3$. |

The unit price for **excavation in rock** is R\$ 21.00/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter of excavated material and includes clearing the vegetation from the area, excavating, loading, transporting up to 1.5 km and unloading.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.17.25.13)

The **area of foundation to be cleaned per meter of dam at section i**, $A_{lfi}$ ($m^2$/m), is given by:

$$A_{lfi} = (m_m + m_j) \times H_{bai} + 10$$

where:

| | |
|---|---|
| $m_m$ | slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $m_j$ | slope of the downstream face, in m; and |
| $H_{bai}$ | dam height at section i, in m. |

The **volume of excavated material and concrete leveling** (without reinforcement steel but with 200 kg cement per m$^3$) **under the core, per meter of dam at section i**, $V_{TFi}$ (m$^3$/m), is given by:

$$V_{TFi} = 0.15 \times H_{bai} + 4.8$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m. |

The **length of grout curtain per meter of dam at section i**, $L_{tf}$ (m/m), is given by:

$$L_{tf} = \frac{1}{3.0} \times (H_{bai} - H_{bl})$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m; and |
| $H_{bl}$ | height of the dam freeboard, in m. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- surface rock excavation: 21.00/ m$^3$
- leveling (dental) concrete: 113.00/m$^3$
- cleaning of rock surface: 39.70/m$^2$
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount as identified in market research.

### Rockfill (account .12.17.25.25)

The **volume of rockfill per meter of dam at section i**, $V_{ei}$ (m$^3$/m), is given by:

$$V_{ei} = \left( \frac{m_m + m_j - 0.3}{2} \right) \times H_{bai}^2 + 0.45 \times H_{bai} + 5$$

where:

| | |
|---|---|
| $m_m$ | slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $m_j$ | slope of the downstream face, in m; and |
| $H_{bai}$ | dam height at section i, in m. |

The unit price of **rockfill** is R$ 10.40/m$^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the rockfill as defined by the design lines of the dam, and includes only spreading and compacting services and basically depends on the source of the material.

Below are the unit prices for rockfill-related services (in Reais, from December 2006 database) which can be used for projects in the south, southeast, central west and northeast regions of Brazil:

- stock handling: 6.98/m$^3$
- excavation in quarries: 14.43/m$^3$
- rockfill transportation: 2.21/m$^3$.km

Whenever an intermediate stockpile has to be used, the cost of the volume in question will relate to the replenishment and transportation services. If there should be a shortage of material, rockfill should be excavated in a quarry, to which the cost of clearing the vegetation from the area, excavation, loading, transportation up to 1.5 km and unloading will be added. When the mandatory excavation sites are further away than 1.5 km, the cost of the transportation will be added to the volume cost.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Clay Core (account .12.17.25.26)

The **volume of clay core per meter of dam at section i**, $V_{ni}$ (m³/m), is given by:

$$V_{ni} = 0.15 \times H_{bai}^2 + 3.55 \times H_{bai} + 2$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m. |

The unit price of the **clay core** is R$ 11.10/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the earthfill as defined by the design lines of the dam, and includes only spreading and compacting services and basically depends on the source of the material.

Below are the unit prices for earthfill-related services (in Reais, from December 2006 database) which can be used for projects in the south, southeast, central west and northeast regions of Brazil:

- stock handling: 6.98/m³
- excavation of soil in borrow areas: 8.66/m³
- transportation of soil: 2.29/m³.km

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Transitions, Filters and Drains (account .12.17.25.29)

The **volume of transitions, filters and drains per meter of dam at section i**, $V_{vi}$ (m³/m), is given by:

$$V_{vi} = 6 \times H_{bai} - 7$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m. |

The unit price of **filters and drains** is R$ 26.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the earth or rockfill as defined by the design lines of the dam, and includes producing artificial sand and gravel by processing the excavated rock, as well as transportation, spreading and compacting. Should natural sand have to be used, a specific market survey should be done or unit prices already charged in the region should be used, adding R$ 9.86/m³ to the cost for the loading, local transportation, unloading and compacting services, based on the same database.

In order to obtain the final price of the compacted material and in view of the fact that the proportion of its component parts and services will considerably affect the unit price of the earthfill, an estimate should be made on a case-by-case basis of the proportion of volumes per type of service and origin of materials, which will give the mean weighted unit price.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Other Costs (account .12.17.25.17)

**Other costs** are estimated as accounting for 2% of the total cost of the corresponding structure.

## CONCRETE-FACED ROCKFILL DAMS (ACCOUNT .12.17.25)

### Basic Data

The main information required for dimensioning and quantification purposes can be obtained from the overall layout, as follows:

- maximum normal water level in the reservoir, $NA_{max}$, from item 4.6, in m;
- mean elevation of the land in section i perpendicular to the longitudinal dam axis, $El_{tei}$, in m;
- thickness of the layer of soil at section i of the dam, $e_{tei}$, in m;
- slope of the upstream face, horizontal distance for a 1.0 m difference in level, $m_m$ in m;
- slope of the downstream face, horizontal distance for a 1.0 m difference in level, $m_j$ in m; and
- distance between sections i and i-1, $\Delta L_i$ in m.

### Considerations and recommendations

**This text relates to dams with a typical cross-section, as shown in** Fig. 5.7.4.04.

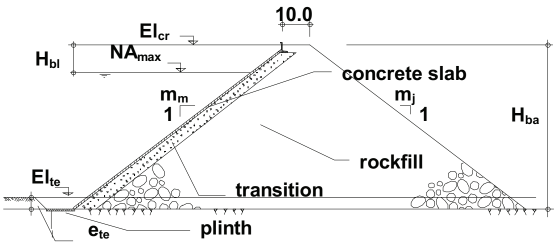



Fig. 5.7.4.04 – Typical cross-section of a concrete-faced rockfill dam.

**Sections should be selected** over contour lines and at important features, like the bottom of saddles, tops of hills, river banks and at any points of contact, such as at intakes or spillways.

The thickness of the layer of soil on the river bed may be different from the mean thickness on the abutments, or there may be no soil at all.

### Quantities per type of service

The **quantities per type of service**, Qtd, are given by the general expression:

$$Qtd = \sum_i \frac{Sec_i + Sec_{i-1}}{2} \times \Delta L_i$$

where:

| | |
|---|---|
| $Sec_i$ | length, area or volume per meter of dam at section i, in m/m, $m^2$/m or $m^3$/m; and |
| $\Delta L_i$ | distance between sections i and i-1, in m. |

The height of the dam in section i, $H_{bai}$ (m), is given by:

$$H_{bai} = El_{cr} - (El_{tei} - e_{tei})$$

where: $El_{cr} = NA_{max} + H_{bl}$

| $H_{bl}$ | For |
|---|---|
| 3.0 | dams with a maximum height of less than 20 m and a reservoir of less than 50 km² |
| 4.0 | all other cases |

where:

| | |
|---|---|
| $El_{cr}$ | elevation of the dam crest; |
| $El_{tei}$ | mean elevation of the land in section i perpendicular to the longitudinal dam axis; |
| $e_{tei}$ | mean thickness of the layer of soil in the dam area, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $H_{bl}$ | height of the dam freeboard, in m. |

### Common excavation in borrow areas (account .12.17.25.12.10)

The **volume of common excavation per meter of dam at section i**, $V_{ti}$ ($m^3/m$), is given by:

$$V_{ti} = \left[ \left( m_m + m_j \right) \times H_{bai} + 33 + e_{tei} \right] \times e_{tei}$$

where:

| m | For |
|---|---|
| 1.3 | low dam at a site with good quality foundations |
| 1.5 | very high dam at a site with poor quality foundations |

where:

| | |
|---|---|
| $m_m$ | slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $m_j$ | slope of the downstream face, in m; |
| $H_{bai}$ | dam height at section i, in m; and |
| $e_{tei}$ | thickness of the layer of soil at section i of the dam, in m. |

The unit price of common excavation is R$ 7.60/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface rock excavation (account .12.17.25.12.11)

If there is any need to excavate below the shells or if cutoff trenches are required, this volume should be estimated and added to this account.

The **volume of rock excavated from quarries**, $V_{rp}$ ($m^3$), is given by:

$$V_{rp} = \frac{\sum V_{ej}}{1.3} + \sum V_{cj} - 0.9 \times \left( \sum V_{rj} + \sum V_{sj} \right) \geq 0$$

where:

| | |
|---|---|
| $V_{ej}$ | volume of rockfill for structures j, in $m^3$; |
| 1.3 | coefficient assuming 30% increase in volume of rock after excavation; |
| $V_{cj}$ | volume of concrete for structures j, in $m^3$; |
| 0.9 | coefficient considering a 10% loss; |
| $V_{rj}$ | volume of excavated rock for structures j, in $m^3$; and |
| $V_{sj}$ | volume of rock excavated for underground structures j, in $m^3$. |

The unit price for **excavation in rock** is R$ 21.00/m$^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter of excavated material and includes clearing the vegetation from the area, excavating, loading, transporting up to 1.5 km and unloading.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.17.25.13)

The area of foundation to be cleaned **per meter of dam at section i**, $A_{lfi}$ (m$^2$/m), is given by:

$$A_{lfi} = (m_m + m_j) \times H_{bai} + 13$$

where:

| | |
|---|---|
| $m_m$ | slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $m_j$ | slope of the downstream face, in m; and |
| $H_{bai}$ | dam height at section i, in m. |

The **length of rock anchors per meter of dam at section i**, $L_{tfci}$ (m/m), is given by:

$$L_{tfci} = 13.3$$

The **length of the grout holes per meter of dam at section i**, $L_{tfi}$ (m/m), is given by:

$$L_{tfi} = \frac{1}{3.0} \times (H_{bai} - H_{bl})$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m; and |
| $H_{bl}$ | height of the dam freeboard, in m. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- surface rock excavation: 21.00/ m$^3$
- leveling (dental) concrete: 113.00/m$^3$
- cleaning of rock surface: 39.70/m$^2$
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Rockfill (account .12.17.25.25)

The **volume of rockfill per meter of dam at section i**, $V_{ei}$ (m$^3$/m), is given by:

$$V_{ei} = \left( \frac{m_m + m_j - 0.035}{2} \right) \times H_{bai}^2 + 2.47 \times H_{bai} - 12$$

where:

| | |
|---|---|
| $m_m$ | slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $m_j$ | slope of the downstream face, in m; and |
| $H_{bai}$ | dam height at section i, in m. |

The unit price of **rockfill** is R$ 10.40/m$^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the rockfill as defined by the design lines of the dam, and includes only spreading and compacting services and basically depends on the source of the material.

Below are the unit prices for rockfill-related services (in Reais, from December 2006 database) which can be used for projects in the south, southeast, central west and northeast regions of Brazil:

- stock handling: 6.98/m$^3$
- excavation in quarries: 14.43/m$^3$
- extra transportation of rockfill: 2.29/m$^3$.km

Whenever an intermediate stockpile has to be used, the cost of the volume in question will relate to the replenishment and transportation services. If there should be a shortage of material, rockfill should be excavated from a quarry, to which the cost of clearing the vegetation from the area, excavation, loading, transportation up to 1.5 km and unloading will be added. When the mandatory excavation sites are further away than 1.5 km, the cost of the transportation will be added to the volume cost.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account .12.17.25.14)

The **volume of concrete per meter of dam at section i**, $V_{cbi}$ (m$^3$/m), is given by:

$$V_{cbi} = V_{cti} + V_{cli} + V_{cni} + V_{cdi}$$

where

$$V_{cti} = 3.19$$

$$V_{cli} = \sqrt{1+m_m^2} \times \left(0.00179 \times H_{bai}^2 + 0.29 \times H_{bai} - 0.8\right)$$

$V_{cni} = 3.85$ $\qquad$ $V_{cdi} = 2.75$

where:

| | |
|---|---|
| $V_{cti}$ | volume of concrete for the parapet, in m$^3$/m; |
| $V_{cli}$ | volume of concrete for slabs, in m$^3$/m; |
| $V_{cni}$ | volume of concrete for the plinth, in m$^3$/m; |
| $V_{cdi}$ | volume of leveling concrete, in m$^3$/m; |
| $m_m$ | slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m; and |
| $H_{bai}$ | dam height at section i, in m. |

The amounts of cement and reinforcement steel are:

| | cement (kg/m$^3$) | reinforcement steel (kg/m$^3$) |
|---|---|---|
| parapet | 300 | 100 |
| slab and plinth | 250 | 80 |
| leveling | 200 | 0 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- parapet: 474.00/$m^3$
- slab and plinth: 234.00/$m^3$
- leveling: 113.00/$m^3$

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Transitions, Filters and Drains (account .12.17.25.29)

The **volume of the bed of crushed rock per meter of dam at section i**, $V_{vi}$ ($m^3$/m), is given by:

$$V_{vi} = 0.0175 \times H_{bai}^2 + 4.9 \times H_{bai} - 14$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m. |

The unit price of **transitions** is R$ 26.00/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter measured using the cross-section of the earth or rockfill as defined by the design lines of the dam, and includes producing artificial sand and gravel by processing the excavated rock, as well as transportation, spreading and compacting. Should natural sand have to be used, a specific market survey should be done or unit prices already charged in the region should be used, adding R$ 9.86/$m^3$ to the cost for the loading, local transportation, unloading and compacting services, based on the same database.

In order to obtain the final price of the compacted material and in view of the fact that the proportion of its component parts and services will considerably affect the unit price of the earthfill, an estimate should be made on a case-by-case basis of the proportion of volumes per type of service and origin of materials, which will give the mean weighted unit price.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Other Costs (account .12.17.25.17)

**Other costs** are estimated as accounting for 2% of the total cost of the corresponding structure.

## CONVENTIONAL CONCRETE GRAVITY DAMS (ACCOUNT .12.17.26)

### Basic Data

The main information required for dimensioning and quantification purposes can be obtained from the overall layout, as follows:

- maximum normal water level in the reservoir, $NA_{max}$, from item 4.6, in m;
- mean elevation of the land in section i perpendicular to the longitudinal dam axis, $El_{tei}$, in m;
- thickness of the layer of soil at section i of the dam, $e_{tei}$, in m;
- slope of the downstream face, horizontal distance for a 1.0 m difference in level, $m_j$, in m;
- distance between sections i and i-1, $\Delta L_i$, in m;
- elevation of the bottom of the approach channel to the diversion sluiceways, $El_{ca}$, from item 5.7.3, when applicable, in m;
- mean elevation of the land in the sluiceway area, $El_{te}$, when applicable, in m;
- height of sluiceways, $H_{ad}$, in m, from item 5.7.3, when applicable;
- width of one sluiceway, $B_{1ad}$ in m, from item 5.7.3, when applicable;
- number of sluiceways, $N_{ad}$, from item 5.7.3, when applicable; and
- total width of sluiceways, $B_{ad}$ in m, from item 5.7.3, when applicable.

### Considerations and recommendations

**This text** relates to dams with a typical cross-section, as shown in Fig. 5.7.4.05.

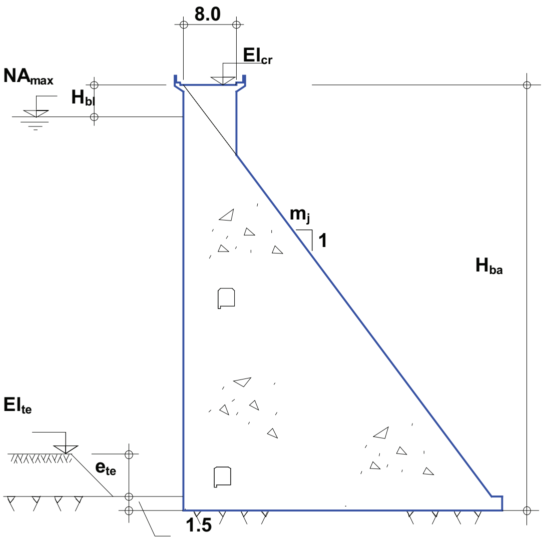



Fig. 5.7.4.05 – Typical cross-section of a conventional concrete gravity dam.

**Sections should be selected** over contour lines and at important features, like the bottom of saddles, tops of hills, river banks and at any points of contact, such as at intakes or spillways.

The thickness of the **layer of soil** on the river bed may be different from the mean thickness on the abutments, or there may be no soil at all.

### Quantities per type of service

The **quantities per type of service**, Qtd, are given by the general expression:

$$Qtd = \sum_i \frac{Sec_i + Sec_{i-1}}{2} \times \Delta L_i$$

where:

| | |
|---|---|
| $Sec_i$ | length, area or volume per meter of dam at section i, in m/m, $m^2/m$ ou $m^3/m$; and |
| $\Delta L_i$ | distance between sections i and i-1, in m. |

The height of the dam in section i, $H_{bai}$ (m), is given by:

$$H_{bai} = El_{cr} - (El_{tei} - e_{tei} - 1.5)$$

where: $El_{cr} = NA_{max} + H_{bl}$

where:

| | |
|---|---|
| $El_{cr}$ | elevation of the dam crest, in m; |
| $El_{tei}$ | mean elevation of the land in section i perpendicular to the longitudinal dam axis, in m; |
| $e_{te}$ | mean thickness of the layer of soil in the dam area, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $H_{bl}$ | 3.0 m – height of the dam freeboard. |

The **height of the dam in the section with the sluiceways**, $H_{ba}$ (m), when applicable, is given by:

$$H_{ba} = El_{cr} - (El_{ca} - 1.5)$$

where:

| | |
|---|---|
| $El_{cr}$ | elevation of the dam crest, in m; |
| $El_{ca}$ | mean elevation of the bottom of the approach channel to the sluiceways, in m; |

### Common Excavation (account .12.17.26.12.10)

The **volume of common excavation per meter of dam at section i**, $V_{ti}$ ($m^3$/m), is given by:

$$V_{ti} = (m_j \times H_{bai} + 20 + e_{tei}) \times e_{tei}$$

where:

| | |
|---|---|
| $m_j$ | 0.75 – slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m, for good quality foundations; |
| $H_{bai}$ | dam height at section i, in m; and |
| $e_{te}$ | mean thickness of the layer of soil in the dam area, in m. |

The unit price of common excavation is R\$ 7.60/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation (account .12.17.26.12.11)

The **volume of excavated rock per meter of dam at section i**, $V_{ri}$ ($m^3$/m), is given by:

$$V_{ri} = m_j \times H_{bai} \times 1.5$$

where:

| | |
|---|---|
| $m_j$ | slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; and |
| $H_{bai}$ | dam height at section i, in m. |

If there are sluiceways through the body of the dam, the **extra volume of material excavated in rock**, $V_{res}$ ($m^3$), should be defined by the expression:

$$V_{res} = \left[ m_j \times H_{ba} \times (El_{te} - e_{te} - El_{ca}) + 17 \right] \times B_{ad}$$

where:

| | |
|---|---|
| $m_j$ | slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $H_{ba}$ | height of the dam at the section with the sluiceway, in m; |
| $El_{te}$ | mean elevation of the land in the sluiceway area, in m; |
| $e_{te}$ | mean thickness of the layer of soil in the dam area, in m; |
| $El_{ca}$ | mean elevation of the bottom of the approach channel to the sluiceways, in m; |
| $B_{ad}$ | total width of sluiceways, in m. |

The unit price of **excavation in rock** is R$ 21.00/m$^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.17.26.13)

The **area of foundation to be cleaned per meter of dam at section i**, $A_{lfi}$ (m$^2$/m), is given by:

$$A_{lfi} = m_j \times H_{bai}$$

where:

| | |
|---|---|
| $m_j$ | slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; and |
| $H_{bai}$ | dam height at section i, in m. |

The **length of the grout holes per meter of dam at section i**, $L_{tfi}$ (m/m), is given by:

$$L_{tfi} = \frac{1}{3.0} \times (H_{bai} - H_{bl})$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m; and |
| $H_{bl}$ | 3.0 m – height of the dam freeboard. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m$^2$
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

## Concrete (account .12.17.26.14)

The **volume of concrete per meter of dam at section i**, $V_{cbi}$ ($m^3/m$), is given by:

$$V_{cbi} = V_{cmi} + V_{cti}$$

where

$$V_{cmi} = \frac{m_j}{2} \times H_{bai}{}^2 + \frac{32}{m_j} + 1.13 \times m_j - 6.7$$

$$V_{cti} = 2.5$$

where:

| | |
|---|---|
| $V_{cmi}$ | volume of concrete for the body of the dam, in $m^3/m$; |
| $V_{cti}$ | volume of concrete for the parapet, in $m^3/m$; |
| $m_j$ | slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; and |
| $H_{bai}$ | dam height at section i, in m. |

If there are sluiceways through the body of the dam, the **extra volume of concrete for the sluiceways**, $V_{cad}$ ($m^3$), should be defined by the expression:

$$V_{cad} = V_{cac} + V_{cpl} - V_{cae} + V_{cat}$$

where:

$$V_{cac} = (0.24 \times H_{ad} + 15) \times B_{ad}$$

$$V_{cpl} = (0.16 \times H_{ad}^2 + 2.7 \times H_{ad} + 8) \times (N_{ad} + 1) \times e_{pa}$$

$$V_{cae} = (0.38 \times H_{ad} + 0.2) \times H_{ad} \times N_{ad} \times B_{1ad}$$

$$V_{cat} = \frac{\left[NA_{max} - \left(EL_{ca} + \frac{H_{ad}}{2}\right)\right] \times B_{1ad} \times H_{ad}}{(2 \times H_{ad} + B_{1ad}) \times 6} \times H_{ad} \times N_{ad} \times B_{1ad}$$

where:

| | |
|---|---|
| $V_{cac}$ | volume of concrete for part of the sluiceway sills, in $m^3$; |
| $V_{cpl}$ | volume of concrete for the sluiceway walls upstream from the face of the dam, in $m^3$; |
| $V_{cae}$ | volume of concrete for the sluiceway inlets, in $m^3$; |
| $V_{cat}$ | volume of concrete for the sluiceway plugs, in $m^3$ |
| $H_{ad}$ | height of sluiceways, in m; |
| $B_{ad}$ | total width of sluiceways, in m; |
| $e_{pa}$ | thickness of the sluiceway walls, in m; |
| $N_{ad}$ | number of sluiceways; and |
| $B_{1ad}$ | width of one sluiceway, in m. |

The calculation for subtracting the **volume of empty space for the sluiceways** ($V_{adu}$), is given by:

$$V_{adu} = N_{ad} \times H_{ad} \times B_{1ad} \times m_j \times \left(H_{ba} - \frac{H_{ad}}{2}\right)$$

where:

| | |
|---|---|
| $H_{ad}$ | height of sluiceways, in m; |
| $m_j$ | slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $H_{ba}$ | height of the dam, in m; |
| $N_{ad}$ | number of sluiceways; and |
| $B_{1ad}$ | width of one sluiceway, in m. |

In the case in question, when the river is diverted through the dam, the **volume of concrete with quantities of cement and reinforcement steel greater** than the quantities for the dam, $V_{cen}$ ($m^3$), is given by:

$$V_{cen} = V_{cet} + V_{ces} + V_{cep}$$

where:

$$V_{cet} = m_j \times (H_{ba} - H_{ad} - 1.5) \times 0.25 \times H_{ad} \times B_{ad}$$

$$V_{ces} = 1.5 \times m_j \times H_{ba} \times B_{ad}$$

$$V_{cep} = m_j \times \left( H_{ba} - \frac{H_{ad}}{2} - 1{,}5 \right) \times H_{ad} \times (N_{ad} + 1) \times e_{pa} - V_{cae}$$

where:

| | |
|---|---|
| $V_{cet}$ | volume of concrete for slabs above the sluiceways, in $m^3$; |
| $V_{ces}$ | volume of concrete for the sluiceway sills, in $m^3$; |
| $V_{cep}$ | volume of concrete for the walls between the sluiceways, in $m^3$; |
| $m_j$ | slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $H_{ba}$ | height of the dam in the sluiceway area, in m; |
| $H_{ad}$ | height of sluiceways, in m; |
| $B_{ad}$ | Total width of sluiceways, in m; |
| $e_{pa}$ | thickness of the sluiceway walls, in m; |
| $N_{ad}$ | number of sluiceways; and |
| $V_{cae}$ | volume of concrete for the sluiceway inlets, in $m^3$. |

The amounts of cement and reinforcement steel are:

| | cement (kg/m³) | reinforcement steel (kg/m³) |
|---|---|---|
| parapet | 300 | 70 |
| body of the dam and sluiceway inlet | 200 | 10 |
| sill and walls for the sluiceways | 250 | 80 |
| plugs | 220 | 20 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- parapet: 474.00/$m^3$
- body of the dam and sluiceway inlet: 113.00/$m^3$
- sill and walls for the sluiceways: 174.00/$m^3$
- plugs: 174.00/$m^3$

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Other Costs (account .12.17.26.17)

**Other costs** are estimated as accounting for 2% of the total cost of the corresponding structure.

## ROLLER COMPACTED CONCRETE DAMS (ACCOUNT .12.17.26)

### Basic Data

The main information required for dimensioning and quantification purposes can be obtained from the overall layout, as follows:

- slope of the downstream face, horizontal distance for a 1.0 m difference in level, $m_j$ in m; maximum normal water level in the reservoir, $NA_{max}$, from item 4.6, in m;
- mean elevation of the land in section i perpendicular to the longitudinal dam axis, $El_{tei}$, in m;
- thickness of the layer of soil at section i of the dam, $e_{tei}$ in m;
- distance between sections i and i-1, $\Delta L_i$ in m;
- elevation of the bottom of the approach channel to the diversion sluiceways, $El_{ca}$, from item 5.7.3, when applicable, in m;
- mean elevation of the land in the sluiceway area, $El_{te}$, when applicable, in m;
- height of sluiceways, $H_{ad}$ in m, from item 5.7.3, when applicable;
- width of one sluiceway, $B_{1ad}$ in m, from item 5.7.3, when applicable;
- number of sluiceways, $N_{ad}$, from item 5.7.3, when applicable; and
- total width of sluiceways, $B_{ad}$ in m, from item 5.7.3, when applicable.

### Considerations and recommendations

**This text** relates to dams with a typical cross-section, as shown in Fig. 5.7.4.06.

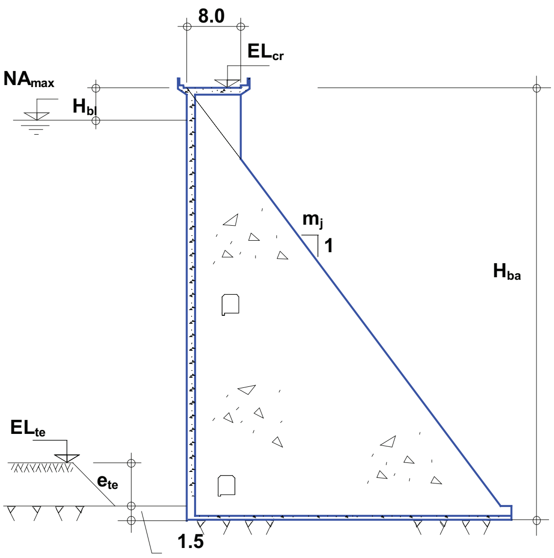



Fig. 5.7.4.06 – Typical cross-section of a roller compacted concrete dam.

**Sections should be selected** over contour lines and at important features, like the bottom of saddles, tops of hills, river banks and at any points of contact, such as at intakes or spillways.

The thickness of the **layer of soil** on the river bed may be different from the mean thickness on the abutments, or there may be no soil at all.

### Quantities per type of service

The **quantities per type of service**, Qtd, are given by the general expression:

$$Qtd = \sum_i \frac{Sec_i + Sec_{i-1}}{2} \times \Delta L_i$$

where:

| | |
|---|---|
| $Sec_i$ | length, area or volume per meter of dam at section i, in m/m, $m^2/m$ or $m^3/m$; and |
| $\Delta L_i$ | distance between sections i and i-1, in m. |

The **height of the dam in section i**, $H_{bai}$ (m), is given by:

$$H_{bai} = El_{cr} - (El_{tei} - e_{tei} - 1.5)$$

where: $El_{cr} = NA_{max} + H_{bl}$

where:

| | |
|---|---|
| $El_{cr}$ | elevation of the dam crest; |
| $El_{tei}$ | mean elevation of the land in section i perpendicular to the longitudinal dam axis; |
| $e_{te}$ | mean thickness of the layer of soil in the dam area, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir; and |
| $H_{bl}$ | 3.0 m – height of the dam freeboard. |

The **height of the dam in the section with the sluiceways**, $H_{ba}$ (m), when applicable, is given by:

$$H_{ba} = El_{cr} - (El_{ca} - 1.5)$$

where:

| | |
|---|---|
| $El_{cr}$ | elevation of the dam crest; and |
| $El_{ca}$ | mean elevation of the bottom of the approach channel to the sluiceways. |

### Common Excavation (account .12.17.26.12.10)

The **volume of common excavation per meter of dam at section i**, $V_{ti}$ ($m^3/m$), is given by:

$$V_{ti} = (m_j \times H_{bai} + 20 + e_{tei}) \times e_{tei}$$

where:

| | |
|---|---|
| $m_j$ | slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $H_{bai}$ | dam height at section i, in m; and |
| $e_{tei}$ | thickness of the layer of soil at section i of the dam, in m. |

The unit price of **common excavation** is R$ 7.60/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation (account .12.17.26.12.11)

The **volume of excavated rock per meter of dam at section i**, $V_{ri}$ ($m^3/m$), is given by:

$$V_{ri} = m_j \times H_{bai} \times 1.5$$

where:

| | |
|---|---|
| $m_j$ | slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; and |
| $H_{bai}$ | dam height at section i, in m. |

If there are sluiceways through the body of the dam, the **extra volume of material excavated in rock**, $V_{res}$ ($m^3$), should be taken into account as shown below:

$$V_{res} = \left[m_j \times H_{ba} \times \left(El_{te} - e_{te} - El_{ca}\right) + 17\right] \times B_{ad}$$

where:

| | |
|---|---|
| $m_j$ | slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $H_{ba}$ | height of the dam in the section with the sluiceway, in m; |
| $El_{te}$ | mean elevation of the land in the sluiceway area; |
| $e_{te}$ | mean thickness of the layer of soil in the dam area, in m; |
| $El_{ca}$ | mean elevation of the bottom of the approach channel to the sluiceways; and |
| $B_{ad}$ | total width of sluiceways, in m. |

The unit price of **excavation in rock** is R$ 21.00/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the dam. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.17.26.13)

The area of foundation to be cleaned **per meter of dam at section i**, $A_{lfi}$ ($m^2/m$), is given by:

$$A_{lfi} = m_j \times H_{bai}$$

where:

| | |
|---|---|
| $m_j$ | slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $H_{bai}$ | dam height at section i, in m. |

The **length of the grout holes per meter of dam at section i**, $L_{tfi}$ (m/m), is given by:

$$L_{tfi} = \frac{1}{3.0} \times \left(H_{bai} - H_{bl}\right)$$

where:

| | |
|---|---|
| $H_{bai}$ | dam height at section i, in m; and |
| $H_{bl}$ | 3.0 m – height of the dam freeboard. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- leveling (dental) concrete: 113.00/$m^3$
- cleaning of rock surface: 39.70/$m^2$
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account .12.17.26.14)

The **volume of concrete per meter of dam at section i**, $V_{cbi}$ ($m^3$/m), is given by:

$$V_{cbi} = V_{cdi} + V_{cpi} + V_{cci} + V_{cti} + V_{cri}$$

where

$$V_{cdi} = m_j \times H_{bai} \times 0.5$$

$$V_{cpi} = H_{bai} \times 0.0794 \times \sqrt{H_{bai} - H_{bl}} \geq 0$$

$$V_{cci} = 8.0 \qquad V_{cti} = 2.5$$

$$V_{cri} = \frac{m_j}{2} \times H_{bai}^2 - \left(0.5 \times m_j + 0.0794 \times \sqrt{H_{bai} - H_{bl}}\right) \times H_{bai} + \frac{32}{m_j} + 1.13 \times m_j - 14.7$$

where:

| | |
|---|---|
| $V_{cdi}$ | volume of leveling concrete, in $m^3$/m; |
| $V_{cpi}$ | volume of concrete for the face, in $m^3$/m; |
| $V_{cci}$ | volume of concrete for the crest, in $m^3$/m; |
| $V_{cti}$ | volume of concrete for the parapet, in $m^3$/m; |
| $V_{cri}$ | volume of roller compacted concrete, in $m^3$/m; |
| $m_j$ | slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $H_{bl}$ | height of the dam freeboard, in m; |
| $H_{bai}$ | dam height at section i, in m. |

If there are sluiceways through the body of the dam, the **extra volume of concrete for the sluiceways**, $V_{cad}$ ($m^3$), should be taken into account as shown below:

$$V_{cad} = V_{cac} + V_{cpl} - V_{cae} + V_{cat} - V_{adu}$$

where:

$$V_{cac} = (0.24 \times H_{ad} + 15) \times B_{ad}$$

$$V_{cpl} = \left(0.16 \times H_{ad}^2 + 2.7 \times H_{ad} + 8\right) \times (N_{ad} + 1) \times e_{pa}$$

$$V_{cae} = (0.38 \times H_{ad} + 0.2) \times H_{ad} \times N_{ad} \times B_{1ad}$$

$$V_{cat} = \frac{\left[NA_{max} - \left(EL_{ca} + \frac{H_{ad}}{2}\right)\right] \times B_{1ad} \times H_{ad}}{(2 \times H_{ad} + B_{1ad}) \times 6} \times H_{ad} \times N_{ad} \times B_{1ad}$$

$$V_{adu} = N_{ad} \times H_{ad} \times B_{1ad} \times m_j \times \left(H_{ba} - \frac{H_{ad}}{2}\right)$$

where:

| | |
|---|---|
| $V_{cac}$ | Volume of concrete for part of the sluiceway sill, in $m^3$; |
| $V_{cpl}$ | Volume of concrete for the sluiceway walls upstream from the face of the dam, in $m^3$; |
| $V_{cae}$ | Volume of concrete for the sluiceway inlets, in $m^3$; |
| $V_{cat}$ | Volume of concrete for the sluiceway plugs, in $m^3$; |
| $V_{adu}$ | Volume of empty space in the sluiceways (before plugging), in $m^3$; |
| $H_{ad}$ | height of sluiceways, in m; |
| $B_{ad}$ | total width of sluiceways, in m; |
| $e_{pa}$ | thickness of the sluiceway walls, in m; |
| $N_{ad}$ | number of sluiceways; and |
| $B_{1ad}$ | width of one sluiceway, in m. |

In the case in question, when the river is diverted through the dam, the **volume of concrete with quantities of cement and reinforcement steel greater** than the quantities for the dam, $V_{cen}$ ($m^3$), is given by:

$$V_{cen} = V_{cet} + V_{ces} + V_{cep}$$

where:

$$V_{cet} = m_j \times (H_{ba} - H_{ad} - 1.5) \times 0.25 \times H_{ad} \times B_{ad}$$

$$V_{ces} = 1{,}5 \times m_j \times H_{ba} \times B_{ad}$$

$$V_{cep} = m_j \times \left(H_{ba} - \frac{H_{ad}}{2} - 1.5\right) \times H_{ad} \times (N_{ad} + 1) \times e_{pa} - V_{cae}$$

where:

| | |
|---|---|
| $V_{cet}$ | volume of concrete for slabs above the sluiceways, in $m^3$; |
| $V_{ces}$ | volume of concrete for the sluiceway sills, in $m^3$; |
| $V_{cep}$ | volume of concrete for the walls between the sluiceways, in $m^3$; |
| $m_j$ | slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $H_{ba}$ | height of the dam in the sluiceway area, in m; |
| $H_{ad}$ | height of sluiceways, in m; |
| $B_{ad}$ | Total width of sluiceways, in m; |
| $e_{pa}$ | thickness of the sluiceway walls, in m; |
| $N_{ad}$ | number of sluiceways; and |
| $V_{cae}$ | volume of concrete for the sluiceway inlets, in $m^3$. |

The volume of **conventional concrete to replace roller-compacted concrete** for the dam, $V_{ccs}$ ($m^3$), is given by:

$$V_{ccs} = m_j \times (H_{ba} \times d_6 - \frac{d_6^2}{2} - \frac{H_{ba}}{2}) \times B_{ad} - V_{cae}$$

where:

$d_6 = 1.25 \times H_{ad} + 1.5$

where:

| | |
|---|---|
| $m_j$ | slope of the downstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $H_{ba}$ | height of the dam in the sluiceway area, in m; |
| $B_{ad}$ | total width of sluiceways, in m; |
| $V_{cae}$ | volume of concrete for the sluiceway inlets to be subtracted from the dam, in $m^3$; |
| $H_{ad}$ | height of sluiceways, in m; and |
| $d_6$ | secondary dimension, in m. |

The amounts of cement and reinforcement steel are:

| | cement (kg/m³) | reinforcement steel (kg/m³) |
|---|---|---|
| parapet | 300 | 70 |
| face | 250 | |
| crest | 250 | 50 |
| leveling | 200 | |
| roller compacted | 100 | |
| sluiceway sills and inlets | 280 | 60 |
| plugs | 220 | 20 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- parapet: 474.00/$m^3$
- face: 234.00/$m^3$
- crest: 113.00/$m^3$
- leveling: 113.00/$m^3$
- roller compacted: 71.00/$m^3$
- sluiceway sills and inlets: 174.00/$m^3$
- sluiceway walls: 174.00/$m^3$
- plugs: 128.00/$m^3$

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Other Costs (account .12.17.26.17)

**Other costs** are estimated as accounting for 2% of the total cost of the corresponding structure.

## CONCRETE RETAINING AND TRANSITION WALLS (ACCOUNT .12.17.27)

### Retaining Walls

The **basic data required for dimensioning purposes** are:

- topographic information;
- geological information;
- mean elevation of the land in the wall area, $El_{te}$;
- mean thickness of the layer of soil in the wall area, $e_{te}$, in m;
- maximum normal water level in the reservoir, $NA_{max}$, from item 4.4;
- height of the dam freeboard, $H_{bl}$ in m, from item 5.7.4;
- slope of the upstream face, $m_m$, in m, and downstream face, $m_j$, in m, horizontal distance for a 1.0 m difference in level, from item 5.7.4.

### Considerations and recommendations

This text relates to retaining walls with a typical cross-section, as shown in Fig. 5.7.4.07.

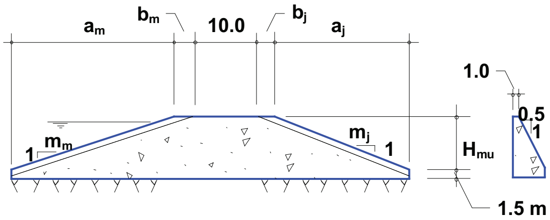



Fig. 5.7.4.07 – Typical cross-section of a retaining wall.

Retaining walls are only recommended for **heights** lower than 11m. For heights greater than 11m, the use of transition walls is recommended.

### Height of the wall

The **height of the wall**, $H_{mu}$ (m), is given by:

$$H_{mu} = NA_{max} + H_{bl} - (El_{te} - e_{te})$$

where:

| | |
|---|---|
| $NA_{max}$ | maximum normal water level in the reservoir; |
| $H_{bl}$ | height of the dam freeboard, in m; |
| $El_{te}$ | mean elevation of the land in the wall area; and |
| $e_{te}$ | mean thickness of the layer of soil in the wall area, in m. |

### Common Excavation (account .12.17.27.12.10)

The **volume of common excavation**, $V_{tmu}$ ($m^3$), is included in the dam account, except for earthfill dams, where it is given by:

$$V_{tmu} = (1.375 \times H_{mu}^2 + 13.25 \times H_{mu} + 15.5) \times e_{te}$$

where:

| | |
|---|---|
| $H_{mu}$ | height of the wall, in m; and |
| $e_{te}$ | mean thickness of the layer of soil in the wall area, in m. |

The unit price of **common excavation** is R$ 7.60/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the wall. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation (account .12.17.27.12.11)

The **volume excavated in rock**, $V_{rmu}$ (m³), is given by:

$$V_{rmu} = 0.375 \times m \times H_{mu}^2 + (2.25 \times m + 7.5) \times H_{mu} + 1.5 \times m + 15$$

where: $m = m_m + m_j$

where:

| | |
|---|---|
| m | secondary variable; |
| $m_m$ | slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $m_j$ | slope of the downstream face, in m; and |
| $H_{mu}$ | height of the wall, in m. |

The unit price of **excavation in rock** is R$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the wall. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.17.27.13)

The **area of foundation to be cleaned**, $A_{lf}$ (m²), is included in the dam account, except for earthfill dams, where it is given by:

$$A_{lf} = 0.25 \times m \times H_{mu}^2 + (1.5 \times m + 5) \times H_{mu} + m + 10$$

where: $m = m_m + m_j$

where:

| | |
|---|---|
| m | secondary variable; |
| $m_m$ | slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $m_j$ | slope of the downstream face, in m; and |
| $H_{mu}$ | height of the wall, in m. |

**Foundation treatment**, $L_{tf}$ (m), is considered as part of the dam.

The unit price for **foundation cleaning** is R$ 39.70/m$^2$ (December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per square meter measured using the design line, and includes the service per se and the supply of inputs and equipment, which will depend on the kind of surface in question.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account .12.17.27.14)

The **volume of concrete for a retaining wall**, $V_{cmu}$ (m$^3$), is given by:

$$V_{cmu} = m \times \frac{H_{mu}^3}{12} + (0.875 \times m + 2.5) \times H_{mu}^2 + (1.5 \times m + 12.5) \times H_{mu} + 15$$

where: $m = m_m + m_j$

where:

| | |
|---|---|
| m | secondary variable; |
| $m_m$ | slope of the upstream face, horizontal distance for a 1.0 m difference in level, in m; |
| $m_j$ | slope of the downstream face, in m; and |
| $H_{mu}$ | height of the wall, in m. |

The amounts of cement and reinforcement steel are:

| | cement (kg/m$^3$) | reinforcement steel (kg/m$^3$) |
|---|---|---|
| structural concrete | 280 | 50 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit price for **concrete without cement** is R$ 113.00/m$^3$ (December 2006 database) and is valid for projects in the south, southeast, central west and northeast regions of Brazil. This is the unit price per cubic meter of volume of the wall and includes all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment.

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Transition Walls

The basic data used for this item are:

- topographic information;
- geological information;
- mean elevation of the land in the wall area, $El_{te}$;

- mean thickness of the layer of soil in the wall area, $e_{te}$ in m;
- maximum normal water level in the reservoir, $NA_{max}$, from item 4.6;
- height of the dam freeboard, $H_{bl}$ in m, from item 5.7.4;
- slope of the upstream face, $m_m$ in m, and downstream face, $m_j$ in m, horizontal distance for a 1.0 m difference in level, from item 5.7.4.

### Considerations and recommendations

This text relates to transition walls with a typical cross-section, as shown in Fig. 5.7.4.08.

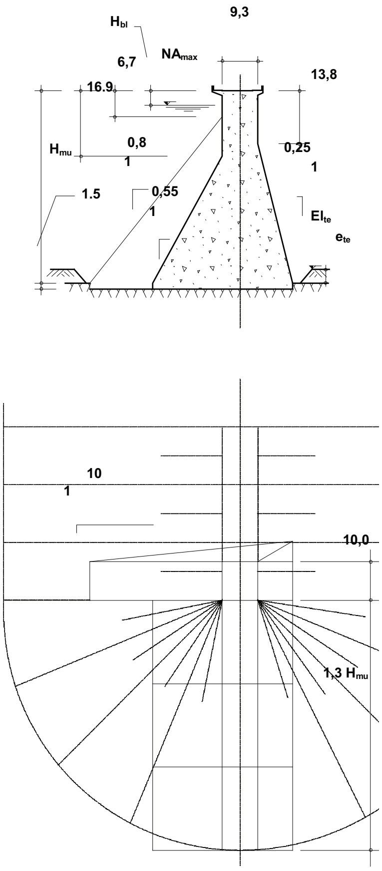



Fig. 5.7.4.08 – Typical cross-section of a transition wall.

Transition walls are recommended for heights of 11m or over. When the height is less than 11m, the use of retaining walls is recommended. However, when the wall is adjoining an intake, a transition wall should be used no matter how high it is.

### Height of the wall

The **height of the wall**, $H_{mu}$ (m), is given by:

$$H_{mu} = NA_{max} + H_{bl} - (El_{te} - e_{te})$$

where:

| | |
|---|---|
| $NA_{max}$ | maximum normal water level in the reservoir; |
| $H_{bl}$ | height of the dam freeboard, in m; |
| $El_{te}$ | mean elevation of the land in the wall area, in m; and |
| $e_{te}$ | mean thickness of the layer of soil in the wall area, in m. |

### Common Excavation (account .12.17.27.12.10)

The **volume of common excavation**, $V_{tmu}$ (m³), is included in the dam account, except for earthfill dams, where it is given by:

$$V_{tmu} = (1.1 \times H_{mu}^2 + 6.1 \times H_{mu} + 5) \times e_{te}$$

where:

| | |
|---|---|
| $H_{mu}$ | height of the wall, in m; and |
| $e_{te}$ | mean thickness of the layer of soil in the wall area, in m; |

The unit price of **common excavation** is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the wall. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation (account .12.17.27.12.11)

The **volume excavated in rock**, $V_{rmu}$ (m³), is given by:

$$V_{rmu} = (1.1 \times H_{mu}^2 + 6.1 \times H_{mu} + 5) \times 1.5$$

where:

| | |
|---|---|
| $H_{mu}$ | height of the wall, in m. |

The unit price of **excavation in rock** is R$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the wall. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.17.27.13)

The **area of foundation to be cleaned**, $A_{lf}$ (m²), is included in the dam account, except for earthfill dams, where it is given by:

$$A_{lf} = 1.1 \times H_{mu}^2 + 6.1 \times H_{mu} + 5$$

where:

| | |
|---|---|
| $H_{mu}$ | height of the wall, in m. |

**Foundation treatment**, $L_{tf}$ (m), is part of the dam account.

The unit price for **foundation cleaning** is R$ 39.70/m² (December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per square meter measured using the design line, and includes the service per se and the supply of inputs and equipment, which will depend on the kind of surface in question.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account .12.17.27.14)

The **volume of concrete for transition walls**, $V_{cmu}$ (m³), is given by:

$$V_{cmu} = 0.53837 \times H_{mu}^3 + 2.1778 \times H_{mu}^2 + 146.8 \times H_{mu} + 425$$

where:

| | |
|---|---|
| $H_{mu}$ | height of the wall, in m. |

The amounts of cement and reinforcement steel are:

| | cement (kg/m³) | reinforcement steel (kg/m³) |
|---|---|---|
| structural concrete | 210 | 5 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit price for **concrete without cement** is R$ 113.00/m³ (December 2006 database) and is valid for projects in the south, southeast, central west and northeast regions of Brazil. This is the unit price per cubic meter of volume of the wall and includes all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

## 5.7.5 Spillways

### GATED SURFACE SPILLWAYS WITH A HIGH OGEE CREST (ACCOUNT .12.18.28)

The main **information required for dimensioning purposes** can be obtained from the overall layout and item 5.1.2. (Hydrometeorological Data) as shown below:

- coefficient for determining the initial height of the gates, $k_v$;
- height of the gates, $H_{cp}$ in m;
- design flood through the spillway, $Q_v$, in m³/s, from item 5.1.2.;

- 100-year flood flow, $Q_c$, in $m^3/s$, from item 5.1.2.;
- maximum normal water level in the reservoir, $NA_{max}$, from item 4.6.;
- elevation of the bottom of the approach channel to the sluiceways, $El_{ca}$, from item 5.7.3., when applicable;
- elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, $El_{cv}$;
- maximum water level in the downstream channel, $NA_{xcr}$, from item 5.1.2.;
- water level in the downstream channel for a 100-year flood, $NA_{ccr}$, from item 5.1.2.;
- elevation of the bottom of the downstream channel, $El_{cr}$; and
- height of sluiceways, $H_{ad}$ in m, from item 5.7.3., when applicable.

The **information required for quantification purposes** is:

- mean elevation of the land in the area of the spillway, including the energy dissipator, $El_{te}$, in m;
- mean elevation of the land in the stilling basin or ski jump area, exclusively, $El_{tde}$, in m;
- mean thickness of the layer of soil in the area of the spillway per se, $e_{te}$, in m;
- mean elevation of the land at section i – 0, 1 and 2 from Fig. 5.7.5.05 – perpendicular to the longitudinal axis of the approach channel, $El_{tai}$, in m;
- mean elevation of the land at section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the downstream channel, $El_{tri}$, in m;
- mean length of the approach channel in the part with no sluiceways, $L_{ca}$, in m;
- mean length of the approach channel in the part with sluiceways, $L_{cad}$, in m, when applicable;
- mean length of the downstream channel, $L_{cr}$, in m;
- thickness of the concrete lining for the stilling basin sill, $e_c$, in m, when applicable;
- width of one sluiceway, $B_{1ad}$ in m, from item 5.7.3., when applicable;
- total width of sluiceways, $B_{ad}$ in m, from item 5.7.3., when applicable; and
- number of sluiceways, $N_{ad}$, from item 5.7.3., when applicable.

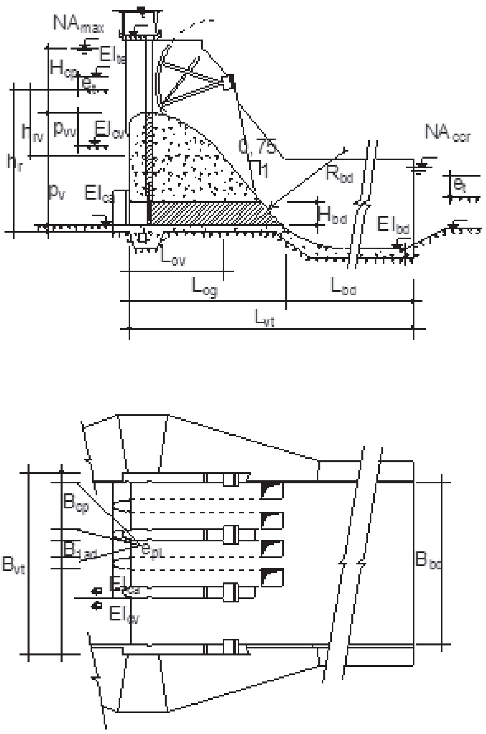



Fig. 5.7.5.01 – Typical cross-section and plan of a surface spillway with a high ogee crest and a stilling basin.

## Considerations and recommendations

This text relates to spillways with a typical cross-section as shown in Fig. 5.7.5.01 and 5.7.5.02.

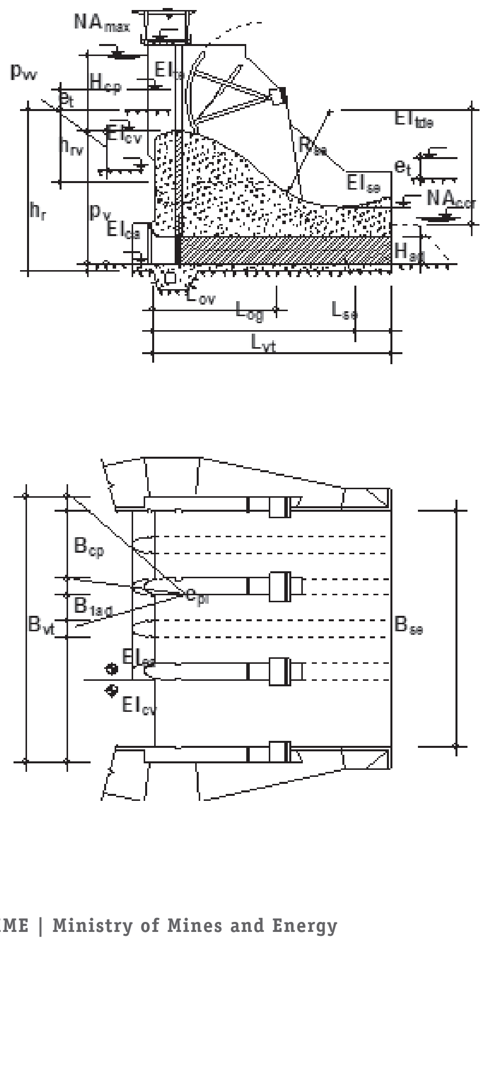



Fig. 5.7.5.02 – Typical cross-section and plan of a gated spillway with a high ogee crest and a ski jump.

An initial estimate of the **height of the gates**, $H_{cp}$ (m), is given by (COPEL, 1996):

$$H_{cp} = k_v \times Q_v^{0.4} \leq 21.0m$$

where:

| $k_v$ | For spillways with |
|---|---|
| 0.6 | two gates |
| 0.5 | three gates |
| 0.4 | five gates |
| 0.3 | ten gates |

where:

| | |
|---|---|
| $k_v$ | coefficient for determining the initial height of the gates; and |
| $Q_v$ | design flow through the spillway, in $m^3/s$. |

Intermediate $k_v$ values can be used. In spillways when it is known that the ogee crest will be submerged, this coefficient should be raised.

When the spillway is not all on the river bed, the **elevation of the foundations** at the two ends of the structure are often different, in which case an mean of these elevations should be used for the elevation of the bottom of the approach channel to the spillway, excluding the sluiceways.

The following applies for **spillways with no sluiceways**:

$$El_{ca} = El_{cv}$$

where:

| | |
|---|---|
| $El_{ca}$ | elevation of the bottom of the approach channel to the sluiceways, in m; and |
| $El_{cv}$ | elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m. |

The thickness of the **layer of soil** on the river bed may be different from the thickness on the abutments, and there may often be no soil at all.

When **dimensioning the stilling basin**, the recommended range for the Froude number before the hydraulic jump is betwen 4.5 and 9.0, in order to ensure its stability. One way of raising the Froude number to 4.5 would be to lower the bottom of the stilling basin beyond the result given by the calculation. Likewise, one way to reduce the Froude number to 9.0 would be to reduce the width of the stilling basin.

In the absence of more accurate information, use 1.0 m as the **thickness of the concrete lining for the bottom of the stilling basin**.

### Discharge Coefficient

The **initial discharge coefficient**, $C'_d$, for the typical cross-section recommended can be obtained from Graph 5.7.5.01 (BuRec, 1977) or from the equivalent expressions:

for: $z \leq 0.475$:

$$C'_d = 2.535 \times z^3 - 3.61 \times z^2 + 1.96 \times z + 1.702$$

for: $0.475 < z \leq 1.2$:

$$C'_d = 0.145 \times z^3 - 0.475 \times z^2 + 0.559 \times z + 1.916$$

for: $1.2 < z \leq 3.0$:

$$C'_d = -0.0072 \times z^2 + 0.0442 \times z + 2.112$$

for: $z > 3.0$:

$$C'_d = 2.18$$

where:

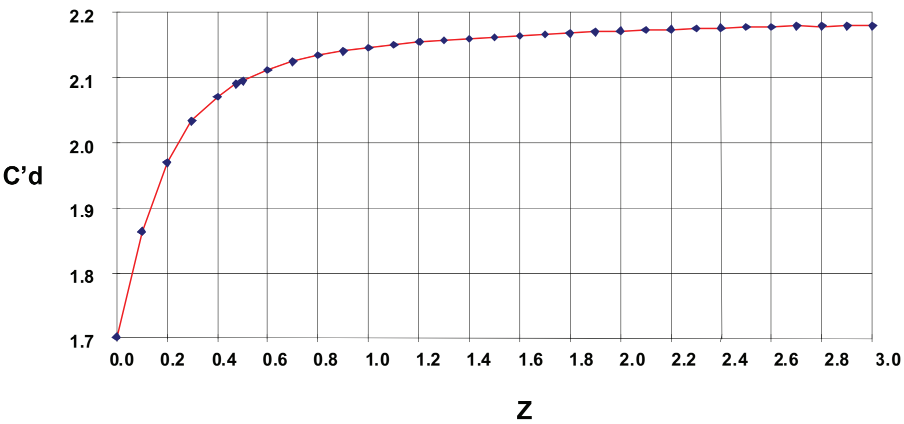
$$z = \frac{0.7 \times p_v + 0.3 \times p_{vv}}{H_{cp}}$$

$$p_v = NA_{max} - H_{cp} - El_{ca}$$

$$p_{vv} = NA_{max} - H_{cp} - El_{cv}$$

where:

| | |
|---|---|
| z | adimensional parameter; |
| $p_v$ | vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m; |
| $p_{vv}$ | vertical distance between the ogee sill and the bottom of the spillway approach channel, sluiceways excluded, in m; |
| $H_{cp}$ | height of the gates, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel to the sluiceways, in m; and |
| $El_{cv}$ | elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m. |




Graph 5.7.5.01 – Initial Discharge Coefficient.

The **reduction coefficient for the discharge coefficient when the outlet is submerged**, $k_c$, can be obtained from Graph 5.7.5.02 (Bureau of Reclamation, 1977) or from the equivalent expressions:

for: $-4 \times u + 7 \times w + 2.6 \geq 0$ (section I):

$$k_c = -0.952 \times \left(\frac{1}{u}\right)^2 + 0.956 \times \left(\frac{1}{u}\right) + 0.767 \leq 1$$

for: $u < 3.6\, e - 4 \times u + 7 \times w + 2.6 < 0$ (section II):

$$k_c = 1.058 - \frac{4 \times (u+5)}{860 \times w} \leq 1$$

for: $u \geq 3.6$ (section III):

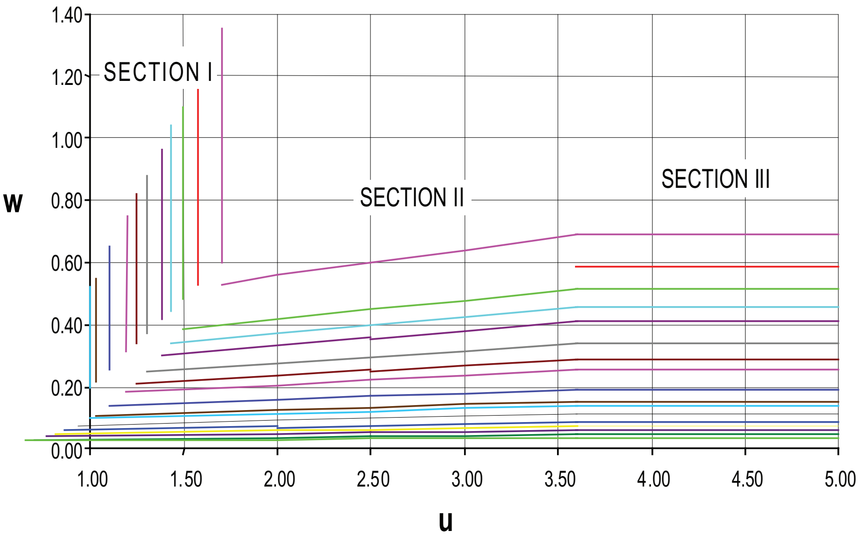
$$k_c = 1.058 - \frac{4}{100 \times w} \leq 1$$

where:

$$u = \frac{NA_{max} - El_{cr}}{H_{cp}} \qquad w = \frac{NA_{max} - NA_{xcr}}{H_{cp}}$$

where:

| | |
|---|---|
| u, w | adimensional parameters; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_{cr}$ | elevation of the bottom of the downstream channel; , in m; |
| $H_{cp}$ | height of the gates, in m; and |
| $NA_{xcr}$ | maximum water level in the downstream channel, in m. |




Graph 5.7.5.02 – Reduction coefficient for the discharge coefficient when the outlet is submerged.

The **corrected discharge coefficient**, $C_d$, is given by:

$$C_d = k_c \times C'_d$$

where:

| | |
|---|---|
| $k_c$ | reduction coefficient for the discharge coefficient; and |
| $C'_d$ | initial discharge coefficient. |

## Dimensions of the gates and spillway

The **useful width of the openings**, $B_{uvt}$ (m), is given by:

$$B_{uvt} = \frac{Q_v}{C_d \times H_{cp}^{3/2}}$$

where:

| | |
|---|---|
| $Q_v$ | design flood through the spillway, in m³/s; |
| $C_d$ | discharge coefficient; and |
| $H_{cp}$ | height of the gates, in m. |

The **real width of the openings**, $B_{rvt}$ (m), is given by:

$$B_{rvt} = B_{uvt} + 0.2 \times H_{cp}$$

where:

| | |
|---|---|
| $B_{uvt}$ | useful width of the openings, in m; |
| 0.2 | contraction coefficient at the end pillars; and |
| $H_{cp}$ | height of the gates, in m. |

The **number of gates**, $N_{cp}$, is given by:

$$N_{cp} = \text{int}\left(\frac{B_{rvt}}{H_{cp}} + 0.999\right)$$

where:

| | |
|---|---|
| $B_{rvt}$ | real width of the openings, in m; |
| $H_{cp}$ | height of the gates, in m; and |
| int(x) | function that returns the integer part of x. |

The **width of the gates**, $B_{cp}$ (m), is given by:

$$B_{cp} = 0.05 \times \text{int}\left(\frac{1}{0.05} \times \frac{B_{rvt}}{N_{cp}} + 0.5\right)$$

where:

| | |
|---|---|
| $B_{rvt}$ | real width of the openings, in m; and |
| $N_{cp}$ | number of gates. |

The **thickness of the pillars**, $e_{pl}$ (m), is given by:

$$e_{pl} = 0.12 \times H_{cp} + 2.4$$

where:

| | |
|---|---|
| $H_{cp}$ | height of the gates, in m. |

The **total width of the spillway**, $B_{vt}$ (m), perpendicular to flow, is given by:

$$B_{vt} = (N_{cp} + 1) \times e_{pl} + N_{cp} \times B_{cp}$$

where:

| | |
|---|---|
| $N_{cp}$ | number of gates; |
| $e_{pl}$ | thickness of the pillars, in m; and |
| $B_{cp}$ | width of the gates, in m. |

The **length of the ogee crest** in the direction of flow is given by:

$L_{ov}$ (m), for the part with no sluiceways:

$$L_{ov} = 1.46 \times H_{cp}^{0.46} \times (p_{vv} + 1.5)^{0.54} + 0.27 \times H_{cp}$$

ou $L_{og}$ (m), for the part with sluiceways:

$$L_{og} = 1.46 \times H_{cp}^{0.46} \times (p_{v} + 1.5)^{0.54} + 0.27 \times H_{cp}$$

where:

| | |
|---|---|
| $H_{cp}$ | height of the gates, in m; and |
| $p_v$ | vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m; and |
| $p_{vv}$ | vertical distance between the ogee sill and the bottom of the spillway approach channel, sluiceways excluded, in m. |

The **total length** of the spillway, $L_{vt}$ (m), can be obtained from one of the following:

- for spillways with a stilling basin and sluiceways: $L_{vt} = L_{og} + L_{bd}$
- for spillways with a stilling basin and no sluiceways: $L_{vt} = L_{ov} + L_{bd}$
- for spillways with a ski jump and sluiceways: $L_{vt} = L_{og} + L_{se}$
- for spillways with a ski jump and no sluiceways: $L_{vt} = L_{ov} + L_{se}$

where:

| | |
|---|---|
| $L_{og}$ | length of the ogee crest with sluiceways, in m; |
| $L_{ov}$ | length of the ogee crest with no sluiceways, in m; |
| $L_{bd}$ | length of the stilling basin, defined below, in m; and |
| $L_{se}$ | length of the ski jump at the foundations, defined below, in m. |

Stilling Basin

The **width of the stilling basin**, $B_{bd}$ (m), is given by:

$$B_{bd} = (N_{cp} - 1) \times e_{pl} + N_{cp} \times B_{cp}$$

where:

| | |
|---|---|
| $N_{cp}$ | number of gates; |
| $B_{cp}$ | width of the gates, in m; and |
| $e_{pl}$ | thickness of the pillars, in m. |

The **stilling basin depth** is determined iteratively, based on the Froude number before the hydraulic jump, in section one of Fig. 5.7.5.03, for the 100-year flood flow .

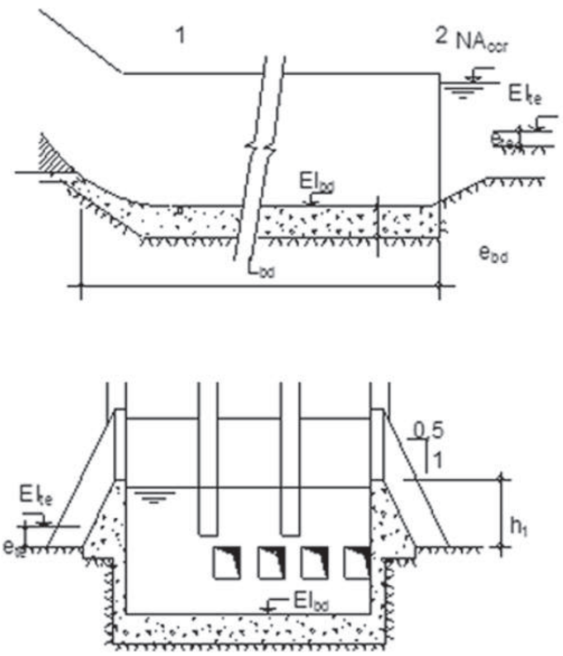



Fig. 5.7.5.03 – Typical cross-section of a stilling basin.

An elevation is picked for the bottom of the stilling basin and its suitability is checked by calculating the **velocity**, $v_1$ (m/s), the **depth of discharge**, $y_1$ (m), and the **Froude number**, $Fr_1$, before the hydraulic jump, **depth of discharge** after the hydraulic jump, $y_2$ (m), and finally the elevation of the **bottom of the stilling basin**.

Should the elevation of the bottom of the stilling basin be different from the figure first picked, the calculations should be redone until the level of precision required is reached.

$$v_1 = \sqrt{k \times 2 \times g \times (NA_{max} - El_{bd})} \qquad y_1 = \frac{Q_c}{B_{bd} \times v_1}$$

$$Fr_1 = \frac{v_1}{\sqrt{g \times y_1}} \qquad y_2 = \left(\frac{y_1}{2}\right) \times \left(\sqrt{1 + 8 \times Fr_1^2} - 1\right)$$

$$El_{bd} = NA_{ccr} - y_2$$

where:

| | |
|---|---|
| $v_1$ | mean velocity of discharge before the hydraulic jump, in m/s; |
| k | 0.9 – reduction coefficient for the energy head; |
| g | 9.81 m/s$^2$ – acceleration due to gravity; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_{bd}$ | elevation of the bottom of the stilling basin, in m; |
| $y_1$ | depth of discharge before the hydraulic jump, in m; |
| $Q_c$ | 100-year flood flow, in m$^3$/s; |
| $B_{bd}$ | width of the stilling basin, in m; |
| $Fr_1$ | Froude number before the hydraulic jump; |
| $y_2$ | depth of discharge after the hydraulic jump, in m; and |
| $NA_{ccr}$ | water level in the downstream channel for a 100-year flood, in m. |

The **radius of curvature at the stilling basin inlet**, $R_{bd}$ (m), is given by:

$$R_{bd} = 6 \times y_1 \text{ (Peterka)}$$

where:

| | |
|---|---|
| $y_1$ | depth of discharge before the hydraulic jump, in m. |

The **length of the stilling basin**, $L_{bd}$ (m), is given by:

$$L_{bd} = 6 \times y_2 + 0.75 \times (El_{ca} - El_{bd}) + 0.5 \times R_{bd} - 1.1$$

where:

| | |
|---|---|
| $y_2$ | depth of discharge after the hydraulic jump, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel, in m; |
| $El_{bd}$ | elevation of the bottom of the stilling basin, in m; |
| $R_{bd}$ | radius of curvature at the stilling basin inlet, in m. |

### Ski Jump

The **width of the ski jump**, $B_{se}$ (m), is given by:

$$B_{se} = (N_{cp} - 1) \times e_{pl} + N_{cp} \times B_{cp}$$

where:

| | |
|---|---|
| $N_{cp}$ | number of gates; |
| $B_{cp}$ | width of the gates, in m; and |
| $e_{pl}$ | thickness of the pillars, in m. |

The **elevation of the ski jump sill**, $El_{se}$, is given by:

$$El_{se} = NA_{ccr} + 1.0 \geq El_{cr}$$

where:

| | |
|---|---|
| $NA_{ccr}$ | water level in the downstream channel for a 100-year flood, in m; |
| $El_{cr}$ | elevation of the bottom of the downstream channel, in m. |

If the sluiceways are built in the body of the spillway, the **elevation of the sill of the ski jump**, $El_{se}$, should be higher than the top of the sluiceways, as shown in Fig. 5.7.5.04:

$$El_{se} \geq El_{ca} + 1.25 \times H_{ad}$$

where:

| | |
|---|---|
| $El_{ca}$ | elevation of the bottom of the approach channel, in m; |
| $H_{ad}$ | height of sluiceways, in m. |

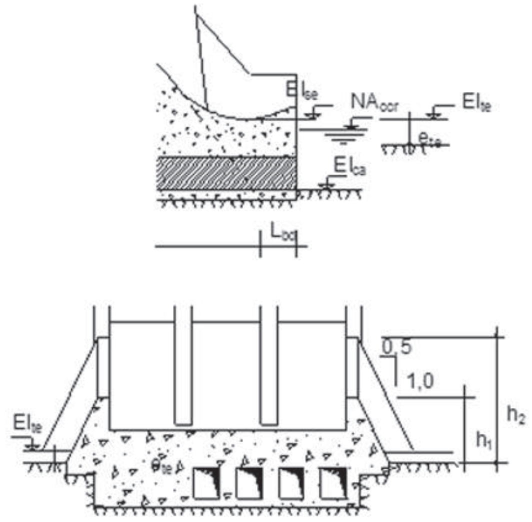



Fig. 5.7.5.04 – Typical cross-section of a ski jump.

The **radius of curvature of the ski jump**, $R_{se}$ (m), is given by:

$R_{se}$ = 6 x y

where:

$$y = \frac{Q_c}{B_{se} \times v} \qquad v = \sqrt{k \times 2 \times g \times (NA_{max} - El_{se})}$$

where:

| | |
|---|---|
| y | depth of the water column at the ski jump, in m; |
| $Q_c$ | 100-year flood flow, in $m^3/s$; |
| $B_{se}$ | width of the ski jump, in m; |
| v | velocity of the water column at the ski jump, in m/s; |
| k | 0.9 – reduction coefficient for the energy head; |
| g | 9.81 $m/s^2$ – acceleration due to gravity; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_{se}$ | elevation of the ski jump sill, in m. |

The **length of the ski jump at the foundations**, $L_{se}$ (m), is given by:

$$L_{se} = d_{16} + 1.286 \times R_{se} - d_{12} \geq 0$$

where:

$$d_{16} = 1.46 \times H_{cp}^{0.46} \times (p_v - h_5)^{0.54}$$

$$d_{12} = 1.46 \times H_{cp}^{0.46} \times (p_v - 1.5)^{0.54}$$

$$h_5 = El_{se} + 0.6 \times R_{se} - El_{cr}$$

where:

| | |
|---|---|
| $R_{se}$ | radius of curvature of the ski jump, in m; |
| $H_{cp}$ | height of the gates, in m; |
| $p_v$ | vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m; |
| $El_{se}$ | elevation of the ski jump sill, in m; |
| $El_{cr}$ | elevation of the bottom of the downstream channel, in m; and |
| $d_i$ | effective distances, in m. |

### Common Excavation (account .12.19.28.12.10)

The **volume of common excavation** for the spillway, $V_{tvt}$ ($m^3$), is given by:

$$V_{tvt} = V_{tca} + V_{tes} + V_{tcr}$$

where:

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

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

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

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

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

$$B_{ca} = B_{vt} - 2 \times (e_{pl} - 1.0)$$

$$V_{tad} = (L_{cad} - L_{ca}) \times e_{te} \times (B_{ad} - e_{pl})$$

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

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

for the stilling basin: $B_{cr} = B_{bd} + 2 \times 1.0$

for the ski jump: $B_{cr} = B_{se} + 2 \times 1.0$

where:

| | |
|---|---|
| $V_{tca}$ | volume of common excavation for the approach channel, in $m^3$; |
| $V_{tes}$ | volume of common excavation for the structure, in $m^3$; |
| $V_{tcr}$ | volume of common excavation for the downstream channel, in $m^3$; |
| $V_{tai}$ | volume of common excavation per meter at section i of the approach channel, in $m^3/m$; |
| $L_{ca}$ | mean length of the approach channel at the part with no sluiceways, in m; |
| $V_{tad}$ | extra volume of common excavation for the approach channel because of the sluiceways, in $m^3$; |
| $B_{vt}$ | total width of the spillway, in m; |
| $e_{te}$ | mean thickness of the layer of soil in the area of the spillway per se, in m; |
| $L_{vt}$ | total length of the spillway, in m; |
| $V_{tri}$ | volume of common excavation per meter at section i for the downstream channel, in $m^3/m$; |
| $L_{cr}$ | mean length of the downstream channel, in m; |
| $B_{ca}$ | width of the bottom of the approach channel, in m; |
| $h_{rai}$ | mean depth of excavation in rock at section i of the approach channel, in m; |
| $El_{tai}$ | mean elevation of the land at section i of the approach channel; |
| $El_{cv}$ | elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m; |
| $e_{pl}$ | thickness of the pillars, in m; |
| $L_{cad}$ | mean length of the approach channel in the part with sluiceways, in m; |
| $B_{ad}$ | total width of sluiceways, in m; |
| $B_{cr}$ | width of the bottom of the downstream channel, in m; |
| $h_{rri}$ | mean depth of excavation in rock at section i of the downstream channel, in m; |
| $El_{tri}$ | mean elevation of the land at section i perpendicular to the longitudinal axis of the downstream channel, in m; |
| $El_{cr}$ | elevation of the bottom of the downstream channel, in m; |
| $B_{bd}$ | width of the stilling basin, in m; and |
| $B_{se}$ | width of the ski jump, in m. |

The unit price of **common excavation** is R$ 7.60/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price

per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation (account .12.18.28.12.11)

Fig. 5.7.5.05 shows typical cross-sections for the excavation of approach and downstream channels.

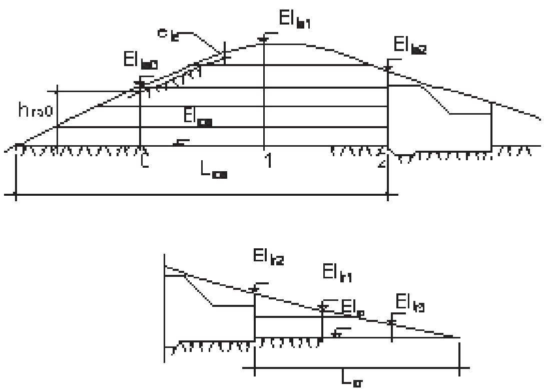


Fig. 5.7.5.05 – Excavation for approach and downstream channels.

The **volume of excavated rock** for the spillway, $V_{rvt}$ ($m^3$), is given by:

$$V_{rvt} = V_{rca} + V_{rog} + V_{rpj} + V_{rde} + V_{rcr}$$

where:

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

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

$$V_{rai} = (B_{ca} - 6 + 0.6 \times h_{rai}) \times h_{rai}$$

$$V_{rri} = (B_{cr} - 6 + 0.6 \times h_{rri}) \times h_{rri}$$

$$h_r = El_{te} - e_{te} - (El_{ca} - 1.5)$$

$$h_{rv} = El_{te} - e_{te} - (El_{cv} - 1.5)$$

for spillways without sluiceways:

$$V_{rog} = (L_{ov} \times h_{rv} + 23) \times B_{vt}$$

$$V_{rad} = 0$$

and with a stilling basin:

$$V_{rpj} = d_1 \times \left( \frac{h_{rv} + h_{rb}}{2} + 0.167 \times H_{cp} \right) \times (B_{bd} + 2.0)$$

and with a ski jump:

$$V_{rpj} = d_7 \times \left( \frac{h_{rv} + h_{rs}}{2} + 0.167 \times H_{cp} \right) \times (B_{se} + 2.0)$$

$$V_{rde} = d_8 \times h_{rs} \times (B_{se} + 2.0)$$

for spillways with sluiceways:

$$V_{rog} = (L_{ov} \times h_{rv} + 23) \times (B_{vt} - B_{ad}) + (L_{og} \times h_r + 23) \times B_{ad}$$

$$V_{rad} = \frac{L_{cad} + L_{ca}}{2} \times (El_{cv} - El_{ca}) \times (B_{ad} - e_{pl}) - V_{tad}$$

and with a stilling basin:

$$V_{rpj} = d_1 \times \left( \frac{h_{rv} + h_{rb}}{2} + 0.167 \times H_{cp} \right) \times (B_{bd} + 2.0 - B_{ad}) + d_2 \times \left( \frac{h_r + h_{rb}}{2} + 0.167 \times H_{cp} \right) \times B_{ad}$$

$$d_2 = 0.75 \times [El_{ca} - 1.5 - (El_{bd} - e_c)]$$

and with a ski jump:

$$V_{rpj} = d_7 \times \left( \frac{h_{rv} + h_{rs}}{2} + 0.167 \times H_{cp} \right) \times (B_{se} + 2.0 - B_{ad})$$

$$V_{rsd} = d_8 \times h_{rs} \times (B_{se} + 2.0 - B_{ad}) + L_{se} \times h_r \times B_{ad}$$

for any spillway with a stilling basin:

$$V_{rde} = V_{rbd} + V_{rmc} + V_{rbe}$$

$$V_{rbd} = L_{bd} \times h_{rb} \times (B_{bd} + 2.0)$$

$$V_{rmc} = 2 \times (L_{bd} + d_4) \times 0.5 \times h_1 \times 1.5 + 2 \times (d_3 - d_4) \times 0.5 \times \frac{h_1 + h_2}{2} \times 1.5$$

$$V_{rbe} = 2 \times (L_{bd} + d_4) \times 0.3 \times h_{re}^2$$

$$h_{rb} = El_{te} - e_{te} - (El_{bd} - e_c)$$

$$h_1 = NA_{ccr} + 2.0 - (El_{tde} - e_{te}) \geq 0$$

$$h_2 = NA_{max} - 0.88 \times H_{cp} - (El_{tde} - e_{te}) \geq 0$$

$$d_1 = 0.75 \times [El_{cv} - 1.5 - (El_{bd} - e_c)]$$

$$d_3 = 0.75 \times [NA_{max} - 1.83 \times H_{cp} - (El_{bd} - e_c)]$$

$$d_4 = 0.75 \times (y_2 + 2.0 + e_c - 0.95 \times H_{cp}) \leq d_3$$

$$h_{re} = El_{tde} - e_{te} - (NA_{ccr} - 5.0) \geq 0$$

for any spillway with a ski jump:

$$h_{rs} = El_{te} - e_{te} - \left(El_{se} - 0.25 \times H_{cp}\right)$$

$$d_7 = 1.46 \times H_{cp}^{0.46} \times \left[\left(p_v - h_4\right)^{0.54} - \left(p_{vv} + 1.5\right)^{0.54}\right]$$

$$h_4 = El_{se} - 0.25 \times H_{cp} - El_{ca}$$

$$d_8 = 0.986 \times R_{se} - 0.188 \times H_{cp}$$

where:

| | |
|---|---|
| $V_{rca}$ | volume of excavated rock for the approach channel, in $m^3$; |
| $V_{rog}$ | volume of excavated rock in the ogee crest area, in $m^3$; |
| $V_{rpj}$ | volume of excavated rock in the area of the downstream face of the ogee crest, in $m^3$; |
| $V_{rde}$ | volume of excavated rock in the area of the stilling basin or ski jump, in $m^3$; |
| $V_{rcr}$ | volume of excavated rock in the downstream channel, in $m^3$; |
| $V_{rai}$ | volume of excavated rock per meter at section i of the approach channel, in $m^3/m$; |
| $L_{cad}$ | mean length of the approach channel in the part with sluiceways, in m; |
| $V_{rad}$ | extra volume of excavation in rock for the approach channel due to the sluiceways, in $m^3$; |
| $V_{rri}$ | volume of excavated rock per meter at section i for the downstream channel, in $m^3/m$; |
| $L_{cr}$ | mean length of the downstream channel, in m; |
| $B_{ca}$ | width of the bottom of the approach channel, in m; |
| $h_{rai}$ | mean depth of excavation in rock at section i of the approach channel, in m; |
| $B_{cr}$ | width of the bottom of the downstream channel, in m; |
| $h_{rri}$ | mean depth of excavation in rock at section i of the downstream channel, in m; |
| $h_r$ | mean depth of the excavation in rock for the ogee crest in the part with sluiceways, in m; |
| $El_{te}$ | mean elevation of the land in the area of the spillway per se, in m; |
| $e_{te}$ | mean thickness of the layer of soil in the area of the spillway per se, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel to the sluiceways, in m; |
| $h_{rv}$ | mean depth of excavation in rock at the part of the ogee crest with no sluiceways, in m; |
| $El_{cv}$ | elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m; |
| $L_{ov}$ | length of the ogee crest at the part with no sluiceways, in m; |
| $B_{vt}$ | total width of the spillway, in m; |
| $h_{rb}$ | mean depth of excavation in rock for the stilling basin, in m; |
| $H_{cp}$ | height of the gates, in m; |
| $B_{bd}$ | width of the stilling basin, in m; and |
| $L_{bd}$ | length of the stilling basin, in m; |
| $h_{rs}$ | mean depth of excavation in rock for the ski jump at the part with no sluiceways, in m; |
| $B_{se}$ | width of the ski jump, in m; |
| $B_{ad}$ | total width of sluiceways, in m; |
| $L_{og}$ | length of the ogee crest at the part with sluiceways, in m; |
| $L_{ca}$ | mean length of the approach channel at the part with no sluiceways, in m; |
| $e_{pl}$ | thickness of the pillars, in m; |
| $V_{tad}$ | extra volume of common excavation for the approach channel because of the sluiceways, in $m^3$; |
| $El_{bd}$ | elevation of the bottom of the stilling basin, in m; |
| $e_c$ | thickness of the concrete lining for the stilling basin sill, in m; |
| $L_{se}$ | length of the ski jump at the foundations, in m; |
| $V_{rbd}$ | volume of excavated rock in the stilling basin area, in $m^3$; |
| $V_{rmc}$ | volume of excavated rock in the area of the buttress walls for the stilling basin or ski jump, in $m^3$; |
| $V_{rbe}$ | volume of excavated rock for berms in the stilling basin area, in $m^3$; |
| $h_{re}$ | mean depth of excavation in rock in berms in the stilling basin area, in m; |
| $NA_{ccr}$ | water level in the downstream channel for a 100-year flood, in m; |
| $El_{tde}$ | mean elevation of the land in the stilling basin or ski jump area, in m; |

| | |
|---|---|
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $y_2$ | depth of discharge after the hydraulic jump, in m; |
| $El_{se}$ | elevation of the ski jump sill, in m; |
| $p_v$ | vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m; |
| $p_{vv}$ | vertical distance between the ogee sill and the bottom of the spillway approach channel, sluiceways excluded, in m; |
| $R_{se}$ | radius of curvature of the ski jump, in m; and |
| $d_i$, $h_i$ | secondary dimensions, in m. |

The unit price of **excavation in rock** is R$ 21.00/m$^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.18.28.13)

The **area of foundation to be cleaned** for the spillway, $A_{lf}$ (m$^2$), is given by:

$$A_{lf} = B_{vt} \times L_{vt}$$

where:

| | |
|---|---|
| $B_{vt}$ | total width of the spillway, in m; and |
| $L_{vt}$ | total length of the spillway, in m. |

The **length of the grout holes** and **drainage line** for the spillway, $L_{tf}$ (m), is given by:

$$L_{tf} = \frac{B_{vt}}{3.0} \times L_{1tf}$$

$$L_{1tf} = 1.5 \times (NA_{max} - El_{ca}) \leq 40m$$

where:

| | |
|---|---|
| $B_{vt}$ | total width of the spillway, in m; |
| $L_{1tf}$ | length of one grout hole, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel; and |
| 3.0 | space between the grout holes, in m. |

The **total length of the rock anchors** in the stilling basin, $L_{tfc}$ (m), when necessary, is given by:

$$L_{tfc} = B_{bd} \times L_{bd}$$

where:

| | |
|---|---|
| $B_{bd}$ | width of the stilling basin, in m; and |
| $L_{bd}$ | length of the stilling basin, in m. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m$^2$
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account .12.18.28.14)

The **volume of concrete** for the spillway, $V_{cvt}$ (m$^3$), is given by (COPEL, 1983 and 1996):

$$V_{cvt} = V_{cog} + V_{cpj} + V_{cpl} + V_{cpo} + V_{cde} + V_{cmv} + V_{cmc}$$

where:

$$V_{cpl} = (1.98 \times H_{cp}^2 + 6.0 \times H_{cp} + 6) \times (N_{cp} + 1) \times e_{pl}$$

$$V_{cpo} = 6.0 \text{ x } B_{vt}$$

for spillways without sluiceways:

$$V_{cog} = [0.944 \times H_{cp}^{0.46} \times (p_{vv} + 1.5)^{1.54} + 0.27 \times p_{vv} \times H_{cp} - 0.007 \times H_{cp}^2 + 0.40 \times H_{cp} + 18] \times B_{vt}$$

and with a stilling basin:

$$V_{cpj} = d_1 \times 0.167 \times H_{cp} \times (B_{bd} + 2.0)$$

and with a ski jump:

$$V_{cpj} = 1.25 \times H_{cp} \times h_5 \times (B_{se} + 2.0)$$

$$V_{csb} = 0$$

$$h_5 = El_{cv} - 1.5 - (El_{se} - 0.25 \times H_{cp}) \geq 0$$

for spillways with sluiceways:

$$V_{cog} = [0.944 \times H_{cp}^{0.46} \times (p_{vv} + 1.5)^{1.54} + 0.27 \times p_{vv} \times H_{cp}] \times (B_{vt} - B_{ad}) + [0.944 \times H_{cp}^{0.46} \times (p_v + 1.5)^{1.54} + $$
$$+ 0.27 \times p_v \times H_{cp}] \times B_{ad} + (-0.007 \times H_{cp}^2 + 0.40 \times H_{cp} + 18) \times B_{vt}$$

and with a stilling basin:

$$V_{cpj} = d_1 \times 0.167 \times H_{cp} \times (B_{bd} + 2.0 - B_{ad}) + d_2 \times 0.167 \times H_{cp} \times B_{ad}$$

$$d_2 = 0.75 \times [El_{ca} - 1.5 - (El_{bd} - e_c)]$$

and with a ski jump:

$$V_{cpj} = 1.25 \times H_{cp} \times h_5 \times (B_{se} + 2.0 - B_{ad})$$

$$V_{csb} = \left(\frac{d_9}{2} + L_{se}\right) \times (h_4 + 1.5) \times (B_{se} + 2.0)$$

$$d_9 = 1.46 \times H_{cp}^{0.46} \times \left[ (p_v + 1.5)^{0.54} - (p_v - h_4)^{0.54} \right]$$

$$h_4 = El_{se} - 0.25 \times H_{cp} - El_{ca}$$

for any spillway with a stilling basin:

$$V_{cde} = \left(L_{bd} \times e_c + 0.036 \times R_{bd}^2 + 0.375 \times e_c^2\right) \times (B_{bd} + 2.0)$$

$$V_{cmv} = 2 \times \left[ (d_5 \times 0.95 \times H_{cp} \times 1.0) + \frac{d_6^2}{2 \times 0.75} \right] + 2 \times L_{bd} \times (y_2 + 2.0) \times 1.0$$

$$V_{cmc} = 2 \times \left( L_{bd} + \frac{d_3 + d_4}{2} \right) \times \left(0.25 \times h_1^2 + 0.75 \times h_1\right) + 2 \times \frac{d_3 - d_4}{2} \times \left(0.25 \times h_2^2 + 0.75 \times h_2\right)$$

$$d_1 = 0.75 \times \left[ El_{cv} - 1.5 - (El_{bd} - e_c) \right]$$

$$d_5 = 0.75 \times \left[ NA_{max} - 2.0 \times H_{cp} - (El_{bd} - e_c) \right]$$

$$d_6 = 0.75 \times (y_2 + 2.0 + e_c - 0.95 \times H_{cp})$$

$$d_3 = 0.75 \times \left[ NA_{max} - 1.83 \times H_{cp} - (El_{bd} - e_c) \right]$$

$$d_4 = 0.75 \times (y_2 + 2.0 + e_c - 0.95 \times H_{cp}) \leq d_3$$

$$h_1 = NA_{ccr} + 2.0 - (El_{tde} - e_{te}) \geq 0$$

$$h_2 = NA_{max} - 0.88 \times H_{cp} - (El_{tde} - e_{te}) \geq 0$$

for any spillway with a ski jump:

$$V_{cde} = V_{csd} + V_{csb}$$

$$V_{csd} = (0.116 \times R_{se}^2 + 0.247 \times H_{cp} \times R_{se} - 0.023 \times H_{cp}^2) \times (B_{se} + 2.0)$$

$$V_{cmv} = 2 \times (d_{11} \times 1.6 \times y \times 1.0 + d_{13} \times 0.95 \times H_{cp} \times 1.0)$$

$$V_{cmc} = 2 \times \left( d_{11} + \frac{d_{10}}{2} \right) \times \left(0.25 \times h_3^2 + 0.75 \times h_3\right) + 2 \times \frac{d_{10}}{2} \times \left(0.25 \times h_2^2 + 0.75 \times h_2\right)$$

$$d_{10} = 0.75 \times [NA_{max} - 0.878 \times H_{cp} - (El_{se} - 1.6 \times y)] \geq 0$$

$$d_{11} = d_{12} + L_{se} - 1.46 \times H_{cp} - d_{13}$$

$$d_{12} = 1.46 \times H_{cp}^{0.46} \times (p_v + 1.5)^{0.54}$$

$$d_{13} = 0.75 \times [NA_{max} - 1.05 \times H_{cp} - (El_{se} - 1.6 \times y)] \geq 0$$

$$h_3 = El_{se} - 1.6 \times y - (El_{tde} - e_{te}) \geq 0$$

$$h_2 = NA_{max} - 0.88 \times H_{cp} - (El_{tde} - e_{te}) \geq 0$$

where:

| | |
|---|---|
| $V_{cog}$ | volume of concrete for the ogee crest, in $m^3$; |
| $V_{cpj}$ | volume of concrete for the downstream face of the ogee crest, in $m^3$; |
| $V_{cpl}$ | volume of concrete for the pillars, in $m^3$; |
| $V_{cpo}$ | volume of concrete for the bridge, in $m^3$; |
| $V_{cde}$ | volume of concrete for the stilling basin or ski jump, in $m^3$; |
| $V_{cmv}$ | volume of concrete for the vertical lining of the stilling basin or ski jump, in $m^3$; |
| $V_{cmc}$ | volume of concrete for the walls of the stilling basin or ski jump, in $m^3$; |
| $H_{cp}$ | height of the gates, in m; |
| $N_{cp}$ | number of gates; |
| $e_{pl}$ | thickness of the pillars, in m; |
| $B_{vt}$ | total width of the spillway, in m; |
| $p_{vv}$ | vertical distance between the ogee sill and the bottom of the spillway approach channel, sluiceways excluded, in m; |
| $B_{bd}$ | width of the stilling basin, in m; |
| $B_{se}$ | width of the ski jump, in m; |
| $B_{ad}$ | total width of sluiceways, in m; |
| $p_v$ | vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel to the sluiceways, in m; |
| $El_{bd}$ | elevation of the bottom of the stilling basin, in m; |
| $e_c$ | thickness of the concrete lining for the stilling basin sill, in m; |
| $L_{se}$ | length of the ski jump at the foundations, in m; |
| $El_{se}$ | elevation of the ski jump sill, in m; |
| $L_{bd}$ | length of the stilling basin, in m; |
| $R_{bd}$ | radius of curvature at the stilling basin inlet, in m; |
| $y_2$ | depth of discharge after the hydraulic jump, in m; |
| $El_{cv}$ | elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $NA_{ccr}$ | water level in the downstream channel for a 100-year flood, in m; |
| $El_{tde}$ | mean elevation of the land in the stilling basin or ski jump area, in m; |
| $e_{te}$ | mean thickness of the layer of soil in the area of the spillway per se, in m; |
| $V_{csd}$ | volume of concrete for the ski jump deflector baffle, in $m^3$; |
| $V_{csb}$ | volume of concrete below the deflector baffle, in $m^3$; |
| $R_{se}$ | radius of curvature of the ski jump, in m; |
| y | flow depth in the ski jump, in m; and |
| $d_i$, $h_i$ | secondary dimensions, in m. |

If there are sluiceways through the body of the spillway, the **extra volume of concrete for the sluiceways**, $V_{cad}$ ($m^3$), should be taken into account as shown below:

$$V_{cad} = V_{cac} + V_{cpl} - V_{cae}$$

where:

$$V_{cac} = (0.24 \times H_{ad} + 2) \times B_{ad}$$

$$V_{cpl} = (0.16 \times H_{ad}^2 + 2.7 \times H_{ad} + 8) \times (N_{ad} + 1) \times e_{pl}$$

$$V_{cae} = (0.38 \times H_{ad} + 0.2) \times H_{ad} \times N_{ad} \times B_{1ad}$$

where:

| | |
|---|---|
| $V_{cac}$ | volume of concrete for part of the sluiceway sills, in $m^3$; |
| $V_{cpl}$ | volume of concrete for the sluiceway walls upstream from the face of the dam, in $m^3$; |
| $V_{cae}$ | volume of concrete for the sluiceway inlets, in $m^3$; |

| | |
|---|---|
| $H_{ad}$ | height of sluiceways, in m; |
| $B_{ad}$ | total width of sluiceways, in m; |
| $N_{ad}$ | number of sluiceways; |
| $e_{pl}$ | thickness of the walls, in m; and |
| $B_{1ad}$ | width of one sluiceway, in m. |

Still considering the case where the diversion is through sluiceways in the spillway, the **volume of concrete for larger amounts of cement and reinforcement steel** than for the ogee crest of the spillway, $V_{cen}$ ($m^3$), is given by:

$$V_{cen} = V_{cet} + V_{ces} + V_{cep}$$

in the case of a stilling basin:

$$V_{cet} = (0.27 \times H_{cp} + d_{14}) \times 0.25 \times H_{ad} \times B_{ad}$$

$$V_{ces} = (0.27 \times H_{cp} + d_{15}) \times 1.5 \times B_{ad}$$

$$V_{cep} = \left(0.27 \times H_{cp} + \frac{d_{14} + d_{15}}{2}\right) \times H_{ad} \times (N_{ad} + 1) \times e_{pl}$$

$$d_{14} = 1.46 \times H_{cp}^{0.46} \times (p_v - H_{ad})^{0.54}$$

$$d_{15} = 1.46 \times H_{cp}^{0.46} \times p_v^{0.54}$$

and for a ski jump:

$$V_{cet} = (L_{vt} - d_0) \times 0.25 \times H_{ad} \times B_{ad}$$

$$V_{ces} = L_{vt} \times 1.5 \times B_{ad}$$

$$V_{cep} = \left(L_{vt} - \frac{d_0}{2}\right) \times H_{ad} \times (N_{ad} + 1) \times e_{pl}$$

$$d_0 = (L_{vt} - 0.27 \times H_{cp} - d_{16} - 0.836 \times R_{se} + 0.15 \times H_{cp}) \times \frac{H_{ad}}{h_4}$$

$$d_{16} = 1.46 \times H_{cp}^{0.46} \times (p_v - h_4)^{0.54}$$

$$h_4 = El_{se} - 0.25 \times H_{ad} - El_{ca}$$

where:

| | |
|---|---|
| $V_{cet}$ | volume of concrete for the sluiceway roofs, in $m^3$; |
| $V_{ces}$ | volume of concrete for the sluiceway sills, in $m^3$; |
| $V_{cep}$ | volume of concrete for the sluiceway walls, in $m^3$; |
| $H_{cp}$ | height of the spillway gates, in m; |
| $H_{ad}$ | height of sluiceways, in m; |
| $B_{ad}$ | total width of sluiceways, in m; |
| $N_{ad}$ | number of sluiceways; |
| $e_{pl}$ | thickness of the walls, in m; |
| $p_v$ | vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m; |
| $L_{vt}$ | total length of the spillway, in m; |
| $R_{se}$ | radius of curvature of the ski jump, in m; |
| $El_{se}$ | elevation of the ski jump sill, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel to the sluiceways, in m; |
| $d_i$, $h_i$ | secondary dimensions, in m. |

The amounts of cement and reinforcement steel are:

| | cement (kg/m³) | reinforcement steel (kg/m³) |
|---|---|---|
| ogee crest, buttress, beneath the deflector baffle, sluiceway sills and inlets | 200 | 20 |
| downstream face of the ogee crest, stilling basin and deflector baffle | 250 | 50 |
| walls and walls | 250 | 80 |
| bridge | 300 | 100 |
| cement and reinforcement steel with greater unit quantities | 250 | 80 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of spillway (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- ogee crest, buttress, beneath the deflector baffle: 113.00/$m^3$
- walls, walls, stilling basin and deflector baffle: 200.00/$m^3$
- bridge: 474.00/$m^3$
- sluiceway sills and inlets and concrete with greater unit quantities: 174.00/$m^3$

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

### Tainter Gates (account .12.18.28.23.16)

The **acquisition cost of each surface tainter gate** for the spillway, $C_{cp}$ (R$), – FOB cost excluding tranportation and insurance, assembly and testing and provisions for taxes payable, depending on the current tax regime – can be obtained from the expression below (or from Graph B.21, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $2.2 \leq z \leq 178$: $C_{cp} = 193.95 \times z^{0.5406}$

where:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$$

$H_x = H_{cp}$

where:

| | |
|---|---|
| z | parameter, in $m^4$; |
| $B_{cp}$ | width of the gates, in m; |
| $H_{cp}$ | height of the gates, in m; and |
| $H_x$ | maximum hydrostatic load on the gate sill, in m. |

The following percentages must be added to the FOB price:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for taxes and charges payable on the equipment.

### Stoplogs

The **acquisition cost of each stoplog** for the spillway, $C_{sl}$ (R$), – FOB cost – can be obtained from the expression below (or from Graph B.24, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.34 \le z \le 177$: $C_{sl} = 72.9 \times z^{0.716}$

where: $$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$$

where:

| | |
|---|---|
| z | parameter, in $m^4$; |
| $B_{cp}$ | width of the gates, in m; |
| $H_{cp}$ | height of the gates, in m; and |
| $H_x$ | maximum hydrostatic load on the gate sill, in m. |

The cost of transportation and insurance, assembly and testing, taxes and charges payable on the equipment must be added to the FOB cost.

The **overall acquisition cost of fixed parts and parts embedded in the concrete** of the stoplogs for the spillway, $C_{gpf}$ (R$), – FOB cost – is given by the expression below. The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

$$C_{gpf} = N_{cp} \times \left[2 \times (H_{cp} + H_{bl}) + B_{cp}\right] \times 2084.80$$

where:

| | |
|---|---|
| $N_{cp}$ | number of gates; |
| $H_{cp}$ | height of the gates, in m; |
| $H_{bl}$ | height of the spillway freeboard, in m; and |
| $B_{cp}$ | width of the gates, in m. |

The cost of transportation and insurance, assembly and testing, taxes and charges payable on the equipment must be added to the FOB cost.

### Crane (account .12.18.28.23.20)

The **acquisition cost of the gantry crane** for the spillway, $C_{pcr}$ (R$), – FOB cost – can be obtained from the expressions below (or from Graph B.26, annex B, as a function of the dimensions of the stoplog and the maximum hydrostatic load on its sill). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.34 \le z \le 13.15$: $C_{pcr} = 141.34 \times z^{0.3555}$

valid for $13.15 < z \le 176.43$: $C_{pcr} = -0.0082 \times z^2 + 6.8982 \times z + 263.93$

where: $$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$$

where:

| | |
|---|---|
| z | parameter, in $m^4$; |
| $B_{cp}$ | width of the gates, in m; |
| $H_{cp}$ | height of the gates, in m; and |
| $H_x$ | maximum hydrostatic load on the gate sill, in m. |

The cost of transportation and insurance, assembly and testing, taxes and charges payable on the equipment must be added to the FOB cost.

### Other Costs (account .12.18.28.17)

**Other costs** are estimated as accounting for 2% of the total cost of the corresponding structure.

## GATED SURFACE ABUTMENT SPILLWAYS (ACCOUNT .12.18.28)

### Basic Data

The main **information required for dimensioning purposes** can be obtained from the overall layout and from item 5.1.2. (Hydrometeorological Data) as follows:

- coefficient for determining the initial height of the gates, $k_v$;
- height of the gates $H_{cp}$, in m;
- design flood through the spillway, $Q_v$, in $m^3/s$, from item 5.1.2.;
- 100-year flood flow, $Q_c$, in $m^3/s$, from item 5.1.2.;
- maximum normal water level in the reservoir, $NA_{max}$, from item 4.6.;
- elevation of the bottom of the approach channel, $El_{ca}$, from item 5.7.3. , in m;
- slope of the upstream face of the ogee crest, horizontal distance for a 1.0 m difference in level, $m_m$ in m;
- slope of the chute, tangent of the absolute value of the angle with the horizontal, $i_{cl}$;, in m/m;
- water level in the downstream channel for a 100-year flood, $NA_{ccr}$, from item 5.1.2, in m.

The **main information used for quantification** purposes is:

- mean elevation of the land in the area of the spillway per se, including the energy dissipator, $El_{te}$, in m;
- mean elevation of the land in the chute area, exclusively, $El_{tc}$, in m;
- mean elevation of the land in the stilling basin or ski jump area, exclusively, $El_{tde}$, in m;
- mean thickness of the layer of soil in the area of the spillway per se, $e_{te}$, in m;
- mean elevation of the land at section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the approach channel, $El_{tai}$, in m;
- mean elevation of the land at section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the downstream channel, $El_{tri}$, in m;
- elevation of the bottom of the downstream channel, $El_{cr}$, in m;
- mean length of the approach channel, $L_{ca}$, in m;
- mean length of the downstream channel, $L_{cr}$, in m; and
- thickness of the concrete lining for the stilling basin sill, $e_c$, in m, when applicable.

### Considerations and recommendations

This text relates to spillways with a typical cross-section as shown in Fig. 5.7.5.06 and Fig. 5.7.5.07.

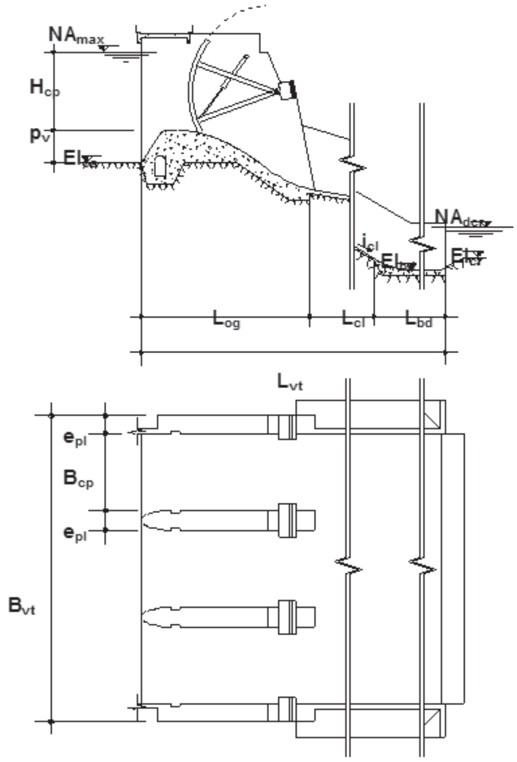



Fig. 5.7.5.06 – Typical cross-section of a gated surface abutment spillway with a stilling basin.

An initial estimate of the **height of the gates**, $H_{cp}$ (m), is given by:

$$H_{cp} = k_v \times Q_v^{0.4} \leq 21.0m$$

where:

| $k_v$ | For spillways with |
|---|---|
| 0.65 | two gates |
| 0.55 | three gates |
| 0.45 | five gates |
| 0.35 | ten gates |

where:

| | |
|---|---|
| $k_v$ | coefficient for determining the initial height of the gates; and |
| $Q_v$ | design flow through the spillway, in m³/s. |

Intermediate values can be used for $k_v$.

In the absence of more accurate information, 10% is the recommended **slope of the chute**.

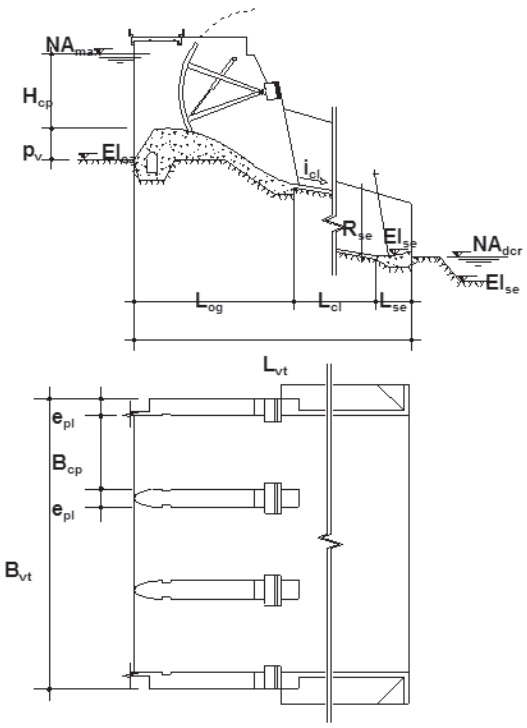



Fig. 5.7.5.07 – Typical cross-section of a gated surface abutment spillway with a ski jump.

When **dimensioning the stilling basin**, the recommended range for the Froude number before the hydraulic jump is betwen 4.5 and 9.0, in order to ensure its stability. One way of raising the Froude number to 4.5 would be to lower the bottom of the stilling basin beyond the result given by the calculation. Likewise, one way to reduce the Froude number to 9.0 would be to reduce the width of the stilling basin.

In the absence of more accurate information, use 1.0 m as the **thickness of the concrete lining for the bottom of the stilling basin**.

### Discharge Coefficient

The **discharge coefficient**, $C_d$, for the typical cross-section recommended, can be obtained from Graph 5.7.5.03 (Bureau of Reclamation 1977) or from the equivalent expressions:

for $m_m$=1.0 and 0.100 < z ≤ 0.524:

$$C_d = 1.9507 \times z^3 - 2.9011 \times z^2 + 1.5498 \times z + 1.8274$$

for $m_m$=1.0 and 0.524 < z ≤ 0.813:

$$C_d = 0.1592 \times z^3 - 0.4409 \times z^2 + 0.4248 \times z + 1.9984$$

for $m_m$=1.0 and 0.813 < z ≤ 1.800:

$$C_d = 0.0159 \times z + 2.1256$$

for $m_m$=0.67 and 0.100 < z ≤ 0.497:

$$C_d = 2.5495 \times z^3 - 3.6032 \times z^2 + 1.8832 \times z + 1.7678$$

$m_m$=0.67 and 0.497 < z ≤ 0.759:

$$C_d = 0.2261 \times z^3 - 0.6256 \times z^2 + 0.6137 \times z + 1.9481$$

$m_m$=0.67 and 0.759 < z ≤ 1.800:

$$C_d = 0.0242 \times z^3 - 0.1143 \times z^2 + 0.1775 \times z + 2.0734$$

$m_m$=0.33 and 0.100 < z ≤ 0.505:

$$C_d = 2.4283 \times z^3 - 3.5181 \times z^2 + 1.9125 \times z + 1.7265$$

$m_m$=0.33 and 0.505 < z ≤ 0.755:

$$C_d = 0.2514 \times z^3 - 0.6927 \times z^2 + 0.6896 \times z + 1.9033$$

$m_m$=0.33 and 0.755 < z ≤ 1.800:

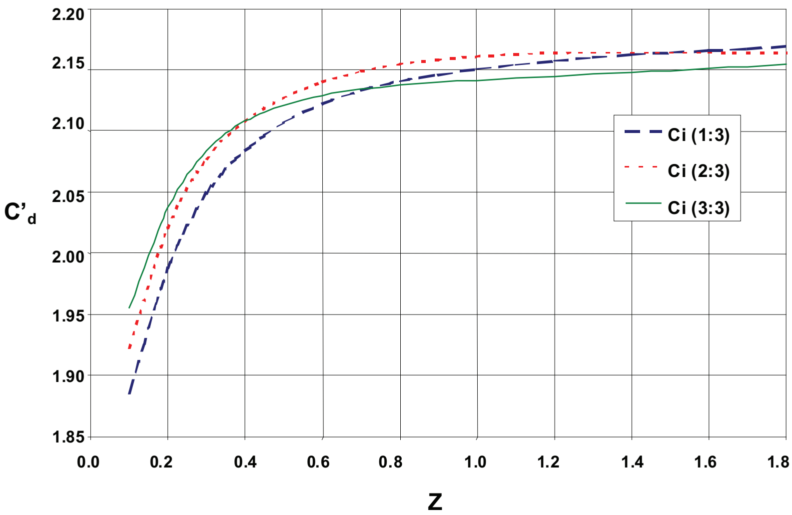
$$C_d = 0.02 \times z^3 - 0.0985 \times z^2 + 0.1782 \times z + 2.0508$$

where:

$$z = \frac{p_v}{H_{cp}} \qquad p_v = NA_{max} - H_{cp} - El_{ca}$$

where:

| | |
|---|---|
| z | adimensional parameter; |
| $p_v$ | difference in height between the ogee crest and the bottom of the approach channel, in m; |
| $H_{cp}$ | height of the gates, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel, in m. |




Graph 5.7.5.03 – Discharge coefficient for spillways with an inclined upstream face.

### Dimensions of the gates and spillway

The **real width of the openings**, $B_{rvt}$ (m), is given by:

$$B_{rvt} = \frac{Q_v}{C_d \times H_{cp}^{3/2}}$$

where:

| | |
|---|---|
| $Q_v$ | design flood through the spillway, in m³/s; |
| $C_d$ | discharge coefficient; and |
| $H_{cp}$ | height of the gates, in m. |

The **number of gates**, $N_{cp}$, is given by:

$$N_{cp} = int\left(\frac{B_{rvt}}{H_{cp}} + 0,999\right)$$

where:

| | |
|---|---|
| $B_{rvt}$ | real width of the openings, in m; |
| $H_{cp}$ | height of the gates, in m; and |
| int(x) | function that returns the integer part of x. |

The **width of the gates**, $B_{cp}$ (m), is given by:

$$B_{cp} = 0.05 \times int\left(\frac{1}{0.05} \times \frac{B_{rvt}}{N_{cp}} + 0.5\right)$$

where:

| | |
|---|---|
| $B_{rvt}$ | real width of the openings, in m; |
| $N_{cp}$ | number of gates; and |
| int(x) | function that returns the integer part of x. |

The **thickness of the pillars**, $e_{pl}$ (m), is given by:

$$e_{pl} = 0.12 \times H_{cp} + 2.4$$

where:

| | |
|---|---|
| $H_{cp}$ | height of the gates, in m. |

The **total width of the spillway**, $B_{vt}$ (m), perpendicular to flow, is given by:

$$B_{vt} = (N_{cp} + 1) \times e_{pl} + N_{cp} \times B_{cp}$$

where:

| | |
|---|---|
| $N_{cp}$ | number of gates; |
| $e_{pl}$ | thickness of the pillars, in m; and |
| $B_{cp}$ | width of the gates, in m. |

The **width of the chute**, $B_{cl}$ (m), is given by:

$$B_{cl} = (N_{cp} - 1) \times e_{pl} + N_{cp} \times B_{cp}$$

where:

| | |
|---|---|
| $N_{cp}$ | number of gates; |
| $e_{pl}$ | thickness of the pillars, in m; and |
| $B_{cp}$ | width of the gates, in m. |

The **length of the ogee crest** in the direction of flow, $L_{og}$ (m), is given by:

$$L_{og} = 1.66 \times H_{cp} + m_m \times p_v$$

where:

| | |
|---|---|
| $H_{cp}$ | height of the gates, in m; |
| $m_m$ | slope of the upstream face of the ogee crest, horizontal distance for a 1.0 m difference in level; and |
| $p_v$ | difference in height between the ogee crest and the bottom of the approach channel, in m. |

The **length of the chute**, $L_{cl}$ (m), is given by:

for the stilling basin: $L_{cl} = \dfrac{NA_{max} - 1.69 \times H_{cp} - El_{bd}}{i_{cl}}$

for the ski jump: $L_{cl} = \frac{NA_{max} - 1.69 \times H_{cp} - El_{se} - 0.03 \times R_{se}}{i_{cl}}$

where:

| | |
|---|---|
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $H_{cp}$ | height of the gates, in m; |
| $El_{bd}$ | elevation of the bottom of the stilling basin, defined below, in m; |
| $El_{se}$ | elevation of the ski jump sill, defined below, in m; |
| $R_{se}$ | radius of curvature of the ski jump, defined below, in m; and |
| $i_{cl}$ | slope of the chute. |

The **total length** of the spillway, $L_{vt}$ (m), is given by:

for the stilling basin: $L_{vt} = L_{og} + L_{cl} + L_{bd}$

for the ski jump: $L_{vt} = L_{og} + L_{cl} + L_{se}$

where:

| | |
|---|---|
| $L_{og}$ | length of the ogee crest, in m; |
| $L_{cl}$ | length of the chute, in m; |
| $L_{bd}$ | length of the stilling basin, defined below, in m; and |
| $L_{se}$ | length of the ski jump at the foundations, defined below, in m. |

## Stilling Basin

The **width of the stilling basin**, $B_{bd}$ (m), is given by:

$B_{bd} = B_{cl}$

where:

| | |
|---|---|
| $B_{cl}$ | width of the chute, in m. |

The **stilling basin depth** is determined iteratively, based on the Froude number before the hydraulic jump, in section one of Fig. 5.7.5.03, for the 100-year flood flow .

An elevation is picked for the bottom of the stilling basin and its suitability is checked by calculating the **velocity**, $v_1$ (m/s), the **depth of discharge**, $y_1$ (m), and the **Froude number**, $Fr_1$, before the hydraulic jump, **depth of discharge** after the hydraulic jump, $y_2$ (m), and finally the elevation of the **bottom of the stilling basin**.

Should the elevation of the bottom of the stilling basin be different from the figure first picked, the calculations should be redone until the level of precision required is reached.

$$v_1 = \sqrt{k \times 2 \times g \times (NA_{max} - El_{bd})}$$

$$y_1 = \frac{Q_c}{B_{bd} \times v_1} \qquad Fr_1 = \frac{v_1}{\sqrt{g \times y_1}}$$

$$y_2 = \left(\frac{y_1}{2}\right) \times \left(\sqrt{1 + 8 \times Fr_1^2} - 1\right)$$

$$El_{bd} = NA_{ccr} - y_2$$

where:

| | |
|---|---|
| $v_1$ | mean velocity of discharge before the hydraulic jump, in m/s; |
| k | 0.9 – reduction coefficient for the energy head; |

| | |
|---|---|
| g | 9.81 m/s$^2$ – acceleration due to gravity; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_{bd}$ | elevation of the bottom of the stilling basin, in m; |
| $y_1$ | depth of discharge before the hydraulic jump, in m; |
| $Q_c$ | 100-year flood flow, in m$^3$/s; |
| $B_{bd}$ | width of the stilling basin, in m; |
| $Fr_1$ | Froude number before the hydraulic jump; |
| $y_2$ | depth of discharge after the hydraulic jump, in m; and |
| $NA_{ccr}$ | water level in the downstream channel for a 100-year flood, in m. |

The **radius of curvature at the stilling basin inlet**, $R_{bd}$ (m), is given by (PETERKA):

$$R_{bd} = 6 \times y_1$$

where:

| | |
|---|---|
| $y_1$ | depth of discharge before the hydraulic jump, in m. |

The **length of the stilling basin**, $L_{bd}$ (m), is given by:

$$L_{bd} = 6 \times y_2 + R_{bd} \times \tan\left[\frac{a\tan(i_{cl})}{2}\right]$$

where:

| | |
|---|---|
| $y_2$ | depth of discharge after the hydraulic jump, in m; |
| $i_{cl}$ | slope of the chute; and |
| $R_{bd}$ | radius of curvature at the stilling basin inlet, in m. |

### Ski Jump

The **width of the ski jump**, $B_{se}$ (m), is given by:

$$B_{se} = B_{cl}$$

where:

| | |
|---|---|
| $B_{cl}$ | width of the chute, in m. |

The **elevation of the ski jump sill**, $El_{se}$, is given by:

$$El_{se} = NA_{ccr} + 1.0 \geq El_{cr}$$

where:

| | |
|---|---|
| $NA_{ccr}$ | water level in the downstream channel for a 100-year flood, in m; |
| $El_{cr}$ | elevation of the bottom of the downstream channel, in m. |

The **radius of curvature of the ski jump**, $R_{se}$ (m), is given by (PETERKA):

$$R_{se} = 6 \times y$$

where:

$$y = \frac{Q_c}{B_{se} \times v}$$

$$v = \sqrt{k \times 2 \times g \times (NA_{max} - El_{se})}$$

where:

| | |
|---|---|
| y | depth of the water column at the ski jump, in m; |
| $Q_c$ | 100-year flood flow, in m$^3$/s; |
| $B_{se}$ | width of the ski jump, in m; |

| | |
|---|---|
| v | velocity of the water column at the ski jump, in m/s; |
| k | 0.9 – reduction coefficient for the energy head; |
| g | 9.81 m/s$^2$ – acceleration due to gravity; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; and |
| $El_{se}$ | elevation of the ski jump sill, in m. |

The **length of the ski jump**, $L_{se}$ (m), is given by:

$$L_{se} = 0.80 \times R_{se} + 1.5$$

where:

| | |
|---|---|
| $R_{se}$ | radius of curvature of the ski jump, in m. |

### Common Excavation (account .12.19.30.12.10)

The **volume of common excavation** for the spillway, $V_{tvt}$ (m$^3$), is given by:

$$V_{tvt} = V_{tca} + V_{tes} + V_{tcr}$$

where:

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

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

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

$$B_{ca} = B_{vt} - 2 \times \left( e_{pl} - 1.0 \right)$$

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

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

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

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

for the stilling basin: $B_{cr} = B_{bd} + 2 \times 1.0$

for the ski jump: $B_{cr} = B_{se} + 2 \times 1.0$

where:

| | |
|---|---|
| $V_{tca}$ | volume of common excavation for the approach channel, in m$^3$; |
| $V_{tes}$ | volume of common excavation for the structure, in m$^3$; |
| $V_{tcr}$ | volume of common excavation for the downstream channel, in m$^3$; |
| $V_{tai}$ | volume of common excavation per meter at section i of the approach channel, in m$^3$/m; |
| $L_{ca}$ | length of the approach channel, in m; |
| $B_{ca}$ | width of the bottom of the approach channel, in m; |
| $h_{rai}$ | depth of excavation in rock at section i of the approach channel, in m; |
| $e_{te}$ | mean thickness of the layer of soil in the area of the spillway per se, in m; |
| $El_{tai}$ | mean elevation of the land at section i of the approach channel, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel, in m; |
| $B_{vt}$ | total width of the spillway, in m; |
| $e_{pl}$ | thickness of the pillars, in m; |
| $L_{vt}$ | total length of the spillway, in m; |

| | |
|---|---|
| $V_{tri}$ | volume of common excavation per meter at section i for the downstream channel, in $m^3/m$; |
| $L_{cr}$ | length of the downstream channel, in m; |
| $B_{cr}$ | width of the bottom of the downstream channel, in m; |
| $h_{rri}$ | depth of excavation in rock at section i of the downstream channel, in m; |
| $El_{tri}$ | mean elevation of the land at section i perpendicular to the longitudinal axis of the downstream channel, in m; |
| $El_{cr}$ | elevation of the bottom of the downstream channel, in m; |
| $B_{bd}$ | width of the stilling basin, in m; and |
| $B_{se}$ | width of the ski jump, in m. |

The unit price of **common excavation** is R$ 7.60/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation (account .12.19.30.12.11)

Figure 5.7.5.05 shows typical cross-sections for the excavation of approach and downstream channels.

The **volume excavated in rock**, $V_{rvt}$ ($m^3$), is given by:

$$V_{rvt} = V_{rca} + V_{rog} + V_{rcl} + V_{rbc} + V_{rde} + V_{rbe} + V_{rcr}$$

where:

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

$$V_{rai} = (B_{ca} - 6 + 0.6 \times h_{rai}) \times h_{rai}$$

$$V_{rog} = L_{og} \times \left[ El_{te} - e_{te} - (El_{ca} - 2) \right] \times B_{vt}$$

$$V_{rcl} = L_{cl} \times \left[ El_{tc} - e_{te} - (El_{tc} - 0.7) \right] \times (B_{cl} + 2)$$

$$V_{rbc} = 2 \times L_{cl} \times 0.3 \times h_{rc}^2$$

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

$$V_{rri} = (B_{cr} - 6 + 0.6 \times h_{rri}) \times h_{rri}$$

$$h_{rc} = El_{tc} - e_{te} - (El_{cm} - 0.7) \ \geq \ 0$$

for the stilling basin:

$$V_{rde} = L_{bd} \times \left[ El_{te} - e_{te} - (El_{bd} - e_c) \right] \times (B_{bd} + 2)$$

$$V_{rbe} = 2 \times L_{bd} \times 0.3 \times h_{re}^2$$

$$El_{cm} = \frac{NA_{max} - 1.69 \times H_{cp} + El_{bd}}{2}$$

$$h_{re} = El_{tde} - e_{te} - (NA_{ccr} - 5.0) \geq 0$$

for the ski jump:

$$V_{rde} = L_{se} \times \left[El_{te} - e_{te} - (El_{se} - 2)\right] \times (B_{se} + 2)$$

$$V_{rbe} = 2 \times L_{se} \times 0.3 \times h_{rs}^2$$

$$El_{cm} = \frac{NA_{max} - 1.69 \times H_{cp} + El_{se}}{2}$$

$$h_{rs} = El_{tde} - e_{te} - (El_{se} - 2.0) \geq 0$$

where:

| | |
|---|---|
| $V_{rca}$ | volume of rock excavated for the approach channel, in $m^3$; |
| $V_{rog}$ | volume of rock excavated in the ogee crest area, in $m^3$; |
| $V_{rcl}$ | volume of rock excavated in the chute area, in $m^3$; |
| $V_{rbc}$ | volume of rock excavated for berms in the chute section, in $m^3$; |
| $V_{rde}$ | volume of rock excavated in the area of the stilling basin or ski jump, in $m^3$; |
| $V_{rbe}$ | volume of rock excavated for berms in the stilling basin or ski jump section, in $m^3$; |
| $V_{rcr}$ | volume of rock excavated for the downstream channel, in $m^3$; |
| $V_{rai}$ | volume of excavated rock per meter at section i of the approach channel, in $m^3/m$; |
| $L_{ca}$ | length of the approach channel, in m; |
| $B_{ca}$ | width of the bottom of the approach channel, in m; |
| $h_{rai}$ | depth of excavation in rock at section i of the approach channel, in m; |
| $L_{og}$ | length of the ogee crest, in m; |
| $El_{te}$ | mean elevation of the land in the area of the spillway per se, in m; |
| $e_{te}$ | mean thickness of the layer of soil in the area of the spillway per se, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel, in m; |
| $B_{vt}$ | total width of the spillway, in m; |
| $L_{cl}$ | length of the chute, in m; |
| $El_{cm}$ | mean elevation of the chute, in m; |
| $B_{cl}$ | width of the chute, in m; |
| $h_{rc}$ | depth of excavation in rock in the chute area, in m; |
| $V_{rri}$ | volume of excavated rock per meter at section i for the downstream channel, in $m^3/m$; |
| $L_{cr}$ | length of the downstream channel, in m; |
| $B_{cr}$ | width of the bottom of the downstream channel, in m; |
| $h_{rri}$ | depth of excavation in rock at section i of the downstream channel, in m; |
| $El_{tc}$ | mean elevation of the land in the chute area, exclusively, in m; |
| $L_{bd}$ | length of the stilling basin, in m; |
| $El_{bd}$ | elevation of the bottom of the stilling basin, in m; |
| $e_c$ | Thickness of the concrete lining for the stilling basin sill, in m; |
| $B_{bd}$ | width of the stilling basin, in m; |
| $h_{re}$ | depth of excavation in rock in the stiling basin area, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $H_{cp}$ | height of the gates, in m; |
| $El_{tde}$ | mean elevation of the land in the stilling basin or ski jump area, exclusively, in m; |
| $NA_{ccr}$ | water level in the downstream channel for a 100-year flood, in m; |

| | |
|---|---|
| $L_{se}$ | length of the ski jump, in m; |
| $El_{se}$ | elevation of the ski jump sill, in m; |
| $B_{se}$ | width of the ski jump, in m; and |
| $h_{rs}$ | depth of excavation in rock in the ski jump area, in m. |

The unit price of **excavation in rock** is R$ 21.00/m$^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.19.30.13)

The area of foundation to be cleaned, $A_{lf}$ (m$^2$), is given by:

$$A_{lf} = B_{vt} \times L_{vt}$$

where:

| | |
|---|---|
| $B_{vt}$ | total width of the spillway, in m; and |
| $L_{vt}$ | total length of the spillway, in m. |

The length of the **grout holes** and the **drainage line**, $L_{tf}$ (m), is given by:

$$L_{tf} = \frac{B_{vt}}{3.0} \times L_{1tf}$$

$$L_{1tf} = 1.5 \times (NA_{max} - El_{ca}) \le 40m$$

where:

| | |
|---|---|
| $B_{vt}$ | total width of the spillway, in m; |
| $L_{1tf}$ | length of one grout hole, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel, in m; |
| 3.0 | space between the grout holes, in m. |

The **total length of the rock anchors** in the stilling basin, $L_{tfc}$(m), when necessary, is given by:

$$L_{tfc} = B_{bd} \times L_{bd}$$

where:

| | |
|---|---|
| $B_{bd}$ | width of the stilling basin, in m; and |
| $L_{bd}$ | length of the stilling basin, in m. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m²
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account .12.19.30.14)

The **volume of concrete** for the spillway, $V_{cvt}$ ($m^3$), is given by:

$$V_{cvt} = V_{cog} + V_{cpl} + V_{cpo} + V_{ccl} + V_{cde} + V_{cmv}$$

where:

$$V_{cog} = (0.165 \times H_{cp}^2 + 0.67 \times p_v \times H_{cp} + 0.84 \times p_v^2 + 32) \times B_{vt}$$

$$V_{cpl} = (N_{cp} + 1) \times e_{pl} \times (1.85 \times H_{cp}^2 + 7.1 \times H_{cp} + 15)$$

$$V_{cpo} = 6.0 \times B_{vt}$$

$$V_{ccl} = L_{cl} \times [0.7 \times B_{cl} + 2 \times (H_{cl} + 0.7) \times 1.0]$$

$$H_{cl} = 0.95 \times H_{cp}$$

for the stilling basin:

$$V_{cde} = L_{bd} \times e_c \times (B_{bd} + 2.0)$$

$$V_{cmv} = 2 \times \left[ L_{bd} \times (2.0 + y_2 + e_c) + \frac{d_1^2}{2 \times i_{cl}} \right] \times 1.0$$

$$d_1 = 2.0 + y_2 - H_{cl}$$

for the ski jump:

$$V_{cde} = (0.12 \times R_{se}^2 + 0.93 \times R_{se} + 0.53) \times (B_{se} + 2.0)$$

$$V_{cmv} = 2 \times L_{se} \times H_{cl} \times 1.0$$

where:

| | |
|---|---|
| $V_{cog}$ | volume of concrete for the ogee crest, in m³; |
| $V_{cpl}$ | volume of concrete for the pillars, in m³; |
| $V_{cpo}$ | volume of concrete for the bridge, in m³; |
| $V_{ccl}$ | volume of concrete for the chute, including walls, in m³; |
| $V_{cde}$ | volume of concrete for the stilling basin or ski jump, including walls, in m³; |
| $H_{cp}$ | height of the gates, in m; |
| $p_v$ | Difference in height between the ogee crest and the bottom of the approach channel, in m; |
| $B_{vt}$ | total width of the spillway, in m; |
| $N_{cp}$ | number of gates; |
| $e_{pl}$ | Thickness of the pillars, in m; |
| $L_{cl}$ | length of the chute, in m; |
| $B_{cl}$ | width of the chute, in m; |

| | |
|---|---|
| $H_{cl}$ | height of the wall of the chute, in m; |
| $L_{bd}$ | length of the stilling basin, in m; |
| $e_c$ | Thickness of the concrete lining for the stilling basin sill, in m; |
| $B_{bd}$ | width of the stilling basin, in m; |
| $y_2$ | depth of discharge after the hydraulic jump, in m; |
| $i_{cl}$ | slope of the chute; |
| $R_{se}$ | radius of curvature of the ski jump, in m; |
| $B_{se}$ | Width of the ski jump, in m; |
| $L_{se}$ | length of the ski jump, in m; and |
| $d_i$ | secondary dimension, in m. |

The amounts of cement and reinforcement steel are:

| | cement (kg/m³) | reinforcement steel (kg/m³) |
|---|---|---|
| ogee crest with stilling basin | 200 | 20 |
| ogee crest with ski jump | 250 | 50 |
| pillars, chute, stilling basin and walls | 270 | 80 |
| pillars, chute, ski jump and walls | 250 | 80 |
| bridge | 300 | 100 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of spillway (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- ogee crest: 113.00/m³
- pillars, chute, stilling basin, ski jump and walls: 200.00/m³
- bridge: 474.00/m³

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Tainter Gates (account .12.18.28.23.16)

The **acquisition cost of each surface tainter gate** for the spillway, $C_{cp}$ (R$), – FOB cost excluding tranportation and insurance, assembly and testing and provisions for taxes payable, depending on the current tax regime – can be obtained from the expression below (or from Graph B.21, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $2.2 \leq z \leq 178$: $C_{cp} = 193.95 \times z^{0.5406}$

where:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000} \qquad H_x = H_{cp}$$

where:

| | |
|---|---|
| z | parameter, in m$^4$; |
| $B_{cp}$ | width of the gates, in m; |
| $H_{cp}$ | height of the gates, in m; and |
| $H_x$ | maximum hydrostatic load on the gate sill, in m. |

The following percentages must be added to the FOB price:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for taxes and charges payable on the equipment.

### Stoplogs (account .12.18.28.23.17)

The **acquisition cost of each stoplog** for the spillway, $C_{sl}$ (R$),– FOB cost – can be obtained from the expression below (or from Graph B.24, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

valid for $0.34 \leq z \leq 177$: $C_{sl} = 72.9 \times z^{0.716}$

where: $z = \dfrac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$

where:

| | |
|---|---|
| z | parameter, in m$^4$; |
| $B_{cp}$ | width of the gates, in m; |
| $H_{cp}$ | height of the gates, in m; and |
| $H_x$ | maximum hydrostatic load on the gate sill, in m. |

It is recommended that one stoplog be used for every ten tainter gates.

The cost of transportation and insurance, assembly and testing, taxes and charges payable on the equipment must be added to the FOB cost.

The **total acquisition cost of fixed parts and parts embedded in the concrete** for the spillway's stoplogs, $C_{gpf}$ (R$), – FOB cost – is given by the expression below. The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

$$C_{gpf} = N_{cp} \times \left[ 2 \times (H_{cp} + H_{bl}) + B_{cp} \right] \times 2084.80$$

where:

| | |
|---|---|
| $N_{cp}$ | number of gates; |
| $H_{cp}$ | height of the gates, in m; |
| $H_{bl}$ | height of the spillway freeboard, in m; and |
| $B_{cp}$ | width of the gates, in m. |

The cost of transportation and insurance, assembly and testing, taxes and charges payable on the equipment must be added to the FOB cost.

### Crane (account .12.18.28.23.20)

The **acquisition cost of the gantry crane** for the spillway, $C_{pcr}$ (R$), – FOB cost – can be obtained from the expressions below (or from Graph B.26, annex B, as a function of the dimensions of the stoplog and the maximum hydrostatic load on its sill). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

for $0.34 \le z \le 13.15$: $C_{pcr} = 141.34 \times z^{0.3555}$

for $13.15 < z \le 176.43$: $C_{pcr} = -0.0082 \times z^2 + 6.8982 \times z + 263.93$

where: $z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$

where:

| | |
|---|---|
| z | parameter, in $m^4$; |
| $B_{cp}$ | width of the gates, in m; |
| $H_{cp}$ | height of the gates, in m; and |
| $H_x$ | maximum hydrostatic load on the gate sill, in m. |

The cost of transportation and insurance, assembly and testing, taxes and charges payable on the equipment must be added to the FOB cost.

### Other Costs (account .12.18.28.17)

**Other costs** are estimated as accounting for 2% of the total cost of the corresponding structure.

## UNGATED SURFACE SPILLWAYS WITH A HIGH OGEE CREST (ACCOUNT .12.18.28)

### Basic Data

The main **information required for dimensioning** purposes can be obtained from the overall layout and from item 5.1.2. (Hydrometeorological Data), as follows:

- design flood through the spillway, $Q_v$ in m³/s, from item 5.1.2.;
- 100-year flood flow, $Q_c$ in m³/s, from item 5.1.2.;
- maximum water level of the reservoir under design flood conditions, $NA_{xmx}$, from item 5.1.2.;
- maximum normal water level in the reservoir, $NA_{max}$, from item 4.6.;
- elevation of the bottom of the approach channel to the sluiceways, $El_{ca}$, from item 5.7.3., when applicable;
- elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, $El_{cv}$;
- maximum water level in the downstream channel, $NA_{xcr}$, from item 5.1.2.;
- water level in the downstream channel for a 100-year flood, $NA_{ccr}$, from item 5.1.2.;
- elevation of the bottom of the downstream channel, $El_{cr}$;
- number of gaps in the bridge, when applicable; and
- height of sluiceways, $H_{ad}$ in m, from item 5.7.3., when applicable.

The **main information required for quantification** purposes is:

- mean elevation of the land in the area of the spillway per se, including the energy dissipator, $El_{te}$, in m;
- mean elevation of the land in the stilling basin or ski jump area, exclusively, $El_{tde}$, in m;
- mean thickness of the layer of soil in the area of the spillway per se, $e_{te}$, in m;
- mean elevation of the land at section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the approach channel, $El_{tai}$, in m;
- mean elevation of the land at section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the downstream channel, $El_{tri}$, in m;
- mean length of the approach channel at the part with no sluiceways, $L_{ca}$, in m;
- mean length of the approach channel in the part with sluiceways, $L_{cad}$, in m, when applicable;
- mean length of the downstream channel, $L_{cr}$, in m;
- thickness of the concrete lining for the stilling basin sill, $e_c$, in m, when applicable;
- width of one sluiceway, $B_{1ad}$ in m, from item 5.7.3., when applicable;
- total width of sluiceways, $B_{ad}$ in m, from item 5.7.3., when applicable;
- number of sluiceways, $N_{ad}$, from item 5.7.3., when applicable;
- thickness of the sluiceway walls, $e_{pl}$, in m, from item 5.7.3., when applicable;
- existence of a bridge.

### Considerations and recommendations

This text relates to spillways with a typical cross-section, as shown in Fig. 5.7.5.08 and 5.7.5.09.

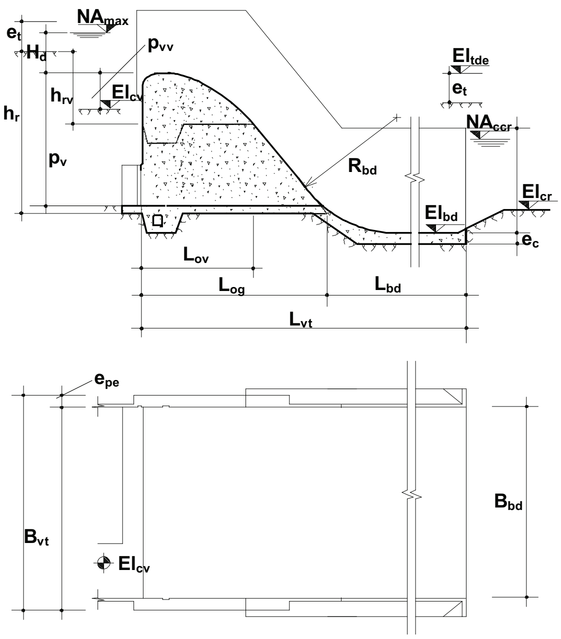



Fig. 5.7.5.08 – Typical cross-section and plan of an ungated surface spillway with a high ogee crest and a stilling basin.

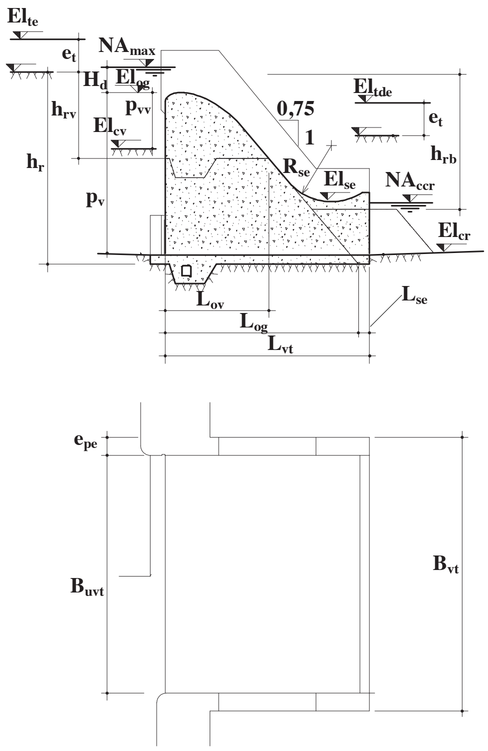



Fig. 5.7.5.09 – Typical cross-section and plan of an ungated surface spillway with a high ogee crest and a ski jump.

When the spillway is not all on the river bed, the **elevation of the foundations** at the two ends of the structure are often different, in which case an mean of these elevations should be used for the elevation of the bottom of the approach channel to the spillway, excluding the sluiceways.

The following should be true for **spillways with no sluiceways**: $El_{ca} = El_{cv}$

where:

| | |
|---|---|
| $El_{ca}$ | elevation of the bottom of the approach channel to the sluiceways, in m; and |
| $El_{cv}$ | elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m. |

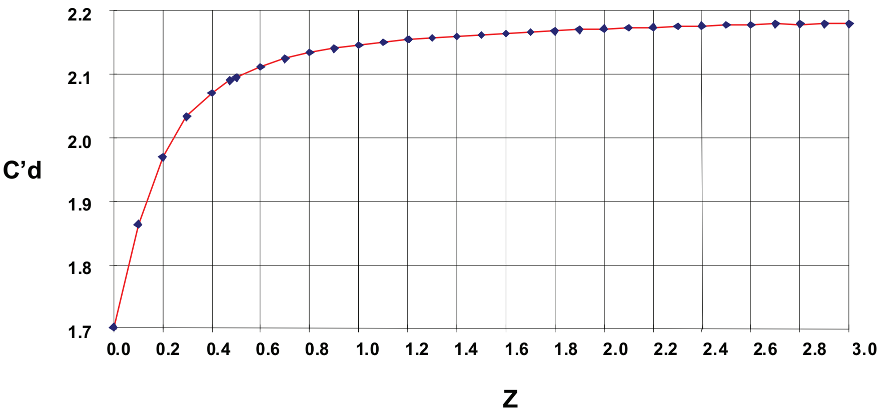
The thickness of the **layer of soil** on the river bed may be different from the thickness on the abutments, and there may often be no soil at all.

When **dimensioning the stilling basin**, the recommended range for the Froude number before the hydraulic jump is betwen 4.5 and 9.0, in order to ensure its stability. One way of raising the Froude number to 4.5 would be to lower the bottom of the stilling basin beyond the result given by the calculation. Likewise, one way to reduce the Froude number to 9.0 would be to reduce the width of the stilling basin.

In the absence of more accurate information, use 1.0 m as the **thickness of the concrete lining for the bottom of the stilling basin**.

### Discharge Coefficient

The **initial discharge coefficient**, $C'_d$, for the recommended typical cross-section, can be obtained from Graph 5.7.5.01 or from the equivalent expressions (Bureau of Reclamation, 1977 ):




Graph 5.7.5.01 – Initial Discharge Coefficient.

for $z \leq 0.475$: $C'_d = 2.535 \times z^3 - 3.61 \times z^2 + 1.96 \times z + 1.702$

for $0.475 < z \leq 1.2$: $C'_d = 0.145 \times z^3 - 0.475 \times z^2 + 0.559 \times z + 1.916$

for $1.2 < z \leq 3.0$: $C'_d = -0.0072 \times z^2 + 0.0442 \times z + 2.112$

for $z > 3.0$: $C'_d = 2.18$

where:

$$z = \frac{0.7 \times p_v + 0.3 \times p_{vv}}{H_d}$$

$p_v = NA_{max} - El_{ca}$ $\qquad$ $p_{vv} = NA_{max} - El_{cv}$

$H_d = NA_{xmx} - NA_{max}$

where:

| | |
|---|---|
| $z$ | adimensional parameter; |
| $p_v$ | vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m; |
| $p_{vv}$ | vertical distance between the ogee sill and the bottom of the spillway approach channel, sluiceways excluded, in m; |
| $H_d$ | maximum energy head on the spillway crest, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel to the sluiceways, in m; |
| $El_{cv}$ | elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m; and |
| $NA_{xmx}$ | maximum water level of the reservoir under design flood conditions, in m. |

The **reduction coefficient for the discharge coefficient when the outlet is submerged**, $k_c$, can be obtained from Graph 5.7.5.02 (Bureau of Reclamation, 1977) or from the equivalent expressions:

for Section I: $-4 \times u + 7 \times w + 2.6 \geq 0$:

$$k_c = -0.952 \times \left(\frac{1}{u}\right)^2 + 0.956 \times \left(\frac{1}{u}\right) + 0.767 \leq 1$$

for Section II $u < 3.6\,e - 4 \times u + 7 \times w + 2.6 < 0$:

$$k_c = 1.058 - \frac{4 \times (u + 5)}{860 \times w} \leq 1$$

for Section III $u \geq 3.6$:

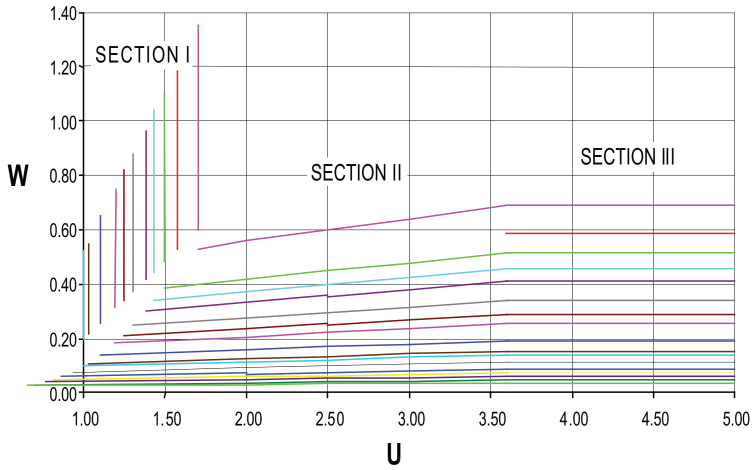
$$k_c = 1.058 - \frac{4}{100 \times w} \leq 1$$

where:

$$u = \frac{NA_{xmx} - El_{cr}}{H_d} \qquad w = \frac{NA_{xmx} - NA_{xcr}}{H_d}$$

where:

| | |
|---|---|
| u, w | adimensional parameters; |
| $NA_{xmx}$ | maximum water level of the reservoir under design flood conditions, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel, in m; |
| $H_d$ | maximum energy head on the spillway crest, in m; and |
| $NA_{xcr}$ | maximum water level in the downstream channel, in m. |




Graph 5.7.5.02 – Reduction coefficient for the discharge coefficient when the outlet is submerged.

The **corrected discharge coefficient**, $C_d$, is given by:

$$C_d = k_c \times C'_d$$

where:

| | |
|---|---|
| $k_c$ | reduction coefficient for the discharge coefficient; and |
| $C'_d$ | initial discharge coefficient. |

Spillway Dimensions

The **useful width** of the spillway perpendicular to flow, $B_{uvt}$ (m), is given by:

$$B_{uvt} = \frac{Q_v}{C_d \times H_d^{3/2}}$$

where:

| | |
|---|---|
| $Q_v$ | design flood through the spillway, in m³/s; |
| $C_d$ | discharge coefficient; and |
| $H_d$ | maximum energy head on the spillway crest, in m. |

The **thickness of the end pillars**, $e_{pe}$ (m), is given by:

$$e_{pe} = 0.12 \times H_d + 2.4$$

where:

| | |
|---|---|
| $H_d$ | maximum energy head on the spillway crest, in m. |

The **total width of the spillway**, $B_{vt}$ (m), perpendicular to flow, is given by:

$$B_{vt} = 0.05 \times \text{int}\left[\left(B_{uvt} + 2 \times e_{pe}\right) \times \frac{1}{0.05} + 0.5\right]$$

where:

| | |
|---|---|
| $N_{vãos}$ | number of openings in the spillway; |
| 2.0 | thickness of the intermediate pillars, in m; |
| $e_{pe}$ | thickness of the end pillars, in m; |
| $B_{vãos}$ | width of an opening, in m. |

The length of the ogee crest in the direction of flow, $L_{og}$ (m), is given by:

for the part with no sluiceways: $L_{ov} = 1.46 \times H_d^{0.46} \times (p_{vv} + 1.5)^{0.54} + 0.27 \times H_d$

for the part with sluiceways: $L_{og} = 1.46 \times H_d^{0.46} \times (p_v + 1.5)^{0.54} + 0.27 \times H_d$

where:

| | |
|---|---|
| $H_d$ | maximum energy head on the spillway crest, in m; and |
| $p_v$ | difference in height between the ogee crest and the bottom of the approach channel, in m. |

The **total length** of the spillway, $L_{vt}$ (m), is given by:

for spillways with a stilling basin and sluiceways: $L_{vt} = L_{og} + L_{bd}$

for spillways with a stilling basin and no sluiceways: $L_{vt} = L_{ov} + L_{bd}$

for spillways with a ski jump and sluiceways: $L_{vt} = L_{og} + L_{se}$

for spillways with a ski jump and no sluiceways: $L_{vt} = L_{ov} + L_{se}$

where:

| | |
|---|---|
| $L_{og}$ | length of the ogee crest with sluiceways, in m; |
| $L_{ov}$ | length of the ogee crest with no sluiceways, in m; |
| $L_{bd}$ | length of the stilling basin, defined below, in m; and |
| $L_{se}$ | length of the ski jump at the foundations, defined below, in m. |

### Stilling Basin

The **width of the stilling basin**, $B_{bd}$ (m), is given by:

$$B_{bd} = B_{uvt}$$

where:

| | |
|---|---|
| $B_{uvt}$ | useful width of the spillway, in m. |

The **stilling basin depth** is determined iteratively, based on the Froude number before the hydraulic jump, in section one of Fig. 5.7.5.03, for the 100-year flood flow .

An elevation is picked for the bottom of the stilling basin and its suitability is checked by calculating the **velocity**, $v_1$ (m/s), the **depth of discharge**, $y_1$ (m), and the **Froude number**, $Fr_1$, before the hydraulic jump, **depth of discharge** after the hydraulic jump, $y_2$ (m), and finally the elevation of the **bottom of the stilling basin**.

Should the elevation of the bottom of the stilling basin be different from the figure first picked, the calculations should be redone until the level of precision required is reached.

$$v_1 = \sqrt{k \times 2 \times g \times (NA_{xmx} - El_{bd})}$$

$$y_1 = \frac{Q_c}{B_{bd} \times v_1}$$

$$Fr_1 = \frac{v_1}{\sqrt{g \times y_1}}$$

$$y_2 = \left(\frac{y_1}{2}\right) \times \left(\sqrt{1 + 8 \times Fr_1^2} - 1\right)$$

$$El_{bd} = NA_{ccr} - y_2$$

where:

| | |
|---|---|
| $v_1$ | mean velocity of discharge before the hydraulic jump, in m/s; |
| k | 0.9 – reduction coefficient for the energy head; |
| g | 9.81 m/s$^2$ – acceleration due to gravity; |
| $NA_{xmx}$ | maximum water level of the reservoir under design flood conditions, in m; |
| $El_{bd}$ | elevation of the bottom of the stilling basin, in m; |
| $y_1$ | depth of discharge before the hydraulic jump, in m; |
| $Q_c$ | 100-year flood flow, in m$^3$/s; |
| $B_{bd}$ | width of the stilling basin, in m; |
| $Fr_1$ | Froude number before the hydraulic jump; |
| $y_2$ | depth of discharge after the hydraulic jump, in m; and |
| $NA_{ccr}$ | water level in the downstream channel for a 100-year flood, in m. |

The **radius of curvature at the stilling basin inlet**, $R_{bd}$ (m), is given by (PETERKA):

$$R_{bd} = 6 \times y_1$$

where:

| | |
|---|---|
| $y_1$ | depth of discharge before the hydraulic jump, in m. |

The **length of the stilling basin**, $L_{bd}$ (m), is given by:

$$L_{bd} = 6 \times y_2 + 0.75 \times (El_{ca} - El_{bd}) + 0.5 \times R_{bd} - 1.1$$

where:

| | |
|---|---|
| $y_2$ | depth of discharge after the hydraulic jump, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel, in m; |
| $El_{bd}$ | elevation of the bottom of the stilling basin, in m; and |
| $R_{bd}$ | radius of curvature at the stilling basin inlet, in m. |

### Ski Jump

The **width of the ski jump**, $B_{se}$ (m), is given by:

$$B_{se} = B_{uvt}$$

where:

| | |
|---|---|
| $B_{uvt}$ | useful width of the spillway, in m. |

The **elevation of the ski jump sill**, $El_{se}$, is given by:

$$El_{se} = NA_{ccr} + 1.0 \geq El_{cr}$$

where:

| | |
|---|---|
| $NA_{ccr}$ | water level in the downstream channel for a 100-year flood, in m; and |
| $El_{cr}$ | elevation of the bottom of the downstream channel, in m. |

If the sluiceways are built in the body of the spillway, the **elevation of the sill of the ski jump**, $El_{se}$, should be higher than the top of the sluiceways, as shown in Fig. 5.7.5.04:

$$El_{se} \geq El_{ca} + 1.25 \times H_{ad}$$

where:

| | |
|---|---|
| $El_{ca}$ | elevation of the bottom of the approach channel, in m; |
| $H_{ad}$ | height of sluiceways, in m. |

The **radius of curvature of the ski jump**, $R_{se}$ (m), is given by (PETERKA):

$$R_{se} = 6 \text{ x } y$$

where:

$$y = \frac{Q_c}{B_{se} \times v}$$

$$v = \sqrt{k \times 2 \times g \times (NA_{xmx} - El_{se})}$$

where:

| | |
|---|---|
| $y$ | depth of the water column at the ski jump, in m; |
| $Q_c$ | 100-year flood flow, in $m^3/s$; |
| $B_{se}$ | width of the ski jump, in m; |
| $v$ | velocity of the water column at the ski jump, in m/s; |
| $k$ | 0.9 – reduction coefficient for the energy head; |
| $g$ | 9.81 $m/s^2$ – acceleration due to gravity; |
| $NA_{xmx}$ | maximum water level of the reservoir under design flood conditions, in m; and |
| $El_{se}$ | elevation of the ski jump sill, in m; |

The **length of the ski jump in the foundations**, $L_{se}$ (m), is given by:

$$L_{se} = d_{16} + 1.286 \times R_{se} - d_{12} \quad \geq 0$$

where:

$$d_{16} = 1.46 \times H_d^{0.46} \times (p_v - h_5)^{0.54}$$

$$d_{12} = 1.46 \times H_d^{0.46} \times (p_v + 1.5)^{0.54}$$

$$h_5 = El_{se} + 0.6 \times R_{se} - El_{ca}$$

where:

| | |
|---|---|
| $R_{se}$ | radius of curvature of the ski jump, in m; |
| $H_d$ | maximum energy head on the spillway crest, in m; |
| $p_v$ | difference in height between the ogee crest and the bottom of the approach channel, in m; |
| $El_{se}$ | elevation of the ski jump sill, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel, in m; and |
| $d_i, h_i$ | secondary dimensions, in m. |

### Common Excavation (account .12.19.30.12.10)

The **volume of common excavation**, $V_{tvt}$ ($m^3$), is given by:

$$V_{tvt} = V_{tca} + V_{tes} + V_{tcr}$$

where:

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

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

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

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

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

$$B_{ca} = B_{vt} - 2 \times (e_{pe} - 1.0)$$

$$V_{tad} = (L_{cad} - L_{ca}) \times e_{te} \times (B_{ad} - e_{pl})$$

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

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

for the stilling basin: $B_{cr} = B_{bd} + 2 \times 1.0$

for the ski jump: $B_{cr} = B_{se} + 2 \times 1.0$

where:

| | |
|---|---|
| $V_{tca}$ | volume of common excavation for the approach channel, in $m^3$; |
| $V_{tes}$ | volume of common excavation for the structure, in $m^3$; |
| $V_{tcr}$ | volume of common excavation for the downstream channel, in $m^3$; |
| $V_{tai}$ | volume of common excavation per meter at section i of the approach channel, in $m^3/m$; |
| $L_{ca}$ | length of the approach channel in the part with no sluiceways, in m; |
| $V_{tad}$ | extra volume of common excavation for the approach channel because of the sluiceways, in $m^3$; |
| $B_{vt}$ | total width of the spillway, in m; |
| $e_{te}$ | mean thickness of the layer of soil in the area of the spillway per se, in m; |
| $L_{vt}$ | total length of the spillway, in m; |
| $V_{tri}$ | volume of common excavation per meter at section i for the downstream channel, in $m^3/m$; |
| $L_{cr}$ | length of the downstream channel, in m; |
| $B_{ca}$ | width of the bottom of the approach channel, in m; |
| $h_{rai}$ | depth of excavation in rock at section i of the approach channel, in m; |
| $El_{tai}$ | mean elevation of the land at section i of the approach channel; |
| $El_{cv}$ | elevation of the bottom of the spillway approach channel, sluiceways excluded; |
| $e_{pe}$ | thickness of the end pillars of the spillway, in m; |
| $L_{cad}$ | mean length of the approach channel in the part with sluiceways, in m; |
| $B_{ad}$ | total width of sluiceways, in m; |
| $e_{pl}$ | thickness of the sluiceway walls, in m; |
| $B_{cr}$ | width of the bottom of the downstream channel, in m; |
| $h_{rri}$ | depth of excavation in rock at section i of the downstream channel, in m; |
| $El_{tri}$ | mean elevation of the land at section i perpendicular to the longitudinal axis of the downstream channel, in m; |
| $El_{cr}$ | elevation of the bottom of the downstream channel, in m; |
| $B_{bd}$ | width of the stilling basin, in m; and |
| $B_{se}$ | width of the ski jump, in m. |

The unit price of **common excavation** is R$ 7.60/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price

per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation (account .12.19.30.12.11)

Fig. 5.7.5.10 shows typical cross-sections with excavation for the approach and downstream channels.

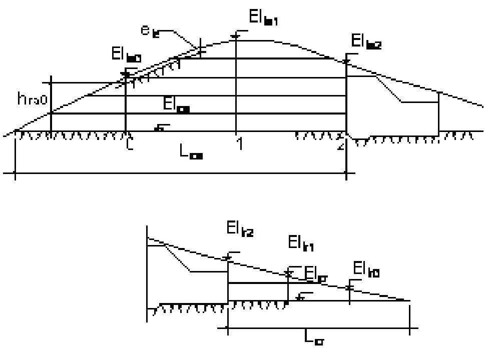


Fig. 5.7.5.10 – Excavation for approach and downstream channels.

The **volume of excavated rock** for the spillway, $V_{rvt}$ (m³), is given by:

$$V_{rvt} = V_{rca} + V_{rog} + V_{rpj} + V_{rde} + V_{rcr}$$

where:

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

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

$$V_{rai} = (B_{ca} - 6 + 0,6 \times h_{rai}) \times h_{rai}$$

$$V_{rri} = (B_{cr} - 6 + 0,6 \times h_{rri}) \times h_{rri}$$

$$h_r = El_{te} - e_{te} - (El_{ca} - 1.5)$$

$$h_{rv} = El_{te} - e_{te} - (El_{cv} - 1.5)$$

for spillways without sluiceways:

$$V_{rog} = (L_{ov} \times h_{rv} + 23) \times B_{vt}$$

$$V_{rad} = 0$$

and with a stilling basin:

$$V_{rpj} = d_1 \times \left( \frac{h_{rv} + h_{rb}}{2} + 0.167 \times H_d \right) \times (B_{bd} + 2.0)$$

and with a ski jump:

$$V_{rpj} = d_7 \times \left( \frac{h_{rv} + h_{rs}}{2} + 0.167 \times H_d \right) \times (B_{se} + 2.0)$$

$$V_{rde} = d_8 \times h_{rs} \times (B_{se} + 2.0)$$

for spillways with sluiceways:

$$V_{rog} = (L_{ov} \times h_{rv} + 23) \times (B_{vt} - B_{ad}) + (L_{og} \times h_r + 23) \times B_{ad}$$

$$V_{rad} = \frac{L_{cad} + L_{ca}}{2} \times (El_{cv} - El_{ca}) \times (B_{ad} - e_{pl}) - V_{tad}$$

and with a stilling basin:

$$V_{rpj} = d_1 \times \left( \frac{h_{rv} + h_{rb}}{2} + 0.167 \times H_d \right) \times (B_{bd} + 2.0 - B_{ad}) + d_2 \times \left( \frac{h_r + h_{rb}}{2} + 0.167 \times H_d \right) \times B_{ad}$$

$$d_2 = 0.75 \times \left[ El_{ca} - 1.5 - (El_{bd} - e_c) \right]$$

and with a ski jump:

$$V_{rpj} = d_7 \times \left( \frac{h_{rv} + h_{rs}}{2} + 0.167 \times H_d \right) \times (B_{se} + 2.0 - B_{ad})$$

$$V_{rde} = d_8 \times h_{rs} \times (B_{se} + 2.0 - B_{ad}) + L_{se} \times h_r \times B_{ad}$$

for any spillway with a stilling basin:

$$V_{rde} = V_{rbd} + V_{rmc} + V_{rbe}$$

$$V_{rbd} = L_{bd} \times h_{rb} \times (B_{bd} + 2.0)$$

$$V_{rmc} = 2 \times (L_{bd} + d_4) \times 0.5 \times h_1 \times 1.5 + 2 \times (d_3 - d_4) \times 0.5 \times \frac{h_1 + h_2}{2} \times 1.5$$

$$V_{rbe} = 2 \times (L_{bd} + d_4) \times 0.3 \times h_{re}^2$$

$$h_{rb} = El_{te} - e_{te} - (El_{bd} - e_c)$$

$$h_1 = NA_{ccr} + 2.0 - (El_{tde} - e_{te}) \geq 0$$

$$h_2 = NA_{max} + 0.12 \times H_d - (El_{tde} - e_{te}) \geq 0$$

$$d_1 = 0.75 \times \left[ El_{cv} - 1.5 - (El_{bd} - e_c) \right]$$

$$d_3 = 0.75 \times \left[ NA_{max} - 0.83 \times H_d - (El_{bd} - e_c) \right]$$

$$d_4 = 0.75 \times (y_2 + 2.0 + e_c - 0.95 \times H_d) \leq d_3$$

$$h_{re} = El_{tde} - e_{te} - (NA_{ccr} - 5.0) \geq 0$$

for any spillway with a ski jump:

$$h_{rs} = El_{te} - e_{te} - (El_{se} - 0.25 \times H_d)$$

$$d_7 = 1.46 \times H_d^{0.46} \times \left[ (p_v - h_4)^{0.54} - (p_{vv} + 1.5)^{0.54} \right]$$

$$h_4 = El_{se} - 0.25 \times H_d - El_{ca}$$

$$d_8 = 0.986 \times R_{se} - 0.188 \times H_d$$

where:

| | |
|---|---|
| $V_{rca}$ | volume of rock excavated for the approach channel, in $m^3$; |
| $V_{rog}$ | volume of rock excavated in the ogee crest area, in $m^3$; |
| $V_{rpj}$ | volume of rock excavated in the area of the downstream face of the ogee crest, in $m^3$; |
| $V_{rde}$ | volume of rock excavated in the area of the stilling basin or ski jump, in $m^3$; |
| $V_{rcr}$ | volume of rock excavated for the downstream channel, in $m^3$; |
| $V_{rai}$ | volume of excavated rock per meter at section i of the approach channel, in $m^3/m$; |
| $L_{cad}$ | mean length of the approach channel in the part with sluiceways, in m; |
| $V_{rad}$ | extra volume of excavation in rock for the approach channel due to the sluiceways, in $m^3$; |
| $V_{rri}$ | volume of excavated rock per meter at section i for the downstream channel, in $m^3/m$; |
| $L_{cr}$ | mean length of the downstream channel, in m; |
| $B_{ca}$ | width of the bottom of the approach channel, in m; |
| $h_{rai}$ | depth of excavation in rock at section i of the approach channel, in m; |
| $B_{cr}$ | width of the bottom of the downstream channel, in m; |
| $h_{rri}$ | depth of excavation in rock at section i of the downstream channel, in m; |
| $h_r$ | mean depth of the excavation in rock for the ogee crest in the part with sluiceways, in m; |
| $El_{te}$ | mean elevation of the land in the area of the spillway per se, in m; |
| $e_{te}$ | mean thickness of the layer of soil in the spillway area, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel, in m; |
| $h_{rv}$ | mean depth of excavation in rock at the part of the ogee crest with no sluiceways, in m; |
| $El_{cv}$ | elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m; |
| $L_{ov}$ | length of the ogee crest at the part with no sluiceways, in m; |
| $B_{vt}$ | total width of the spillway, in m; |
| $h_{rb}$ | mean depth of excavation in rock for the stilling basin, in m; |
| $H_d$ | maximum energy head on the spillway crest, in m; |
| $B_{bd}$ | width of the stilling basin, in m; |
| $L_{bd}$ | length of the stilling basin, in m; |
| $h_{rs}$ | mean depth of excavation in rock for the ski jump at the part with no sluiceways, in m; |
| $B_{se}$ | width of the ski jump, in m; |
| $B_{ad}$ | total width of sluiceways, in m; |
| $L_{og}$ | length of the ogee crest at the part with sluiceways, in m; |
| $L_{ca}$ | mean length of the approach channel at the part with no sluiceways, in m; |
| $e_{pl}$ | thickness of the sluiceway walls, in m; |
| $V_{tad}$ | extra volume of common excavation for the approach channel because of the sluiceways, in $m^3$; |
| $El_{bd}$ | elevation of the bottom of the stilling basin, in m; |
| $e_c$ | thickness of the concrete lining for the stilling basin sill, in m; |
| $L_{se}$ | length of the ski jump at the foundations, in m; |
| $V_{rbd}$ | volume of excavated rock in the stilling basin area, in $m^3$; |
| $V_{rmc}$ | volume of excavated rock in the area of the buttress walls for the stilling basin or ski jump, in $m^3$; |
| $V_{rbe}$ | volume of excavated rock for berms in the stilling basin area, in $m^3$; |
| $h_{re}$ | mean depth of excavation in rock in berms in the stilling basin area, in m; |
| $NA_{ccr}$ | water level in the downstream channel for a 100-year flood, in m; |
| $El_{tde}$ | mean elevation of the land in the stilling basin or ski jump area, in m; |

| | |
|---|---|
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $y_2$ | depth of discharge after the hydraulic jump, in m; |
| $El_{se}$ | elevation of the ski jump sill, in m; |
| $p_v$ | vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m; |
| $p_{vv}$ | vertical distance between the ogee sill and the bottom of the spillway approach channel, in m; |
| $R_{se}$ | radius of curvature of the ski jump, in m; and |
| $d_i$, $h_i$ | secondary dimensions, in m. |

The unit price of **excavation in rock** is R$ 21.00/m$^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.19.30.13)

The **area of foundation to be cleaned**, $A_{lf}$ (m$^2$), is given by:

$$A_{lf} = B_{vt} \times L_{vt}$$

where:

| | |
|---|---|
| $B_{vt}$ | total width of the spillway, in m; and |
| $L_{vt}$ | total length of the spillway, in m. |

The **length of the grout holes** and the **drainage line** of the spillway, $L_{tf}$ (m), is given by:

$$L_{tf} = \frac{B_{vt}}{3.0} \times L_{1tf}$$

$$L_{1tf} = 1.5 \times (NA_{xmx} - El_{cv}) \leq 40m$$

where:

| | |
|---|---|
| $B_{vt}$ | total width of the spillway, in m; |
| $L_{1tf}$ | length of one grout hole, in m; |
| $NA_{xmx}$ | maximum water level of the reservoir under design flood conditions, in m; |
| $El_{cv}$ | elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m; and |
| 3.0 | space between the grout holes, in m. |

The **total length of the rock anchors** in the stilling basin, $L_{tfc}$ (m), when necessary, is given by:

$$L_{tfc} = B_{bd} \times L_{bd}$$

where:

| | |
|---|---|
| $B_{bd}$ | width of the stilling basin, in m; and |
| $L_{bd}$ | length of the stilling basin, in m. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/$m^2$
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account .12.19.30.14)

The **volume of concrete** for the spillway, $V_{cvt}$ ($m^3$), is given by (COPEL, 1983 and 1996):

$$V_{cvt} = V_{cog} + V_{cpj} + V_{cpl} + V_{cpo} + V_{cde} + V_{cmv} + V_{cmc}$$

where:

$$V_{cpl} = 2 \times (1.21 \times H_d^2 + 18.4 \times H_d + 25) \times e_{pe}$$

$$V_{cpo} = 6 \times B_{vt}$$

for spillways without sluiceways:

$$V_{cog} = [0.944 \times H_d^{0.46} \times (p_{vv} + 1.5)^{1.54} + 0.27 \times p_{vv} \times H_d - 0.007 \times H_d^2 + 0.40 \times H_d + 18] \times B_{vt}$$

and with a stilling basin:

$$V_{cpj} = d_1 \times 0.167 \times H_d \times (B_{bd} + 2.0)$$

and with a ski jump:

$$V_{cpj} = 1.25 \times H_d \times h_5 \times (B_{se} + 2.0)$$

$$V_{csb} = 0$$

for spillways with sluiceways:

$$V_{cog} = [0.944 \times H_d^{0.46} \times (p_{vv} + 1.5)^{1.54} + 0.27 \times p_{vv} \times H_d] \times (B_{vt} - B_{ad}) +$$
$$+ [0.944 \times H_d^{0.46} \times (p_v + 1.5)^{1.54} + 0.27 \times p_v \times H_d] \times B_{ad} +$$
$$+ (-0.007 \times H_d^2 + 0.40 \times H_d + 18) \times B_{vt}$$

and with a stilling basin:

$$V_{cpj} = d_1 \times 0.167 \times H_d \times (B_{bd} + 2.0 - B_{ad}) + d_2 \times 0.167 \times H_d \times B_{ad}$$

$$d_2 = 0.75 \times [El_{ca} - 1.5 - (El_{bd} - e_c)]$$

and with a ski jump:

$$V_{cpj} = 1.25 \times H_d \times h_5 \times (B_{se} + 2.0 - B_{ad})$$

$$V_{csb} = \left(\frac{d_9}{2} + L_{se}\right) \times (h_4 + 1.5) \times (B_{se} + 2.0)$$

$$d_9 = 1{,}46 \times H_d^{0.46} \times \left[ \left(p_v + 1.5\right)^{0.54} - \left(p_v - h_4\right)^{0.54} \right]$$

$$h_4 = El_{se} - 0.25 \times H_d - El_{ca}$$

for any spillway with a stilling basin:

$$V_{cde} = \left(L_{bd} \times e_c + 0.036 \times R_{bd}^2 - 0.375 \times e_c^2\right) \times (B_{bd} + 2.0)$$

$$V_{cmv} = 2 \times \left[ \left(d_5 \times 0.95 \times H_d \times 1.0\right) + \frac{d_6^2}{2 \times 0.75} \right] + 2 \times L_{bd} \times \left(y_2 + 2.0\right) \times 1.0$$

$$V_{cmc} = 2 \times \left( L_{bd} + \frac{d_3 + d_4}{2} \right) \times \left(0.25 \times h_1^2 + 0.75 \times h_1\right) + 2 \times \frac{d_3 - d_4}{2} \times \left(0.25 \times h_2^2 + 0.75 \times h_2\right)$$

$$d_1 = 0.75 \times \left[ El_{cv} - 1.5 - \left(El_{bd} - e_c\right) \right]$$

$$d_5 = 0.75 \times \left[ NA_{max} - 1.0 \times H_d - \left(El_{bd} - e_c\right) \right]$$

$$d_6 = 0.75 \times \left(y_2 + 2.0 + e_c - 0.95 \times H_d\right)$$

$$d_3 = 0.75 \times \left[ NA_{max} - 0.83 \times H_d - \left(El_{bd} - e_c\right) \right]$$

$$d_4 = 0.75 \times \left(y_2 + 2.0 + e_c - 0.95 \times H_d\right) \leq d_3$$

$$h_1 = NA_{ccr} + 2.0 - \left(El_{tde} - e_{te}\right) \geq 0$$

$$h_2 = NA_{max} + 0.12 \times H_d - \left(El_{tde} - e_{te}\right) \geq 0$$

for any spillway with a ski jump:

$$V_{cde} = V_{csd} + V_{csb}$$

$$V_{csd} = (0.116 \times R_{se}^2 + 0.247 \times H_d \times R_{se} - 0.023 \times H_d^2) \times \left(B_{se} + 2.0\right)$$

$$V_{cmv} = 2 \times (d_{11} \times 1.6 \times y \times 1.0 + d_{13} \times 0.95 \times H_d \times 1.0)$$

$$V_{cmc} = 2 \times \left( d_{11} + \frac{d_{10}}{2} \right) \times \left(0.25 \times h_3^2 + 0.75 \times h_3\right) + 2 \times \frac{d_{10}}{2} \times \left(0.25 \times h_2^2 + 0.75 \times h_2\right)$$

$$h_5 = El_{cv} - 1.5 - \left(El_{se} - 0.25 \times H_d\right) \geq 0$$

$$d_{10} = 0.75 \times [NA_{max} + 0.12 \times H_d - \left(El_{se} - 1.6 \times y\right)] \geq 0$$

$$d_{11} = d_{12} + L_{se} - 1.46 \times H_d - d_{13}$$

$$d_{12} = 1.46 \times H_d^{0.46} \times \left(p_v + 1.5\right)^{0.54}$$

$$d_{13} = 0.75 \times [NA_{max} - 0.05 \times H_d - \left(El_{se} - 1.6 \times y\right)] \geq 0$$

$$h_3 = El_{se} - 1.6 \times y - \left(El_{tde} - e_{te}\right) \geq 0$$

$$h_2 = NA_{max} + 0.12 \times H_d - \left(El_{tde} - e_{te}\right) \geq 0$$

where:

| | |
|---|---|
| $V_{cog}$ | volume of concrete for the ogee crest, in $m^3$; |
| $V_{cpo}$ | volume of concrete for the bridge, in $m^3$; |
| $V_{cpj}$ | volume of concrete for the downstream face of the ogee crest, in $m^3$; |
| $V_{cpl}$ | volume of concrete for the pillars, in $m^3$; |
| $V_{cde}$ | volume of concrete for the stilling basin or ski jump, in $m^3$; |
| $V_{cmv}$ | volume of concrete for the vertical lining of the stilling basin or ski jump, in $m^3$; |
| $V_{cmc}$ | volume of concrete for the walls of the stilling basin or ski jump, in $m^3$; |
| $H_d$ | maximum energy head on the spillway crest, in m; |
| $e_{pe}$ | thickness of the end pillars, in m; |
| $B_{vt}$ | total width of the spillway, in m; |
| $p_{vv}$ | vertical distance between the ogee sill and the bottom of the spillway approach channel, in m; |
| $B_{bd}$ | width of the stilling basin, in m; |
| $B_{se}$ | width of the ski jump, in m; |
| $B_{ad}$ | total width of sluiceways, in m; |
| $p_v$ | difference in height between the ogee crest and the bottom of the approach channel, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel to the sluiceways, in m; |
| $El_{bd}$ | elevation of the bottom of the stilling basin, in m; |
| $e_c$ | thickness of the concrete lining for the stilling basin sill, in m; |
| $L_{se}$ | length of the ski jump at the foundations, in m; |
| $El_{se}$ | elevation of the ski jump sill, in m; |
| $L_{bd}$ | length of the stilling basin, in m; |
| $R_{bd}$ | radius of curvature at the stilling basin inlet, in m; |
| $y_2$ | depth of discharge after the hydraulic jump, in m; |
| $El_{cv}$ | elevation of the bottom of the approach channel to the spillway, excluding the sluiceways, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $NA_{ccr}$ | water level in the downstream channel for a 100-year flood, in m; |
| $El_{tde}$ | mean elevation of the land in the stilling basin or ski jump area, in m; |
| $e_{te}$ | mean thickness of the layer of soil in the area of the spillway per se, in m; |
| $V_{csd}$ | volume of concrete for the ski jump deflector baffle, in $m^3$; |
| $V_{csb}$ | volume of concrete beneath the deflector baffle, in $m^3$; |
| $R_{se}$ | radius of curvature of the ski jump, in m; |
| y | flow depth at the ski jump, in m; and |
| $d_i$, $h_i$ | secondary dimensions, in m. |

If there are sluiceways through the body of the spillway, the **extra volume of concrete for the sluiceways**, $V_{cad}$ ($m^3$), should be taken into account as shown below:

$$V_{cad} = V_{cac} + V_{cpl} - V_{cae}$$

where:

$$V_{cac} = (0.24 \times H_{ad} + 2) \times B_{ad}$$

$$V_{cpl} = (0.16 \times H_{ad}^2 + 2.7 \times H_{ad} + 8) \times (N_{ad} + 1) \times e_{pa}$$

$$V_{cae} = (0.38 \times H_{ad} + 0.2) \times H_{ad} \times N_{ad} \times B_{1ad}$$

where:

| | |
|---|---|
| $V_{cac}$ | volume of concrete for part of the sluiceway sills, in $m^3$; |
| $V_{cpl}$ | volume of concrete for the sluiceway walls upstream from the face of the dam, in $m^3$; |
| $V_{cae}$ | volume of concrete for the sluiceway inlets, in $m^3$; |
| $H_{ad}$ | height of sluiceways, in m; |
| $B_{ad}$ | total width of sluiceways, in m; |
| $N_{ad}$ | number of sluiceways; |
| $e_{pa}$ | thickness of the sluiceway walls, in m; and |
| $B_{1ad}$ | width of one sluiceway, in m. |

Still considering the case where the diversion is through sluiceways in the spillway, the **volume of concrete for larger amounts of cement and reinforcement steel** than for the ogee crest of the spillway, $V_{cen}$ ($m^3$), is given by:

$$V_{cen} = V_{cet} + V_{ces} + V_{cep}$$

for a stilling basin:

$$V_{cet} = (0.27 \times H_d + d_{14}) \times 0.25 \times H_{ad} \times B_{ad}$$

$$V_{ces} = (0.27 \times H_d + d_{15}) \times 1.5 \times B_{ad}$$

$$V_{cep} = \left(0.27 \times H_d + \frac{d_{14} + d_{15}}{2}\right) \times H_{ad} \times (N_{ad} + 1) \times e_{pa}$$

$$d_{14} = 1.46 \times H_d^{0.46} \times (p_v - H_{ad})^{0.54}$$

$$d_{15} = 1.46 \times H_d^{0.46} \times p_v^{0.54}$$

for a ski jump:

$$V_{cet} = (L_{vt} - d_0) \times 0.25 \times H_{ad} \times B_{ad}$$

$$V_{ces} = L_{vt} \times 1.5 \times B_{ad}$$

$$V_{cep} = \left(L_{vt} - \frac{d_0}{2}\right) \times H_{ad} \times (N_{ad} + 1) \times e_{pa}$$

$$d_0 = (L_{vt} - 0.27 \times H_d - d_{16} - 0.836 \times R_{se} + 0.15 \times H_d) \times \frac{H_{ad}}{h_4}$$

$$d_{16} = 1.46 \times H_d^{0.46} \times (p_v - h_4)^{0.54}$$

$$h_4 = El_{se} - 0.25 \times H_{ad} - El_{ca}$$

where:

| | |
|---|---|
| $V_{cet}$ | volume of concrete for the sluiceway roofs, in $m^3$; |
| $V_{ces}$ | volume of concrete for the sluiceway sills, in $m^3$; |
| $V_{cep}$ | volume of concrete for the sluiceway walls, in $m^3$; |
| $H_d$ | hydrostatic load on the ogee crest, in m; |
| $H_{ad}$ | height of sluiceways, in m; |
| $B_{ad}$ | total width of sluiceways, in m; |
| $N_{ad}$ | number of sluiceways; |
| $e_{pa}$ | thickness of the end pillars of the spillway, in m; |
| $p_v$ | vertical distance between the ogee sill and the bottom of the sluicway approach channel, in m; |
| $L_{vt}$ | total length of the spillway, in m; |
| $R_{se}$ | radius of curvature of the ski jump, in m; |
| $El_{se}$ | elevation of the ski jump sill, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel to the sluiceways, in m; |
| $d_i$, $h_i$ | secondary dimensions, in m. |

The amounts of cement and reinforcement steel are:

| | cement (kg/m³) | reinforcement steel (kg/m³) |
|---|---|---|
| ogee crest, buttress, beneath the deflector baffle, sluiceway sills and inlets | 200 | 20 |
| plugs | 220 | 20 |
| stilling basin and deflector baffle | 270 | 50 |
| pillars and walls | 270 | 80 |
| bridge | 300 | 100 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of spillway (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- ogee crest, buttress, beneath the deflector baffle: 113.00/$m^3$
- pillars, walls, stilling basin and deflector baffle: 200.00/$m^3$
- sluiceway sills and inlets and concrete with greater unit quantities: 113.00/$m^3$

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

## UNGATED SURFACE ABUTMENT SPILLWAYS (ACCOUNT .12.18.28)

### Basic Data

The main **information required for dimensioning** purposes can be obtained from the overall layout and from item 5.1.2. (Hydrometeorological Data), as follows:

- design flood through the spillway, $Q_v$, in $m^3/s$, from item 5.1.2.;
- 100-year flood flow, $Q_c$, in $m^3/s$, from item 5.1.2.;
- maximum water level of the reservoir under design flood conditions, $NA_{xmx}$, from item 5.1.2.;
- maximum normal water level in the reservoir, $NA_{max}$, from item 4.6.;
- elevation of the bottom of the approach channel, $El_{ca}$, from item 5.7.3.;
- slope of the upstream face of the ogee crest, horizontal distance for a 1.0 m difference in level, $m_m$ in m;

- slope of the chute, tangent of the absolute value of the angle with the horizontal, $i_{cl}$; and
- water level in the downstream channel for a 100-year flood flow, $NA_{ccr}$, from item 5.1.2.
- elevation of the ski jump sill, $El_{se}$, when applicable;

The **main information used for quantification** purposes is:

- mean elevation of the land in the area of the spillway per se, including the energy dissipator, $El_{te}$, in m;
- mean elevation of the land in the chute area, exclusively, $El_{tc}$, in m;
- mean elevation of the land in the stilling basin or ski jump area, exclusively, $El_{tde}$, in m;
- mean thickness of the layer of soil in the area of the spillway per se, $e_{te}$ in m;
- mean elevation of the land at section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the approach channel, $El_{tai}$, in m;
- mean elevation of the land at section i – 0, 1 and 2 – perpendicular to the longitudinal axis of the downstream channel, $El_{tri}$, in m;
- elevation of the bottom of the downstream channel, $El_{cr}$, in m;
- mean length of the approach channel, $L_{ca}$, in m;
- mean length of the downstream channel, $L_{cr}$, in m; and
- thickness of the concrete lining for the stilling basin sill, $e_c$, in m, when applicable.

### Considerations and recommendations

This text relates to spillways with a typical cross-section as shown in Fig. 5.7.5.11 or 5.7.5.12.

Use 10% as the **slope of the chute** in the absence of more accurate information.

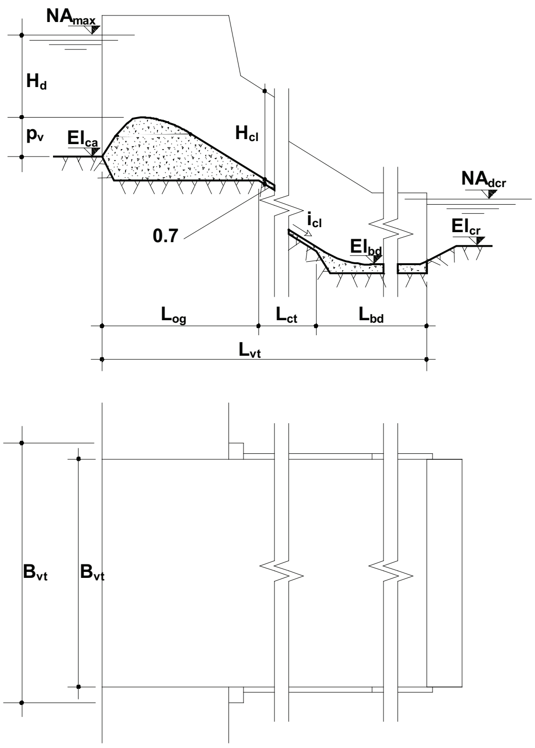



Fig. 5.7.5.11 – Typical cross-section and plan of an ungated abutment spillway with a stilling basin.

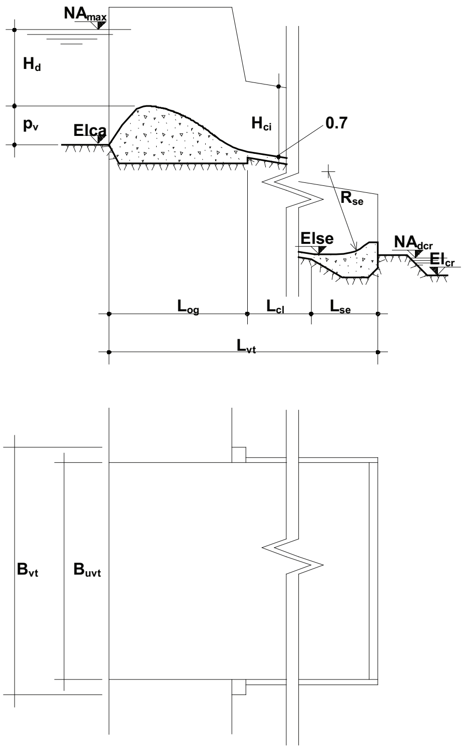



Fig. 5.7.5.12 – Typical cross-section of an ungated abutment spillway with a ski jump.

When **dimensioning the stilling basin**, the recommended range for the Froude number before the hydraulic jump is betwen 4.5 and 9.0, in order to ensure its stability. One way of raising the Froude number to 4.5 would be to lower the bottom of the stilling basin beyond the result given by the calculation. Likewise, one way to reduce the Froude number to 9.0 would be to reduce the width of the stilling basin.

In the absence of more accurate information, use 1.0 m as the **thickness of the concrete lining for the bottom of the stilling basin**.

### Discharge Coefficient

The **discharge coefficient**, $C_d$, for the typical cross-section recommended can be obtained from Graph 5.7.5.03 (Bureau of Reclamarion, 1977) or from the equivalent expressions:

for $m_m$=1.0 and $0.100 < z \leq 0.524$:

$$C_d = 1.9507 \times z^3 - 2.9011 \times z^2 + 1.5498 \times z + 1.8274$$

for $m_m$=1.0 and $0.524 < z \leq 0.813$:

$$C_d = 0.1592 \times z^3 - 0.4409 \times z^2 + 0.4248 \times z + 1.9984$$

for $m_m$=1.0 and $0.813 < z \leq 1.800$:

$$C_d = 0.0159 \times z + 2.1256$$

for $m_m$=0.67 and $0.100 < z \leq 0.497$:

$$C_d = 2.5495 \times z^3 - 3.6032 \times z^2 + 1.8832 \times z + 1.7678$$

$m_m$=0.67 and $0.497 < z \leq 0.759$:

$$C_d = 0.2261 \times z^3 - 0.6256 \times z^2 + 0.6137 \times z + 1.9481$$

for $m_m$=0.67 and 0.759 < z ≤ 1.800:

$$C_d = 0.0242 \times z^3 - 0.1143 \times z^2 + 0.1775 \times z + 2.0734$$

for $m_m$=0.33 and 0.100 < z ≤ 0.505:

$$C_d = 2.4283 \times z^3 - 3.5181 \times z^2 + 1.9125 \times z + 1.7265$$

for $m_m$=0.33 and 0.505 < z ≤ 0.755:

$$C_d = 0.2514 \times z^3 - 0.6927 \times z^2 + 0.6896 \times z + 1.9033$$

$m_m$=0.33 and 0.755 < z ≤ 1.800:

$$C_d = 0.02 \times z^3 - 0.0985 \times z^2 + 0.1782 \times z + 2.0508$$

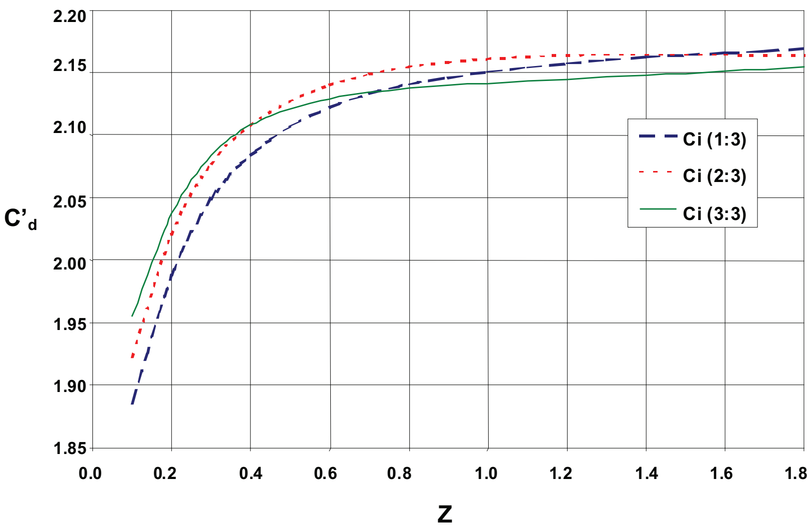
where:

$$z = \frac{p_v}{H_d}$$

$$p_v = NA_{max} - El_{ca} \qquad H_d = NA_{xmx} - NA_{max}$$

where:

| | |
|---|---|
| z | adimensional parameter; |
| $p_v$ | difference in height between the ogee crest and the bottom of the approach channel, in m; |
| $H_d$ | energy head on the spillway crest, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel, in m; and |
| $NA_{xmx}$ | maximum water level of the reservoir under design flood conditions, in m. |




Graph 5.7.5.03 – Discharge coefficient for spillways with an inclined upstream face.

### Spillway Dimensions

The **useful width** of the spillway, perpendicular to flow, $B_{uvt}$ (m), is given by:

$$B_{uvt} = \frac{Q_v}{C_d \times H_d^{3/2}}$$

where:

| | |
|---|---|
| $Q_v$ | design flood through the spillway, in $m^3/s$; |
| $C_d$ | discharge coefficient; and |
| $H_d$ | maximum energy head on the spillway crest, in m. |

The **thickness of the end pillars**, $e_{pe}$ (m), is given by:

$$e_{pe} = 0.12 \times H_d + 2.4$$

where:

| | |
|---|---|
| $H_d$ | maximum energy head on the spillway crest, in m. |

The **total width of the spillway**, $B_{vt}$ (m), perpendicular to flow, is given by:

$$B_{vt} = 0.05 \times \text{int}\left[ \left( B_{uvt} + 2 \times e_{pe} \right) \times \frac{1}{0.05} + 0.5 \right]$$

where:

| | |
|---|---|
| $B_{uvt}$ | useful width of the spillway, in m; |
| $e_{pe}$ | thickness of the end pillars, in m; and |
| int(x) | function that returns the integer part of x. |

The **width of the chute**, $B_{cl}$ (m), is given by:

$$B_{cl} = B_{uvt}$$

where:

| | |
|---|---|
| $B_{uvt}$ | useful width of the spillway, in m. |

The **length of the ogee crest** in the direction of flow, $L_{og}$ (m), is given by:

$$L_{og} = 1.66 \times H_d + m_m \times p_v$$

where:

| | |
|---|---|
| $H_d$ | maximum energy head on the spillway crest, in m; |
| $m_m$ | slope of the upstream face of the ogee crest, horizontal distance for a 1.0 m difference in level; and |
| $p_v$ | difference in height between the ogee crest and the bottom of the approach channel, in m. |

The **length of the chute**, $L_{cl}$ (m), is given by:

for the stilling basin:

$$L_{cl} = \frac{NA_{max} - 0.69 \times H_d - El_{bd}}{i_{cl}}$$

for the ski jump:

$$L_{cl} = \frac{NA_{max} - 0.69 \times H_d - El_{se} - 0.03 \times R_{se}}{i_{cl}}$$

where:

| | |
|---|---|
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $H_d$ | maximum energy head on the spillway crest, in m; |
| $El_{bd}$ | elevation of the bottom of the stilling basin, defined below, in m; |
| $El_{se}$ | elevation of the ski jump sill, defined below, in m; |
| $R_{se}$ | radius of curvature of the ski jump, defined below, in m; and |
| $i_{cl}$ | slope of the chute. |

The **total length** of the spillway, $L_{vt}$ (m), is given by:

for the stilling basin: $L_{vt} = L_{og} + L_{cl} + L_{bd}$

for the ski jump: $L_{vt} = L_{og} + L_{cl} + L_{se}$

where:

| | |
|---|---|
| $L_{og}$ | length of the ogee crest, in m; |
| $L_{cl}$ | length of the chute, in m; |
| $L_{bd}$ | length of the stilling basin, defined below, in m; and |
| $L_{se}$ | length of the ski jump at the foundations, defined below, in m. |

### Stilling Basin

The **width of the stilling basin**, $B_{bd}$ (m), is given by:

$$B_{bd} = B_{uvt}$$

where:

| | |
|---|---|
| $B_{uvt}$ | useful width of the spillway, in m. |

The **stilling basin depth** is determined iteratively, based on the Froude number before the hydraulic jump, in section one of Fig. 5.7.5.03, for the 100-year flood flow .

An elevation is picked for the bottom of the stilling basin and its suitability is checked by calculating the **velocity**, $v_1$ (m/s), the **depth of discharge**, $y_1$ (m), and the **Froude number**, $Fr_1$, before the hydraulic jump, **depth of discharge** after the hydraulic jump, $y_2$ (m), and finally the elevation of the **bottom of the stilling basin**.

Should the elevation of the bottom of the stilling basin be different from the figure first picked, the calculations should be redone until the level of precision required is reached.

$$v_1 = \sqrt{k \times 2 \times g \times (NA_{xmx} - El_{bd})}$$

$$y_1 = \frac{Q_c}{B_{bd} \times v_1}$$

$$Fr_1 = \frac{v_1}{\sqrt{g \times y_1}}$$

$$y_2 = \left(\frac{y_1}{2}\right) \times \left(\sqrt{1 + 8 \times Fr_1^2} - 1\right)$$

where:

| | |
|---|---|
| $v_1$ | mean velocity of discharge before the hydraulic jump, in m/s; |
| k | 0.9 – reduction coefficient for the energy head; |
| g | 9.81 $m/s^2$ – acceleration due to gravity; |
| $NA_{xmx}$ | maximum water level of the reservoir under design flood conditions, in m; |
| $El_{bd}$ | elevation of the bottom of the stilling basin, in m; |
| $y_1$ | mean depth of discharge in section 1, before the hydraulic jump, in m; |
| $Q_c$ | 100-year flood flow, in $m^3/s$; |
| $B_{bd}$ | width of the stilling basin, in m; |
| $Fr_1$ | Froude number before the hydraulic jump; |
| $y_2$ | depth of discharge after the hydraulic jump, in m; and |
| $NA_{ccr}$ | water level in the downstream channel for a 100-year flood, in m. |

The **radius of curvature at the stilling basin inlet**, $R_{bd}$ (m), is given by (PETERKA):

$$R_{bd} = 6 \times y_1$$

where:

| | |
|---|---|
| $y_1$ | depth of discharge before the hydraulic jump, in m. |

The length **of the stilling basin**, $L_{bd}$ (m), is given by:

$$L_{bd} = 6 \times y_2 + R_{bd} \times \tan\left[\frac{\arctan(i_{cl})}{2}\right]$$

where:

| | |
|---|---|
| $y_2$ | depth of discharge after the hydraulic jump, in m; |
| $i_{cl}$ | slope of the chute, tangent of the absolute value of the angle with the horizontal; and |
| $R_{bd}$ | radius of curvature at the stilling basin inlet, in m. |

### Ski Jump

The **width of the ski jump**, $B_{se}$ (m), is given by:

$$B_{se} = B_{cl}$$

where:

| | |
|---|---|
| $B_{cl}$ | width of the chute, in m. |

The **elevation of the ski jump sill**, $El_{se}$, is given by:

$$El_{se} = NA_{ccr} + 1.0 \geq El_{cr}$$

where:

| | |
|---|---|
| $NA_{ccr}$ | water level in the downstream channel for a 100-year flood, in m; and |
| $El_{cr}$ | elevation of the bottom of the downstream channel, in m. |

The **radius of curvature of the ski jump**, $R_{se}$ (m), is given by:

$$R_{se} = 6 \times y$$

where:

$$y = \frac{Q_c}{B_{se} \times v} \qquad v = \sqrt{k \times 2 \times g \times (NA_{xmx} - El_{se})}$$

where:

| | |
|---|---|
| $y$ | depth of the water column at the ski jump, in m; |
| $Q_c$ | 100-year flood flow, in $m^3/s$; |
| $B_{se}$ | width of the ski jump, in m; |
| $v$ | velocity of the water column at the ski jump, in m/s; |
| $k$ | 0.9 – reduction coefficient for the energy head; |
| $g$ | 9.81 $m/s^2$ – acceleration due to gravity; |
| $NA_{xmx}$ | maximum water level of the reservoir under design flood conditions, in m; and |
| $El_{se}$ | elevation of the ski jump sill, in m. |

The **length of the ski jump**, $L_{se}$ (m), is given by:

$$L_{se} = 0.80 \times R_{se} + 1.5$$

where:

| | |
|---|---|
| $R_{se}$ | radius of curvature of the ski jump, in m. |

### Common Excavation (account .12.19.30.12.10)

The **volume of common excavation** for the spillway, $V_{tvt}$ ($m^3$), is given by:

$$V_{tvt} = V_{tca} + V_{tes} + V_{tcr}$$

where:

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

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

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

$$B_{ca} = B_{vt} - 2 \times \left( e_{pl} - 1.0 \right)$$

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

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

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

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

for the stilling basin: $B_{cr} = B_{bd} + 2 \times 1.0$

for the ski jump: $B_{cr} = B_{se} + 2 \times 1.0$

where:

| | |
|---|---|
| $V_{tca}$ | volume of common excavation for the approach channel, in $m^3$; |
| $V_{tes}$ | volume of common excavation for the structure, in $m^3$; |
| $V_{tcr}$ | volume of common excavation for the downstream channel, in $m^3$; |
| $V_{tai}$ | volume of common excavation per meter at section i of the approach channel, in $m^3/m$; |
| $L_{ca}$ | length of the approach channel, in m; |
| $B_{ca}$ | width of the bottom of the approach channel, in m; |
| $h_{rai}$ | depth of excavation in rock at section i of the approach channel, in m; |
| $e_{te}$ | mean thickness of the layer of soil in the area of the spillway, in m; |
| $El_{tai}$ | mean elevation of the land at section i of the approach channel, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel, in m; |
| $B_{vt}$ | total width of the spillway, in m; |
| $e_{pl}$ | thickness of the pillars, in m; |
| $L_{vt}$ | total length of the spillway, in m; |
| $V_{tri}$ | volume of common excavation per meter at section i for the downstream channel, in $m^3/m$; |
| $L_{cr}$ | length of the downstream channel, in m; |
| $B_{cr}$ | width of the bottom of the downstream channel, in m; |
| $h_{rri}$ | depth of excavation in rock at section i of the downstream channel, in m; |
| $El_{tri}$ | mean elevation of the land at section i perpendicular to the longitudinal axis of the downstream channel, in m; |
| $El_{cr}$ | elevation of the bottom of the downstream channel, in m; |
| $B_{bd}$ | width of the stilling basin, in m; and |
| $B_{se}$ | width of the ski jump, in m. |

The unit price of **common excavation** is R$ 7.60/m$^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation (account .12.19.30.12.11)

Figure 5.7.5.10 shows typical cross-sections for the excavation of approach and downstream channels.

The **volume excavated in rock**, $V_{rtv}$ (m$^3$), is given by:

$$V_{rvt} = V_{rca} + V_{rog} + V_{rcl} + V_{rbc} + V_{rde} + V_{rbe} + V_{rcr}$$

where:

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

$$V_{rai} = (B_{ca} - 6 + 0.6 \times h_{rai}) \times h_{rai}$$

$$V_{rog} = L_{og} \times [El_{te} - e_{te} - (El_{ca} - 2)] \times B_{vt}$$

$$V_{rcl} = L_{cl} \times [El_{tc} - e_{te} - (El_{tc} - 0.7)] \times (B_{cl} + 2)$$

$$V_{rbc} = 2 \times L_{cl} \times 0.3 \times h_{rc}^2$$

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

$$V_{rri} = (B_{cr} - 6 + 0.6 \times h_{rri}) \times h_{rri}$$

$$h_{rc} = El_{tc} - e_{te} - (El_{cm} - 0.7) \geq 0$$

for the stilling basin:

$$V_{rde} = L_{bd} \times [El_{te} - e_{te} - (El_{bd} - e_c)] \times (B_{bd} + 2)$$

$$V_{rbe} = 2 \times L_{bd} \times 0.3 \times h_{re}^2$$

$$El_{cm} = \frac{NA_{max} - 0.69 \times H_d + El_{bd}}{2}$$

$$h_{re} = El_{tde} - e_{te} - (NA_{ccr} - 5.0) \geq 0$$

for the ski jump:

$$V_{rde} = L_{se} \times \left[ El_{te} - e_{te} - (El_{se} - 2) \right] \times (B_{se} + 2)$$

$$V_{rbe} = 2 \times L_{se} \times 0.3 \times h_{rs}^2$$

$$El_{cm} = \frac{NA_{max} - 0.69 \times H_d + El_{se}}{2}$$

$$h_{rs} = El_{tde} - e_{te} - (El_{se} - 2.0) \;\; \geq \;\; 0$$

where:

| | |
|---|---|
| $V_{rca}$ | volume of rock excavated for the approach channel, in $m^3$; |
| $V_{rog}$ | volume of rock excavated in the ogee crest area, in $m^3$; |
| $V_{rcl}$ | volume of rock excavated in the chute area, in $m^3$; |
| $V_{rbc}$ | volume of rock excavated for berms in the chute section, in $m^3$; |
| $V_{rde}$ | volume of rock excavated in the area of the stilling basin or ski jump, in $m^3$; |
| $V_{rbe}$ | volume of rock excavated for berms in the stilling basin or ski jump section, in $m^3$; |
| $V_{rcr}$ | volume of rock excavated for the downstream channel, in $m^3$; |
| $V_{rai}$ | volume of excavated rock per meter at section i of the approach channel, in $m^3/m$; |
| $L_{ca}$ | Length of the approach channel, in m; |
| $B_{ca}$ | width of the bottom of the approach channel, in m; |
| $h_{rai}$ | Depth of excavation in rock at section i of the approach channel, in m; |
| $L_{og}$ | length of the ogee crest, in m; |
| $El_{te}$ | mean elevation of the land in the area of the spillway per se, in m; |
| $e_{te}$ | mean thickness of the layer of soil in the area of the spillway per se, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel, in m; |
| $B_{vt}$ | total width of the spillway, in m; |
| $L_{cl}$ | length of the chute, in m; |
| $El_{cm}$ | mean elevation of the chute, in m; |
| $B_{cl}$ | width of the chute, in m; |
| $h_{rc}$ | Depth of excavation in rock in the chute area, in m; |
| $V_{rri}$ | volume of excavated rock per meter at section i for the downstream channel, in $m^3/m$; |
| $L_{cr}$ | Length of the downstream channel, in m; |
| $B_{cr}$ | width of the bottom of the downstream channel, in m; |
| $h_{rri}$ | Depth of excavation in rock at section i of the downstream channel, in m; |
| $El_{tc}$ | mean elevation of the land in the chute area, exclusively, in m; |
| $L_{bd}$ | length of the stilling basin, in m; |
| $El_{bd}$ | elevation of the bottom of the stilling basin, in m; |
| $e_c$ | Thickness of the concrete lining for the stilling basin sill, in m; |
| $B_{bd}$ | width of the stilling basin, in m; |
| $h_{re}$ | depth of excavation in rock in the stiling basin area, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $H_d$ | maximum energy head on the spillway crest, in m; |
| $El_{rde}$ | mean elevation of the land in the stilling basin or ski jump area, exclusively, in m; |
| $NA_{ccr}$ | water level in the downstream channel for a 100-year flood, in m; |
| $L_{se}$ | length of the ski jump, in m; |
| $El_{se}$ | elevation of the ski jump sill, in m; |
| $B_{se}$ | width of the ski jump, in m; and |
| $h_{rs}$ | depth of excavation in rock in the ski jump area, in m. |

The unit price of **excavation in rock** is R$ 21.00/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price

per cubic meter calculated above the excavation line of the spillway. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.19.30.13)

The **area of foundation to be cleaned**, $A_{lf}$ ($m^2$), is given by:

$$A_{lf} = B_{vt} \times L_{vt}$$

where:

| | |
|---|---|
| $B_{vt}$ | total width of the spillway, in m; and |
| $L_{vt}$ | total length of the spillway, in m. |

The **length of the grout holes** and the **drainage line**, $L_{tf}$ (m), is given by:

$$L_{tf} = \frac{B_{vt}}{3.0} \times L_{1tf}$$

$$L_{1tf} = 1.5 \times (NA_{xmx} - El_{ca}) \leq 40m$$

where:

| | |
|---|---|
| $B_{vt}$ | total width of the spillway, in m; |
| $L_{1tf}$ | length of one grout hole, in m; |
| $NA_{xmx}$ | maximum water level of the reservoir under design flood conditions, in m; |
| $El_{ca}$ | elevation of the bottom of the approach channel, in m; and |
| 3.0 | space between the grout holes, in m. |

The **total length of the rock anchors** in the stilling basin, $L_{tfc}$(m), when necessary, is given by:

$$L_{tfc} = B_{bd} \times L_{bd}$$

where:

| | |
|---|---|
| $B_{bd}$ | width of the stilling basin, in m; and |
| $L_{bd}$ | length of the stilling basin, in m. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/$m^2$
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account .12.19.30.14)

The **volume of concrete** for the spillway, $V_{cvt}$ ($m^3$), is given by:

$$V_{cvt} = V_{cog} + V_{cpl} + V_{cpo} + V_{ccl} + V_{cde} + V_{cmv}$$

where:

$$V_{cog} = (0.165 \times H_d^2 + 0.67 \times p_v \times H_d + 0.84 \times p_v^2 + 32) \times B_{vt}$$

$$V_{cpl} = 2 \times \left(1.21 \times H_d^2 + 18.4 \times H_d + 25\right) \times e_{pe}$$

$$V_{cpo} = 6.0 \times B_{vt}$$

$$V_{ccl} = L_{cl} \times \left[0.7 \times B_{cl} + 2 \times \left(H_{cl} + 0.7\right) \times 1.0\right]$$

$$H_{cl} = 0.95 \times H_d$$

for the stilling basin:

$$V_{cde} = L_{bd} \times e_c \times \left(B_{bd} + 2.0\right)$$

$$V_{cmv} = 2 \times \left[L_{bd} \times \left(2.0 + y_2 + e_c\right) + \frac{d_1^2}{2 \times i_{cl}}\right] \times 1.0$$

$$d_1 = 2.0 + y_2 - H_{cl}$$

for the ski jump:

$$V_{cde} = \left(0.12 \times R_{se}^2 + 0.93 \times R_{se} + 0.53\right) \times \left(B_{se} + 2.0\right)$$

$$V_{cmv} = 2 \times L_{se} \times H_{cl} \times 1.0$$

where:

| | |
|---|---|
| $V_{cog}$ | volume of concrete for the ogee crest, in $m^3$; |
| $V_{cpl}$ | volume of concrete for the pillars, in $m^3$; |
| $V_{cpo}$ | volume of concrete for the bridge, in $m^3$; |
| $V_{ccl}$ | Volume of concrete for the chute, including walls, in $m^3$; |
| $V_{cde}$ | Volume of concrete for the stilling basin or ski jump, including walls, in $m^3$; |
| $H_d$ | maximum energy head on the spillway crest, in m; |
| $p_v$ | difference in height between the ogee crest and the bottom of the approach channel, in m; |
| $B_{vt}$ | total width of the spillway, in m; |
| $e_{pe}$ | thickness of the end pillars, in m; |
| $L_{cl}$ | length of the chute, in m; |
| $B_{cl}$ | width of the chute, in m; |
| $H_{cl}$ | height of the wall for the chute, in m; |
| $L_{bd}$ | length of the stilling basin, in m; |
| $e_c$ | thickness of the concrete lining for the stilling basin sill, in m; |
| $B_{bd}$ | width of the stilling basin, in m; |
| $y_2$ | depth of discharge after the hydraulic jump, in m; |
| $i_{cl}$ | slope of the chute; |
| $R_{se}$ | radius of curvature of the ski jump, in m; |
| $B_{se}$ | Width of the ski jump, in m; |
| $L_{se}$ | length of the ski jump, in m; and |
| $d_i$ | secondary dimensions, in m. |

The amounts of cement and reinforcement steel are:

| | cement (kg/m³) | reinforcement steel (kg/m³) |
|---|---|---|
| ogee crest | 200 | 40 |
| pillars, chute, stilling basin, ski jump and walls | 250 | 80 |
| bridge | 300 | 100 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian Reais per cubic meter of spillway (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- ogee crest: 113.00/$m^3$
- pillars, chute , walls and stilling basin: 200.00/$m^3$
- bridge: 474.00/m

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

## 5.7.6 Intake (account .12.19)

### HEADRACE CANAL (.12.19.31)

#### Basic Data

The main **information required for dimensioning** purposes is:

- length of the canal, $L_{cn}$ in m;
- length of the section lined with concrete, $L_c$ in m;
- total maximum turbine flow, $Q_t$, in m³/s, from item 5.7.2.;
- mean elevation of the land along the canal axis, $El_{te}$;
- mean thickness of the layer of soil, $e_{te}$, in m;
- maximum normal water level in the reservoir, $NA_{max}$, from item 4.4.; and
- minimum water level in the reservoir, $NA_{min}$, from item 5.3.

The **information required for quantification** purposes is:

- thickness of the concrete lining, $e_c$, in m.

#### Considerations and Recommendations

Three typical cross-sections are used for design purposes:

- canal with a **compound** trapezoidal cross-section excavated in soil and rock and with the minimum water level **above** the elevation of the top of the bedrock, as per Fig. 5.7.6.01. (Case 1)
- canal with a **compound** trapezoidal cross-section excavated in soil and rock and with the minimum water level **below** the elevation of the top of the bedrock, as per Fig. 5.7.6.02. (Case 2)
- canal with a **simple** trapezoidal cross-section excavated in soil only, as per Fig. 5.7.6.03. (Case 3)

The freeboard of the canal is set at 2.0 m.

The canals may or may not be lined. The lining serves two main purposes: to reduce seepage along the canal and to increase the velocity of discharge, allowing the cross-section of the canal to be smaller. Lining the canal tends to be economically advantageous when the canal is long.

When the canal is excavated through rock, a concrete lining should be used. When the canal is excavated through soil, it can be lined with a clay blanket.

#### Basic Cross-Section

**Maximum reservoir drawdown**, d (m), is given by:

$$d = NA_{max} - NA_{min}$$

where:

| | |
|---|---|
| $NA_{max}$ | maximum normal water level in the reservoir, in m; and |
| $NA_{min}$ | minimum water level in the reservoir, in m. |

The **depth of flow**, $y_m$ (m), is given by:

$$y_m = \sqrt{\frac{Q_t}{3}}$$

where:

| | |
|---|---|
| $Q_t$ | total maximum turbine flow, in m³/s. |

The **elevation of the canal bottom**, $El_{cn}$, is given by:

$$El_{cn} = NA_{min} - y_m$$

where:

| | |
|---|---|
| $NA_{min}$ | minimum water level in the reservoir, in m; and |
| $y_m$ | depth of flow, in m. |

The **width of the canal bottom**, $B_{cn}$ (m), is given by:

$$B_{cn} = 1.5 \times y_m$$

where:

| | |
|---|---|
| $y_m$ | depth of flow, in m. |

## Canal excavated through soil and rock (Case 1)

Flow is through a compound cross-section of soil and rock when:

$$El_{cn} < El_{te} - e_{te}$$

$$NA_{min} > El_{te} - e_{te}$$

where:

| | |
|---|---|
| $El_{cn}$ | elevation of the canal bottom, in m; |
| $El_{te}$ | mean elevation of the land along the canal axis, in m; |
| $e_{te}$ | thickness of the layer of soil, in m; and |
| $NA_{min}$ | minimum water level in the reservoir, in m. |

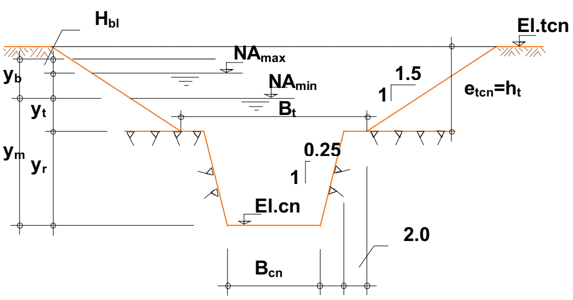



Fig. 5.7.6.01 – Typical cross-section of a headrace canal excavated through soil and rock (case 1).

The **height of the water column in the part of the canal cut through soil**, $y_t$ (m), is given by:

$$y_t = e_{te} - (El_{te} - NA_{min})$$

where:

| | |
|---|---|
| $e_{te}$ | thickness of the layer of soil, in m; |
| $El_{te}$ | elevation of the land along the canal axis, in m; and |
| $NA_{min}$ | minimum water level in the reservoir, in m. |

The **depth of excavation in soil for the canal**, $h_t$ (m), is given by:

$$h_t = e_{te}$$

where:

| | |
|---|---|
| $e_{te}$ | thickness of the layer of soil, in m. |

The **depth of the water in the part of the canal cut through rock**, $y_r$ (m), is given by:

$$y_r = y_m - y_t$$

where:

| | |
|---|---|
| $y_m$ | depth of flow, in m; and |
| $y_t$ | height of the water column in the part of the canal cut through soil, in m. |

The **total area of the canal cross-section**, $A_{cn}$ ($m^2$), is given by:

$$A_{cn} = A_{rcn} + A_{tcn}$$

where:

$$A_{rcn} = (1.5 \times y_m + 0.25 \times y_r) \times y_r$$

$$A_{tcn} = (B_{cn} + 0.5 \times y_r + 1.5 \times y_t + 4) \times y_t$$

where:

| | |
|---|---|
| $A_{rcn}$ | flow cross-section through rock, in $m^2$; |
| $A_{tcn}$ | flow cross-section through soil, in $m^2$; |
| $y_m$ | depth of flow, in m; |
| $y_r$ | depth of the water in the part of the canal cut through rock, in m; |
| $B_{cn}$ | width of the canal bottom, in m; and |
| $y_t$ | depth of the water in the part of the canal cut through soil, in m. |

**The head loss in the headrace canal**, $h_c$ (m), is given by:

$$h_c = \left[(L_{cn} - L_c) \times n^2 + L_c \times n_c^2\right] \times \frac{v_{cn}^2}{R_h^{4/3}}$$

where:

$$v_{cn} = \frac{Q_t}{A_{cn}}$$

$$R_h = \frac{A_{cn}}{B_{cn} + 2.06 \times y_r + 4 + 3.61 \times y_t}$$

| n | Type of Lining |
|---|---|
| 0.035 | for canals cut through rock |
| 0.025 | for canals cut through soil |
| 0.014 | for canals lined with concrete |
| 0.010 | for canals lined with a clay blanket |

where:

| | |
|---|---|
| n | Manning's coefficient for the unlined section; |
| $n_c$ | Manning's coefficient for the lined section; |
| $L_{cn}$ | length of the canal, in m. |
| $L_c$ | length of the lined section of the canal, in m. |
| $v_{cn}$ | mean velocity of discharge in the canal, in m/s; |
| $R_h$ | hydraulic radius, in m; |
| $Q_t$ | total maximum turbine flow, in $m^3/s$; |
| $A_{cn}$ | total flow cross-section of the canal, in $m^2$; |
| $B_{cn}$ | width of the canal bottom, in m; |
| $y_r$ | depth of the water in the part of the canal cut through rock, in m; and |
| $y_t$ | depth of the water in the part of the canal cut through soil, in m. |

The **slope of the canal bottom**, i (m/m), is given by:

$$i = \frac{h_c}{L_{cn}}$$

where:

| | |
|---|---|
| $h_c$ | head loss in the canal, in m; and |
| $L_{cn}$ | length of the canal, in m. |

For long headrace canals (over 3.0 km long), a forebay of volume $V_{cg}$ ($m^3$) is included at the intake, given by:

$$V_{cg} = \frac{300 \times Q_t}{N_g}$$

where:

| | |
|---|---|
| $Q_t$ | total maximum turbine flow, in $m^3/s$; and |
| $N_g$ | number of generating units in a plant. |

### Common Excavation (account .12.19.31.10)

The **volume of common excavation**, $V_{tcn}$ ($m^3$), is given by:

$$V_{tcn} = (B_{cn} + 0.5 \times h_r + 1.5 \times h_t + 4) \times h_t \times L_{cn}$$

where: $h_r = y_r$

where:

| | |
|---|---|
| $B_{cn}$ | width of the canal bottom, in m; |
| $y_r$ | depth of the water in the part of the canal cut through rock, in m; |
| $h_t$ | depth of excavation in soil, in m; |
| $h_r$ | depth of excavation in rock, in m; and |
| $L_{cn}$ | length of the canal, in m. |

The unit price of **common excavation** is R$ 7.60/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the canal. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation(account .12.19.31.12.11).

The **volume of excavation in rock**, $V_{rcn}$ ($m^3$), is given by:

$$V_{rcn} = (B_{cn} + 0.25 \times h_r) \times h_r \times L_{cn}$$

where: $h_r = y_r$

where:

| | |
|---|---|
| $B_{cn}$ | width of the canal bottom, in m; |
| $y_r$ | depth of the water in the part of the canal cut through rock, in m; |
| $h_r$ | depth of excavation in rock; and |
| $L_{cn}$ | length of the canal, in m. |

The unit price of **excavation in rock** is R$ 21.00/m$^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the canal. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account.12.19.31.14)

The **volume of concrete**, $V_{ccn}$ (m$^3$), is given by:

$$V_{ccn} = \left[B_{cn} + 2.06 \times y_r + 4.0 + 3.61 \times (d + y_t + 2)\right] \times e_c \times L_c$$

where:

| | |
|---|---|
| $B_{cn}$ | width of the canal bottom, in m; |
| $y_r$ | depth of the water in the part of the canal cut through rock, in m; |
| d | maximum reservoir drawdown, in m; |
| $y_t$ | depth of the water in the part of the canal cut through soil, in m; |
| $e_c$ | thickness of the concrete lining, in m; and |
| $L_c$ | length of the section of canal lined with concrete, in m. |

The amounts of cement and reinforcement steel are:

| | cement (kg/m$^3$) | reinforcement steel (kg/m$^3$) |
|---|---|---|
| conventional concrete | 275 | 50 |
| shotcrete | 300 | 70 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- conventional concrete: 234.00/m$^3$
- shotcrete: 128.00/m$^3$

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Canal excavated through rock (Case 2)

The flow is through a compound cross-section, but predominantly of rock, when:

$$NA_{min} \leq El_{te} - e_{te}$$

where:

| | |
|---|---|
| $NA_{min}$ | minimum water level in the reservoir, in m; |
| $El_{te}$ | elevation of the land along the canal axis, in m; and |
| $e_{te}$ | thickness of the layer of soil, in m. |

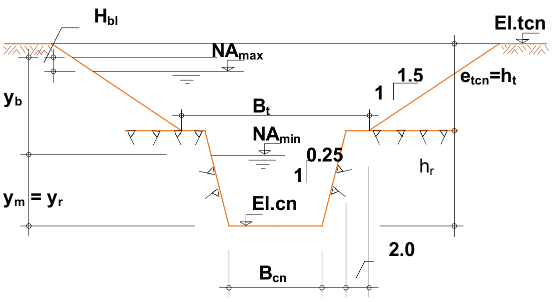



Fig. 5.7.6.02 – Typical cross-section of a headrace canal through rock (case 2).

The **depth of the water in the part of the canal cut through soil**, $y_t$ (m), is given by:

$$y_t = 0$$

The **depth of excavation in soil for the canal**, $h_t$ (m), is given by:

$$h_t = e_{te}$$

where:

| | |
|---|---|
| $e_{te}$ | thickness of the layer of soil, in m. |

The **depth of the water in the part of the canal cut through rock**, $y_r$ (m), is given by:

$$y_r = y_m$$

where:

| | |
|---|---|
| $y_m$ | depth of flow, in m. |

The **depth of excavation in rock**, $h_r$ (m), is given by:

$$h_r = y_r + El_{te} - e_{te} - NA_{min}$$

where:

| | |
|---|---|
| $y_r$ | depth of the water in the part of the canal cut through rock, in m; |
| $El_{te}$ | elevation of the land along the canal axis, in m; |
| $e_{te}$ | thickness of the layer of soil, in m; and |
| $NA_{min}$ | minimum water level in the reservoir, in m. |

The **total flow cross-section through the canal**, $A_{cn}$ ($m^2$), is given by:

$$A_{cn} = (1.5 \times y_m + 0.25 \times y_r) \times y_r$$

where:

| | |
|---|---|
| $y_m$ | depth of flow, in m; and |
| $y_r$ | depth of the water in the part of the canal cut through rock, in m. |

The **head loss in the headrace canal**, $h_c$ (m), is given by:

$$h_c = [(L_{cn} - L_c) \times n^2 + L_c \times n_c^2] \times \frac{v_{cn}^2}{R_h^{4/3}}$$

where:

$$v_{cn} = \frac{Q_t}{A_{cn}}$$

$$R_h = \frac{A_{cn}}{B_{cn} + 2.06 \times y_r}$$

where:

| n | Type of Lining |
|---|---|
| 0.035 | for canals cut through rock |
| 0.025 | for canals cut through soil |
| 0.014 | for canals lined with concrete |
| 0.010 | for canals lined with a clay blanket |

where:

| | |
|---|---|
| n | Manning's coefficient for the unlined section; |
| $n_c$ | Manning's coefficient for the lined section; |
| $L_{cn}$ | length of the canal, in m; |
| $L_c$ | length of the lined section of the canal, in m; |
| $v_{cn}$ | mean velocity of discharge in the canal, in m/s; |
| $R_h$ | hydraulic radius, in m; |
| $Q_t$ | total maximum turbine flow, in $m^3/s$; |
| $A_{cn}$ | total flow cross-section of the canal, in $m^2$; |
| $B_{cn}$ | width of the canal bottom, in m; and |
| $y_r$ | depth of the water in the part of the canal cut through rock, in m. |

The **slope of the canal bottom**, i (m/m), is given by:

$$i = \frac{h_c}{L_{cn}}$$

where:

| | |
|---|---|
| $h_c$ | head loss in the canal, in m; and |
| $L_{cn}$ | length of the canal, in m. |

For long headrace canals (over 3.0 km long), a forebay of volume $V_{cg}$ ($m^3$) is included at the intake, given by:

$$V_{cg} = \frac{300 \times Q_t}{N_g}$$

where:

| | |
|---|---|
| $Q_t$ | total maximum turbine flow, in m³/s; and |
| $N_g$ | number of generating units in a plant. |

Common Excavation (account .12.19.31.12.10)

The **volume of common excavation**, $V_{tcn}$ (m³), is given by:

$$V_{tcn} = (B_{cn} + 0.5 \times h_r + 1.5 \times h_t + 4) \times h_t \times L_{cn}$$

where:

| | |
|---|---|
| $B_{cn}$ | width of the canal bottom, in m; |
| $h_r$ | depth of excavation in rock, in m; |
| $h_t$ | depth of excavation in soil, in m; and |
| $L_{cn}$ | length of the canal, in m. |

The unit price of **common excavation** is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the canal. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves a favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower%.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation (account .12.19.31.12.11)

The **volume of excavation in rock** , $V_{rcn}$ (m³), is given by:

$$V_{rcn} = (B_{cn} + 0.25 \times h_r) \times h_r \times L_{cn}$$

where:

| | |
|---|---|
| $B_{cn}$ | width of the canal bottom, in m; |
| $h_r$ | depth of excavation in rock, in m; and |
| $L_{cn}$ | length of the canal, in m. |

The unit price of **excavation in rock** is R$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the canal. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves a favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account.12.19.31.14)

The **volume of concrete**, $V_{ccn}$ ($m^3$), is given by:

$$V_{ccn} = \left[B_{cn} + 2.06 \times h_r + 4.0 + 3.61 \times (d_1 + y_t)\right] \times e_c \times L_c$$

where:

$$d_1 = NA_{max} - (El_{te} - e_{te}) + 2$$

where:

| | |
|---|---|
| $B_{cn}$ | width of the canal bottom, in m; |
| $h_r$ | depth of excavation in rock, in m; |
| $d_1$ | secondary variable, in m; |
| $y_t$ | depth of the water in the part of the canal cut through soil; in m; |
| $e_c$ | thickness of the concrete lining, in m; |
| $L_c$ | length of the section of canal lined with concrete, in m; |
| $NA_{max}$ | maximum level of the reservoir, in m; |
| $El_{te}$ | elevation of the land along the canal axis, in m; and |
| $e_{te}$ | thickness of the layer of soil, in m. |

The **amounts of cement and reinforcement steel** are:

| | cement (kg/m$^3$) | reinforcement steel (kg/m$^3$) |
|---|---|---|
| conventional concrete | 275 | 50 |
| shotcrete | 300 | 70 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- conventional concrete: 234.00/m$^3$
- shotcrete: 128.00/m$^3$

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Canal excavated through soil (Case 3)

The canal is excavated through soil only when:

$$El_{cn} \geq El_{te} - e_{te}$$

where:

| | |
|---|---|
| $El_{cn}$ | elevation of the canal bottom; |
| $El_{te}$ | elevation of the land along the canal axis; and |
| $e_{te}$ | thickness of the layer of soil, in m. |

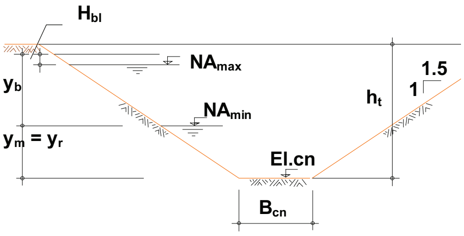



Fig. 5.7.6.03 – Typical cross-section of a headrace canal cut through soil (Case 3).

The **depth of the water in a canal cut through soil**, $y_t$ (m), is given by:

$$y_t = y_m$$

where:

| | |
|---|---|
| $y_m$ | depth of flow, in m. |

The **depth of excavation for the canal**, $h_t$ (m), is given by:

$$h_t = El_{te} - El_{cn}$$

where:

| | |
|---|---|
| $El_{te}$ | elevation of the land along the canal axis, in m; and |
| $El_{cn}$ | elevation of the canal bottom, in m. |

The total flow cross-section of the canal, $A_{cn}$ (m²), is given by:

$$A_{cn} = (B_{cn} + 1.5 \times y_t) \times y_t$$

where:

| | |
|---|---|
| $B_{cn}$ | width of the canal bottom, in m; and |
| $y_t$ | depth of the water in the part of the canal cut through soil, in m. |

**The head loss in the headrace canal**, $h_c$ (m), is given by:

$$h_c = \left[(L_{cn} - L_c) \times n^2 + L_c \times n_c^2\right] \times \frac{v_{cn}^2}{R_h^{4/3}}$$

where:

$$v_{cn} = \frac{Q_t}{A_{cn}}$$

$$R_h = \frac{A_{cn}}{B_{cn} + 3.61 \times y_t}$$

| n | Type of Lining |
|---|---|
| 0.025 | for canals cut through soil |
| 0.014 | for canals lined with concrete |
| 0.010 | for canals lined with a clay blanket |

where:

| | |
|---|---|
| $n$ | Manning's coefficient for the unlined section; |
| $n_c$ | Manning's coefficient for the lined section; |
| $L_{cn}$ | length of the canal, in m. |
| $L_c$ | length of the lined section of the canal, in m. |
| $v_{cn}$ | mean velocity of discharge in the canal, in m/s; |
| $R_h$ | hydraulic radius, in m; |
| $Q_t$ | total turbine flow, in m³/s; |
| $A_{cn}$ | total flow cross-section of the canal, in m²; |
| $B_{cn}$ | width of the canal bottom, in m; and |
| $y_t$ | depth of the water in the part of the canal cut through soil, in m. |

The **slope of the canal bottom**, i (m/m), is given by:

$$i = \frac{h_c}{L_{cn}}$$

where:

| | |
|---|---|
| $h_c$ | head loss in the canal, in m; and |
| $L_{cn}$ | length of the canal, in m. |

For long headrace canals (over 3.0 km long), a forebay of volume $V_{cg}$ ($m^3$) is included at the intake, given by:

$$V_{cg} = \frac{300 \times Q_t}{N_g}$$

where:

| | |
|---|---|
| $Q_t$ | total turbine flow, in m³/s; and |
| $N_g$ | number of generating units in a plant. |

### Common Excavation (account .12.19.31.12.10)

The **volume of common excavation**, $V_{tcn}$ ($m^3$), is given by:

$$V_{tcn} = (B_{cn} + 1.5 \times h_t) \times h_t \times L_{cn}$$

where:

| | |
|---|---|
| $B_{cn}$ | width of the canal bottom, in m; |
| $h_t$ | depth of excavation in soil, in m; and |
| $L_{cn}$ | length of the canal, in m. |

The unit price of **common excavation** is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the canal. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account.12.19.31.14)

The **volume of concrete**, $V_{ccn}$ ($m^3$), is given by:

$$V_{ccn} = \left[B_{cn} + 3.61 \times (d + y_t + 2)\right] \times e_c \times L_c$$

where:

| | |
|---|---|
| $B_{cn}$ | width of the canal bottom, in m; |
| d | maximum reservoir drawdown, in m; |
| $y_t$ | depth of the water in the part of the canal cut through soil; in m; |
| $e_c$ | thickness of the concrete lining, in m; and |
| $L_c$ | length of the section of canal lined with concrete, in m. |

The amounts of **cement and reinforcement steel** are:

| | cement (kg/m³) | reinforcement steel (kg/m³) |
|---|---|---|
| conventional concrete | 275 | 50 |
| shotcrete | 300 | 70 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- conventional concrete: 234.00/$m^3$
- shotcrete: 128.00/$m^3$

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

## GRAVITY INTAKE (ACCOUNT .12.19.30)

This item applies for projects with penstocks, whether they be pressure penstocks or not, and with Pelton, Francis or Kaplan turbines with a steel spiral casing.

The intake recommended for this item is shown in Fig. 5.7.6.04.

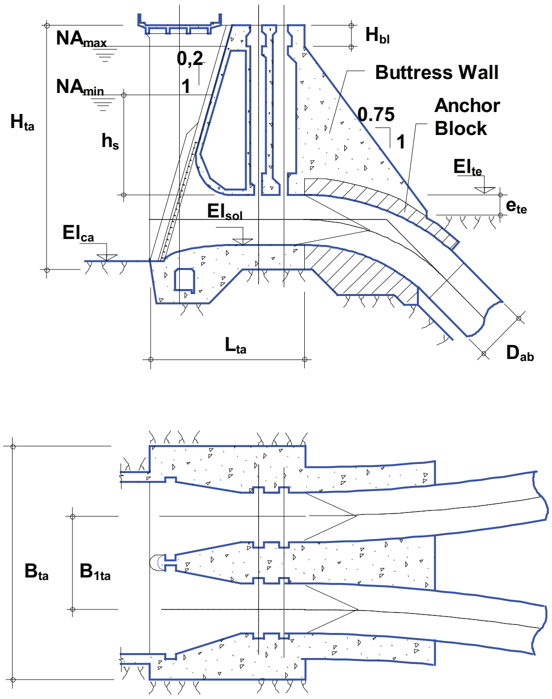



Fig 5.7.6.04 – Typical cross-section and plan of a gravity intake.

## Basic Data

The main **information required for dimensioning** purposes is as follows:

- number of generating units, $N_g$, from item 5.7.2.;
- number of generating units per pressure penstock or tunnel, $N_f$, from item 5.7.2., when applicable;
- internal diameter of the conduit associated to the intake, $D_{ab}$, in m;
- maximum normal water level in the reservoir, $NA_{max}$, from item 4.4; and
- elevation of the intake sill, $El_{sol}$.

The main **information required for quantification** purposes is as follows:

- total maximum turbine flow, $Q_t$, in m$^3$/s, from item 5.7.2.;
- mean elevation of the land in the structure area, $El_{te}$, in m; and
- mean thickness of the layer of soil in the intake area, $e_{te}$, in m.

## Dimensions of the intake

The **number of openings in the intake**, $N_{at}$, is:

for pressure penstocks or tunnels: $N_{at} = \frac{N_g}{N_f}$

for intake penstocks: $N_{at} = 1$

where:

| | |
|---|---|
| $N_g$ | number of generating units; and |
| $N_f$ | number of generating units per intake conduit. |

The **height of the intake**, $H_{ta}$ (m), is given by:

$$H_{ta} = NA_{max} - El_{sol} + H_{bl} + 2.5$$

where:

| | |
|---|---|
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $H_{bl}$ | 4.0 m – freeboard of the intake; and |
| $El_{sol}$ | elevation of the intake sill, in m. |

The **width of the block of a unit perpendicular to flow**, $B_{1ta}$ (m), is given by:

$$B_{1ta} = 1.2 \times D_{ab} + 1.2$$

where:

| | |
|---|---|
| $D_{ab}$ | internal diameter of the intake conduit, in m. |

The **total width**, $B_{ta}$ (m), is given by:

$$B_{ta} = N_{at} \times B_{1ta} + 2 \times 2.0$$

where:

| | |
|---|---|
| $N_{at}$ | number of openings in the intake; |
| $B_{1ta}$ | width of the block of a unit, in m; and |
| 2.0 | extra thickness for the end pillars, in m. |

The **length of the intake at its base**, in the direction of flow, $L_{ta}$ (m), is given by:

$$L_{ta} = 9.2 + 0.20 \times H_{ta}$$

where:

| | |
|---|---|
| $H_{ta}$ | height of the intake, in m; and |
| 0.2 | slope of the upstream face. |

### Common Excavation (account .12.19.30.12.10)

The volume of common excavation, $V_{tta}$ ($m^3$), is given by:

$$V_{tta} = B_{ta} \times L_{ta} \times e_{te}$$

where:

| | |
|---|---|
| $B_{ta}$ | total width, in m; |
| $L_{ta}$ | length of the intake at the foundations, in m; and |
| $e_{te}$ | mean thickness of the layer of soil in the intake area, in m. |

The unit price of **common excavation** is R$ 7.60/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the intake. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation (account .12.19.30.12.11)

The volume of excavation in rock, $V_{rta}$ ($m^3$), is given by:

$$V_{rta} = L_{ta} \times \left[ El_{te} - e_{te} - (El_{sol} - 2.5) \right] \times B_{ta}$$

where:

| | |
|---|---|
| $B_{ta}$ | total width, in m; |
| $L_{ta}$ | length of the intake at the foundations, in m; |
| $e_{te}$ | mean thickness of the layer of soil in the intake area, in m; |
| $El_{te}$ | mean elevation of the land in the intake area, in m; and |
| $El_{sol}$ | elevation of the intake sill, in m. |

The unit price of **excavation in rock** is R$ 21.00/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the intake. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.19.30.13)

The **area of foundation to be cleaned**, $A_{lf}$ ($m^2$), is given by:

$$A_{lf} = B_{ta} \times L_{ta}$$

where:

| | |
|---|---|
| $B_{ta}$ | total width, in m; and |
| $L_{ta}$ | length of the intake at the foundations, in m. |

The **foundation treatment** entails a drainage line immediately downstream from the grout curtain, with the **total length of each grout hole**, $L_{tf}$ (m), being given by:

$$L_{tf} = \frac{B_{ta}}{3.0} \times L_{1tf}$$

$$L_{1tf} = 1.5 \times (NA_{max} - El_{sol}) \leq 40m$$

where:

| | |
|---|---|
| $B_{ta}$ | total width of the intake, in m; |
| $L_{1tf}$ | length of the grout holes, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_{sol}$ | elevation of the sill, in m; |
| 3.0 | space between the grout holes, in m. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m²
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account .12.19.30.14)

The **volume of concrete for the intake**, $V_{cta}$ (m³), is given by:

$$V_{cta} = V_{ctp} + N_{at} \times V_{ctb} + V_{ctc}$$

where:

$$V_{ctp} = 2 \times (2.0 \times L_{ta} + 10.0) \times H_{ta}$$

$$V_{ctc} = 0.375 \times (H_{ta} - D_{ab} - 9.3)^2 \times B_{ta}$$

$$V_{ctb} = 1{,}3 \times e^z$$

$$z = (0.0460 - 0.00167 \times D_{ab}) \times (H_{ta} - 104.0) + 10.16$$

where:

| | |
|---|---|
| $V_{ctp}$ | volume of concrete for the end walls, in m³; |
| $V_{ctb}$ | volume of concrete for the block of the unit, in m³; |
| $V_{ctc}$ | volume of concrete for the downstream buttress wall, in m³; |
| $L_{ta}$ | length of the intake at the foundations, in m; |
| $z_i$ | parameter, in m³; |
| $N_{at}$ | number of openings, in m; |
| $D_{ab}$ | internal diameter of the intake conduit, in m; and |
| $H_{ta}$ | height of the intake, in m. |

The amounts of cement and reinforcement steel are:

| | cement (kg/m³) | reinforcement steel (kg/m³) |
|---|---|---|
| external wall | 200 | 40 |
| block of the unit | 300 | 60 |
| buttress wall | 200 | 20 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- external wall: 128.00/m³
- block of the unit: 174.00/m³
- buttress wall: 129.00/m³

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Emergency Gates (account .12.19.30.23.16)

The **acquisition cost of each emergency gate** for the intake, $C_{cp}$ (R$), including the respective operating system and fixed and embedded parts – FOB cost excluding tranportation and insurance, assembly and testing and provisions for taxes payable, depending on the current tax regime – can be obtained from the expression below (or from Graph B.23, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

for $0.13 \leq z \leq 9.17$:

$$C_{cp} = -4.3986 \times z^2 + 124.79 \times z + 110.2$$

for $9.17 < z \leq 125.39$:

$$C_{cp} = -0.128 \times z^2 + 57.311 \times z + 369.83$$

where:

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

where:

| | |
|---|---|
| z | parameter, in $m^4$; |
| $B_{cp}$ | width of the gate, in m; |
| $H_{cp}$ | height of the gate, in m; |
| $H_x$ | maximum hydrostatic load on the gate sill, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_{sol}$ | elevation of the intake sill, in m. |

The following percentages must be added to the FOB price:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for taxes and charges payable on the equipment.

### Stoplogs (account .12.19.30.23.17)

The **acquisition cost of each stoplog** for the intake, $C_{sl}$ (R$), – FOB cost – can be obtained from the expression below (or from Graph B.25, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil:

for $0.16 \leq z \leq 54.43$:

$$C_{sl} = 72.896 \times z^{0.716}$$

where: $z = \frac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$

where:

| | |
|---|---|
| z | parameter, in $m^4$; |
| $B_{cp}$ | width of the gate, in m; |
| $H_{cp}$ | height of the gate, in m; and |
| $H_x$ | maximum hydrostatic load on the gate sill, in m. |

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

The **overall acquisition cost of fixed parts and parts embedded in the concrete** of the stoplogs for the inlet, $C_{gpf}$ (R$), – FOB cost – is given by the expression below. The figures are valid for December 2006 and for projects anywhere in Brazil:

$$C_{gpf} = 2 \times N_{at} \times (H_{ta} - 1.0) \times 2084.80$$

where:

| | |
|---|---|
| $N_{at}$ | number of openings in the intake; and |
| $H_{ta}$ | height of the intake, in m. |

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

### Crane (account .12.19.30.23.20)

The **acquisition cost of the gantry crane** for the intake, $C_{pcr}$ (R$), – FOB cost – is given by the expression below (or from Graph B.27, annex B, as a function of the dimensions of the emergency fixed-wheel gate and the hydrostatic load on its sill). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

for $0.13 \le z \le 54.35$:

$$C_{pcr} = -0.71 \times z^2 + 97.3 \times z + 57.78$$

where: $z = \dfrac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$

where:

| | |
|---|---|
| z | parameter, in $m^4$; |
| $B_{cp}$ | width of the gate, in m; |
| $H_{cp}$ | height of the gate, in m; and |
| $H_x$ | maximum hydrostatic load on the bottom of the gate, in m. |

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost.

### Trash Racks and Trash Rack Cleaners (account .12.19.30.23.21)

The **overall acquisition cost of the set of trash racks** and respective embedded parts, $C_{gr}$ (R$), – FOB cost – can be obtained from Graph B28, annex B, as a function of their dimensions or by the equivalent expression below. The figures are valid for December 2006 and for projects anywhere in Brazil:

valid for $2 \le (B \times H) \le 750$: $C_{gr} = 5.35 \times B \times H$

where:

| | |
|---|---|
| B | width of the trash racks, in m; |
| H | height of the trash racks, in m; |
| 1.0 | velocity of discharge in the section with the trash racks, in m/s. |

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost.

### Other Costs (account .12.19.30.17)

The cost of other services is estimated as being 2% of the total cost of the intake.

## PROJECT WITH INTEGRAL INTAKE POWERHOUSE (ACCOUNT .12.19.30)

This item applies to projects equipped with Kaplan turbines with a semi-spiral casing made of concrete.

The intake recommended for this item can be seen in Fig. 5.8.6.04a.

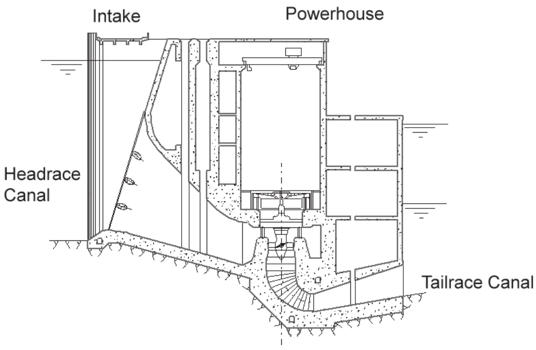



Fig. 5.7.6.04a – Typical cross-section of a project with an integral intake powerhouse.

### Basic Data

The main **information required for dimensioning** purposes is as follows:

- number of generating units, $N_g$, from item 5.7.2.;
- maximum normal water level in the reservoir, $NA_{max}$, from item 4.4.;
- elevation of the intake sill, $El_{sol}$; and
- width of the block of a unit for the powerhouse, $B_{1cf}$, in m, from item 5.7.2.

The main **information required for quantification** purposes is as follows:

- total maximum turbine flow, $Q_t$, in m³/s, from item 5.7.2.;
- volume of soil excavated, $V_{tta}$, in $m^3$;
- volume of surface rock excavation, $V_{rta}$, in $m^3$;
- volume of concrete, $V_{cta}$, in $m^3$;
- the area of foundation to be cleaned, $A_{lf}$, in $m^2$;
- width of the emergency gate, $L_{cp}$ in m;
- height of the emergency gate, $H_{cp}$ in m;
- maximum hydrostatic load on the sill of the emergency gate, $H_x$ in m; and
- height of the stoplog, $H_{sl}$, in m.

### Dimensions of the intake

The **number of openings in the intake**, $N_{at}$, is given by:

$$N_{at} = N_g$$

where:

| | |
|---|---|
| $N_g$ | number of generating units. |

The **height of the intake**, $H_{ta}$ (m), is given by:

$$H_{ta} = NA_{max} - El_{sol} + H_{bl} + 2.5$$

where:

| | |
|---|---|
| $NA_{max}$ | maximum normal water level in the reservoir; |
| $H_{bl}$ | 4.0m, height of the intake freeboard, in m; and |
| $El_{sol}$ | elevation of the intake sill, in m. |

The **width of the block of a unit perpendicular to flow**, $B_{1ta}$ (m), is given by:

$$B_{1ta} = B_{1cf}$$

where:

| | |
|---|---|
| $B_{1cf}$ | width of the block of a unit for the powerhouse, in m. |

The **total width**, $B_{ta}$ (m), is given by:

$$B_{ta} = N_g \times B_{1ta} + 2 \times 2.0$$

where:

| | |
|---|---|
| $N_g$ | number of generating units; |
| $B_{1ta}$ | width of the block of a unit, in m; and |
| 2.0 | extra thickness for the end pillars, in m. |

The lengthwise horizontal projection **of the base of the intake**, in the direction of flow, $L_{ta}$ (m), is given by:

$$L_{ta} = 9.2 + 0.20 \times H_{ta}$$

where:

| | |
|---|---|
| $H_{ta}$ | height of the intake, in m; and |
| 0.2 | slope of the upstream face. |

### Common Excavation (account .12.19.30.12.10)

The volume of common excavation should be determined from the project design.

The unit price of **common excavation** is R$ 7.60/m$^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the intake. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation (account .12.19.30.12.11)

The volume of excavation in rock should be determined from the project design.

The unit price of **excavation in rock** is R$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the intake. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.19.30.13)

The area of foundation to be cleaned should be determined from the project design.

The **foundation treatment** should include a drainage line immediately downstream from a grout curtain, with the **length of each grout hole**, $L_{tf}$ (m), given by:

$$L_{tf} = \frac{B_{ta}}{3.0} \times L_{1tf}$$

$$L_{1tf} = 1.5 \times (NA_{max} - El_{sol}) \leq 40m$$

where:

| | |
|---|---|
| $B_{ta}$ | total width of the intake, in m; |
| $L_{1tf}$ | length of the grout holes, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_{sol}$ | elevation of the sill, in m; and |
| 3.0 | space between the grout holes, in m. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m²
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account .12.19.30.14)

The **volume of concrete for the intake**, $V_{cta}$ ($m^3$), should be defined from the project design.

The amounts of cement and reinforcement steel are:

| | cement ($kg/m^3$) | reinforcement steel ($kg/m^3$) |
|---|---|---|
| external wall | 200 | 40 |
| block of the unit | 300 | 60 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- intake: 214.00/$m^3$
- external walls: 128.00/$m^3$

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Emergency Gates (account .12.19.30.23.16)

The **acquisition cost of each emergency gate** for the intake, $C_{cp}$ (R$), including the respective operating system and fixed and embedded parts – FOB cost excluding tranportation and insurance, assembly and testing and provisions for taxes payable, depending on the current tax regime – can be obtained from the expression below (or from Graph B.23, annex B, as a function of its dimensions and maximum hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil

for: $0.13 \le z \le 9.17$: $C_{cp} = -4.3986 \times z^2 + 124.79 \times z + 110.2$ and

for: $9.17 \le z \le 125.39$: $C_{cp} = -0.128 \times z^2 + 57.311 \times z + 369.83$

where:

$$z = \frac{B_{cp}^2 \times H_{cp} \times H_{xe}}{1000} \qquad H_{xe} = \frac{NA_{max} - El_{td}}{3}$$

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

$$n_v = \text{int}\left(\frac{D_{td}}{4.5} + 0.9\right)$$

where:

| | |
|---|---|
| z | parameter, in $m^4$; |
| $B_{cp}$ | width of the gates in the diversion tunnel, in m; |
| $H_{cp}$ | height of the gates in the diversion tunnel, in m; |
| $H_x$ | maximum hydrostatic load on the sill of the diversion tunnel gate, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $D_{td}$ | internal diameter of the tunnels, in m; |
| $n_v$ | number of openings in the inlet to each diversion tunnel; and |
| int(x) | function that returns the integer part of x. |

The following percentages should be added to the FOB cost:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for taxes and charges payable on the equipment.

### Stoplogs (account .12.19.30.23.17)

The **acquisition cost of each stoplog** for the intake, $C_{sl}$ (R$), – FOB cost – can be obtained from the expression below (or from Graph B.25, annex B, as a function of its dimensions and hydrostatic load). The figures are valid for December 2006 and for projects anywhere in Brazil:

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

where:

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

where:

| | |
|---|---|
| z | parameter, in $m^4$; |
| $B_{cp}$ | width of the gates in the diversion tunnel, in m; |
| $H_{cp}$ | height of the gates in the diversion tunnel, in m; and |
| $H_x$ | maximum hydrostatic load on the sill of the diversion tunnel gate, in m. |

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

The **overall acquisition cost of fixed parts and parts embedded in the concrete** of the stoplogs for the inlet, $C_{gpf}$ (R$), – FOB cost – is given by the expression below. The figures are valid for December 2006 and for projects anywhere in Brazil:

$$C_{gpf} = 2 \times N_v \times N_{td} \times H_{td} \times 2084.80$$

where: $H_{td} = 2.5 \times H_{cp}$

where:

| | |
|---|---|
| $N_v$ | number of openings in the inlet to each diversion tunnel; |
| $N_{td}$ | number of diversion tunnels; |
| $H_{td}$ | height of the inlet structure from the sill, in m; and |
| $H_{cp}$ | height of the gates in the diversion tunnel, in m. |

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

### Crane (account .12.19.30.23.20)

The **acquisition cost of the gantry crane**, $C_{pcr}$ (R$), for the intake – FOB cost – is given by the expression below (or from Graph B.27, annex B, as a function of the dimensions of the emergency fixed-wheel gate and the hydrostatic load on its sill). The figures are valid for December 2006 and for projects anywhere in Brazil:

for $0.13 \le z \le 54.35$: $C_{pcr} = -0.71 \times z^2 + 97.3 \times z + 57.78$

where: $z = \dfrac{B_{cp}^2 \times H_{cp} \times H_x}{1000}$

where:

| | |
|---|---|
| z | parameter, in $m^4$; |
| $B_{cp}$ | width of the gate, in m; |
| $H_{cp}$ | height of the gate, in m; and |
| $H_x$ | maximum hydrostatic load on the bottom of the gate, in m. |

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

### Trash Racks and Trash Rack Cleaners (account .12.19.30.23.21)

The **overall acquisition cost of the set of trash racks** and respective embedded parts, $C_{gr}$ (R$), – FOB cost – can be obtained from Graph B28, annex B, as a function of their dimensions or by the equivalent expression below. The figures are valid for December 2006 and for projects anywhere in Brazil:

Valid for $2 \le (B \times H) \le 750$: $C_{gr} = 5.35 \times B \times H$

where:

| | |
|---|---|
| B | width of the trash racks, in m; |
| H | height of the trash racks, in m; and |
| 1.0 | velocity of discharge in the section with the trash racks, in m/s. |

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

### Other Costs (account .12.19.30.17)

The cost of other services is estimated as being 2% of the total cost of the intake.

## HEADRACE TUNNEL

The main **information required for dimensioning** purposes is as follows:

- maximum turbine flow, $Q_t$, in $m^3/s$, from item 5.7.2.;
- length of the tunnel, $L_{ad}$, in m;
- length of the section lined with structural concrete, $L_c$, in m;
- length of the section lined with shotcrete, $L_{cp}$, in m;

The **information required for quantification** purposes is as follows:

- geological conditions of the area crossed by the tunnels;
- maximum normal water level in the reservoir, $NA_{max}$, from item 4.4.;

- elevation of the intake sill, $El_{sol}$; and
- length of the section where the rock is to be treated, $L_{pt}$, in m.

### Considerations and Recommendations

**This text** relates to headrace tunnels with a typical cross-section, as shown in Fig. 5.7.6.05.

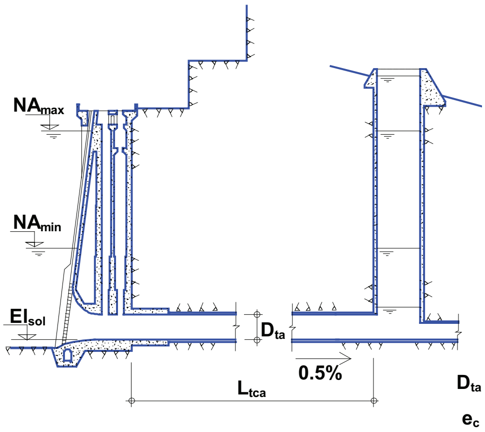



Fig. 5.7.6.05 Typical longitudinal section of a headrace tunnel.

When the diameter of the headrace tunnels exceeds the **maximum diameter**, the mean velocity of discharge must be increased to the limit, and if necessary the tunnel should be lined with shotcrete to raise the velocity limit.

When the diameter of the headrace tunnels is lower than the **minimum diameter**, the mean velocity can be reduced or the tunnel can be partially replaced by a canal or a surface penstock.

The **slope of the tunnel** should be no more than 0.005 m/m.

### Diameter of the headrace tunnel

Initially, the **mean velocity of discharge**, $v_{ad}$ (m/s), can be taken as:

$$v_{ad} = 0{,}8 \times v_{max}$$

where:

| $v_{max}$ | Type of Lining |
|---|---|
| 2.2 | for an unlined tunnel; |
| 3.0 | for a tunnel lined with shotcrete; and |
| 4.5 | for a tunnel lined with structural concrete. |

where:

| | |
|---|---|
| $v_{max}$ | mean velocity of discharge limit for the tunnel, in m/s. |

The **internal diameter of the headrace tunnel**, $D_{ad}$ (m), is given by:

$$D_{ad} = \sqrt{\frac{Q_t}{0.8927 \times v_{ad}}}$$

where:

$$2.5 \le D_{ad} \le 15.0\,m$$

where:

| | |
|---|---|
| $Q_t$ | maximum turbine flow, in m³/s; and |
| $v_{ad}$ | mean velocity of discharge, in m/s. |

The **total head loss along the tunnel**, $h_a$ (m), is given by:

$$h_a = h_o + h_f$$

where:

$$h_o = \Sigma k_{oi} \times \frac{v_{ad}^2}{2 \times g}$$

$$h_f = 6.23 \times [(L_{ad} - L_c - L_{cp}) \times n^2 + L_c \times n_{cr}^2 + L_{cp} \times n_{cp}^2] \times \frac{v_{ad}^2}{D_{ad}^{4/3}}$$

For $(r_i/D_{ad}) < 5$

$$k_{oi} = 0.2147 \times \left(\frac{r_i}{D_{ad}}\right)^{-0.5718} \times (0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i)$$

For $(r_i/D_{ad}) \geq 5$

$$k_{oi} = \left(0.08 - 0.002 \times \left(\frac{r_i}{D_{ad}} - 5\right)\right) \times (0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i)$$

where:

| | |
|---|---|
| $h_o$ | head loss at bends, in m; |
| $h_f$ | continuous head loss, in m; |
| $v_{ad}$ | mean velocity of discharge in the tunnel, in m/s; |
| g | 9.81 m/s² – acceleration due to gravity; |
| $k_{oi}$ | coefficient of head loss at bends in the tunnel; |
| $r_i$ | radius of curvature of the section, in meters; |
| $L_{ad}$ | length of the tunnel, in m; |
| $L_c$ | length of the section lined with structural concrete, in m; |
| $L_{cp}$ | length of the section lined with shotcrete, in m; |
| n | 0.035, Manning's coefficient for the unlined section; |
| $n_{cr}$ | 0.012, Manning's coefficient for the section lined with structural concrete; |
| $n_{cp}$ | 0.022, Manning's coefficient for the section lined with shotcrete; |
| $D_{ad}$ | internal diameter of the tunnel, in m; and |
| θ | deflection of the tunnel axis, in radians. |

### Underground Excavation of Rock (account .12.19.32.12.12)

The **volume rock excavated underground** for the headrace tunnel, $V_{sad}$ ($m^3$), is given by:

$$V_{sad} = V_{sae} + V_{san}$$

where:

$$V_{sae} = 0.8927 \times (D_{ad} + 2 \times e_c)^2 \times L_c$$

$$V_{san} = 0.8927 \times D_{ad}^2 \times (L_{ad} - L_c)$$

$$e_c = k_g \times [0.091 \times D_{ad}^{0,62} + 0.0034 \times (H - 30)]$$

$$H = NA_{max} - El_{sol}$$

| $k_g$ | geological conditions |
|---|---|
| 1.0 | good |
| 1.4 | average |
| 2.0 | poor or no information |

where:

| | |
|---|---|
| $V_{sae}$ | volume of rock excavated underground for the section lined with structural concrete, in $m^3$; |
| $V_{san}$ | volume of rock excavated underground for the unlined section and the section lined with shotcrete, in $m^3$; |
| $D_{ad}$ | internal diameter of the tunnel, in m; |
| $e_c$ | thickness of the structural concrete lining, in m; |
| $k_g$ | coefficient to represent geological conditions; |
| H | hydrostatic load in the tunnel, in m; |
| $e_{cp}$ | 0.05 m, thickness of the shotcrete lining; |
| $L_{ad}$ | length of the tunnel, in m; |
| $L_c$ | length of the tunnel lined with structural concrete, in m; |
| $L_{cp}$ | length of the tunnel lined with shotcrete, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; and |
| $El_{sol}$ | elevation of the intake sill, in m. |

The unit price of **underground excavation**, $P_{us}$ (R$/m^3$), (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil, can be obtained from the expression below (or from Graph B33, annex B, as a function of the excavated area). This is the price per cubic meter measured using the excavation line of the project design and includes excavating, loading, transportation up to 1.5 km and unloading:

valid for $4 \leq A_{se} \leq 300$: $P_{us} = 474.08 \times A_{se}^{-0.3987}$

where: $A_{se} = 0.8927 \times D_{ad}^2$

where:

| | |
|---|---|
| $A_{se}$ | area of the excavated area, in $m^2$. |

A careful appraisal should be made of the different situations where the tunnels form a representative part of the cost estimate, paying special attention to the geological conditions in the region and particularly long tunnels.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Rock Cleaning and Treatment (account .12.19.32.13)

The **total length of the rock anchors**, $L_{tfp}$ (m), is given by:

$$L_{tfp} = 11.9 \times D_{ad} \times L_{pt}$$

where:

| | |
|---|---|
| $D_{ad}$ | internal diameter of the tunnel, in m; and |
| $L_{pt}$ | length of the section where the rock is to be treated, in m. |

The unit price of **rock anchors is** R$ 241.00/m (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per meter measured using the excavation line and includes the service per se and the supply of inputs and equipment.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account .12.19.32.14)

The **volume of concrete**, $V_{cad}$ ($m^3$), is given by:

$$V_{cad} = V_{cap} + V_{cae}$$

where:

$$V_{cap} = 2.57 \times D_{ad} \times L_{cp} \times e_{cp}$$

$$V_{cae} = 0.8927 \times \left[ (D_{ad} + 2 \times e_c)^2 - D_{ad}^2 \right] \times L_c$$

where:

| | |
|---|---|
| $V_{cap}$ | volume of shotcrete, in $m^3$; |
| $V_{cae}$ | volume of structural concrete for lining, in $m^3$; |
| $D_{ad}$ | internal diameter of the tunnel, in m; |
| $L_{cp}$ | length of the tunnel lined with shotcrete, in m; |
| $e_{cp}$ | 0.05, mean thickness of the shotcrete, in m; |
| $e_c$ | thickness of the structural concrete lining, in m; and |
| $L_c$ | length of tunnel lined with structural concrete, in m. |

The amounts of cement and reinforcement steel are:

| | cement (kg/m³) | reinforcement steel (kg/m³) |
|---|---|---|
| conventional concrete | 250 | 50 |
| shotcrete | 300 | 70 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- conventional concrete: 234.00/$m^3$
- shotcrete: 378.00/$m^3$

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

## SURGE TANK (ACCOUNT .12.19.33)

The use of surge tanks should fulfill the criteria set out in item 5.5.2.

### Basic Data

The main **information required for dimensioning** this item can be obtained from the overall layout and item 4.4, Formulation of Cascade Options and 5.3 Energy Studies, as follows:

- mean velocity of discharge in the headrace tunnel, $v_{ad}$, in m/s, from item 5.7.6;
- length of the headrace tunnel, $L_{ad}$, in m, from item 5.7.6;
- diameter of the headrace tunnel, $D_{ad}$, in m, from item 5.7.6;
- head loss in the headrace tunnel, $h_a$, in m, from item 5.7.6;
- maximum normal water level in the reservoir, $NA_{max}$, from item 4.4, in m;
- minimum water level in the reservoir, $NA_{min}$, from item 5.3, in m;
- elevation of the center line of the turbine distributor, $El_d$, from item 5.7.2, in m;
- elevation of the intake sill, $El_{sol}$, in m;
- elevation of the land in the surge tank area, $El_{te}$, in m;
- mean thickness of the layer of soil in the surge tank area, $e_{te}$ in m.

### Considerations

This text relates to surge tanks with a typical cross-section, as shown in Fig. 5.7.6.06 or 5.7.6.07.

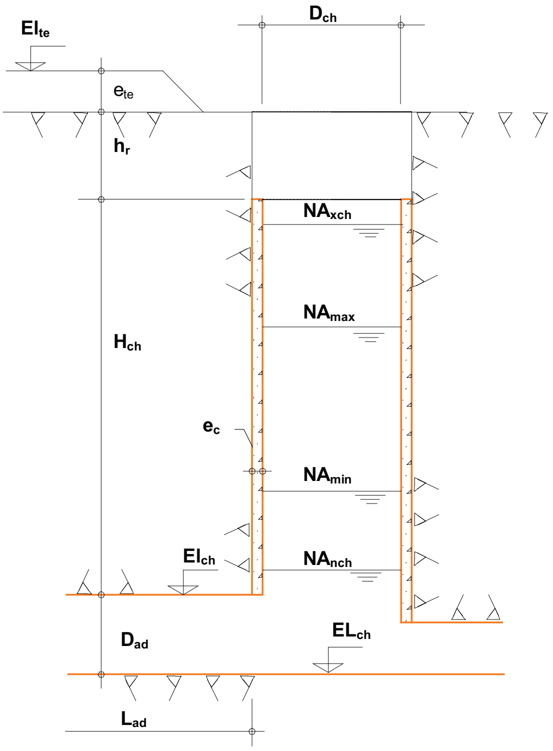



Fig. 5.7.6.06 – Typical cross-section of a surge tank.

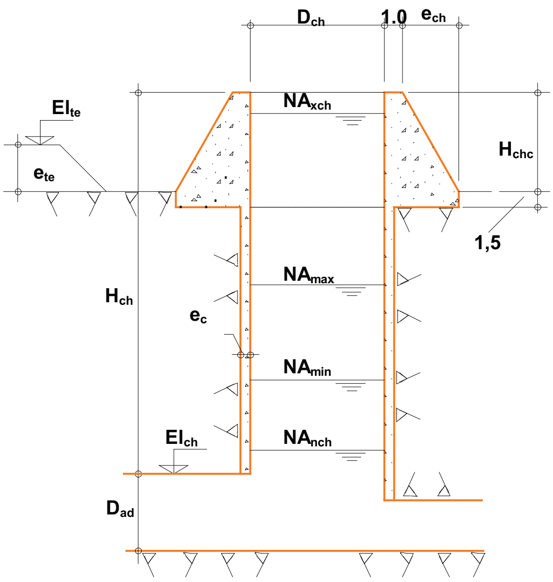



Fig. 5.7.6.07 – Typical cross-section of a surge tank with maximum oscillation exceeding the top of the bedrock.

## Surge Tank Dimensions

The **cross-sectional area of the surge tank**, $A_{ch}$ ($m^2$), is given by:

$$A_{ch} = 1.25 \times \frac{v_{ad}^2}{2 \times g} \times \frac{L_{ad} \times A_{ad}}{(H_d - h_e - h_a) \times (h_e + h_a)}$$

where:

$$A_{ad} = 0.8927 \times D_{ad}^2 \qquad H_d = NA_{min} - El_d$$

$$h_e = 0.20 \times \frac{v_{ad}^2}{2 \times g}$$

where:

| | |
|---|---|
| $v_{ad}$ | mean velocity of flow in the headrace tunnel, in m/s; |
| g | 9.81 $m/s^2$, acceleration due to gravity; |
| $L_{ad}$ | length of the headrace tunnel, in m; |
| $A_{ad}$ | cross-sectional area of the headrace tunnel, in $m^2$; |
| $H_d$ | minimum static head, in m; |
| $h_e$ | head loss at the intake inlet, in m; |
| $h_a$ | head loss in the headrace tunnel, in m; |
| $D_{ad}$ | diameter of the headrace tunnel, in m; |
| $NA_{min}$ | minimum water level in the reservoir, in m; and |
| $El_d$ | elevation of the center line of the turbine distributor, in m. |

The **internal diameter of the surge tank**, $D_{ch}$ (m), is given by:

$$D_{ch} = \sqrt{\frac{4 \times A_{ch}}{\pi}}$$

where:

| | |
|---|---|
| $A_{ch}$ | cross-sectional area of the surge tank, in $m^2$. |

The **maximum oscillation in the surge tank**, $Y_{max}$ (m), is given by:

$$Y_{max} = v_{ad} \times \sqrt{\frac{L_{ad} \times A_{ad}}{g \times A_{ch}}}$$

where:

| | |
|---|---|
| $v_{ad}$ | mean velocity of flow in the headrace tunnel, in m/s; |
| $L_{ad}$ | length of the headrace tunnel, in m; |
| $A_{ad}$ | cross-sectional area of the headrace tunnel, in $m^2$; |
| g | 9.81 $m/s^2$, acceleration due to gravity; and |
| $A_{ch}$ | cross-sectional area of the surge tank, in $m^2$. |

The **maximum and minimum water levels in the surge tank**, $NA_{xch}$ and $NA_{nch}$, are given by:

$$NA_{xch} = NA_{max} - \frac{2}{3} \times (h_e + h_a) + Y_{max}$$

$$NA_{nch} = NA_{min} + 2 \times (h_e + h_a) - Y_{max}$$

where:

| | |
|---|---|
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $h_e$ | head loss at the intake inlet, in m; |
| $h_a$ | head loss in the headrace tunnel, in m; |
| $Y_{max}$ | maximum oscillation in the surge tank, in m; and |
| $NA_{min}$ | minimum water level in the reservoir, in m. |

The **elevation of the bottom of the surge tank**, $El_{ch}$, is given by:

$$El_{ch} = El_{sol} - 0.005 \times L_{ad} + D_{ad} \quad \leq \; NA_{nch} - 1{,}0$$

where:

| | |
|---|---|
| $El_{sol}$ | elevation of the intake sill, in m; |
| $L_{ad}$ | length of the headrace tunnel, in m; |
| $D_{ad}$ | diameter of the headrace tunnel, in m; and |
| $NA_{nch}$ | minimum water level in the surge tank, in m. |

The **height of the surge tank**, $H_{ch}$ (m), is given by:

$$H_{ch} = NA_{xch} + 1.0 - El_{ch}$$

where:

| | |
|---|---|
| $NA_{xch}$ | maximum water level in the surge tank, in m; |
| $El_{ch}$ | elevation of the bottom of the surge tank, in m; and |
| 1.0 | freeboard, in m. |

When the maximum water level in the surge tank exceeds the top of the bedrock (Fig. 5.7.6.07), the **height of the surge tank above the top of the bedrock**, $H_{chc}$ (m), is given by:

For $NA_{xch} + 1.0 > El_{te} - e_{te}$ : $H_{chc} = NA_{xch} + 1.0 - (El_{te} - e_{te})$

where:

| | |
|---|---|
| $NA_{xch}$ | maximum water level in the surge tank; |
| $El_{te}$ | elevation of the land along the surge tank axis, in m; and |
| $e_{te}$ | mean thickness of the layer of soil in the surge tank area, in m. |

In the same situation, the **extra thickness of concrete at the base of the buttress wall**, $e_{ch}$ (m), is given by:

$$e_{ch} = \frac{D_{ch}}{2}\left(\sqrt{\frac{\sigma_c + p_s}{\sigma_c - p_s}} - 1\right)$$

where:

$$p_s = \frac{NA_{xch} - (El_{te} - e_{te})}{10}$$

where:

| | |
|---|---|
| $D_{ch}$ | internal diameter of the surge tank, in m; |
| $\sigma_c$ | 10 kg/cm² – tensile stress capacity of concrete; |
| $p_s$ | working pressure, in kg/cm²; |
| $NA_{xch}$ | maximum water level in the surge tank, in m; |
| $El_{te}$ | elevation of the land along the surge tank axis, in m; and |
| $e_{te}$ | mean thickness of the layer of soil in the surge tank area, in m. |

When the maximum water level in the surge tank does not exceed the top of the bedrock (Fig. 5.8.6.06), the **mean depth of rock above the top of the surge tank**, $h_r$ (m), is given by:

For $NA_{xch} + 1.0 \leq El_{te} - e_{te}$ : $h_r = El_{te} - e_{te} - (NA_{xch} + 1.0)$

where:

| | |
|---|---|
| $El_{te}$ | elevation of the land in the surge tank area; |
| $e_{te}$ | thickness of the layer of soil in the surge tank area, in m; and |
| $NA_{xch}$ | maximum water level in the surge tank. |

In either case, the **thickness of the concrete lining**, $e_c$ (m), is given by:

$$e_c = 0.00274 \times D_{ch}^2 + 0.018 \times D_{ch} + 0.10$$

where:

| | |
|---|---|
| $D_{ch}$ | internal diameter of the surge tank, in m. |

## Common Excavation (account .12.19.33.12.10)

The **volume of common excavation**, $V_{tch}$ (m³), is given by one of the expressions below:

$$\text{if } H_{chc} > 0: \ V_{tch} = \frac{\pi}{4} \times \left[D_{ch} + 2 \times (1 + e_{ch} + e_{te})\right]^2 \times e_{te}$$

$$\text{if } H_{chc} \leq 0: \ V_{tch} = \frac{\pi}{4} \times \left[D_{ch} + 2 \times (e_c + e_{te})\right]^2 \times e_{te}$$

where:

| | |
|---|---|
| $D_{ch}$ | internal diameter of the surge tank, in m; |
| $e_{ch}$ | extra thickness of concrete at the base of the buttress wall, in m; |
| $e_{te}$ | thickness of the layer of soil in the surge tank area, in m; and |
| $e_c$ | thickness of the concrete lining, in m. |

The unit price of **common excavation** is R$ 7.60/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 surge tank. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Underground Excavation of Rock (account .12.19.33.12.12)

The **volume of rock excavated underground**, $V_{sch}$ ($m^3$), is given by one of the expressions below:

if $H_{chc} > 0$:

$$V_{sch} = \frac{\pi}{4} \times (D_{ch} + 2 \times e_c)^2 \times (H_{ch} - H_{chc}) + \pi \times (D_{ch} + e_c + e_{ch} + 1) \times (e_{ch} + 1 - e_c) \times 1.5$$

if $H_{chc} \leq 0$:

$$V_{sch} = \frac{\pi}{4} \times (D_{ch} + 2 \times e_c)^2 \times (h_r + H_{ch})$$

where:

| | |
|---|---|
| $D_{ch}$ | internal diameter of the surge tank, in m; |
| $e_c$ | thickness of the concrete lining, in m; |
| $H_{ch}$ | height of the surge tank, in m; |
| $H_{chc}$ | height of the surge tank above the top of the bedrock, in m; |
| $e_{ch}$ | extra thickness of concrete at the base of the buttress wall, in m; and |
| $h_r$ | mean depth of rock above the top of the surge tank, in m. |

The unit price of **underground excavation**, $P_{us}$ (R\$/$m^3$), (from December 2006 database), including excavating, loading, transportation up to 1.5 km and unloading, can be obtained from the expression below (or from Graph B33, annex B, as a function of the excavated area) and can be used for projects in the south, southeast, central west and northeast regions of Brazil:

valid for $4 \leq A_{se} \leq 300$: $P_{us} = 474.08 \times A_{se}^{-0.3987}$

where: $A_{se} = \frac{\pi}{4} \times (D_{ch} + 2 \times e_c)^2$

where:

| | |
|---|---|
| $A_{se}$ | excavation cross-sectional area, in $m^2$; |
| $D_{ch}$ | internal diameter of the surge tank, in m; and |
| $e_c$ | thickness of the concrete lining, in m. |

A careful appraisal should be made of the different situations where the tunnels form a representative part of the cost estimate, paying special attention to the geological conditions in the region and particularly long tunnels.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.19.33.13)

The **area of foundation to be cleaned**, $A_{lf}$ ($m^2$), is given by one of the expressions below:

if $H_{chc} > 0$:

$$A_{lf} = \pi \times (D_{ch} + 2 \times e_c) \times (H_{ch} - H_{chc}) + \pi \times (D_{ch} + e_c + e_{ch} + 1) \times (e_{ch} + 1 - e_c)$$

if $H_{chc} \leq 0$:

$$A_{lf} = \pi \times (D_{ch} + 2 \times e_c) \times H_{ch}$$

where:

| | |
|---|---|
| $D_{ch}$ | internal diameter of the surge tank, in m; |
| $e_c$ | thickness of the concrete lining, in m; |
| $H_{ch}$ | height of the surge tank, in m; |
| $H_{chc}$ | height of the surge tank above the top of the bedrock, in m; and |
| $e_{ch}$ | extra thickness of concrete at the base of the buttress wall, in m. |

The **length of contact grouting and consolidation grouting holes**, $L_{tf}$ (m), is given by one of the expressions below:

if $H_{chc} > 0$: $L_{tf} = 2 \times \pi \times (D_{ch} + 2 \times e_c) \times (H_{ch} - H_{chc})$

if $H_{chc} \leq 0$: $L_{tf} = 2 \times \pi \times (D_{ch} + 2 \times e_c) \times H_{ch}$

where:

| | |
|---|---|
| $D_{ch}$ | internal diameter of the surge tank, in m; |
| $e_c$ | thickness of the concrete lining, in m; |
| $H_{ch}$ | height of the surge tank, in m; and |
| $H_{chc}$ | height of the surge tank above the top of the bedrock, in m. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m$^2$
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account .12.19.33.14)

The **volumes of structural concrete**, $V_{cch}$ (m$^3$), and **shotcrete**, $V_{cp}$ (m$^3$), are given by one of the expressions below:

if $H_{chc} > 0$:

$$V_{cch} = \left(\frac{A_1 + A_2}{2} - A_{ch}\right) \times H_{chc} + (A_0 - A_{ch}) \times H_{ch} + (A_2 - A_0) \times 1.5$$

where:

$$A_0 = \frac{\pi}{4} \times (D_{ch} + 2 \times e_c)^2$$

$$A_1 = \frac{\pi}{4} \times (D_{ch} + 2.0)^2$$

$$A_2 = \frac{\pi}{4} \times (D_{ch} + 2.0 + 2 \times e_{ch})^2$$

$$V_{cp} = 0$$

if $H_{chc} \leq 0$:

$$V_{cch} = \pi \times (D_{ch} + e_c) \times e_c \times H_{ch}$$

$$V_{cp} = \pi \times (D_{ch} + e_c) \times e_c \times h_r$$

The recommended thickness of shotcrete lining ($e_c$) is 0.10 m.

where:

| | |
|---|---|
| $A_i$ | secondary areas, in $m^2$; |
| $A_{ch}$ | cross-sectional area of the surge tank, in $m^2$; |
| $D_{ch}$ | internal diameter of the surge tank, in m; |
| $H_{chc}$ | height of the surge tank above the top of the bedrock, in m; |
| $e_{ch}$ | extra thickness of concrete at the base of the buttress wall, in m; |
| $e_c$ | thickness of the concrete lining, in m; |
| $H_{ch}$ | height of the surge tank, in m; and |
| $h_r$ | mean depth of rock above the top of the surge tank, in m. |

The amounts of cement and reinforcement steel are:

| | cement (kg/$m^3$) | reinforcement steel (kg/$m^3$) |
|---|---|---|
| conventional concrete | 250 | 50 |
| shotcrete | 300 | 70 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- conventional concrete: 214.00/$m^3$
- shotcrete: 378.00/$m^3$

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

## SURFACE PRESSURE PENSTOCKS (ACCOUNT .12.19.34)

### Basic Data

The main **information required for dimensioning** this item can be obtained from the overall layout of the project and the energy studies, as follows:

- horizontal projection of each section, $L_{h1}$, $L_{h2}$ and $L_{h}$, in m;
- slope of each section, $\alpha_1$, $\alpha_2$ and $\alpha_3$, in degrees;
- length of the headrace tunnel, $L_{ad}$, in m, when applicable;
- capacity of one generating unit, $P_1$ in MW, from item 5.7.2.;
- coefficient k, product of the yield of the generating unit by acceleration due to gravity, from item 2.1.;
- number of generating units, $N_g$, from item 5.7.2.;
- number of generating units per pressure penstock, $N_f$;
- maximum normal water level in the reservoir, $NA_{max}$, from item 4.4.;
- minimum water level in the reservoir, $NA_{min}$, from item 5.3.;
- lowest elevation of the land in the intake area, $El_{ten}$;
- mean thickness of the layer of soil in the pressure penstock area, $e_{te}$ in m;
- normal water level in the tailrace canal, $NA_{fu}$, from item 4.4.;
- elevation of the center line of the turbine distributor, $El_d$, from item 5.7.2.;
- maximum net head, $H_1$, in m, from item 5.3.;
- head loss in the headrace canal, $h_c$, in m, from item 5.7.6., when applicable;
- head loss in the headrace tunnel, $h_a$, in m, from item 5.7.6., when applicable; and
- diameter of the inlet to the spiral casing, A, in m, from item 5.7.2., when applicable.

The main **information required for quantification** purposes is as follows:

- maximum water level in the tailrace canal, $NA_{xfu}$, from item 5.1.2.;
- maximum water level in the surge tank, $NA_{xch}$, from item 5.7.6., when applicable;
- length of the section through a tunnel, $L_t$, in m, when applicable;
- volume of common excavation, $V_{ttf}$, in $m^3$; and
- volume of surface rock excavation, $V_{rtf}$, in $m^3$.

### Considerations and Recommendations

A **section** is understood as being the stretch between two deflections in the vertical plane, counting from the outlet of the intake, point 0, to the inlet of the powerhouse, point 4, as shown in Figure 5.7.6.08.

For methodological reasons, the **length of section 3**, $L_3$, is obtained from the other data.

In this text, the term **bifurcation** is also used for sections that split into three and for separators.

The **horizontal projection** of sections 1 and 4, $L_{h1}$ and $L_{h4}$, must be long enough to match the curve.

$$L_{h1} \geq 4 \times D_b \times tg\frac{\alpha_2 - \alpha_1}{2} \qquad L_{h4} \geq 4 \times D_b \times tg\frac{\alpha_3}{2}$$

where:

| | |
|---|---|
| $D_b$ | internal diameter of the pressure penstock defined below, in m; and |
| $\alpha_i$ | slope of section i, in degrees. |

For **penstocks with just three sections**, the following must be true:

$\alpha_3 = \alpha_2$ and $L_{h2} = 1.0$ m

where:

| | |
|---|---|
| $L_{h2}$ | horizontal projection of section 2, in m; and |
| $\alpha_i$ | slope of section i, in degrees. |

### Internal diameter and mean velocity of discharge

The **number of pressure penstocks**, $N_t$, is given by:

$$N_t = \frac{N_g}{N_f}$$

where:

| | |
|---|---|
| $N_g$ | number of generating units; and |
| $N_f$ | number of generating units per pressure penstock. |

The **internal diameter of the pressure penstock**, $D_b$ (m), can be determined by:

$$D_b = 14.2 \times \frac{(N_f \times P_1)^{0.43}}{H_{b1}^{0.65}}$$

where: $H_{b1} = NA_{max} - NA_{fu}$

where:

| | |
|---|---|
| $N_f$ | number of generating units per pressure penstock; |
| $P_1$ | capacity of one generating unit, in MW; |
| $H_{b1}$ | maximum gross head, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir; and |
| $NA_{fu}$ | normal water level in the tailrace canal. |

The **maximum turbine flow of each turbine**, $Q_1$ ($m^3/s$), is given by:

$$Q_1 = \frac{10^6 \times P_1}{k \times H_1}$$

where:

$$k = \rho \times g \times \eta_{t1} \times \eta_{g1} \qquad P_2 = \frac{P_1}{f_p}$$

for Francis turbines:

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

for Pelton and Kaplan turbines:

$$\eta_{t1} = 0.96 \qquad \eta_{g1} = 0.98$$

where:

| | |
|---|---|
| $P_1$ | capacity of one generating unit, in MW; |
| k | coefficient; |
| $H_1$ | maximum net head, in m; |
| $\rho$ | 1000 $kg/m^3$ – specific mass of water; |
| $\eta_{t1}$ | turbine output for maximum net head; |
| $\eta_{g1}$ | generator output for maximum net head; |

| | |
|---|---|
| g | 9.81 m/s$^2$ – acceleration due to gravity; |
| $P_2$ | capacity of one generator, in MVA; and |
| $f_p$ | capacity factor. |

The **maximum flow through each pressure penstock**, $Q_{1f}$ (m$^3$/s), is given by:

$$Q_{1f} = N_f \times Q_1$$

where:

| | |
|---|---|
| $N_f$ | number of generating units per pressure penstock; and |
| $Q_1$ | maximum turbine flow of each turbine, in m$^3$/s. |

The **mean velocity of discharge** through the pressure penstock, $v_b$ (m/s), is given by:

$$v_b = \frac{4 \times Q_{1f}}{\pi \times D_b^2} \leq 7 m/s$$

where:

| | |
|---|---|
| $Q_{1f}$ | maximum flow through each pressure penstock, in m$^3$/s; and |
| $D_b$ | internal diameter of the pressure penstock, in m. |

If this restriction is not fulfilled, the velocity limit should be adopted and the diameter should be recalculated using:

$$D_b = \sqrt{\frac{4}{\pi} \times \frac{Q_{1f}}{7}}$$

where:

| | |
|---|---|
| $Q_{1f}$ | maximum flow through each pressure penstock, in m$^3$/s. |

The **internal diameter of the pressure penstock after a bifurcation** or **the lateral diameter after the separators**, $D_{1b}$ (m), can be determined by:

$$D_{1b} = \frac{D_b}{N_f^{3/8}}$$

where:

| | |
|---|---|
| $D_b$ | internal diameter of the pressure penstock, in m; and |
| $N_f$ | number of generating units per pressure penstock. |

Profile of the pressure penstock

The **elevation of points 0 to 4**, $El_0$ to $El_4$, is given by:

$$El_0 = El_{sol} - i \times L_{ad} + \frac{D_b}{2}$$

$$El_1 = El_0 - L_{h1} \times tg\alpha_1$$

$$El_2 = El_1 - L_{h2} \times tg\alpha_2$$

$$El_3 = E_4 = E_d$$

where:

$$El_{sol} = 0.5 \times int\left(\frac{NA_{min} - h_c - h_s - H_{cp}}{0.5}\right)$$

$$El_{sol} \leq 0.5 \times \text{int}\left(\frac{El_{ten} - e_{te}}{0.5}\right) + 1.0$$

$$h_s = 0.8 \times v_{cp} \times \sqrt{H_{cp}}$$

$$v_{cp} = \frac{Q_{1f}}{B_{cp} \times H_{cp}}$$

for projects with no headrace tunnel:

$B_{cp} = D_b$ $H_{cp} = D_b$

for projects with a headrace tunnel:

$B_{cp} = D_{ad}$ $H_{cp} = D_{ad}$

where:

| | |
|---|---|
| $El_{sol}$ | elevation of the intake sill, in m; |
| $L_{ad}$ | length of the headrace tunnel, in m; |
| $D_b$ | internal diameter of the pressure penstock, in m; |
| $L_{hi}$ | horizontal projection of section i, in m; |
| $\alpha_i$ | slope of section i, in degrees; |
| i | slope of the headrace tunnel, in degrees; |
| $El_d$ | elevation of the center line of the turbine distributor, in m; |
| $NA_{min}$ | normal minimum water level in the reservoir, in m; |
| $h_c$ | head loss in the headrace canal, in m, when applicable; |
| $h_s$ | minimum submergence of the intake (Gordon, 1970), in m; |
| $H_{cp}$ | height of the intake gate, in m; |
| $El_{ten}$ | lowest elevation of the land in the intake area, in m; |
| $e_{te}$ | mean thickness of the layer of soil in the pressure penstock area, in m; |
| $v_{cp}$ | velocity of discharge at the intake gate, in m/s; |
| $Q_{1f}$ | maximum flow through each pressure penstock, in $m^3/s$; |
| $B_{cp}$ | width of the intake gate, in m; |
| $Q_t$ | total maximum turbine flow, in $m^3/s$; and |
| $D_{ad}$ | internal diameter of the headrace tunnel, in m. |

The **horizontal projection of section 3**, $L_{h3}$ (m), is given by:

$$L_{h3} = \frac{El_2 - El_3}{tg\alpha_3}$$

where:

| | |
|---|---|
| $El_i$ | elevation of point i, in m; and |
| $\alpha_3$ | slope of section 3, in degrees. |

The length of the penstock is determined from its profile.

The **length of each section**, $L_i$ (m), of penstock is given by:

$$L_i = \frac{L_h}{\cos \alpha_i}, \; i = 1, 2, 3$$

where:

| | |
|---|---|
| $L_{hi}$ | horizontal projection of section i, in m; and |
| $\alpha_i$ | slope of section i, in degrees. |

The **total length** of the penstock, $L_b$ (m), is given by:

$L_b = L_1 + L_2 + L_3 + L_{h4}$

where:

| | |
|---|---|
| $L_{h4}$ | horizontal projection of section 4, in m; and |
| $L_i$ | length of section i, in m. |

The **horizontal projection** of the penstock, $L_{hb}$ (m), is given by:

$L_{hb} = L_{h1} + L_{h2} + L_{h3} + L_{h4}$

where:

| | |
|---|---|
| $L_{hi}$ | the horizontal projection of section i, in m. |

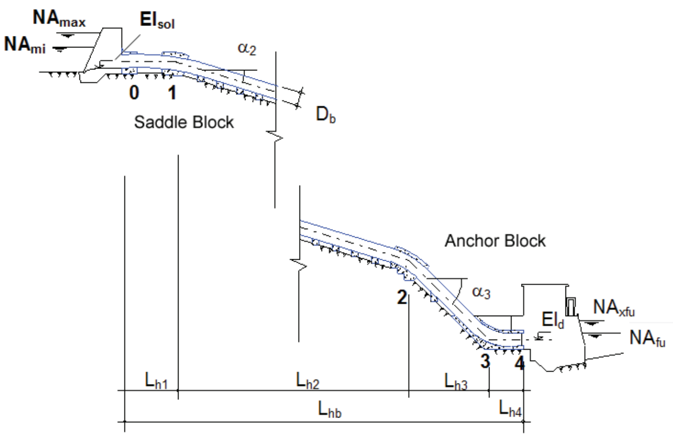



Fig. 5.7.6.08 – Schematic profile of a pressure penstock with four sections.

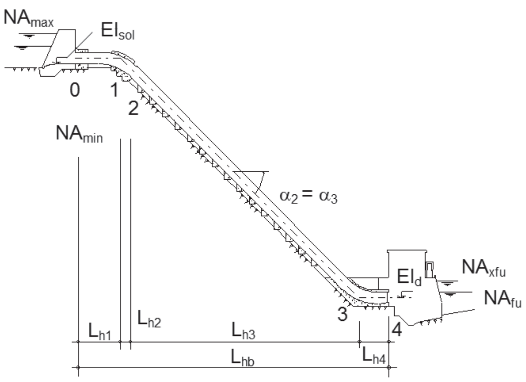



Fig. 5.7.6.09 – Schematic profile of a pressure penstock with three sections.

## Surge Pressure

**Maximum surge pressure** due to water hammer, $h_{sx}$ (m), is given by:

$$h_{sx} = \frac{2 \times L_b \times v_b}{g \times T_c} \leq 0.30 \times H_d$$

where: $H_d = NA_{max} - El_d$

| $T_c$ | For |
|---|---|
| 6 s | short penstocks ($L_b \leq 3 \times H_{b1}$) |
| 10 s | long penstocks ($L_b > 3 \times H_{b1}$) |

where:

| | |
|---|---|
| $L_b$ | total length of the penstock, in m; |
| $v_b$ | mean velocity of flow in the penstock, in m/s; |
| g | 9.81 m/s$^2$ – acceleration due to gravity; |
| $T_c$ | time taken to close the valve or the turbine distributor, in s; |
| $H_d$ | static head, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir; and |
| $El_d$ | elevation of the center line of the turbine distributor, in m. |

When the maximum surge pressure exceeds the limit, the diameter must be redimensioned using:

$$D_b = \sqrt{\frac{4}{\pi} \times \frac{Q_{1f}}{v_b}}$$

with:

$$v_b = \frac{0.30 \times H_{b1} \times g \times T_c}{2 \times L_b}$$

where:

| | |
|---|---|
| $Q_{1f}$ | maximum flow through each pressure penstock, in m$^3$/s; |
| $v_b$ | mean velocity of flow through the pressure penstock, in m/s; |
| $H_{b1}$ | maximum gross head, in m. |
| g | 9.81 m/s$^2$ – acceleration due to gravity; |
| $T_c$ | time taken to close the valve or distributor, in s; and |
| $L_b$ | total length of the pressure penstock, in m. |

The **energy gradient** for maximum surge pressure, $i_s$, is given by:

$$i_s = \frac{h_{sx}}{L_{hb}}$$

where:

| | |
|---|---|
| $h_{sx}$ | maximum surge pressure due to water hammer, in m; and |
| $L_{hb}$ | horizontal projection of the pressure penstock, in m. |

The whole penstock should be below the minimum energy gradient. This can be checked at critical points 1 and 2:

$$El_1 \le NA_{min} - L_{h1} \times i_s - \frac{D_b}{2}$$

$$El_2 \le NA_{min} - (L_{h1} + L_{h2}) \times i_s - \frac{D_b}{2}$$

where:

| | |
|---|---|
| $El_i$ | elevation of points 1 and 2, in m; |
| $NA_{min}$ | minimum water level in the reservoir, in m; |
| $L_{hi}$ | horizontal projection of section i, in m; |
| $i_s$ | energy gradient for maximum surge pressure, in m/m; and |
| $D_b$ | internal diameter of the pressure penstock, in m. |

If any one of these criteria is not fulfilled, the elevation of the critical point should be lowered and the slope of the adjacent sections should be adapted.

## Head Loss

The **total head loss**, $h_p$ (m), from the intake to the pressure penstock, is given by:

$$h_p = h_e + h_a + h_o + h_r + h_b + h_v + h_f$$

where:

$$h_o = \sum k_{oi} \times \frac{v_b^2}{2 \times g}$$

$$h_r = 0.10 \times \frac{(v_a - v_b)^2}{2 \times g}$$

$$h_b = 0.10 \times \frac{v_b^2}{2 \times g}$$

$$h_f = 6.35 \times L_b \times \frac{n^2 \times v_b^2}{D_b^{4/3}}$$

$$v_a = \frac{4}{\pi} \times \frac{Q_1}{A^2}$$

For $(r_i/D_b)<5$

$$k_{oi} = 0.2147 \times \left(\frac{r_i}{D_b}\right)^{-0.5718} \times \left(0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i\right)$$

For $(r_i/ D_b)\geq 5$

$$k_{oi} = \left(0.08 - 0.002 \times \left(\frac{r_i}{D_b} - 5\right)\right) \times \left(0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i\right)$$

$$\delta_i = abs(\alpha_{i+1} - \alpha_i),\ i = 1, 2, 3$$

for projects with no headrace tunnel: $h_e = 0.20 \times \frac{v_b^2}{2 \times g}$

for projects with a headrace tunnel: $h_e = 0.20 \times \frac{v_{ad}^2}{2 \times g}$

with a butterfly valve at the beginning of the pressure penstock: $h_v = 0.20 \times \frac{v_b^2}{2 \times g}$

with a butterfly valve at the end of the pressure penstock: $h_v = 0.20 \times \frac{v_a^2}{2 \times g}$

with a spherical valve: $h_v = 0.03 \times \frac{v_a^2}{2 \times g}$

where:

| | |
|---|---|
| $h_e$ | head loss at the intake, in m; |
| $h_a$ | head loss in the headrace tunnel, in m, when applicable; |
| $h_o$ | head loss at the bends, in m; |

| | |
|---|---|
| $h_r$ | head loss where the diameter is reduced, in m; |
| $h_b$ | head loss at the bifurcation, in m, when applicable; |
| $h_v$ | head loss at the valve, in m, when applicable; |
| $h_f$ | continuous head loss, in m; |
| $k_o$ | coefficient for head loss at bends; |
| $v_b$ | mean velocity of flow in the penstock, in m/s; |
| g | 9.81 $m/s^2$ – acceleration due to gravity; |
| $L_b$ | total length of the penstock, in m; |
| n | 0.010 – Manning's coefficient for steel; |
| $D_b$ | internal diameter of the pressure penstock, in m; |
| $r_i$ | radius of curvature of the section, in m; |
| $\theta$ | deflection of the tunnel axis, in radians. |
| $Q_1$ | maximum turbine flow of each turbine, in $m^3/s$; |
| $v_a$ | mean velocity of flow at the inlet to the spiral casing, in m/s; |
| A | diameter of the inlet to the spiral casing, in m; |
| $\delta_i$ | angle of vertical deflection at points 0 to 3 of the penstock, in degrees; |
| $\alpha_i$ | slope of section i, in degrees; |
| $v_{ad}$ | mean velocity of flow in the headrace tunnel, in m/s; and |
| abs(x) | function that returns the absolute value of x. |

### Common Excavation (account .12.19.34.12.10)

The volume of **common excavation**, $V_{ttf}$ ($m^3$), should be determined from the project design.

The unit price of **common excavation** is R$ 7.60/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the penstock. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation (account .12.19.34.12.11)

The **volume of excavation in rock**, $V_{rtf}$ ($m^3$), should be determined from the project design.

The unit price of **excavation in rock** is R$ 21.00/$m^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the penstock. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Underground Excavation of Rock (account .12.19.34.12.12)

The **volume of rock excavated underground** for pressure penstocks, $V_{stf}$ ($m^3$), is given by:

$$V_{stf} = V_{sb} + V_{st}$$

where:

$$V_{sb} = N_t \times 4 \times (D_b + 2 \times e_c)^3$$

$$V_{st} = N_t \times \frac{\pi}{4} \times [(D_b + 2 \times e_c)^2 \times L_t]$$

where:

| | |
|---|---|
| $V_{sb}$ | volume of rock excavated underground for the butterfly valve housing, in $m^3$, when installed in a separate cavity; |
| $V_{st}$ | volume of rock excavated underground for tunnels, in $m^3$, when applicable; |
| $N_t$ | number of pressure penstocks; |
| $D_b$ | internal diameter of the pressure penstock, in m; |
| $e_c$ | thickness of the concrete lining of the section through a tunnel, in m, as defined below; and |
| $L_t$ | length of the section through a tunnel, in m. |

The unit price of **underground excavation**, $P_{us}$ (R$/$m^3$), (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil, can be obtained from the expression below (or from Graph B33, annex B, as a function of the excavated area). This is the price per cubic meter measured using the excavation line of the project design and includes excavating, loading, transportation up to 1.5 km and unloading:

valid for $4 \leq A_{se} \leq 300$: $P_{us} = 474.08 \times A_{se}^{-0.3987}$

where: $A_{se} = \frac{\pi}{4} \times (D_b + 2 \times e_c)^2$

where:

| | |
|---|---|
| $A_{se}$ | area of the excavated area, in $m^2$; |
| $D_b$ | internal diameter of the pressure penstock, in m; and |
| $e_c$ | thickness of the concrete lining of the section through a tunnel, in m, as defined below. |

A careful appraisal should be made of the different situations where the tunnels form a representative part of the cost estimate, paying special attention to the geological conditions in the region and particularly long tunnels.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.19.34.13)

The **area of foundation to be cleaned**, $A_{lf}$ ($m^2$), is given by:

$$A_{lf} = A_{ls} + A_{lb} + A_{ln} + A_{la} + A_{lt}$$

where:

$$A_{lb} = N_t \times 4.2 \times D_b \times (D_b + 1) \times sen(\alpha_3 - \alpha_2)$$

$$A_{ln} = N_t \times \left[ (4.5 \times D_b + 1.5) \times \tan\frac{\alpha_3}{2} + d_2 \right] \times (D_b + 2)$$

$$A_{la} = N_t \times N_a \times D_b^2$$

$$A_{lt} = N_t \times \pi \times (D_b + 2 \times e_c) \times L_t$$

$$d_2 = \frac{NA_{xcf} - El.d + \frac{D_b}{2} + 2.5}{\tan\alpha_3}$$

for $L_{h1} \leq 1{,}7 \times D_b$ e $L_t = 0$: $A_{ls} = N_t \times \left(2.1 \times D_b^2 + 3 \times D_b + 0.9\right)$

for $L_{h1} > 1{,}7 \times D_b$ e $L_t = 0$: $A_{ls} = N_t \times 4.2 \times D_b \times (D_b + 1) \times \mathrm{sen}(\alpha_2 - \alpha_1)$

where:

| | |
|---|---|
| $A_{ls}$ | the area of foundation to be cleaned for the upper anchor block, in m²; |
| $A_{lb}$ | the area of foundation to be cleaned for the middle anchor block, in m²; |
| $A_{ln}$ | the area of foundation to be cleaned for the lower anchor block, in m²; |
| $A_{la}$ | the area of foundation to be cleaned for the saddle block, in m²; |
| $A_{lt}$ | the area of the tunnel foundation to be cleaned, in m², when applicable; |
| $N_t$ | number of pressure penstocks; |
| $D_b$ | internal diameter of the pressure penstock, in m; |
| $\alpha_i$ | slope of section i, in degrees; |
| $N_a$ | number of saddle blocks per pressure penstock, as defined below; |
| $e_c$ | thickness of the concrete lining of the section through a tunnel, in m, as defined below; |
| $L_t$ | length of the section through a tunnel, in m; |
| $L_{h1}$ | horizontal projection of section 1, in m; and |
| $d_2$ | effective distance, in m. |

The **length of contact grouting and consolidation grouting holes**, $L_{tf}$(m) , is given by:

$$L_{tf} = 1.0 \times A_{lt}$$

| | |
|---|---|
| $A_{lt}$ | the area of the tunnel rock to be cleaned, in m², when applicable. |

The **total length of the rock anchors** for the anchor blocks, $L_{tfp}$ (m), when applicable, is given by:

$$L_{tfp} = 1.0 \times (A_{ls} + A_{lb} + A_{ln})$$

where:

| | |
|---|---|
| $A_{ls}$ | the area of foundation to be cleaned for the upper anchor block, in m²; |
| $A_{lb}$ | the area of foundation to be cleaned for the middle anchor block, in m²; and |
| $A_{ln}$ | the area of foundation to be cleaned for the lower anchor block, in m². |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m²
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account .12.19.34.14)

The **volume of concrete** for the pressure penstocks, $V_{ctf}$ ($m^3$), is given by:

$$V_{ctf} = V_{cs} + V_{cb} + V_{cf} + V_{ca} + V_{ct} + V_{cc} + V_{cv} + V_{ce}$$

where:

$$V_{cb} = N_t \times (0.0072 \times D_b^3 + 0.105 \times D_b^2 + 0.08 \times D_b) \times (\alpha_3 - \alpha_2)$$

$$V_{cf} = N_t \times (0.228 \times D_b^2 + 4.77 \times D_b + 5.3) \times (L_{h4} + d_3)$$

$$V_{ca} = N_t \times N_a \times 0.5 \times D_b^3$$

$$V_{ct} = N_t \times \pi \times (D_b + e_c) \times e_c \times L_{ct}$$

$$V_{cc} = \pi \times (D_b + 3 \times e_c) \times e_c \times 5 \times D_b$$

$$V_{cv} = N_t \times 12 \times D_b^3$$

$$V_{ce} = N_g \times 12 \times A^3$$

$$N_a = \text{int}\left[\frac{d_4}{1.6 \times D_b} + 0.5\right] - 2$$

$$e_c = 0.091 \times D_b^{0.62}$$

$$d_3 = \frac{NA_{xfu} + 1.0 - EI_d}{\text{sen}\alpha_3}$$

$$d_4 = L_b - 4 \times D_b \times \left(\text{tg}\frac{\alpha_2 - \alpha_1}{2} + \text{tg}\frac{\alpha_3 - \alpha_2}{2}\right) - 2 \times D_b - d_3$$

for $L_{h1} \leq 1{,}7 \times D_b \; e L_t = 0$:

$$V_{cs} = N_t \times [0.565 \times D_b^3 + 10.50 \times D_b^2 + 8.4 \times D_b + (0.029 \times D_b^3 + 0{,}42 \times D_b^2 + 0.34 \times D_b) \times \alpha]$$

$$\alpha = \alpha_2 - \alpha_1 - 25 \geq 0$$

for $L_{h1} > 1{,}7 \times D_b \quad e \quad L_t = 0$:

$$V_{cs} = N_t \times (0.0072 \times D_b^3 + 0.105 \times D_b^2 + 0.08 \times D_b) \times (\alpha_2 - \alpha_1)$$

where:

| | |
|---|---|
| $V_{cs}$ | volume of concrete for the upper anchor block, in $m^3$, when applicable; |
| $V_{cb}$ | volume of concrete for the middle anchor block, in $m^3$; |
| $V_{cf}$ | volume of concrete for the lower anchor block, in $m^3$; |
| $V_{ca}$ | volume of concrete for a saddle block, in $m^3$; |
| $V_{ct}$ | volume of concrete for the lining of the section through a tunnel, in $m^3$; |
| $V_{cc}$ | volume of extra concrete for bifurcations, in $m^3$, when required; |
| $V_{cv}$ | volume of concrete for the valve housings at the beginning of the penstock, in $m^3$, when required; |
| $V_{ce}$ | volume of concrete for the valve housings at the end of the penstock, in $m^3$, when required; |
| $N_t$ | number of pressure penstocks; |

| | |
|---|---|
| $D_b$ | internal diameter of the pressure penstock, in m; |
| $\alpha_i$ | slope of section i, in degrees; |
| $L_{hi}$ | horizontal projection of section i, in m; |
| $N_a$ | number of saddle blocks per penstock; |
| $e_c$ | thickness of the concrete lining of the section through a tunnel, in m; |
| $L_t$ | length of the section through a tunnel, in m; |
| $L_{ct}$ | length of the section through a tunnel lined with concrete, in m; |
| N | number of generating units; |
| A | diameter of the inlet to the spiral casing, in m; |
| $NA_{xfu}$ | maximum water level in the tailrace canal, in m; |
| $El_d$ | elevation of the center line of the turbine distributor, in m; |
| $L_b$ | length of the pressure penstocks, in m; |
| $\alpha$ | secondary angle, in degrees; |
| $d_i$ | effective distance, in m; and |
| int(x) | function that returns the integer part of x. |

The amounts of cement and reinforcement steel are:

| | cement ($kg/m^3$) | reinforcement steel ($kg/m^3$) |
|---|---|---|
| lining | 250 | 50 |
| bifurcations and anchor blocks; saddle blocks and valve housings | 270 | 70 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- lining and saddle blocks:128.00/$m^3$
- anchor blocks and valve housings:174.00/$m^3$

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Steel Plates (account .12.19.34.23.23)

The total weight of the steel plates can be calculated from its thickness and the length of the penstock.

All the thicknesses calculated below should be rounded up to whole centimeters, using the expressions below:

$$x = \frac{e}{2.54}$$

if x ≤ 1: $e = 2.54 \times \frac{\text{int}(32 \times x + 0.9999)}{32}$

if 1 < x ≤ 1,5: $e = 2.54 \times \frac{\text{int}(16 \times x + 0.9999)}{16}$

if x > 1,5: $e = 2.54 \times \frac{\text{int}(8 \times x + 0.9999)}{8}$

The **minimum thickness for construction purposes**, $e_{min}$ (cm), is given by:

$$e_{min} = \frac{D_b}{4} + 0{,}127 \geq 0.635 \text{ cm}$$

where:

| | |
|---|---|
| $D_b$ | internal diameter of the pressure penstock, in m. |

The **working pressure**, $p_s$ (kgf/cm$^2$), at points 0 to 4, is given by:

for projects with **no** surge tank:

$p_{s0} = 0.1 \text{ x } (NA_{max} - El_0)$

$p_{s1} = 0.1 \text{ x } (NA_{max} + i_s \text{ x } L_{h1} - El_1)$

$p_{s2} = 0.1 \text{ x } (NA_{max} + i_s \text{ x } (L_{h1} + L_{h2}) - El_2)$

$p_{s3} = 0.1 \text{ x } [NA_{max} + i_s \text{ x } (L_{h1} + L_{h2} + L_{h3}) - El_3]$

$p_{s4} = 0.1 \text{ x } (NA_{max} + h_{sx} - El_4)$

for projects **with** a surge tank:

$p_{s0} = 0.1 \text{ x } (NA_{xch} - El_0)$

$p_{s1} = 0.1 \text{ x } (NA_{xch} + i_s \text{ x } L_{h1} - El_1)$

$p_{s2} = 0.1 \text{ x } (NA_{xch} + i_s \text{ x } (L_{h1} + L_{h2}) - El_2)$

$p_{s3} = 0.1 \text{ x } [NA_{xch} + i_s \text{ x } (L_{h1} + L_{h2} + L_{h3}) - El_3]$

$p_{s4} = 0.1 \text{ x } (NA_{xch} + h_{sx} - El_4)$

where:

| | |
|---|---|
| $NA_{max}$ | maximum normal water level in the reservoir; |
| $El_i$ | elevation of point i; |
| $h_{sx}$ | maximum surge pressure due to water hammer, in m; |
| $NA_{xch}$ | maximum normal water level in the surge tank; |
| $L_{hi}$ | horizontal projection of section i, in m; and |
| $i_s$ | energy gradient. |

The **thickness of the steel plate required at point i**, $e_i$ (cm), can be calculated by:

$$e_i = \frac{100 \times p_{si} \times D_b}{2 \times \sigma_a} + 0.3$$

where:

| | |
|---|---|
| $D_b$ | internal diameter of the pressure penstock, in m; |
| $p_{si}$ | working pressure at point i, in kgf/cm$^2$; |
| $\sigma_a$ | 1200 kgf/cm$^2$, permissible stress in steel; and |
| 0.3 | extra thickness to compensate for corrosion, in cm. |

The **working pressure withstood by the steel plate of minimum thickness**, $p_{sn}$ (kgf/cm²), is given by:

$$p_{sn} = 2 \times \sigma_a \times \frac{e_{min} - 0.3}{100 \times D_b}$$

where:

| | |
|---|---|
| $\sigma_a$ | 1200 kgf/cm², permissible stress in steel; |
| $e_{min}$ | minimum thickness of the steel plate for construction purposes, in cm; and |
| $D_b$ | internal diameter of the pressure penstock, in m. |

The **weight of the steel plates**, $P_c$ (t), can be obtained from one of the cases:

- Case 1: If $p_{s0} \geq p_{sn}$:

$$P_{c1} = k_c \times \frac{e_0 + e_1}{2} \times L_1$$

$$P_{c2} = k_c \times \frac{e_1 + e_2}{2} \times L_2$$

$$P_{c3} = k_c \times \frac{e_2 + e_3}{2} \times L_3$$

$$P_{c4} = k_c \times \frac{e_3 + e_4}{2} \times L_4$$

- Case 2: If $p_{s1} \geq p_{sn} > p_{s0}$:

$$L_{min} = \frac{p_{sn} - p_{so}}{0.1 \times (i_s + tg\alpha_1) \times \cos\alpha_1}$$

$$P_{c1} = k_c \times [e_{min} \times L_{min} + \frac{e_{min} + e_1}{2} \times (L_1 - L_{min})]$$

$P_{c2}$, $P_{c3}$, $P_{c4}$ same as case 1.

$e_0 = e_{min}$

- Case 3: If $p_{s2} > p_{sn} > p_{s1}$:

$$L_{min} = \frac{p_{sn} - p_{s1}}{0.1 \times (i_s + tg\alpha_2) \times \cos\alpha_2}$$

$$P_{c1} = k_c \times e_{min} \times L_1$$

$$P_{c2} = k_c \times [e_{min} \times L_{min} + \frac{e_{min} + e_2}{2} \times (L_2 - L_{min})]$$

$P_{c3}$, $P_{c4}$ same as case 1

$e_0$, $e_1 = e_{min}$

- Case 4: If $p_{s3} \geq p_{sn} > p_{s2}$:

$$L_{min} = \frac{p_{sn} - p_{s2}}{0.1 \times (i_s + tg\alpha_3) \times \cos\alpha_3}$$

$P_{c1}$ same as case 3

$$P_{c2} = k_c \times e_{min} \times L_2$$

$$P_{c3} = k_c \times [e_{min} \times L_{min} + \frac{e_{min} + e_3}{2} \times (L_3 - L_{min})]$$

$P_{c4}$ same as case 1

$e_0$, $e_1$, $e_2$ = $e_{min}$

- Case 5: If $p_{s4} \geq p_{sn} > p_{s3}$:

$$L_{min} = \frac{p_{sn} - p_{s3}}{0.1 \times i_s}$$

$P_{c1}$, $P_{c2}$ same as case 1

$$P_{c3} = k_c \times e_{min} \times L_3$$

$$P_{c4} = k_c \times [e_{min} \times L_{min} + \frac{e_{min} + e_4}{2} \times (L_4 - L_{min})]$$

$e_0$, $e_1$, $e_2$, $e_3$ = $e_{min}$

- Case 6: If $p_{sn} > p_{s4}$

$P_{c1}$, $P_{c2}$, $Pc_3$ same as case 5

$$P_{c4} = k_c \times e_{min} \times L_4$$

$e_0$, $e_1$, $e_2$, $e_3$, $e_4$ = $e_{min}$

where:

$$k_c = \frac{7.842}{100} \times \pi \times D_b$$

where:

| | |
|---|---|
| $p_{si}$ | working pressure at point i, in kgf/cm²; |
| $k_c$ | coefficient; |
| $e_i$ | required thickness of the steel plate at point i, in cm; |
| $L_i$ | length of section i, in m; |
| $L_{min}$ | length of section with minimum thickness, in m; |
| $p_{sn}$ | working pressure withstood by the steel plate of minimum thickness, in kgf/cm²; |
| $i_s$ | energy gradient. |
| $\alpha_i$ | slope of section i, in degrees; |
| $e_{min}$ | minimum thickness of the steel plate for construction purposes, in cm; and |
| 7.842 | specific mass of steel, in kg/cm³; and |
| $D_b$ | internal diameter of the pressure penstock, in m. |

The **total weight of the steel plates**, $P_c$ (t), including a 10% provision for fastening parts, is given by:

$$P_c = 1{,}10 \times N_t \times (P_{c1} + P_{c2} + P_{c3} + P_{c4})$$

where:

| | |
|---|---|
| $N_t$ | number of pressure tunnels; and |
| $P_{ci}$ | weight of the steel lining in section i, in t. |

The **acquisition cost of the steel plate** for the pressure penstock is R$ 4,235.00/t – FOB cost excluding tranportation and insurance, assembly and testing and provisions for taxes payable, depending on the current tax regime. The figures are valid for December 2006 and for projects anywhere in Brazil.

The following percentages must be added to the FOB price:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for taxes and charges payable on the equipment.

### Butterfly Valve (account .12.19.34.23.24)

The **acquisition cost of each butterfly valve**, $C_{vb}$ (R$), for the pressure penstock – FOB cost – is given by the expression below (or obtained from Graph B.29 from annex B, as a function of its diameter and maximum working pressure). The figures are valid for December 2006 and for projects anywhere in Brazil:

$$C_{vb} = 2.528 \times H_x^{0.35} \times K_B$$

where:

$K_B = 1000 \times (9.6 \times D_B^2 + 8.6 \times D_B - 1.85)$, for $0.75 \le D_B \le 2.0$ m, and $10 \le H_x \le 300$

$K_B = 1000 \times (10.2 \times D_B^2 + 9.2 \times D_B - 1.97)$, for $2.5 \le D_B \le 8.0$ m, and $10 \le H_x \le 300$

case **a**, with the valve at the beginning of the penstock, just after the surge tank:

$D_B = D_b$ $\qquad\qquad$ $H_x = NA_{xch} - El_0$

case **b**, with the valve at the end of the penstock:

$D_B = A$ $\qquad\qquad$ $H_x = NA_{max} - El_4 + h_s$

where:

| | |
|---|---|
| $H_x$ | maximum working pressure of the valve, in m; |
| $K_B$ | coefficient; |
| $D_B$ | diameter of the butterfly valve, in m; |
| $D_b$ | internal diameter of the pressure penstock, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $NA_{xch}$ | maximum water level in the surge tank, in m; |
| $El_i$ | elevation of the butterfly valve axis, in m; |
| $h_s$ | maximum surge pressure due to water hammer, in m; and |
| A | diameter of the inlet to the spiral casing, in m. |

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

### Spherical Valve (account .12.19.34.23.24)

The **acquisition cost of a spherical valve**, $C_{ve}$ (R$), for the pressure penstock – FOB cost – is given by the expression below (or obtained from Graph B.30 from annex B, as a function of its diameter and maximum working pressure). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

$$C_{ve} = 2.528 \times H_x^{0.40} \times K_E$$

where:

$K_E = 1000 \times (24.4 \times D_E^2 + 4.4 \times D_E + 12.37)$, for $1.0 \le D_E \le 4{,}0$ m, and $200 \le H_x \le 1500$ m.

$$D_E = A$$

$$H_x = NA_{max} - El_4 + h_s$$

where:

| | |
|---|---|
| $H_x$ | maximum working pressure of the valve, in m; |
| $K_E$ | coefficient; |
| $D_E$ | diameter of the spherical valve, in m; |
| A | diameter of the inlet to the spiral casing, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_4$ | elevation of the spherical valve axis, in m; and |
| $h_s$ | maximum surge pressure due to water hammer, in m. |

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost.

## PRESSURE TUNNELS (ACCOUNT .12.19.34)

### Basic Data

The main **information required for dimensioning** this item can be obtained from the overall layout of the project and the energy studies, as follows:

- horizontal projection of each section, $L_{h1}$, $L_{h2}$ and $L_{h3}$, in m;
- horizontal projection of the steel-lined part of the pressure tunnel, $L_{hb}$, in m;
- length of the headrace tunnel, $L_{ad}$, in m, when applicable;
- capacity of one generating unit, $P_1$, in MW, from item 5.7.2.;
- coefficient k, product of the yield of the generating unit by acceleration due to gravity, from item 2.1.;
- number of generating units, $N_g$, from item 5.7.2.;
- number of generating units per pressure tunnel, $N_f$;
- maximum normal water level in the reservoir, $NA_{max}$, from item 4.4.;
- minimum water level in the reservoir, $NA_{min}$, from item 5.3.;
- lowest elevation of the land in the intake area, $El_{ten}$;
- mean thickness of the layer of soil in the intake area, $e_{te}$ in m;
- normal water level in the tailrace canal, $NA_{fu}$, from item 4.4.;
- elevation of the center line of the turbine distributor, $El_d$, from item 5.7.2.;
- maximum net head, $H_1$, in m, from item 5.3.;
- head loss in the headrace canal, $h_c$, in m, from item 5.7.6., when applicable;
- head loss in the headrace tunnel, $h_a$, in m, from item 5.7.6., when applicable; and
- diameter of the inlet to the spiral casing, A, in m, from item 5.7.2., when applicable.

The main **information required for quantification** purposes is as follows:

- mean hydrostatic load on the tunnel, H in m;
- geological conditions of the area crossed by the tunnels;
- maximum water level in the surge tank, $NA_{xch}$;

- volume of common excavation upstream from the surface powerhouse, $V_{ttf}$, in $m^3$; and
- volume of surface rock excavation upstream from the surface powerhouse, $V_{rtf}$, in $m^3$.

### Considerations and Recommendations

A **section** is understood as being the stretch between two deflections in the vertical plane, counting from the outlet of the intake, point 0, and the inlet of the powerhouse, point 4, as shown in Figure 5.7.6.08.

For methodological reasons, the **slope of section 2**, $\alpha_2$, is obtained from the other data.

In this text, ther term **bifurcation** is also used for sections that split into three and for separators.

The **horizontal projection** of section 1, $L_{h1}$, must be long enough to match the curve.

$$L_{h1} \geq 4 \times D_b \times tg\frac{\alpha_2}{2}$$

where:

| | |
|---|---|
| $D_b$ | internal diameter of the steel-lined part of the pressure tunnel, in m; and |
| $\alpha_2$ | slope of section 2, in degrees. |

### Internal diameter and mean velocity

The **number of pressure tunnels**, $N_t$, is given by:

$$N_t = \frac{N_g}{N_f}$$

where:

| | |
|---|---|
| $N_g$ | number of generating units; and |
| $N_f$ | number of generating units per pressure tunnel. |

The **internal diameter of the steel-lined part of the pressure tunnel**, $D_b$ (m), can be determined by:

$$D_b = 14.2 \times \frac{(N_f \times P_1)^{0.43}}{H_{b1}^{0.65}}$$

where: $H_{b1} = NA_{max} - NA_{fu}$

where:

| | |
|---|---|
| $N_f$ | number of generating units per pressure penstock; |
| $P_1$ | capacity of one generating unit, in MW; |
| $H_{b1}$ | maximum gross head, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; and |
| $NA_{fu}$ | normal water level in the tailrace canal, in m. |

The **internal diameter of the unlined part of the pressure tunnel**, $D_c$ (m), is given by:

$$D_c = 1.1 \times D_b$$

where:

| | |
|---|---|
| $D_b$ | internal diameter of the steel-lined part of the pressure tunnel, in m. |

The **maximum turbine flow of each turbine**, $Q_1$ ($m^3/s$), is given by:

$$Q_1 = \frac{10^6 \times P_1}{k \times H_1}$$

where:

$$k = \rho \times g \times \eta_{t1} \times \eta_{g1} \qquad P_2 = \frac{P_1}{f_p}$$

for Francis turbines:

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

for Pelton and Kaplan turbines:

$$\eta_{t1} = 0.96 \qquad \eta_{g1} = 0.98$$

where:

| | |
|---|---|
| $P_1$ | capacity of one generating unit, in MW; |
| k | coefficient; |
| $H_1$ | maximum net head, in m; |
| ρ | 1000 kg/m³ – specific mass of water; |
| $\eta_{t1}$ | turbine output for maximum net head; |
| $\eta_{g1}$ | generator output for maximum net head; |
| g | 9.81 m/s² – acceleration due to gravity; |
| $P_2$ | capacity of one generator, in MVA; and |
| $f_p$ | capacity factor. |

The **maximum flow in each pressure tunnel**, $Q_{1f}$ (m³/s), is given by:

$$Q_{1f} = N_f \times Q_1$$

where:

| | |
|---|---|
| $N_f$ | number of generating units per pressure tunnel; and |
| $Q_1$ | maximum turbine flow of each turbine, in m³/s. |

The **mean velocity of flow in the steel-lined part** of the tunnel, $v_b$ (m/s), is given by:

$$v_b = \frac{4 \times Q_{1f}}{\pi \times D_b^2} \leq 7 \text{ m/s}$$

where:

| | |
|---|---|
| $Q_{1f}$ | maximum flow in each pressure tunnel, in m³/s; and |
| $D_b$ | internal diameter of the steel-lined part of the pressure tunnel, in m. |

If this restriction is not fulfilled, the velocity limit should be adopted and the diameter should be recalculated using:

$$D_b = \sqrt{\frac{4}{\pi} \times \frac{Q_{1f}}{7}} \qquad D_c = 1.1 \times D_b$$

where:

| | |
|---|---|
| $Q_{1f}$ | maximum flow in each pressure tunnel, in m³/s. |

The **mean velocity of flow** in the unlined part of the tunnel, $v_c$ (m/s), is given by:

$$v_c = \frac{4}{\pi} \times \frac{Q_{1f}}{D_c^2}$$

where:

| | |
|---|---|
| $Q_{1f}$ | maximum flow in each pressure tunnel, in $m^3/s$; and |
| $D_c$ | internal diameter of the unlined part of the pressure tunnel, in m. |

The **internal diameter of the pressure tunnel after a bifurcation** or the **lateral diameter after a separator**, $D_{1b}$ (m), can be determined by:

$$D_{1b} = \frac{D_b}{N_f^{3/8}}$$

where:

| | |
|---|---|
| $D_b$ | internal diameter of the pressure tunnel, in m; and |
| $N_f$ | number of generating units per pressure tunnel. |

### Profile of the pressure tunnel

The **elevation of points 0 to 3**, $El_0$ a $El_3$, is given by:

$$El_0 = El_1 = El_{sol} - i \times L_{ad} + \frac{D_{ab}}{2}$$

$$El_2 = El_3 = El_d$$

where:

$$El_{sol} = 0.5 \times \text{int}\left(\frac{NA_{min} - h_c - h_s - H_{cp}}{0.5}\right)$$

$$El_{sol} \leq 0.5 \times \text{int}\left(\frac{El_{ten} - e_{te}}{0.5}\right) + 1.0$$

$$h_s = 0.8 \times v_{cp} \times \sqrt{H_{cp}}$$

$$B_{cp} = D_{ab} \qquad H_{cp} = D_{ab}$$

for projects with no headrace tunnel and a totally steel-lined pressure tunnel:

$$v_{cp} = \frac{Q_{1f}}{B_{cp} \times H_{cp}} \qquad D_{ab} = D_b$$

for projects with no headrace tunnel and a partially steel-lined pressure tunnel:

$$v_{cp} = \frac{Q_{1f}}{B_{cp} \times H_{cp}} \qquad D_{ab} = D_c$$

for projects with a headrace tunnel:

$$v_{cp} = \frac{Q_t}{B_{cp} \times H_{cp}} \qquad D_{ab} = D_{ad}$$

where:

| | |
|---|---|
| $El_{sol}$ | elevation of the intake sill, in m; |
| i | slope of the headrace tunnel, in m/m, defined in the headrace tunnel studies; |
| $L_{ad}$ | length of the headrace tunnel, in m; |
| $D_{ab}$ | internal diameter at the start of the pressure tunnel, in m; |
| $El_d$ | elevation of the center line of the turbine distributor, in m; |
| $NA_{min}$ | normal minimum water level in the reservoir; |

| | |
|---|---|
| $h_{pc}$ | head loss in the headrace canal, in m, when applicable; |
| $h_s$ | minimum submergence of the intake (Gordon, 1970), in m; |
| $H_{cp}$ | height of the intake gate, in m; |
| $El_{ten}$ | lowest elevation of the land in the intake area; |
| $e_{te}$ | mean thickness of the layer of soil in the intake area, in m; |
| $v_{cp}$ | velocity of flow at the intake gate, in m/s; |
| $B_{cp}$ | height of the intake gate, in m; |
| $Q_{1f}$ | maximum flow in each pressure tunnel, in m³/s; |
| $D_b$ | internal diameter of the steel-lined part of the pressure tunnel, in m; |
| $D_c$ | internal diameter of the unlined part of the pressure tunnel, in m; |
| $Q_t$ | total maximum turbine flow, in m³/s; and |
| $D_{ad}$ | internal diameter of the headrace tunnel, in m. |

The **slope of section 2**, $\alpha_2$ (degrees), is given by:

$$\alpha_2 = \arctan\left(\frac{E_1 - E_2}{L_{h2}}\right)$$

where:

| | |
|---|---|
| $El_i$ | elevation of point i; |
| $L_{h2}$ | horizontal projection of section 2, in m; and |
| arctan | arctangent function. |

The length of the tunnel is determined from its profile.

The **length of section 2**, $L_2$ (m), of the tunnel is given by:

$$L_2 = \frac{L_{h2}}{\cos\alpha_2}$$

where:

| | |
|---|---|
| $L_{h2}$ | horizontal projection of section 2, in m; and |
| $\alpha_2$ | slope of section 2, in degrees. |

The **total length of the tunnel**, $L_t$ (m), is given by:

$$L_t = L_{h1} + L_2 + L_{h3}$$

where:

| | |
|---|---|
| $L_{hi}$ | horizontal projection of section i, in m; and |
| $L_2$ | length of section 2, in m. |

The **horizontal projection** of the tunnel, $L_{ht}$ (m), is given by:

$$L_{ht} = L_{h1} + L_{h2} + L_{h3}$$

where:

| | |
|---|---|
| $L_{hi}$ | length of the horizontal projection of section i, in m. |

It is not advisable to have a very short unlined part. If $L_{ht} - L_{hb} < 5 \times D_b$

then: $L_{hb} = L_{ht}$

where:

| | |
|---|---|
| $L_{ht}$ | horizontal projection of the tunnel, in m; |
| $L_{hb}$ | horizontal projection of the steel-lined part of the tunnel, in m; and |
| $D_b$ | internal diameter of the steel-lined part of the pressure tunnel, in m. |

The **length of the steel-lined part,** $L_b$ (m), and **unlined part** of the pressure tunnel, $L_c$ (m), is given by one of the expressions below:

- **case a** – when steel lining begins in section 1:

$$L_{b1} = L_{hb} - L_{h2} - L_{h3} \qquad L_b = L_{b1} + L_2 + L_{h3}$$

$$L_c = L_t - L_b$$

- **case b** – when steel lining begins in section 2:

$$L_{b2} = \frac{L_{hb} - L_{h3}}{\cos \alpha_2} \qquad L_b = L_{b2} + L_{h3}$$

$$L_c = L_t - L_b$$

- **case c** – when steel lining begins in section 3:

$$L_b = L_{hb} \qquad L_c = L_t - L_b$$

where:

| | |
|---|---|
| $L_{b1}$ | length of the steel-lined part in section 1, in m; |
| $L_{hb}$ | horizontal projection of the steel=lined part of the pressure tunnel, in m; |
| $L_{h2}$ | horizontal projection of section 2, in m; |
| $L_{h3}$ | horizontal projection of section 3, in m; |
| $L_2$ | length of section 2, in m; |
| $L_t$ | total length of the tunnel, in m; |
| $L_{b2}$ | length of the steel-lined part in section 2, in m; and |
| $\alpha_2$ | slope of section 2, in degrees. |

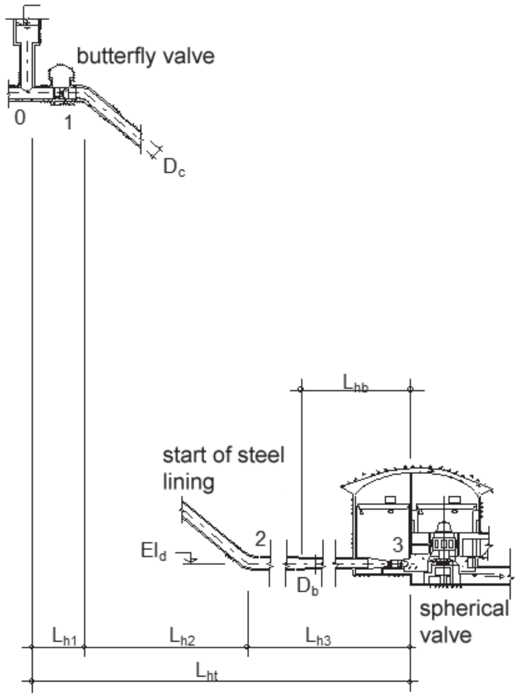



Fig. 5.7.6.10 Cross-section of a pressure tunnel.

### Surge Pressure

The **maximum surge pressure** due to water hammer, $h_{sx}$ (m), is given by:

$$h_{sx} = \frac{2 \times (L_b \times v_b + L_c \times v_c)}{g \times T_c} \quad \leq \; 0.30 \times H_d$$

where: $H_d = NA_{max} - El_d$

| $T_c$ | For |
|---|---|
| 6 s | short tunnels ($L_t \leq 3 \times H_{b1}$) |
| 10 s | long tunnels ($L_t > 3 \times H_{b1}$) |

where:

| | |
|---|---|
| $L_c$ | length of the unlined part of the pressure tunnel, in m; |
| $v_c$ | mean velocity of flow in the unlined part of the pressure tunnel, in m/s; |
| $L_b$ | length of the steel-lined part of the pressure tunnel, in m; |
| $v_b$ | mean velocity of flow in the steel-lined part of the pressure tunnel, in m/s; |
| g | 9.81 $m/s^2$ – acceleration due to gravity; |
| $T_c$ | time taken to close the valve or distributor, in s; |
| $L_t$ | total length of the pressure tunnel, in m; |
| $H_d$ | static head, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; and |
| $El_d$ | elevation of the center line of the turbine distributor, in m. |

When maximum surge pressure exceeds the limit, the diameter must be redimensioned using:

$$D_b = \sqrt{\frac{4}{\pi} \times \frac{Q_{1f}}{v_b}}$$

where: $v_b = \dfrac{0.30 \times H_{b1} \times g \times T_c}{2 \times (L_b + 0.826 \times L_c)}$

where:

| | |
|---|---|
| $Q_{1f}$ | maximum flow in each pressure tunnel, in $m^3/s$; |
| $v_b$ | mean velocity of flow in the steel-lined part of the pressure tunnel, in m/s; |
| $H_{b1}$ | maximum gross head, in m. |
| g | 9.81 $m/s^2$ – acceleration due to gravity; |
| $T_c$ | time taken to close the valve or distributor, in s; |
| $L_b$ | length of the steel-lined part of the pressure tunnel, in m; and |
| $L_c$ | length of the unlined part of the pressure tunnel, in m. |

The **energy gradient** for maximum surge pressure, $i_s$, is given by:

$$i_s = \frac{h_{sx}}{L_{ht}}$$

where:

| | |
|---|---|
| $h_{sx}$ | maximum surge pressure due to water hammer, in m; and |
| $L_{ht}$ | horizontal projection of the pressure penstock, in m. |

### Head loss

The **total head loss**, $h_p$ (m), from the intake to the pressure tunnel, is given by:

$$h_p = h_e + h_c + h_a + h_o + h_b + h_r + h_v + h_f$$

where:

for the unlined part:

$$h_o = \sum k_{oi} \times \frac{v_c^2}{2 \times g} \qquad h_r = 0,10 \times \frac{(v_b - v_c)^2}{2 \times g}$$

For $(r_i/D_{ad}) < 5$

$$k_{oi} = 0.2147 \times \left(\frac{r_i}{D_c}\right)^{-0.5718} \times \left(0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i\right)$$

For $(r_i/D_{ad}) \geq 5$

$$k_{oi} = \left(0.08 - 0.002 \times \left(\frac{r_i}{D_c} - 5\right)\right) \times \left(0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i\right)$$

and with a butterfly valve at the beginning of the pressure tunnel:

$$h_v = 0.20 \times \frac{v_b^2}{2 \times g}$$

for the steel-lined part:

$$h_o = \sum k_{oi} \times \frac{v_b^2}{2 \times g} \qquad h_r = 0.10 \times \frac{(v_a - v_b)^2}{2 \times g}$$

$$h_b = 0.10 \times \frac{v_b^2}{2 \times g} \qquad h_f = 6.35 \times L_t \times \frac{n^2 \times v_b^2}{D_b^{4/3}}$$

$$v_a = \frac{4}{\pi} \times \frac{Q_1}{A^2}$$

For $(r_i/D_{ad}) < 5$

$$k_{oi} = 0.2147 \times \left(\frac{r_i}{D_b}\right)^{-0.5718} \times \left(0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i\right)$$

For $(r_i/D_{ad}) \geq 5$

$$k_{oi} = \left(0.08 - 0.002 \times \left(\frac{r_i}{D_b} - 5\right)\right) \times \left(0.0746 \times \theta_i^3 - 0.4698 \times \theta_i^2 + 1.1928 \times \theta_i\right)$$

with a butterfly valve at the beginning of the pressure tunnel: $h_v = 0.20 \times \frac{v_b^2}{2 \times g}$

with a butterfly valve at the end of the pressure tunnel: $h_v = 0.20 \times \frac{v_a^2}{2 \times g}$

with a spherical valve: $h_v = 0.03 \times \frac{v_a^2}{2 \times g}$

for projects with no headrace tunnel and a totally steel-lined pressure tunnel: $h_e = 0.20 \times \frac{v_b^2}{2 \times g}$

for projects with no headrace tunnel and a partially steel-lined pressure tunnel: $h_e = 0.20 \times \frac{v_c^2}{2 \times g}$

for projects with a headrace tunnel: $h_e = 0.20 \times \frac{v_{ad}^2}{2 \times g}$

where:

| | |
|---|---|
| $h_e$ | head loss at the inlet, in m; |
| $h_a$ | head loss in the headrace tunnel, in m, when applicable; |
| $h_o$ | head loss at the bends, in m; |
| $h_b$ | head loss at the bifurcation, in m, when applicable; |
| $h_c$ | head loss in the headrace canal, in m, when applicable; |
| $h_r$ | head loss at diameter reductions, in m; |
| $h_v$ | head loss at the valves, in m, when applicable; |
| $h_f$ | continuous head loss in the pressure tunnel, in m; |
| $k_{oi}$ | coefficient for head loss at bends i; |
| $r_i$ | radius of curvature of section i, in meters; |
| $v_c$ | velocity in the unlined part of the pressure tunnel, in m/s; |
| g | 9.81 m/s$^2$ – acceleration due to gravity; |
| $v_b$ | velocity in the steel-lined part of the pressure tunnel, in m/s; |
| $v_a$ | velocity at the inlet to the spiral casing, in m/s; |
| $L_t$ | total length of the tunnel, in m; |
| n | 0.010 – Manning's coefficient for steel; |
| $D_b$ | internal diameter of the pressure tunnel in the steel-lined part, in m. |
| $Q_1$ | maximum turbine flow of each turbine, in m$^3$/s; |
| A | diameter of the inlet to the spiral casing, in m; |
| $v_{ad}$ | mean velocity of flow in the headrace tunnel, in m/s; and |
| θ | deflection of the tunnel axis, in radians. |

### Common Excavation (account .12.19.34.12.10)

The **volume of common excavation**, $V_{ttf}$ (m$^3$), upstream from the surface powerhouse should be determined from the project design.

The unit price of **common excavation** is R$ 7.60/m$^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the tunnel. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Surface Rock Excavation (account .12.19.34.12.11)

The **volume of excavation in rock**, $V_{rtf}$ (m$^3$), upstream from the surface powerhouse should be determined from the project design.

The unit price of **excavation in rock** is R$ 21.00/m$^3$ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the tunnel. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Underground Excavation of Rock (account .12.19.34.12.12)

The **volume of rock excavated underground** for pressure tunnels, $V_{stf}$ ($m^3$), is given by:

$$V_{stf} = V_{st} + V_{sb} + V_{se}$$

where:

$$V_{st} = N_t \times \frac{\pi}{4} \times [(D_c + 2 \times e_{cc})^2 \times L_c + (D_b + 2 \times e_c)^2 \times L_b]$$

$$V_{sb} = N_t \times 12 \times (D_b + 2 \times e_c)^3$$

$$V_{se} = N_t \times 12 \times (A + 2 \times e_{ca})^3$$

$$e_{ca} = 0.091 \times A^{0,62}$$

where:

| | |
|---|---|
| $V_{st}$ | volume of rock excavated underground for pressure tunnels, in $m^3$; |
| $V_{sb}$ | volume of rock excavated underground for the butterfly valve housing, in $m^3$, when installed in a separate cavity; |
| $V_{se}$ | volume of rock excavated underground for the spherical valve housing, in $m^3$, when installed in a separate cavity; |
| $N_t$ | number of tunnels; |
| $D_c$ | internal diameter of the unlined part of the pressure tunnel, in m; |
| $D_b$ | internal diameter of the steel-lined part of the pressure tunnel, in m; |
| $e_{cc}$ | thickness of the concrete lining in the unlined part of the pressure tunnel, in m; |
| $e_c$ | thickness of the concrete lining in the steel-lined part of the pressure tunnel, in m; |
| $L_c$ | length of the unlined part of the pressure tunnel, in m; |
| $L_b$ | length of the steel-lined part of the pressure tunnel, in m; |
| A | diameter of the inlet to the spiral casing, in m; and |
| $e_{ca}$ | thickness of the concrete lining at the inlet to the spiral casing, in m. |

The unit price of **underground excavation**, $P_{us}$ (R$/$m^3$), (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil, can be obtained from the expression below (or from Graph B33, annex B, as a function of the excavated area). This is the price per cubic meter measured using the excavation line of the project design and includes excavating, loading, transportation up to 1.5 km and unloading:

valid for $4 \leq A_{se} \leq 300$: $P_{us} = 474.08 \times A_{se}^{-0.4629}$

where: $A_{se} = \frac{\pi}{4} \times (D_b + 2 \times e_c)^2$

where:

| | |
|---|---|
| $A_{se}$ | area of the excavated area, in m²; |
| $D_b$ | internal diameter of the steel-lined part of the pressure tunnel, in m; and |
| $e_c$ | thickness of the concrete lining steel-lined part of the pressure tunnel, in m. |

A careful appraisal should be made of the different situations where the tunnels form a representative part of the cost estimate, paying special attention to the geological conditions in the region and particularly long tunnels.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Foundation Cleaning and Treatment (account .12.19.34.13)

The **area of foundation to be cleaned**, $A_{lf}$ (m²), is given by:

$$A_{lf} = N_t \times \pi \times (D_c + 2 \times e_{cc}) \times L_c + N_t \times \pi \times (D_b + 2 \times e_c) \times L_b$$

where:

| | |
|---|---|
| $N_t$ | number of tunnels; |
| $D_c$ | internal diameter of the unlined part of the pressure tunnel, in m; |
| $D_b$ | internal diameter of the steel-lined part of the pressure tunnel, in m; |
| $e_{cc}$ | thickness of the concrete lining in the unlined part of the pressure tunnel, in m; |
| $e_c$ | thickness of the concrete lining in the steel-lined part of the pressure tunnel, in m; |
| $L_c$ | length of the unlined part of the pressure tunnel, in m; and |
| $L_b$ | length of the steel-lined part of the pressure tunnel, in m. |

The **length of contact grouting and consolidation grouting holes**, $L_{tf}$ (m), is given by:

$$L_{tf} = N_t \times 1.0 \times \pi \times (D_c + 2 \times e_{cc}) \times L_c + N_t \times 1.0 \times \pi \times (D_b + 2 \times e_c) \times L_b$$

| | |
|---|---|
| $N_t$ | number of tunnels; |
| $D_c$ | internal diameter of the unlined part of the pressure tunnel, in m; |
| $D_b$ | internal diameter of the steel-lined part of the pressure tunnel, in m; |
| $e_{cc}$ | thickness of the concrete lining in the unlined part of the pressure tunnel, in m; |
| $e_c$ | thickness of the concrete lining in the steel-lined part of the pressure tunnel, in m; |
| $L_c$ | length of the unlined part of the pressure tunnel, in m; and |
| $L_b$ | length of the steel-lined part of the pressure tunnel, in m. |

The unit prices for **foundation cleaning and treatment** (expressed in Brazilian Reais, from December 2006) to be used for projects in the south, southeast, central west and northeast regions of Brazil, including the supply of inputs and equipment and the service per se, depend on the kind of surface and the equipment to be used, as follows:

- cleaning of rock surface: 39.70/m²
- rotary percussive drilling: 168.00/m
- grouting: 72.00/m
- rock anchors: 241.00/m

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Concrete (account .12.19.34.14)

The **volume of concrete** for pressure tunnels, $V_{ctf}$ (m³), is given by:

$$V_{ctf} = V_{cr} + V_{ct} + V_{cf} + V_{cb} + V_{ce}$$

where:

$$V_{cr} = N_t \times \pi \times (D_c + e_{cc}) \times e_{cc} \times L_c + N_t \times \pi \times (D_b + e_c) \times e_c \times L_b$$

$$V_{cf} = \pi \times (D_b + 3 \times e_c) \times e_c \times 5 \times D_b$$

$$V_{cb} = N_t \times 12 \times D_b^3 \qquad V_{ce} = N_g \times 12 \times A^3$$

$$e_{cc} = k_g \times \left[0.091 \times D_c^{0.62} + 0.0034 \times (H\text{-}30)\right]$$

$$e_c = 0.091 \times D_b^{0.62}$$

| $k_g$ | Geological conditions |
|---|---|
| 1.0 | good |
| 1.4 | average |
| 2.0 | poor or no information |

for $L_c$>0: $V_{ct} = N_t \times \frac{\pi}{2} \times \left[(D_c + 3)^2 - (D_c + 2 \times e_{cc})^2\right] \times D_c$

for $L_c$=0: $V_{ct} = N_t \times \frac{\pi}{2} \times \left[(D_b + 3)^2 - (D_b + 2 \times e_c)^2\right] \times D_b$

where:

| | |
|---|---|
| $V_{cr}$ | volume of concrete for the lining of the pressure tunnels, in m³; |
| $V_{ct}$ | volume of concrete for the transition from the square to circular section after the intake, in m³; |
| $V_{cf}$ | volume of extra concrete for the bifurcations, in m³, when required; |
| $V_{cb}$ | volume of concrete for the valve housing at the beginning of the penstock, in m³, when required; |
| $V_{ce}$ | volume of concrete for the valve housing at the end of the penstock, in m³, when required; |
| $N_t$ | number of pressure tunnels; |
| $D_c$ | internal diameter of the unlined part of the pressure tunnel, in m; |
| $e_{cc}$ | thickness of the concrete lining in the unlined part of the pressure tunnel, in m; |
| $L_c$ | length of the unlined part of the pressure tunnel, in m; |
| $D_b$ | internal diameter of the steel-lined part of the pressure tunnel, in m; |
| $e_c$ | thickness of the concrete lining along the steel-lined part of the pressure tunnel, in m; |
| $L_b$ | length of the steel-lined part of the pressure tunnel, in m; |
| $N_g$ | number of generating units; |
| A | diameter of the inlet to the spiral casing, in m; |
| $k_g$ | coefficient to represent geological conditions; and |
| H | mean hydrostatic load in the pressure tunnel, in m. |

The amounts of cement and reinforcement steel are:

| | cement (kg/m³) | reinforcement steel (kg/m³) |
|---|---|---|
| transitions and lining | 250 | 50 |
| bifurcations and valve housings | 270 | 70 |

The unit price for **cement** is R$ 348.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the manufacture of the concrete, measured from the project drawings, and includes its supply, transportation to the worksite, storage and handling costs.

The unit price of the **reinforcement steel** is R$ 4,327.00/t (December 2006 database) for projects in the south, southeast, central west and northeast regions of Brazil. This price per ton is for the steel used, and includes its supply, transportation to the worksite, storage, preparation and installation of the steel.

The unit prices for **concrete without cement** are expressed in Brazilian reals per cubic meter of dam (December 2006 database) and are valid for projects in the south, southeast, central west and northeast regions of Brazil. They include all the services and inputs required for its manufacture, transportation up to 1.5 km, placing and treatment, and are:

- transitions, lining and bifurcations: 129.00/$m^3$
- valve housings: 174.00/$m^3$

When the construction work demands large production peaks, significant rises and falls, and small volumes of work that make the mobilization and demobilization costs of the contractor proportionally higher, based on the judgement of the cost engineer and in the absence of more accurate information, the unit price of concrete without cement may be up to 10% higher.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### Steel Lining

The total weight of the steel lining is obtained from its thickness of the steel plate and the length of the lined part.

All the thicknesses calculated below should be rounded up to whole centimeters, using the expressions below:

$$x = \frac{e}{2.54}$$

if $x \leq 1$: $e = 2.54 \times \frac{int(32 \times x + 0.9999)}{32}$

if $1 < x \leq 1.5$: $e = 2.54 \times \frac{int(16 \times x + 0.9999)}{16}$

if $x > 1.5$: $e = 2.54 \times \frac{int(8 \times x + 0.9999)}{8}$

The **minimum thickness for construction purposes**, $e_{min}$ (cm), is given by:

$$e_{min} = \frac{D_b}{4} + 0.127 \geq 0.635 \text{ cm}$$

where:

| | |
|---|---|
| $D_b$ | internal diameter of the steel-lined part of the pressure tunnel, in m. |

The **working pressure**, $p_s$ (kgf/cm$^2$), at points 0 to 3 is given by the expressions below, assuming that the steel lining will withstand half the dynamic pressure:

for projects with no surge tank:

$$p_{so} = \frac{0.1 \times (NA_{max} - El_0)}{2}$$

$$p_{s1} = \frac{0.1 \times (NA_{max} + i_s \times L_{h1} - El_1)}{2}$$

$$p_{s2} = \frac{0.1 \times (NA_{max} + i_s \times (L_{h1} + L_{h2}) - El_2)}{2}$$

$$p_{s3} = \frac{0.1 \times (NA_{max} + h_{sx} - El_3)}{2}$$

for projects with a surge tank:

$$p_{so} = \frac{0.1\times (NA_{xch} - El_0)}{2}$$

$$p_{s1} = \frac{0.1\times (NA_{xch} + i_s \times L_{h1} - El_1)}{2}$$

$$p_{s2} = \frac{0.1\times (NA_{xch} + i_s \times (L_{h1} + L_{h2}) - El_2)}{2}$$

$$p_{s3} = \frac{0.1\times (NA_{xch} + h_{sx} - El_3)}{2}$$

where:

| | |
|---|---|
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $NA_{xch}$ | maximum water level in the surge tank, in m; |
| $El_i$ | elevation of point i, in m; |
| $L_{hi}$ | horizontal projection of section i, in m; |
| $h_{sx}$ | maximum surge pressure in the tunnel due to water hammer, in m; and |
| $i_s$ | energy gradient for maximum surge pressure. |

The **thickness of steel lining required at point i**, $e_i$ (cm), can be calculated by:

$$e_i = \frac{100\times p_{si}\times D_b}{2\times \tau_a} + 0.3$$

where:

| | |
|---|---|
| $D_b$ | internal diameter of the steel-lined part of the pressure tunnel, in m; |
| $p_{si}$ | working pressure at point i, in kgf/cm$^2$; |
| $\tau_a$ | 1200 kgf/cm$^2$, permissible stress in steel; and |
| 0.3 | extra thickness to compensate for corrosion, in cm. |

The **working pressure withstood by the minimum-thickness metal plate**, $p_{sn}$ (kgf/cm$^2$), is given by:

$$p_{sn} = 2\times \tau_a \times \frac{e_{min} - 0.3}{100\times D_b}$$

where:

| | |
|---|---|
| $\tau_a$ | 1200 kgf/cm$^2$, permissible stress in steel; |
| $e_{min}$ | minimum thickness of the steel plate for construction purposes, in cm; and |
| $D_b$ | internal diameter of the pressure penstock, in m. |

The expressions for determining the **weight of the steel lining**, $P_b$ (t), will depend on the section where the lining begins.

The **working pressure at point B**, beginning of the steel-lined part of the pressure tunnel, $p_{sB}$ (kgf/cm$^2$), is given by:

$$p_{sB} = \frac{0.1\times (NA_{max} + i_s \times L_c - El_0)}{2}$$

where:

| | |
|---|---|
| $NA_{max}$ | normal maximum water level in the reservoir, in m; |
| $i_s$ | energy gradient for maximum surge pressure; |
| $L_c$ | length of the unlined part of the pressure tunnel, in m; and |
| $El_0$ | elevation of point 0, in m. |

- **case a** – when steel lining begins in section 1:

- **case a1** – if $p_{sB} \geq p_{sn}$:

$$P_{b1} = k_b \times \frac{e_B + e_1}{2} \times L_{b1}$$

$$P_{b2} = k_b \times \frac{e_1 + e_2}{2} \times L_2$$

$$P_{b3} = k_b \times \frac{e_2 + e_3}{2} \times L_{h3}$$

- **case a2** – if $p_{s1} \geq p_{sn} \geq p_{sB}$:

$$L_{min} = 2 \times \frac{p_{sn} - p_{sB}}{0.1 \times i_s}$$

$$P_{b1} = k_b \times e_{min} \times L_{min} + k_b \times \frac{e_{min} + e_1}{2} \times (L_{b1} - L_{min})$$

$P_{b2}$, $P_{b3}$ same as case a1.

$e_B = e_{min}$

- **case a3** – if $p_{s2} \geq p_{sn} > p_{s1}$:

$$L_{min} = 2 \times \frac{p_{sn} - p_{s1}}{0.1 \times \left(\frac{i_s}{2} + tg\alpha_2\right) \times \cos\alpha_2}$$

$$P_{b1} = k_b \times e_{min} \times L_{b1}$$

$$P_{b2} = k_b \times e_{min} \times L_{min} + k_b \times \frac{e_{min} + e_2}{2} \times (L_2 - L_{min})$$

$P_{b3}$ same as case a1.

$e_B = e_1 = e_{min}$

- **case a4** – if $p_{s3} \geq p_{sn} > p_{s2}$:

$$L_{min} = 2 \times \frac{p_{sn} - p_{s2}}{0.1 \times i_s}$$

$P_{b1}$ same as case a3.

$$P_{b2} = k_b \times e_{min} \times L_2$$

$$P_{b3} = k_b \times e_{min} \times L_{min} + k_b \times \frac{e_{min} + e_3}{2} \times (L_{h3} - L_{min})$$

$e_B = e_1 = e_2 = e_{min}$

- **case a5** – if $p_{sn} > p_{s3}$:

$P_{b1}$, $P_{b2}$ same as case a4.

$$P_{b3} = k_b \times e_{min} \times L_{h3}$$

$e_B = e_1 = e_2 = e_3 = e_{min}$

- **case b** – when steel lining begins in section 2:

- **case b1** – if $p_{sB} \geq p_{sn}$:

$$P_{b1} = 0$$

$$P_{b2} = k_b \times \frac{e_B + e_2}{2} \times L_{b2}$$

$$P_{b3} = k_b \times \frac{e_2 + e_3}{2} \times L_{h3}$$

- **case b2** – if $p_{s2} \geq p_{sn} \geq p_{sB}$:

$$L_{min} = 2 \times \frac{p_{sn} - p_{sB}}{0.1 \times \left( \frac{i_s}{2} + \tan\alpha_2 \right) \times \cos\alpha_2}$$

$$P_{b1} = 0$$

$$P_{b2} = k_b \times e_{min} \times L_{min} + k_b \times \frac{e_{min} + e_2}{2} \times (L_2 - L_{min})$$

$P_{b3}$ same as case b1.

$e_B = e_{min}$

- **case b3** – if $p_{s3} \geq p_{sn} > p_{s2}$:

$$L_{min} = 2 \times \frac{p_{sn} - p_{s2}}{0.1 \times i_s}$$

$$P_{b1} = 0$$

$$P_{b2} = k_b \times e_{min} \times L_{b2}$$

$$P_{b3} = k_b \times e_{min} \times L_{min} + k_b \times \frac{e_{min} + e_3}{2} \times (L_{h3} - L_{min})$$

$e_B = e_2 = e_{min}$

- **case b4** – if $p_{sn} > p_{s3}$:

$$P_{b1} = 0$$

$P_{b2}$ same as case b3.

$$P_{b3} = k \times e_{min} \times L_{h3}$$

$e_B = e_2 = e_3 = e_{min}$

- **case c** – when steel lining begins in section 3:

- **case c1** – if $p_{sB} \geq p_{sn}$:

$$P_{b1} = P_{b2} = 0$$

$$P_{b3} = k_b \times \frac{e_B + e_3}{2} \times L_b$$

- **case c2** – if $p_{s3} \geq p_{sn} > p_{sB}$:

$$L_{min} = 2 \times \frac{p_{sn} - p_{sB}}{0.1 \times i_s}$$

$$P_{b1} = P_{b2} = 0$$

$$P_{b3} = k_b \times e_{min} \times L_{min} + k_b \times \frac{e_{min} + e_3}{2} \times (L_b - L_{min})$$

$e_B = e_{min}$

- **case c3** – if $p_{sn} > p_{s3}$:

$$P_{b1} = P_{b2} = 0$$

$$P_{b3} = k_b \times e_{min} \times L_b$$

$e_B = e_3 = e_{min}$

where: $k_b = \frac{7.842}{100} \times \pi \times D_b$

where:

| | |
|---|---|
| $p_{si}$ | working pressure at point i, in kgf/cm²; |
| $k_b$ | coefficient; |
| $e_i$ | thickness of the steel plate at point i, in cm; |
| $L_{bi}$ | length of the steel-lined part of section i where steel lining begins, in m; |
| $L_2$ | length of section 2, in m; |
| $L_{h3}$ | length of section 3, in m; |
| $p_{sn}$ | working pressure withstood by a steel plate of minimum thickness, in kgf/cm²; |
| $i_s$ | energy gradient for maximum surge pressure; |
| $e_{min}$ | minimum thickness of steel lining for construction purposes, in cm; |
| $L_{min}$ | length with minimum thickness, in m; |
| $\alpha_2$ | slope of section 2, in degrees; |
| 7.842 | specific mass of steel, in t/m³; and |
| $D_b$ | internal diameter of the steel-lined part of the pressure tunnel, in m. |

The **total weight of the steel lining**, $P_b$ (t), including a 10% provision for fastening parts, is given by:

$$P_b = 1.10 \times N_t \times (P_{b1} + P_{b2} + P_{b3})$$

where:

| | |
|---|---|
| $N_t$ | number of pressure tunnels; and |
| $P_{bi}$ | weight of the steel lining in section i, in t. |

The **acquisition cost of the steel plate** for the pressure penstock is R$ 4.235.00/t – FOB cost excluding tranportation and insurance, assembly and testing and provisions for taxes payable, depending on the current tax regime. The figures are valid for December 2006 and for projects anywhere in Brazil.

The following percentages must be added to the FOB price:

- 5.0%: for transportation and insurance;
- 8.0%: for assembly and testing; and
- 28.0%: for taxes and charges payable on the equipment.

### Butterfly Valve (account .12.19.34.23.24)

The **acquisition cost of each butterfly valve**, $C_{vb}$ (R$), for the pressure penstock – FOB cost – is given by the expression below (or obtained from Graph B.29, annex B, as a function of its diameter and maximum working pressure). The figures are valid for December 2006 and for projects anywhere in Brazil:

$$C_{vb} = 2.528 \times H_x^{0.35} \times K_B$$

where:

$K_B = 1000 \times (9.6 \times D_B^2 + 8.6 \times D_B - 1.85)$, for $0.75 \leq D_B \leq 2.0$ m, and $10 \leq H_x \leq 300$

$K_B = 1000 \times (10.2 \times D_B^2 + 9.2 \times D_B - 1.97)$, for $2.5 \leq D_B \leq 8.0$ m, and $10 \leq H_x \leq 300$

- case **a**, valve at the beginning of the penstock, just after the surge tank:

$$D_B = D_b \qquad H_x = NA_{xch} - El_0$$

- case **b**, valve at the end of the penstock:

$$D_B = A \qquad H_x = NA_{max} - El_4 + h_s$$

where:

| | |
|---|---|
| $H_x$ | maximum working pressure of the valve, in m; |
| $K_B$ | coefficient; |
| $D_B$ | diameter of the butterfly valve, in m; |
| $D_b$ | internal diameter of the pressure penstock, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $NA_{xch}$ | maximum water level in the surge tank, in m; |
| $El_i$ | elevation of the butterfly valve axis, in m; |
| $h_s$ | maximum surge pressure due to water hammer, in m; and |
| A | diameter of the inlet to the spiral casing, in m. |

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost

### Spherical Valve (account .12.19.34.23.24)

The **acquisition cost of each spherical valve**, $C_{ve}$ (R$), for the pressure penstock – FOB cost – is given by the expression below (or obtained from Graph B.30, annex B, as a function of its diameter and maximum working pressure). The figures are valid for December 2006 and for projects anywhere in Brazil (Eletrosul, 1996):

$$C_{ve} = 2.528 \times H_x^{0.40} \times K_E$$

where:

$K_E = 1000 \times (24.4 \times D_E^2 + 4.4 \times D_E + 12.37)$, for $1.0 \leq D_E \leq 4{,}0$ m, and $200 \leq H_x \leq 1500$ m.

$$D_E = A$$

$$H_x = NA_{max} - El_4 + h_s$$

where:

| | |
|---|---|
| $H_x$ | maximum working pressure of the valve, in m; |
| $K_E$ | coefficient; |
| $D_E$ | diameter of the spherical valve, in m; |

| | |
|---|---|
| A | diameter of the inlet to the spiral casing, in m; |
| $NA_{max}$ | maximum normal water level in the reservoir, in m; |
| $El_4$ | elevation of the axis of the spherical valve, in m; and |
| $h_s$ | maximum surge pressure due to water hammer, in m. |

The cost of transportation and insurance, assembly and testing, and taxes and charges payable on the equipment must be added to the FOB cost.

## TAILRACE CANAL

The main **information required for dimensioning** this item can be obtained from the overall layout and item 5.1.2. (Hydrometeorological Data) and from the dimensioning underway, as follows:

- minimum water level in the tailrace canal, $NA_{fu}$, in m;
- width of the powerhouse, with the exception of ones equipped with Pelton turbines, $B_{cf}$, in m;
- mean length of the tailrace canal, $L_{fu}$, in m;
- total maximum turbine flow, $Q_t$, in $m^3/s$;
- mean velocity of flow in the tailrace canal, ideally lower than 1.5 m/s, $v_{fu}$, in m/s;
- mean elevation of the land in section 0 perpendicular to the tailrace canal, next to the powerhouse, $El_{tf0}$;
- thickness of the layer of soil in section 0, $e_{te0}$, in m;
- mean elevation of the land in section 1 perpendicular to the tailrace canal in the first third, $El_{tf1}$;
- thickness of the layer of soil in section 1, $e_{te1}$, in m;
- mean elevation of the land in section 2 perpendicular to the tailrace canal in the second third, $El_{tf2}$;
- thickness of the layer of soil in section 2, $e_{te2}$, in m; and
- mean lateral slope, horizontal distance for a 1.0 m difference in level, in m.

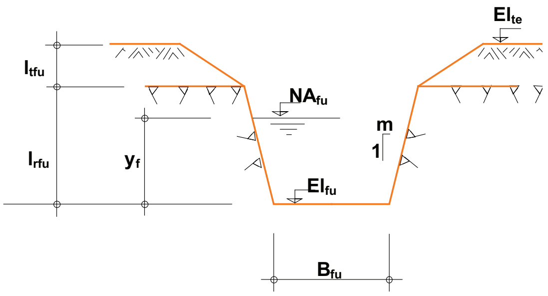



Fig. 5.7.6.11 – Typical cross-section of a tailrace canal.

The methodology expounded below is for short canals with negligible head loss. A canal can be deemed short if its length is no more than three times the width of the canal bottom.

For projects with powerhouses equipped with Pelton turbines, it is better to use the spreadsheet for long intake cannals (576CN.xls), with adaptations.

The mean lateral slope, "m", will depend on the geological conditions and the height of excavation:

| m | Height of excavation |
|---|---|
| 0.3 | ≤ 16 m |
| 0.6 | > 16 m |

For good geological conditions, the lateral slope can be assumed, provided it is possible to excavate without the need for intermediate berms.

### Dimensioning the canal

The **cross-sectional area of flow** in the canal, $A_{fu}$ ($m^2$), is given by:

$$A_{fu} = \frac{Q_t}{v_{fu}}$$

where:

| | |
|---|---|
| $Q_t$ | total maximum turbine flow, in $m^3/s$; and |
| $v_{fu}$ | mean velocity of discharge. |

The **width of the canal bottom**, $B_{fu}$ (m), is given by:

$$B_{fu} = B_{cf} - 2.0$$

where:

| | |
|---|---|
| $B_{cf}$ | width of the powerhouse, with the exception of ones equipped with Pelton turbines, in m. |

The **depth of flow in the canal**, $y_f$ (m), is given by:

$$y_f = \frac{-B_{fu} + \sqrt{B_{fu}^2 + 4 \times m \times A_{fu}}}{2 \times m}$$

where:

| | |
|---|---|
| $B_{fu}$ | width of the canal bottom, in m; |
| $A_{fu}$ | area of the cross-section of discharge in the canal, in $m^2$; and |
| m | lateral slope, horizontal distance for a 1.0 m difference in level. |

The **elevation of the canal bottom**, $El_{fu}$, is given by:

$$El_{fu} = NA_{nfu} - y_f$$

where:

| | |
|---|---|
| $NA_{nfu}$ | minimum water level in the tailrace canal, in m; and |
| $y_f$ | depth of flow of the canal, in m. |

The **head loss in the canal** is negligible.

### Common Excavation (Account .12.19.35.12.10)

The **volume of common excavation**, $V_{tfu}$ ($m^3$), is given by:

$$V_{tfu} = \left( \frac{V_{tf0}}{2} + V_{tf1} + V_{tf2} \right) \times \frac{L_{fu}}{3}$$

where:

$$V_{tfi} = \left[ B_{fu} - 10 \times m + 2 \times (m \times h_{rfi} + e_{tei}) \right] \times e_{tei}$$

$$h_{rfi} = (El_{tei} - El_{fu}) - e_{tei}$$

where:

| | |
|---|---|
| $V_{tfi}$ | volume of common excavation per meter in section i of the canal, in $m^3/m$; |
| $L_{fu}$ | length of the tailrace canal, in m; |
| $B_{fu}$ | width of the canal bottom, in m; |
| m | lateral slope, horizontal distance for a 1.0 m difference in level, in m; |
| $h_{ri}$ | depth of excavation in rock in section i of the canal, in m; |
| $e_{tei}$ | mean thickness of the layer of soil in section i of the canal, in m; |
| $El_{tei}$ | elevation of the land along the canal axis in section i of the canal, in m; and |
| $El_{fu}$ | elevation of the canal bottom, in m. |

The unit price of **common excavation** is R$ 7.60/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the tailrace canal. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the work involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the work involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

#### Surface Rock Excavation (Account .12.19.35.11)

The volume of surface rock excavation, $V_{rfu}$ (m³), is given by:

$$V_{rfu} = \left( \frac{V_{rf0}}{2} + V_{rf1} + V_{rf2} \right) \times \frac{L_{fu}}{3}$$

where:

$$V_{rfi} = (B_{fu} - 10 \times m + m \times h_{rfi}) \times h_{rfi}$$

where:

| | |
|---|---|
| $V_{rfi}$ | volume of surface rock excavation per meter in section i of the canal, in m³/m; |
| $L_{fu}$ | length of the tailrace canal, in m; |
| $B_{fu}$ | width of the canal bottom, in m; |
| m | lateral slope, horizontal distance for a 1.0 m difference in level, in m; and |
| $h_{ri}$ | depth of excavation in rock in section i of the canal, in m. |

The unit price of **excavation in rock** is R$ 21.00/m³ (from December 2006 database), which can be used for projects in the south, southeast, central west and northeast regions of Brazil. This is the price per cubic meter calculated above the excavation line of the tailrace canal. The price includes clearing the vegetation from the area, excavating, loading, transportation up to 1.5 km and unloading. It should be adjusted as required for each project using the following recommendations:

- when the service involves adverse topography, significant differences in ground level, small volumes and restricted working space, based to the judgment of the cost engineer and in the absence of more accurate information, the unit price could be up to 20% higher; and
- when the service involves favorable topography, high productivity, ample working space and large volumes, the unit price could be up to 20% lower.

For projects in the Amazon region, the price should be raised either by 20% or by a different amount identified in market research.

### 5.7.7 Roads, Railroads and Bridges (account .16)

Road maps should be consulted and field trips made, as and when necessary, to identify the need to build bridges, roads or railroads, their length and the construction categories involved.

The cost of this account should be calculated using the unit prices from tables 4.10.10.01 to 4.10.10.03. When an earth landing strip (1km) is deemed necessary, the cost should be assumed to be R$ 500,000.00 per runway. For larger airport facilities, a survey should be undertaken based on the Manual.

## 5.7.8 Indirect Costs (account .17)

Indirect costs include:

- building and maintaining the construction site and workers' camp;
- engineering and socioenvironmental studies;
- owner's administration costs.

The costs can be obtained from graphs or as percentages of the total direct cost, as set out below.

### Construction Site and Workers' Camp (Account .17.21)

The basic data are:

- notify whether or not there is a workers' camp;
- total volume of concrete for construction, $V_{ct}$, in $m^3$;
- total volume of surface rock excavation, $V_{et}$, in $m^3$;
- volume of rock excavated underground, $V_{es}$, in $m^3$;
- total volume of earthfill or rockfill, $V_a$, in $m^3$.

Use spreadsheet 57ope.xls to obtain the building and maintenance cost of the construction site and workers' camp.

In inventory studies, the cost of buildings, and the maintenance and operation of the construction site and workers' camp are a function of the volume of construction work translated by factor F:

for $V_a \leq 30{,}000{,}000\ m^3$

$$F = 30 \times V_{ct} + V_{et} + 5 \times V_{es} + 0.25 \times V_a$$

for $V_a > 30{,}000{,}000\ m^3$

$$F = 30 \times V_{ct} + V_{et} + 5 \times V_{es} + 0.15 \times V_a$$

where:

| | |
|---|---|
| $V_{ct}$ | total volume of concrete for construction, in $m^3$; |
| $V_{et}$ | total volume of surface rock excavation, comprising the sum of the volumes of common excavation, excavation in rock and excavation in borrow areas, in $m^3$, |
| $V_{es}$ | volume of underground excavation, in $m^3$; and |
| $V_a$ | total volume of earthfill or rockfill, in $m^3$. |

### Engineering (Account. 17.22.40)

For this account, which represents the engineering costs, the overall value is taken as a percentage of the total direct cost, as shown below:

- 3.5%: .17.22.40.36 – Basic Project Engineering
- 1.0%: .17.22.40.37 – Special Engineering Services
- 0.5%: .17.22.40.54 – Socioenvironmental Studies and Plans

### Owner's Administration Costs (Account. 17.22.41)

The owner's administration costs are estimated as being 10% of the total direct costs.

## 5.7.9 Interest During Construction (account .18)

The procedure adopted for estimating interest during construction is to distribute the costs according to the construction schedule and apply an interest rate to the entire costs accrued in each year.

Tables 5.7.9.01 and 5.7.9.02 show examples of annual interest rates used to calculate interest during construction (i) at 10% and 12%. Annual interest rates can be obtained from the concession-granting authority.

Based on the estimated construction time, from preparation to generation, the disbursement schedule and respective interest rates should be used during construction.

Table 5.7.9.01 – Interest during construction (for i = 10% p.a.)

| Construction time | Year | | | | | | | Interest | % |
|---|---|---|---|---|---|---|---|---|---|
| (Years) | 1 | 2 | 3 | 4 | 5 | 6 | 7 | calculation | interest |
| 2 | 40 | 60 | | | | | | 109.55 | 10 |
| 3 | 25 | 40 | 35 | | | | | 115.08 | 15 |
| 4 | 15 | 30 | 35 | 20 | | | | 120.89 | 21 |
| 5 | 10 | 20 | 25 | 30 | 15 | | | 126.11 | 26 |
| 6 | 5 | 15 | 20 | 21 | 25 | 14 | | 130.14 | 30 |
| 7 | 3 | 7 | 15 | 20 | 20 | 21 | 14 | 133.22 | 33 |

Table 5.7.9.02 – Interest during construction(for i = 12% p.a.)

| Construction time | Year | | | | | | | Interest | % |
|---|---|---|---|---|---|---|---|---|---|
| (Years) | 1 | 2 | 3 | 4 | 5 | 6 | 7 | calculation | interest |
| 2 | 40 | 60 | | | | | | 111.49 | 11 |
| 3 | 25 | 40 | 35 | | | | | 118.26 | 18 |
| 4 | 15 | 30 | 35 | 20 | | | | 125.44 | 25 |
| 5 | 10 | 20 | 25 | 30 | 15 | | | 131.97 | 32 |
| 6 | 5 | 15 | 20 | 21 | 25 | 14 | | 137.09 | 37 |
| 7 | 3 | 7 | 15 | 20 | 20 | 21 | 14 | 141.03 | 41 |

# 5.8 COMPARISON AND SELECTION OF CASCADE OPTIONS

In the analysis of the different cascade options considered in the Final Studies, the energy benefits that would arise from developing the hydroelectric potential of the river basin according to each cascade should be compared with their building costs and the negative socioenvironmental impacts they would cause, as well as the positive socioenvironmental impacts of each cascade.

The energy-economic implications are assessed by the cost/energy benefit index, while the negative socioenvironmental impacts are expressed by the negative socioenvironmental impact index. The positive socioenvironmental impacts on the river basin are assessed separately by the positive socio-environmental impact index. Items 5.8.1, 5.8.2 and 5.8.3 describe how these three indexes are obtained for each cascade in the Final Studies, while item 5.8.4 sets out how these indexes can be used in a multi-objective approach to rank the cascades and choose the most advantageous one.

## 5.8.1 Cost/Energy Benefit Index

The method used in the Final Studies to obtain the cost/energy benefit index of the different cascades under study is the same as that used in the Preliminary Studies (see item 4.11.1). Clearly, the energy benefits and cost estimates used in the Final Studies must be determined using criteria and procedures compatible with this stage of the studies, which are set out in items 5.3 and 5.7.

## 5.8.2 Negative Socioenvironmental Impact Index (IAn)

The negative socioenvironmental impact index of a cascade option should express the degree of negative impact the set of projects comprising it would have on the study area. In the Final Studies, the index is used to rank the cascades as a function of the objective of **minimizing negative socio-environmental impacts**, thereby providing a valuable input for their comparison and the selection of the best alternative.

As in the Preliminary Studies, the negative socioenvironmental impact index of the cascade option is built up in two steps:

- an impact index for the cascade option on each synthesis component (representing the impact of the group of projects on the component);
- an impact index for the cascade option on the environmental system (representing its aggregate impact index for all the synthesis components).

### Formulating the negative socioenvironmental impact index of cascades on synthesis components

The negative socioenvironmental impact index of each cascade option on each synthesis component should represent the impact of the group of projects that make up the cascade option on the synthesis components in the study area. The impact processes inherent to the group of projects that affect a given sub-area must therefore be considered, including the cumulative and synergistic effects amongst the projects.

The negative socioenvironmental **impact index of each cascade on each synthesis component** in the study area (**IAC**) is obtained from the weighted sum of the impact indexes relative to the sub-areas, which are attributed using the procedures described in item 5.4.:

$$IAC = \Sigma\, I_{SAi} \times P_{SAi} \quad (5.8.2.01)$$

where:

| | |
|---|---|
| $I_{SAi}$ | negative socioenvironmental impact index on a component relative to the group of projects affecting sub-area i; and |
| $P_{SAi}$ | weighting factor relative to each sub-area i, for a given synthesis component. |

In order to keep the **IAC** between zero and one, the weights $\mathbf{P_{SAi}}$ must also be attributed on a continuous scale from zero to one, with the sum of the weights relative to the sub-areas equalling one, per synthesis component.

The weights are used to ensure the relativization of the impact indexes calculated for the sub-areas within the context of the study area. The weighting factors are the same as those established per synthesis component in the Preliminary Studies, taking into account the significance of the socioenvironmental processes existing in each sub-area. Note that the consolidation of the diagnosis (item 5.2) may lead to a review of the weighting factors used.

As a function of the specificities of each synthesis component and each study area, different procedures can be adopted to systematize this weighting amongst sub-areas.

A table can be prepared to calculate the index, as shown below.

**Table 5.8.2.01 – Negative socioenvironmental impact index of a cascade on a synthesis component**

| Sub-area (Weight) / Projects | I (0.07) | II (0.08) | III (0.18) | IV (0.12) | V (0.25) | VI (0.30) | |
|---|---|---|---|---|---|---|---|
| A | | x | | | | | |
| B | | x | x | x | | | |
| C | | | x | | x | | |
| F | | x | | | | | |
| G | x | | | | | | |
| H | x | | | | x | | |
| I | x | x | x | x | x | | |
| M | x | | | | | x | |
| N | x | | x | | | | |
| $Q_2$ | | | | | | x | |
| $I_{SAi}$ | 0.65 | 0.55 | 0.95 | 0.20 | 0.40 | 1.0 | IAC |
| $I_{SAi} \times P_{SAi}$ | 0.045 | 0.044 | 0.171 | 0.024 | 0.10 | 0.30 | 0.684 |

When it comes to the Indigenous Peoples synthesis component, since the spatial unit of analysis is the whole study area, there are no factors used to weight sub-areas. The **IAC** should, then, represent the impact of the group of projects from the cascade in question on this component.

Formulation of a negative impact index for a cascade on the environmental system

A cascade's negative socioenvironmental impact index on the environmental system (IA) should express its total negative socioenvironmental impact on the study area. In other words, it should consider the negative socioenvironmental impacts caused by the cascade on all the synthesis components.

This index is the weighted sum of the impact indexes for the cascade on the synthesis components (IAC) as calculated previously.

$$IA = \Sigma\, IAC_i \times P_{ci} \quad (5.8.2.02)$$

where:

| | |
|---|---|
| $P_{ci}$ | weighting factor for each synthesis component. |

In order to keep the **IA** values between zero and one, the weights $\mathbf{P_{ci}}$ must also be attributed on a continuous scale from zero to one, with the sum of the weights relative to the components equalling one.

The weights are used to ensure the relativization of the impact indexes calculated for the cascade on the synthesis component within the environmental context of the study area. The weighting factors should represent the relative importance of the impact processes of each syntehsis component on the environmental system, which can be measured by the repercussion of these processes on the other components.

The factors are the same as those established in the Preliminary Studies, and may, if necessary, be reviewed as a function of the systemic analysis of the impact processes of the different components undertaken during the Final Studies.

## 5.8.3 Positive Socioenvironmental Impact Index (IAp)

The positive socioenvironmental impact index of a cascade option (IAp) should express the intensity of a positive impact on the study area associated with the building of the group of projects in that cascade. The purpose of obtaining this index is to take account of positive socioenvironmental impacts in the final selection of the best cascade.

The positive socioenvironmental impact index is formulated in two steps:

- a positive socioenvironmental impact index of a cascade option relative to each element in the environmental system selected for evaluation;
- a positive socioenvironmental impact index of a cascade option for the study area (the aggregate of the indexes for each element).

### Formulating the positive socioenvironmental impact index of a cascade on each element selected

The positive socioenvironmental impact index of each cascade option relating to each element (IAE) should represent the impact of the group of projects that make up the cascade on the element in question in the study area.

### Formulating the positive socioenvironmental impact index of a cascade on the environmental system

The positive socioenvironmental impact index of a cascade on the environmental system (IAp) should express its total positive socioenvironmental impact on the study area. In other words, it should consider the positive socioenvironmental impacts caused by the cascade on all the elements.

This index is the weighted sum of the positive socioenvironmental impact indexes for the cascade on each element (IAE) as calculated previously.

$$IAp = \Sigma\, IAEi \times Pei \qquad (5.8.3.01)$$

where:

| | |
|---|---|
| Pei | weighting factor for each element. |

In order to keep the IAp values between zero and one, the weights $P_{ei}$ must also be attributed on a continuous scale from **zero to one**, with the sum of the weights relative to the components equalling one.

The weights are used to ensure the relativization of the positive socioenvironmental impact indexes calculated for the cascades relating to each element in the study area. These weights should represent the relative importance of the impact processes of each element on the environmental system, which can be measured by the repercussion of these processes on the other elements.

## 5.8.4 Selection of One Cascade

First, the cascade options should be compared by means of a graphic representation, where, as in the Preliminary Studies (item 4.11.3), one of the axes represents the cost/energy benefit index and the other the negative socioenvironmental impact index. The best cascade will be chosen from the points near the lower left-hand corner of the graph.

One criterion for ranking the cascades is to use a **preference index**, ***I*** , which is the weighted sum of the cost/energy benefit indexes and negative socioenvironmental impact indexes, taking care first to standardize the cost/energy benefit index by dividing it by the reference unit cost, CUR (item 2.6):

$$I = p_{cb} \times \frac{ICB}{CUR} + p_{an} \times IAn \qquad (5.8.4.01)$$

where:

$p_{cb} + p_{an} = 1$

$p_{cb} \geq 0$ $\quad$ $p_{an} \geq 0$ $\quad$ $p_{ben} \geq 0$

where:

| | |
|---|---|
| $p_{cb}$ | weight to reflect importance relative to the objective of “minimizing the cost/energy benefit index”; |
| ICB | cost/energy benefit index, in R$/MWh; |
| CUR | reference unit cost, in R$/MWh; |
| $p_{an}$ | weight to reflect importance relative to the objective of “minimizing the negative socioenvironmental impact index”; and |
| IAn | negative socioenvironmental impact index. |

In order to rank the cascades as part of a multi-objective approach, the weights $\mathbf{p_{cb}}$ and $\mathbf{p_{an}}$ should be designated taking into account not only the opinion of the experts directly involved in the studies, but also the input given and opinions expressed at the technical meeting held at the end of the Preliminary Studies (item 2.9), so as to reflect the broader context of the analysis and the period during which the studies are undertaken.

Before the final selection of one cascade is made, an additional analysis is recommended, incorporating the positive socioenvironmental impacts on the study area, as represented by the positive socioenvironmental impact indexes, IAp, to the ranking already undertaken.

The closer this index is to **one**, the better the circumstances of the cascade in question with respect to this requirement. In the case of the negative socioenvironmental impact index and cost/energy benefit index, the opposite applies, i.e. the closer the index is to **zero**, the better the cascade in question. In order to aggregate the **IAp** with the preference index ***I***, the IAp complement is introduced to bring the scale of the positive socioenvironmental impact index into line with the other indexes, i.e. **(1 – IAp)**. The **modified preference index *I′*** is given by:

$$I' = (1 - p_{ap}).I + p_{ap}\left(1 - IA_p\right) \qquad (5.8.4.02)$$

where: $0 \leq p_{ap} \leq 1$

| | |
|---|---|
| $p_{ap}$ | weight to reflect the relative importance of the positive socioenvironmenal impacts; and |
| IAp | positive socioenvironmenal impact index. |

The weight $\mathbf{p}_{ap}$ should be defined in the same way as described for weights $\mathbf{p}_{cb}$ and $\mathbf{p}_{an}$. Given that the selection of the best cascade must take account of three objectives, and that the objective of maximizing positive impacts is supplementary to the other two, it is suggested that this latter's weight ($\mathbf{p}_{ap}$) should not exceed 0.25.

Sensitivity analyses should be undertaken and presented for the values given to the three weights ($\mathbf{p}_{cb}$, $\mathbf{p}_{an}$, $\mathbf{p}_{ap}$).

The SINV system can be used to calculate the cost/energy benefit index by using the "Economic-Energy Assessment" function, while the negative and positive socioenvironmental impact indexes can be calculated using the "Calculate Socioenvironmental Impact" function, and the preference and modified preference indexes can be obtained using "Final Multiobject Analysis". This last function is also used to rank the cascades by the two preference indexes, allowing the most advantageous cascade to be identified and sensitivity analyses to be undertaken on the values given to weights $\mathbf{p}_{cb}$, $\mathbf{p}_{an}$, $\mathbf{p}_{ap}$, in order to ensure the robustness of the cascades identified.

# 5.9 SEQUENCE OF CONSTRUCTION OF THE PROJECTS IN THE FINAL SELECTED CASCADE

The studies undertaken at the Inventory stage for defining the sequence of construction of the projects in a cascade option from an exclusively economic perspective are based on the criterion of incremental costs (item 5.9.1). According to this criterion, when considering two projects, the one with the lower incremental cost should be constructed first.

## 5.9.1 Incremental Cost

The incremental cost of a project or group of projects is calculated in much the same way as the cost/energy benefit index is (item 4.11.1). The only difference is the way the firm energy contribution is calculated, which must now assume that the projects already built are those that actually do exist plus those projects that, according to the sequence of construction for the cascade under analysis (item 5.9.2.), are set to be built before the project under analysis.

The "Eliminate" function of the SINV system uses the incremental cost to determine which projects from a cascade option have a ICB that is higher than the reference unit cost (CUR) and which should therefore be eliminated from the cascade.

## 5.9.2 Sequence of Construction from an Economic Perspective

The sequence of construction of the projects in a cascade option, taking into account only economic factors, is obtained by putting the projects from the cascade in increasing order of incremental cost. As calculating these incremental costs depends on knowing the sequence of construction, the sequence must be defined by an iterative process.

Starting out with a blank sequence, the cost/energy benefit indexes of each project and group of projects yet to be included in the sequence are calculated at each stage, assuming the pre-existence only of those projects that already exist and those that have already been included in the sequence, then choosing the project or group of projects with the lowest index to be added next. The process ends when all the projects from the cascade have been included in the sequence of construction.

One way to reduce the time taken to do this is to carry out the iterative process described above, first checking the outcome of adding the new projects one by one. If the resulting list of incremental costs is monotonically increasing, the sequence of construction can be considered complete. If not, the sequence of the series projects with decreasing marginal costs must be redone, testing the simultaneous addition of two or more projects until a complete list of monotonically increasing incremental costs is obtained.

The "Sequencing" function from the SINV system can be used to sequence the projects from the final selected cascade based on the incremental cost of each project.

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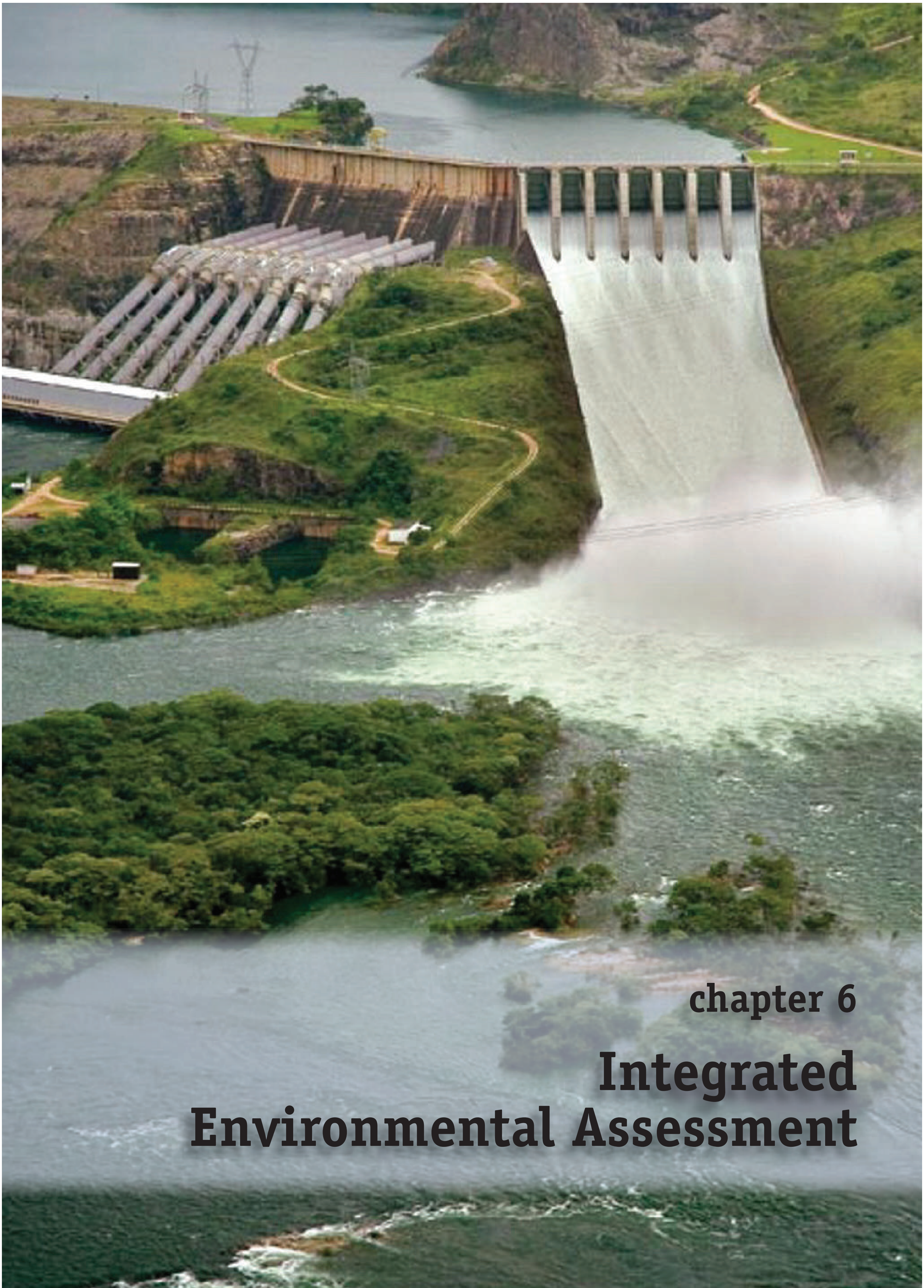


chapter 6

# Integrated Environmental Assessment

# CHAPTER 6

Inventory Studies and Integrated Environmental Assessments (IEA) both have the objective of analyzing river basins, with different yet complementary objectives. While in Inventory Studies the focus is on comparing and selecting the best cascade option for harnessing the hydroelectric potential of the basin, in an Integrated Environmental Assessment the aim is to evaluate the state of the natural and human environments in terms of their capacity to receive the group of hydropower projects that would make up the final cascade selected.

The main focus of the IEA is to assess the status of the environment in the river basin as a consequence of building the group of existing or planned projects. The cumulative and synergistic effects relative to this group of projects are assessed, considering different development scenarios for the river basin and taking into account the period over which the projects are to be built. Guidelines and recommendations are also put forward as inputs for the design and building of the projects and the environmental licensing process, with a view to assuring the socioenvironmental sustainability of the region as measured by sustainability indicators formulated within the ambit of the study.

In order to integrate the IEA procedures with the methodology used in the socioenvironmental studies undertaken as part of the process to select the best cascade option in the Inventory Studies, a number of additions were made to this edition of the Manual, in both the Preliminary and the Final Studies.

In order to meet the requirements of the IEA and make it possible for the socioenvironmental studies undertaken within the ambit of this Manual to be transformed into a stand-alone document to provide information for environmental entities for the future licensing of the projects, the studies relating to the final cascade are consolidated at this stage, highlighting the following:

- the most significant socioenvironmental aspects for the river basin;
- the results of the assessment of cumulative and synergistic effects brought about by the group of projects related to the different synthesis components and to the aspects selected for assessing positive impacts;
- the areas of environmental fragility and socioeconomic potentiality resulting from the building of the projects for the final cascade selected, considering the scenario of future development formulated for the river basin and assuming all the projects are built.

Guidelines and recommendations should also be prepared for the design and building of the future projects as a function of the issues highlighted in the analyses, with a view to assuring the socioenvironmental sustainability of the river basin and reducing the risks and uncertainties inherent to the process of harnessing its hydroelectric potential.

# 6.1 OBJECTIVE

The aim of this stage is to supplement and consolidate the socioenvironmental studies undertaken, supplying a broad picture of the future socioenvironmental status of the river basin once the projects in the final cascade have been built, considering:

- their cumulative and synergistic effects on the natural resources and human groupings;
- the current and potential uses of the water resources for current and future planning purposes, aiming to ensure that the generation of electricity is compatible with the conservation of biodiversity; and
- social diversity and socioeconomic development trends in the river basin.

In situations where the inventory studies for a river basin are being reviewed, those projects already in service must also be taken into account, as must those for which a concession or authorization has already been granted by ANEEL.

The other objectives to be met are:

- to develop sustainability indicators for the river basin, focusing on the water resources and their use for energy generation;
- to demarcate areas of fragility and identify the socioeconomic potentialities that could be leveraged by the building of the hydropower projects;
- to indicate conflicts over the different uses of the land and water resources in the river basin;
- to set down socioenvironmental guidelines and recommendations for the Feasibility Studies of the projects in the final cascade; and
- in the future, the guidelines and recommendations should serve as inputs for: (i) environmental studies in the river basin; (ii) the environmental licensing processes for the projects; (iii) any adjustments to projects or programs; (iv) procedures associated with the expansion of electricity supply; and (v) the building of the hydropower projects in the river basin, such that the risks and uncertainties associated with socioenvironmental development and the harnessing of energy in the basin are kept to a minimum.

## 6.2 STAGES OF THE IEA

The original methodology for IEAs[1] was structured into a number of steps, as shown below:

- Socioenvironmental diagnosis and potential conflicts – this aims to build up an overall picture of the river basin so that the most significant socioenvironmental elements in the current situation and their likely trends can be identified and located. Any existing or potential conflicts that would be exacerbated by building the hydropower projects must also be identified, as well as any conflicts or synergies with policies, plans or programs for the region.
- Distributed Environmental Assessment – this aims to subdivide the river basin into areas with features in common or which stand out from the others, so that the impacts from one or more projects in their vicinity can be identified and assessed, from which a picture of the combined effects on each of them and the effects that extrapolate their boundaries can be built up. In order to assess the impacts, indicators should be used that allow them to be quantified or qualified for the scenarios for different periods in time. For each subdivision, the indicators must be weighted to ensure that the local impacts are ranked by importance. Next, they should be mapped out so that the most sensitive areas can be identified. The local effects capable of leveraging cumulative or synergistic effects with other subdivisions must also be identified.
- Building up scenarios of socioeconomic development, taking into account the state of conservation of the natural resources, with mid- and long-term time frames.
- Integrated Environmental Assessment – at this stage, the interaction between the effects of the different hydropower projects, and between the different variables that characterize the socioenvironmental impacts deemed most significant, is assessed using indicators and simulation models based on the scenarios already built up. The indicators should be such that they permit an analysis to be made of the cumulative and synergistic impacts on the future scenarios. Guidelines and recommendations should be drawn up as a result of this process.
- Public Consultation – with a view to ensuring the involvement of the public throughout the studies, allowing stakeholders to participate and give their feedback on the results, and also to collect inputs and information for the studies themselves, meetings must be held to present, discuss and make contributions to the partial and final results of the IEA. The meetings must be held at venues in all the different states that the river basin occupies.

When this Manual was published, IEAs were a relatively recent addition to the electricity sector planning process, the first having been undertaken in 2005. Between 2006 and 2007, six IEAs were carried out. The procedures were still being developed, with the goal of future consolidation. Further information on these procedures can be obtained from Annex F, which provides a summary of the methodology used in three of the IEAs[2].

The methodology used in the IEAs is consistent with that used in the socioenvironmental studies undertaken in the Preliminary and Final Studies, described in chapters 4 and 5, not only in terms of the procedures used but also the content and scope, despite the difference in the focus of the studies. In order to ensure that the Inventory Studies and IEA are mutually compatible, throughout the description of the procedures for the socioenvironmental studies required for the selection of cascade options, procedures have also been introduced to meet the requirements of the IEA. The procedures for consolidating the studies are set out below.

1 Termo de referência para a Avaliação Ambiental Integrada para a bacia do Rio Uruguai, MMA, 2005.

2 Methodology developed by EPE/Sondotécnica (2007) for the IEAs of the Paranaíba, Doce and Paraíba do Sul river basins. Other examples of methodologies can be found on the EPE website (http://epe.gov.br/Lists/MeioAmbiente/MeioAmbiente.aspx).

## 6.3 INTEGRATION OF THE SOCIOENVIRONMENTAL STUDIES WITH THE IEA

Some of the activities required to meet the objectives of the IEA are already included in the environmental studies at the Planning stage, the Preliminary Studies and the Final Studies, as shown below:

- Planning (chapter 3): establishing a communication channel for the presentation of information about the studies to be undertaken in the river basin with the environmental and river basin entities;
- Preliminary Studies (chapter 4): Data Gathering and Studies (item 4.1), Socioenvironmental Diagnosis (item 4.3) and Assessment of Negative Socioenvironmental Impacts per Project (item 4.8). At the end of the Preliminary Studies, a technical meeting is held by the Ministry of Mines and Energy to present the findings of the studies (item 2.9);
- Final Studies (chapter 5): Consolidation of the Socioenvironmental Diagnosis (item 5.2), Assessment of the Socioenvironmental Impacts of the Projects (item 5.4) Comparison and Selection of Cascades (item 5.8).

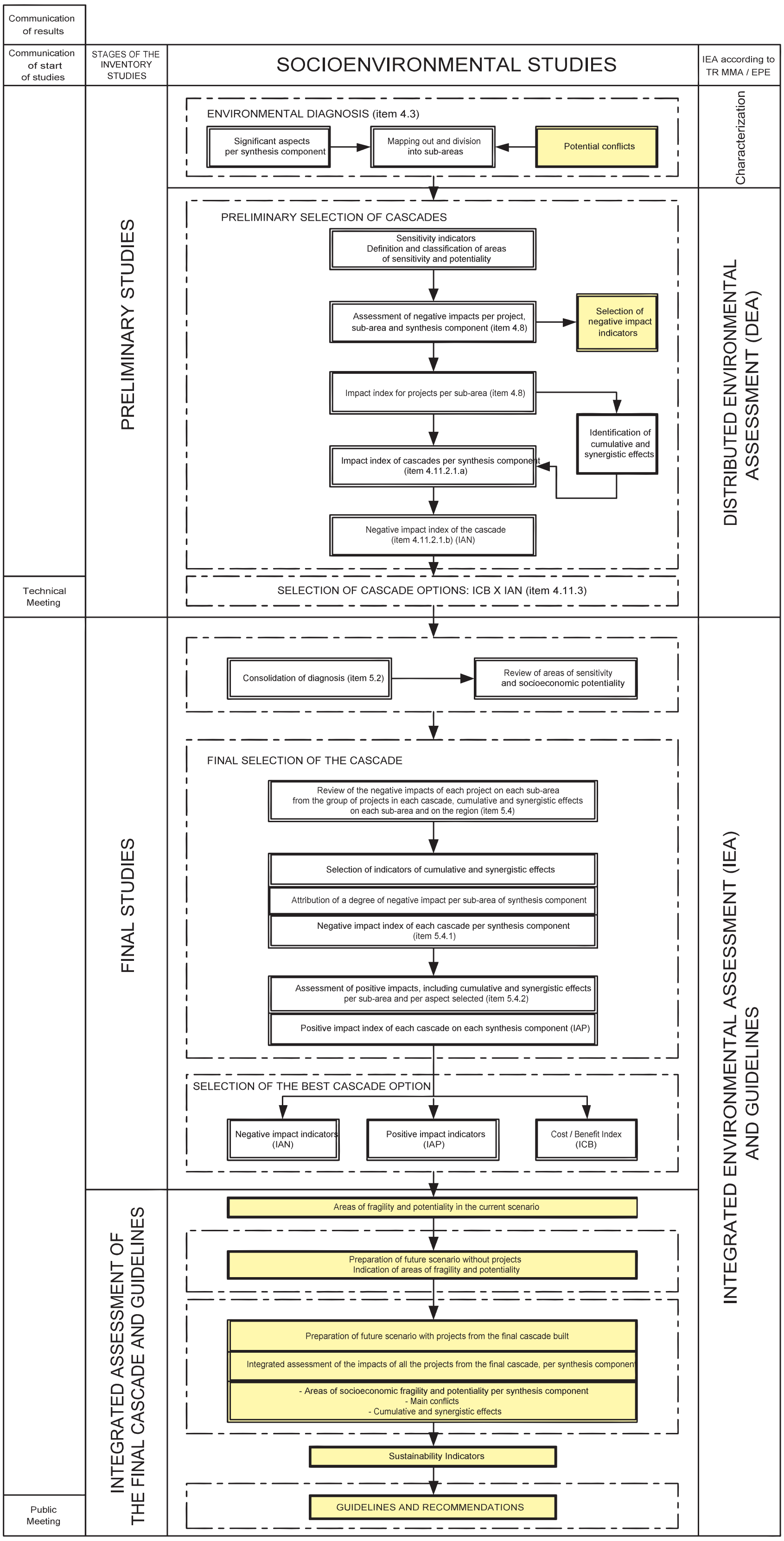
However, in addition to these, further studies are also required to supplement the integrated environmental assessment of the final cascade selected.

The diagram in Figure 6.3.01 shows how the socioenvironmental studies from the previous stages are integrated with the supplementary studies for the IEA. The activities required for the main items in the IEA are highlighted.

As already mentioned, the IEA of the final cascade can be used as a stand-alone document, separate from the Inventory Studies report. Below are all the activities needed to do this, which are split into two sections:

- organization and presentation of the information concerning the final cascade contained in the Preliminary and Final Studies, following the structure set forth for the IEA;
- supplementary activities to fulfill the scope of the IEA.

Figure 6.3.01 – Diagram of integration between the socioenvironmental studies from the Inventory Studies and the IEA



# 6.4 ORGANIZATION OF INFORMATION FROM PREVIOUS STUDIES

## 6.4.1 Environmental diagnosis and potential conflicts

Taking the data and information consolidated in the Socioenvironmental Diagnosis (items 4.3 and 5.2), a brief description should be prepared of the main characteristics of the river basin and the most representative socioenvironmental indicators, giving precedence to the most significant socioenvironmental issues for each synthesis component, the areas of environmental management and the main existing and potential conflicts relating to the harnessing of the hydroelectric potential of the river basin. This description should be illustrated with maps of each synthesis component and their respective sub-areas, with details given of the criteria upon which these subdivisions were based, as well as their main characteristics.

The local and regional conflicts identified in the course of the study must be identified and located on the maps, and associated with the projects and the cascade, while identifying the main parties involved.

## 6.4.2 Main characteristics of the final cascade

A brief description should be provided of the final cascade, accompanied by a map showing where each of its projects is located. The main energy-related features of each project must be included: installed capacity, water levels, regulating volume, and information on the other uses of the waters.

## 6.4.3 Distributed Environmental Assessment (DEA)

### Areas of environmental sensitivity and socioeconomic potentiality

The environmental sensitivity and socioeconomic potentiality indicators must be included, as well as the variables that comprise them and their respective values. The areas of sensitivity and potentiality in the sub-areas for each synthesis component should also be mapped out and classified.

### Main impacts of the projects and the cascade

The negative socioenvironmental impacts of each project on each synthesis component should be presented per sub-area, as identified and assessed in the Preliminary Studies (item 4.8) and reviewed in the Final Studies (item 5.4.1). The impact indicators used and impact indexes attributed to the projects must also be presented.

The main negative and positive impact processes arising from the cascade and affecting each synthesis component and sub-area, as identified and assessed in the Final Studies (items 5.4.1 and 5.4.2), should be presented, as should their cumulative and synergistic effects, highlighting the respective indicators, assessment criteria, negative and positive impact indexes per cascade, and the weights used to formulate the indexes.

# 6.5 SUPPLEMENTARY ACTIVITIES FOR THE IEA

In this section, the procedures for conducting the supplementary activities are described.

## 6.5.1 Areas of fragility and potentiality in the current scenario

The areas of fragility[3] in each synthesis component must be identified and mapped out. These are the areas of sensitivity where the main impact processes arising from the projects in the cascade are located. The areas of fragility are identified by crossing the maps of the areas of sensitivity for each synthesis component with the spatial distribution of the impacts arising from the group of projects per synthesis component[4]. The areas of socioeconomic potentiality must also be identified.

## 6.5.2 Preparation of reference scenario

In order to analyze the final cascade, a long-term scenario of socioeconomic development (20 years) must be built up for the region, to be used as a reference for the analyses. It should take into account the state of conservation of the natural resources, but not the harnessing of the hydroelectric potential proposed for the final cascade. In situations where the inventory studies for a river basin are being reviewed, only those projects already in service and those for which a concession or authorization has already been granted by ANEEL should be considered.

In order to build up the reference scenario, projections should be made of how the group of indicators of economic, social and environmental conditions in the study area and the institutional organization of this area will change over the time frame established. The scenario must be compatible with the scenario created to forecast multiple water uses (item 5.1.3).

The most significant socioenvironmental issues and the topics deemed of priority identified in the previous studies, their future trends and their spatial distribution should be used in the identification and location of the main pressures from socioeconomic development on the river basin: on the water and other natural resources; on land use, especially in the areas of sensitivity and potentiality, in the areas with restrictions on use and areas of environmental management; and on the local population's ways of life and the land organization and dynamics.

The following must be considered:

- the economic, social, environmental and cultural indicators that best represent the issues deemed of relevance in the study area;
- the scenarios of water uses in the river basin considered in item 5.1.3;
- development policies, plans and programs for the study area that cover the same time frame, and their main interactions;
- existing and potential conflicts;

3 The definition of "fragility" used here is as follows: "fragility of the environment is understood as meaning the degree of susceptibility to damage as a result of given actions, and can also be defined as the inversion of the capacity to absorb potential alterations without any loss of quality," (Angel Ramos, cited in Iara Verocai, FEEMA/PETROBRÁS, 1990).

4 The procedure for identifying areas of fragility is based on the methodology used in the IEAs for Doce and Paranaíba river basins. (Sondotécnica, 2007).

- the government's policies for the environment, water resources, social development, and agricultural and industrial development, and all policies relating to international agreements, existing environmental quality standards, and standards established by other instruments, such as zoning;
- the risks and trends for future deterioration in socioeconomic and environmental conditions.

A synthesis map should be prepared that represents the socioenvironmental status in the river basin over the time frame established, indicating the areas of environmental sensitivity and socioeconomic potentiality in the absence of the projects.

## 6.5.3 Integrated environmental assessment of the effects of building all the projects in the final cascade

### Future scenario with all the projects from the cascade built

Based on the reference scenario, a scenario should be developed that assumes that all the projects included in the final cascade are built in 20 years.

### Integrated analysis of the final cascade

The aim of this stage is to present the results of the assessment of the effects of building the projects that make up the final cascade, including the cumulative and synergistic effects, considering the future scenario for development in the region. The main impacts of the cascade on the synthesis components should be identified, as should those resulting from the interactions between the components. The impact indicators used to represent these processes should be listed. The most significant socioenvironmental processes in the river basin under this future scenario should be highlighted and mapped out, emphasizing those that should be addressed in the design and building of the future projects.

The most sensitive aspects of the environmental system and for each synthesis component should be highlighted and the extent of their influence identified. All areas of fragility for each synthesis component should be identified and mapped out, which are obtained by crossing the areas of sensitivity identified for each synthesis component with the area to be impacted by the group of projects per synthesis component. The areas of socioeconomic potentiality should also be identified.

A synthesis map representing the status of the river basin under the forecast scenario should be prepared. The integrated analysis of the final cascade should highlight:

- the areas of fragility relating to the most significant impacts arising from the group of hydropower projects;
- the areas of socioeconomic potentiality;
- the areas where the most significant cumulative and synergistic effects are identified;
- existing and potential conflicts, such as:
  - conflicts over the way the urban and rural populations are resettled;
  - changes in land use, breakdown of social relations and production base;
  - property speculation;
  - interference in archaeological, historical and cultural heritage;
  - areas with conflicts over land use;
  - interference in natural resources for development;
  - loss of tourism potential;
  - loss of natural resources (minerals, biodiversity);

- conflicts over the multiple uses of water resources (navigation, energy generation, withdrawals for human and livestock water supply, dilution of wastewaters, irrigation and flood control); and
- interference in indigenous lands and federal, state and municipal conservation areas.

### 6.5.4 Formulation of socioenvironmental sustainability indicators for the region

Sustainability indicators are formulated for the river basin with the purpose of setting reference parameters for the guidelines and recommendations to be proposed in the IEA.

The formulation of the environmental sustainability indicators must draw on the socioenvironmental conditions set out in the current and prospective scenarios, and are represented by the fragility and potentiality indicators relating to the synthesis components.

These conditions are crossed with the data and information contained in federal, state and municipal legal provisions and standards concerning environmental preservation/conservation, such as ecological/economic zoning, legislation on land use and occupation, regulations on conservation areas and the use of water resources, river basin plans, and policies or plans for the social and economic development of the region. Aside from these references, national and international scientific data on socioenvironmental sustainability, and perceptions of the regional community's aspirations for their living conditions and environmental conservation in the study area should be included amongst the reference elements used to formulate the socioenvironmental sustainability indicators for the river basin.

### 6.5.5 Guidelines and recommendations

The analyses undertaken should provide the data needed to set down guidelines, which should have the following goals:

- to serve as inputs for the socioenvironmental assessments of projects to be analyzed within the ambit of Ten-Year Electricity Sector Expansion Plans;
- to contribute towards the consolidation of a georeferenced information system which, once expanded and made operational, could become a key planning and environmental management tool for the river basin, serving not only the needs of environmental entities, but also, and more importantly, those of the communities living there;
- to contribute towards the design and building of hydropower plants, taking into account the main issues identified, land use and occupation, regional development, critical areas and potentialities; and
- serve as a guide for future environmental studies for hydropower projects and environmental licensing processes for projects at their planning stage or being licensed by the competent environmental authorities.

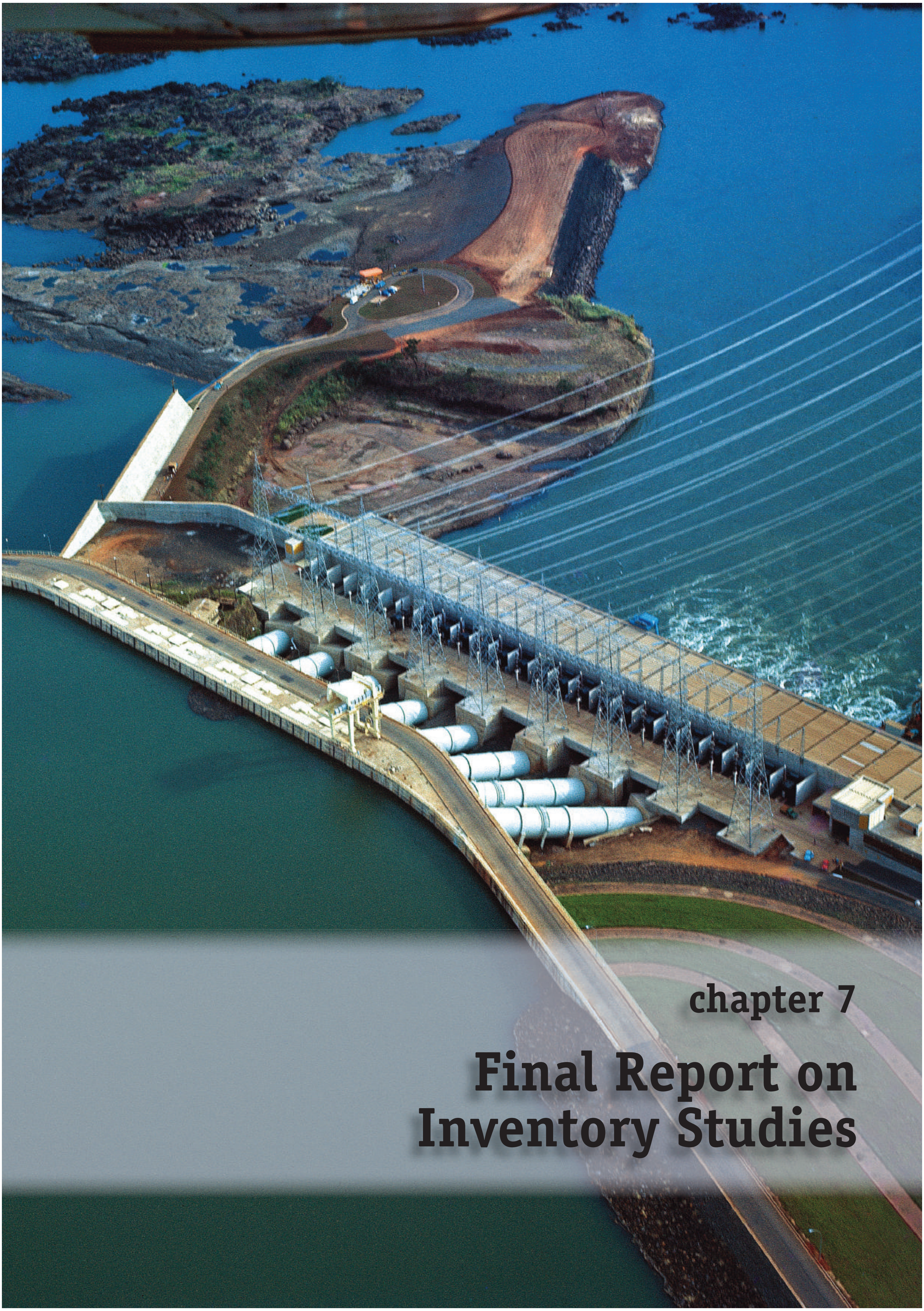
Recommendations should also be made to:

- provide more in-depth or supplementary information for the IEAs when their results contain a high level of uncertainty as to the reliability or suitability of the data and information used, and for undertaking future environmental studies of interest to the electricity industry;
- foster and develop activities to promote integration in the river basin, taking into account the multiple stakeholders in the use of the waters and lands, and the different public and private entities operating in it;

- supplement Feasibility Studies for future projects; and
- assist in the building of the projects.

### 6.5.6 Final Communication of the Studies

At the end of the studies, a public consultation meeting is held by the Ministry of Mines and Energy to present the results of the final cascade selected, as well as the IEA, its guidelines and recommendations.



chapter 7

# Final Report on Inventory Studies

# CHAPTER 7

The Final Report is the document that presents the Final Studies of the Hydropower Inventory Studies of a river basin, the results obtained and the recommendations made for the Feasibility stage. It should include the energy potential of the river basin and the scheme proposed to harness this potential, while also containing a characterization of the projects, the main socioenvironmental issues associated with them, the key points highlighted in the Integrated Environmental Assessment, and the socioenvironmental guidelines and recommendations for the building of the projects, which are also to be used in the Feasibility Studies and licensing processes. Below, basic recommendations are set out to ensure the standardized presentation of the contents and results of Hydropower Inventory Studies.

# 7.1 ORGANIZATION OF THE REPORT

The Final Report on Hydropower Inventory Studies should comprise a General Report and its Appendices, which should be printed in as few volumes as possible.

Ideally, the information from the General Report should be organized into the sections listed at the end of this chapter. The appendices should contain a description of the studies undertaken, in order to characterize them in greater depth and breadth. Depending on their size, the appendices can be compiled and printed in separate volumes.

The volumes should be printed in A4 format (ABNT). Any figures that cannot be adapted to this format can use A3.

The text of the General Report and its appendices should be organized in line with the guidelines set out in Brazilian standard (ABNT) NB-69. It should also meet the requirements of the regulation entity and should be submitted in a hard copy and in digital form.

# 7.2 GENERAL REPORT

## 7.2.1 General information

The General Report should be bound with a letter of presentation, by which the executive entity submits the Final Report for the appreciation of the relevant authority.

This letter should be followed by the title page, which should contain the following minimum information: title of the report, name of the executive entity, and issue date.

Next should come the contents and the lists of appendices, tables, and figures. The contents should contain first- and second-level section headings. The lists of figures and tables should be numbered according to the first-level sections they are part of.

The foreword should provide a presentation of the work. As a rule, it should be brief and should identify or highlight the main characteristics of the studies, the results obtained and the means employed to obtain them. Normally, the foreword will include the references and credits to individuals and organizations that did not participate directly in the work, but still made significant contributions.

After the foreword comes an abstract, containing excerpts from the most important parts of each chapter, providing an overview of the studies undertaken and conclusions reached. The abstract may only contain information from the main text of the report; it cannot include any information from other sources. Some schematic diagrams or condensed tables can be included in the abstract, provided they are pertinent. A compilation of the summarized table of data from the Final Studies must be included in the abstract.

It is common practice to begin the abstract with a brief description of the aims of the study, the location of the river basin, the amount of energy to be harnessed, the number of projects identified in the Final Studies, the total estimated cost of the final cascade selected, and the corresponding unit costs, as well as the information about the socioenvironmental studies and other pertinent information.

The data that has been gathered, analyzed and processed should be described in the General Report or in specific Appendices. In order to make this information available for future use, it should also be submitted in digital form. The formats of the digital files containing the data used and results obtained are available from the concession-granting authority. The use of the correct format ensures that the data can be transferred directly to the database, where all information from Inventory Studies is stored by this authority. This information should include the maps and georeferenced data used or created in the Inventory Studies.

## 7.2.2 Introduction

The introduction should contain the following sections:

- aim of the studies;
- characterization of the area studied;
- previous studies; and
- basic criteria.

In defining the aim of the studies, the reasons behind the execution of the studies must be presented, and the scope of the studies of each of the main rivers and their tributaries must be justified. The recommendations made in the Preliminary Studies must also be included, as should the means by which they were addressed in the Final Studies.

The study area should be characterized by its geographical location, physiography, political and administrative organization, human occupation and economic activity, both current and potential, as well as its main socioenvironmental issues. Maps and graphics should be included in this section to illustrate the points made.

Any prior studies made of the river basin or contiguous areas should be described critically, noting the results obtained, the basic criteria and the methodologies used.

Finally, the basic criteria selected for use in the Inventory Studies should be listed and justified. These criteria correspond to the energy, economic, multiple water use and socioenvironmental parameters employed to define, compare and select the different cascade options. Any variations between the methodology used and that set forth in this manual must be justified and described.

### 7.2.3 Planning

This section should contain a summary of the Planning report, using the sub-items suggested in the Format of the Final Report provided at the end of this chapter.

The basic data gathered, analyzed and processed throughout this and the successive stages of the Inventory Studies should be presented once only, in item 4.2 of the Final Report. This should include a summary of communications with environmental entities and water authorities to notify them about the studies planned.

### 7.2.4 Preliminary Studies

A brief description should be provided of the Preliminary Studies undertaken, based on the list of items proposed. It should also include a summary of the issues highlighted at the technical meeting held at the end of this stage.

As suggested in the previous item, in order to present the information in a more reader-friendly fashion and avoid repeating the same information, all the data gathered, analyzed and processed at this stage and studied in greater depth in the Final Studies should be presented in detail or in summary form, and included in the Appendices in question, and in Section 4, Final Studies.

### 7.2.5 Final Studies

This section contains the consolidated results of the Final Studies, which should be organized into the items suggested below, highlighting the key aspects noted from each section:

#### Cartographic Studies

Description of the aerial and land reconnaissance activities, the topographic and aerial photogrammetric surveys, and the other surveys undertaken to determine the cartographic parameters, following the procedures set forth in this manual. The cartographic knowledge of the study area achieved by the end of the studies should be illustrated in map form. If necessary, details of the studies may be presented in a corresponding Appendix.

### Geological and Geotechnical Studies

Description of the studies undertaken to identify the general geological features of the river basin, and the geology of the reservoirs and dam sites. A general geological map of the river basin should be included, as should a schematic cross-sectional diagram of each site, indicating the main lithological and stratigraphic features identified.

### Hydrometeorological Studies

Description of the analyses undertaken to test the consistency of the hydrometeorological data, and other studies to determine the hydrological parameters to be used in the Inventory Studies, as well as the climatology, sedimentology and water quality investigations and studies undertaken. Figures giving a concise representation of the basic hydrological and climatological information should be included in this section, such as isohyetal maps, maps of drainage areas, hydrographs from the main gauging stations, maps showing the regional extent of hydrological features, including long-term minimum and mean flows, minimum and mean flows for the critical period of the reference system, flood flows for diversion, design flood flows for spillways, sediment transport rates, and others. The basic gauging station network and the equations used to transfer the fluviometric information in terms of time series of flows, as well as the other hydrological data of relevance from the gauging stations used for the project sites should all be presented clearly. The details of the studies should be included in the corresponding Appendix.

### Socioenvironmental Studies

Description of the investigations and studies undertaken to build up an understanding of the environmental system, represented by the aquatic and terrestrial ecosystems, ways of life, territorial organization, regional economy and indigenous peoples; preparation of the socioenvironmental diagnosis, and division of the study area into sub-areas; assessment of the negative and positive socioenvironmental impacts and indexes of the cascades, and the weights used. This section should also contain maps of the synthesis components with the sub-areas marked on them, and a presentation of the areas of sensitivity and potentiality. The environmental assessment of the cascades studied and the projects from the final selected cascade should be presented clearly, with the corresponding Appendix containing the details of these and the other studies undertaken.

### Studies of Multiple Water Uses

The diagnosis of the multiple water uses in the river basin should be presented, as undertaken in the Preliminary Studies and consolidated in the Final Studies, which provided the foundation for the scenarios of multiple water uses formulated for use in the Final Studies. The sources of the information used to formulate these scenarios should be cited and justified. The methodologies used to formulate or adapt the scenarios of multiple water uses (when they are based on sector plans) should also be described in detail.

### Studies of Cascade Options

This section should describe the cascades selected in the Preliminary Studies, and the Final Studies of these cascades and the variations derived from them. The cascades investigated in the Preliminary Studies should be described in brief. The variants resulting from new data and corresponding adjustments should also be described, as should the analysis of the conclusions reached, a list of the cascades selected to go through to the Final Studies, their reformulations and all additional studies undertaken.

For the energy studies, all the simplifications adopted should be listed and justified. When the SINV system is not used, the mathematical model used to simulate the system of power plants must be justified. A table of summarized information should be included for each cascade, indicating its respective firm energy contribution to the reference system and corresponding installed capacity. These tables should also contain a description of the projects making up each cascade: their energy characteristics (live capacity, reference head, installed capacity, etc.) and their cost/energy benefit index.

The socioenvironmental studies should contain a brief description (details to be included in the corresponding Appendix) of the assessment of the negative and positive socioenvironmental impacts of the cascades (item 5.4), highlighting their cumulative and synergistic effects.

For the final layouts, dimensioning and cost estimates, a description should be given of the methodology used to dimension the civil construction work and equipment whenever procedures are used that differ from those set out in this manual. The guiding principles used for designing the layout of the structures for the projects and for determining the quantities and costs should also be provided, making specific reference to any deviation from the criteria and instructions set out in this manual.

Finally, a general, summarized table should be included for each cascade and its respective projects, indicating their location, maximum gross head, live storage of the reservoir, reference capacity and cost estimates, using the OPE format (items 5.5 to 5.7), and including their main socioenvironmental impacts.

#### Comparison and selection of cascades

This section should contain a description of the comparative analyses of the cascades from an energy-economic and socioenvironmental perspective, as shown in item 5.8 of this manual. A summarized table should be included with the cost/energy benefit index, negative socioenvironmental impact index, preference index, positive socioenvironmental impact index and modified preference index of each cascade. This table should also set out the weights used for calculating the preference and modified preference indexes.

A sensitivity analysis of the weights used to calculate the preference and modified preference indexes should also be provided.

### 7.2.6 Characterization of the final selected cascade

This section should contain the studies undertaken of the final selected cascade, including not only the adjustments and refinements made to its projects, but also a characterization of special cases, such as multi-purpose projects, pumped storage facilities and any others identified in the studies. The final results of the Inventory Studies should be presented, consolidated in the final selected cascade, after adjustments, which should represent the most advantageous scheme for the river basin.

A general description should be provided of the cascade proposed, covering energy, geographical and socioenvironmental data. The main negative and positive impact processes associated with the cascade should also be included, highlighting all cumulative and synergistic effects.

At the end of the section, a summarized map should be presented showing the location of the projects and the longitudinal sections of the rivers indicating the maximum normal water levels in the reservoirs, and others showing the areas of resettlement and main interferences brought about by the cascade, highlighting areas of fragility and potentiality in the current scenario.

#### Characterization of the projects

Each project from the final selected cascade should be characterized in this section, including a brief description, a map showing the project's location, a general plan, longitudinal sections of the water intake, powerhouse, spillway and diversion scheme, and cross-sections of the dam axis and intakes, as well as the elevation/area and elevation/volume curves of the reservoir and the rating curve of the tailrace canal. These three curves should be accompanied by lists of the measurements that served as the basis for the plotting of the curves. A table of the mean monthly flows for each site should also be included.

The Technical Form for each project should also be included, using the template from Annex E, so that the information can be transferred to the database held by the concession-granting authority.

### 7.2.7 Integrated environmental assessment of the final selected cascade

Initially, a summary of the long-term scenario for socioeconomic development in the region should be presented, taking account of the state of conservation of the natural resources in the river basin, but not the harnessing of its hydroelectric potential by the projects in the cascade.

Next, a long-term scenario should be presented, assuming that all the projects that make up the final selected cascade have been built and highlighting their main impacts, including cumulative and synergistic effects. The most significant socioenvironmental processes should be mapped out and highlighted, as well as the most significant conflicts in the river basin for this future scenario, emphasizing those that should be addressed in the design and building of the projects in the future. All areas of fragility and socioeconomic potentiality should also be identified and mapped out.

The socioenvironmental sustainability indicators chosen for the river basin that served as a basis for formulating the guidelines and recommendations should be presented.

### 7.2.8 Conclusions and recommendations

This section should not only provide a general overview of the results of the studies, but should also make recommendations for the Feasibility Studies, including specific suggestions for additional studies and surveys. The guidelines and recommendations should be presented for the purposes of electricity sector planning, the design, building and socioenvironmental management of the projects, and the environmental licensing of future projects, established in the IEA.

### 7.2.9 Supplementary Information

At the end of the main sections of the General Report, the following should be added:

- list of the main terms used and their definitions; and
- list of abbreviations and symbols used in the texts and figures.

### 7.2.10 Database

All the information gathered, used and produced in the Inventory Studies that is cited in the previous items and included in the tables and maps must be submitted in digital form, using the format designated by the concession-granting authority. As for the maps and georeferenced information, these should be submitted in files that are compatible with the geoprocessing systems. Details of the format of the files can be obtained from the concession-granting authority.

# 7.3 APPENDICES

The appendices to the General Report should contain the details of the surveys and studies undertaken, and should present the basic data collected in an organized way so that the document can be understood clearly and fully.

The number of appendices will depend on the nature of the work undertaken, but the following will certainly have to be prepared:

- Appendix A – Topographic Studies
- Appendix B – Geological and Geotechnical Studies
- Appendix C – Hydrometeorological Studies
- Appendix D – Socioenvironmental Studies
- Appendix E – Studies into Multiple Water Uses
- Appendix F – Studies of Cascade Options
- Appendix G – Report on the Integrated Environmental Assessment
- Appendix H – Organization and Background on the Studies

## 7.3.1 Appendix A – Topographic Studies

This appendix should list the topographic data and describe the topographic surveys of the river basin, the reservoirs and the dam sites undertaken as part of the Inventory Studies.

The information and data obtained should be described critically, including its technical characteristics and the survey methodology and survey dates. Maps should be prepared indicating the areas covered by aerial photogrammetric surveys and triangulation and leveling networks. The main topographic marks should be listed, with their altitude, geographical coordinates, if available, and instructions for their future location.

## 7.3.2 Appendix B – Geological and Geotechnical Studies

This appendix should list the data and describe the geological and geotechnical investigations undertaken in the river basin, the reservoirs and the dam sites.

The geological photo-interpretations and surface and sub-surface prospecting should be described, as well as the local tests and laboratory tests carried out.

## 7.3.3 Appendix C – Hydrometeorological Studies

This appendix should list the primary hydrological and climatological data on the river basin and the analyses and studies undertaken.

## 7.3.4 Appendix D – Socioenvironmental Studies

This appendix should list the data gathered and describe the surveys and studies undertaken for the socioenvironmental diagnosis of the river basin, the impact assessments and the findings obtained, as well as the extent to which they would interfere with the final selected cascade.

## 7.3.5 Appendix E – Studies of Multiple Water Uses

This appendix should list and critically describe the studies into the different uses of the waters in the river basin, both those undertaken by third parties and those conducted during the Inventory Studies, so that a diagnosis can be prepared and scenarios can be built up for the multiple water uses.

## 7.3.6 Appendix F – Studies of Cascades

This appendix should contain a detailed description of the studies and analyses involved in formulating, assessing, comparing and selecting the cascade options.

In order to broaden the scope of the Final Report on Inventory Studies, brief descriptions should be provided of the Preliminary Studies undertaken for the first selection of cascade options. The cascades selected to go through to the Final Studies should be described and the choice of their location should be justified. The energy dimensioning process should be discussed in great enough detail for the results to be understood. In the assessment of quantities and costs, the main characteristics of the structures taken into account should be indicated for each cascade option. The selection of the cascades to go through to the Final Studies should be illustrated using tables that permit a clear understanding of the reasons for their choice.

When it comes to the Final Studies, in all situations where the models recommended in this manual are not adopted, the mathematical models used should be presented, and their parameters and criteria should be given. In the assessment of quantities and costs, the main features of the structures taken into account should be given, alongside a summary of the cost estimates. Details should be given of the comparison of the cascades, clearly indicating the values of the cost/energy benefit index and corresponding negative and positive socioenvironmental indexes, as well as the weights used when calculating the preference and modified preference indexes.

## 7.3.7 Appendix G – Report on the Integrated Environmental Assessment

This appendix should provide a consolidation of the socioenvironmental studies of the final cascade selected, so that the studies undertaken following the procedures set out in this manual can be transformed into a stand-alone document with the objective of providing information for environmental entities as part of the licensing process for the projects in the future.

As such, this appendix should have the following structure:

### 1. Summary of the studies undertaken for the selection of the cascade options

Here, details should be given of the comparisons between the cascades in the Final Studies, clearly indicating the values of the cost/energy benefit index and corresponding negative and positive socioenvironmental indexes, as well as the weights used when calculating the preference and modified preference indexes.

**2. Description of the main features of the final cascade selected**

**3. Socioenvironmental diagnosis and potential conflicts**

3.1. Characterization elements

3.2. Identification of existing and potential conflicts

3.3. Aspects of relevance

3.4. Sensitivity indicators

3.5. Indicators of socioeconomic potentiality

**4. Distributed Environmental Assessment**

4.1. Mapping of sensitivity and potentiality indicators

4.2. Selection and scope of indicators of cumulative and synergistic impacts

4.3. Assessment of the cumulative and synergistic effects of the cascade per synthesis component and per sub-area

**5. Integrated Environmental Analysis of the final selected cascade**

5.1. Reference scenario

5.1.1. Basic assumptions and models

5.1.2. Reference scenario without the projects being built – areas of sensitivity and potentiality

5.1.3. Reference scenario with the projects built

5.1.3.1. Areas of fragility and potentiality per synthesis component and per project

5.1.3.2. Areas of socioeconomic potentiality per project

5.2. Formulation of socioenvironmental sustainability indicators for the region

5.3. Guidelines and recommendations

**6. Final communication of the studies**

## 7.3.8 Appendix H – Organization and summary of the work undertaken

This appendix should include the data on the operational organization and execution of the Inventory Studies.

The organizational chart of the operational team should be included, as should details of any additional agreements signed with third parties, and a brief summary of the main parts of the work undertaken.

The work schedule should also be included, citing the maximum number of people allocated to the studies and the number of man-hours involved in undertaking them.

The equipment and materials should be described, as should the logistic support and any special services provided by outsourced companies and/or consultants.

Finally, an economic and financial statement of the Inventory Studies should be presented.

# Format of the Final Report on Inventory Studies

| |
|---|
| LETTER OF PRESENTATION |
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| LIST OF APPENDICES, TABLES AND FIGURES |
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| 3.4 SOCIOENVIRONMENTAL DIAGNOSIS |
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| 3.6 TECHNICAL FORM FOR PROJECTS |
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| 3.8 PROJECT LAYOUTS |
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| 3.10 STANDARD ELETROBRAS COST ESTIMATE (ORÇAMENTO PADRÃO ELETROBRÁS) |
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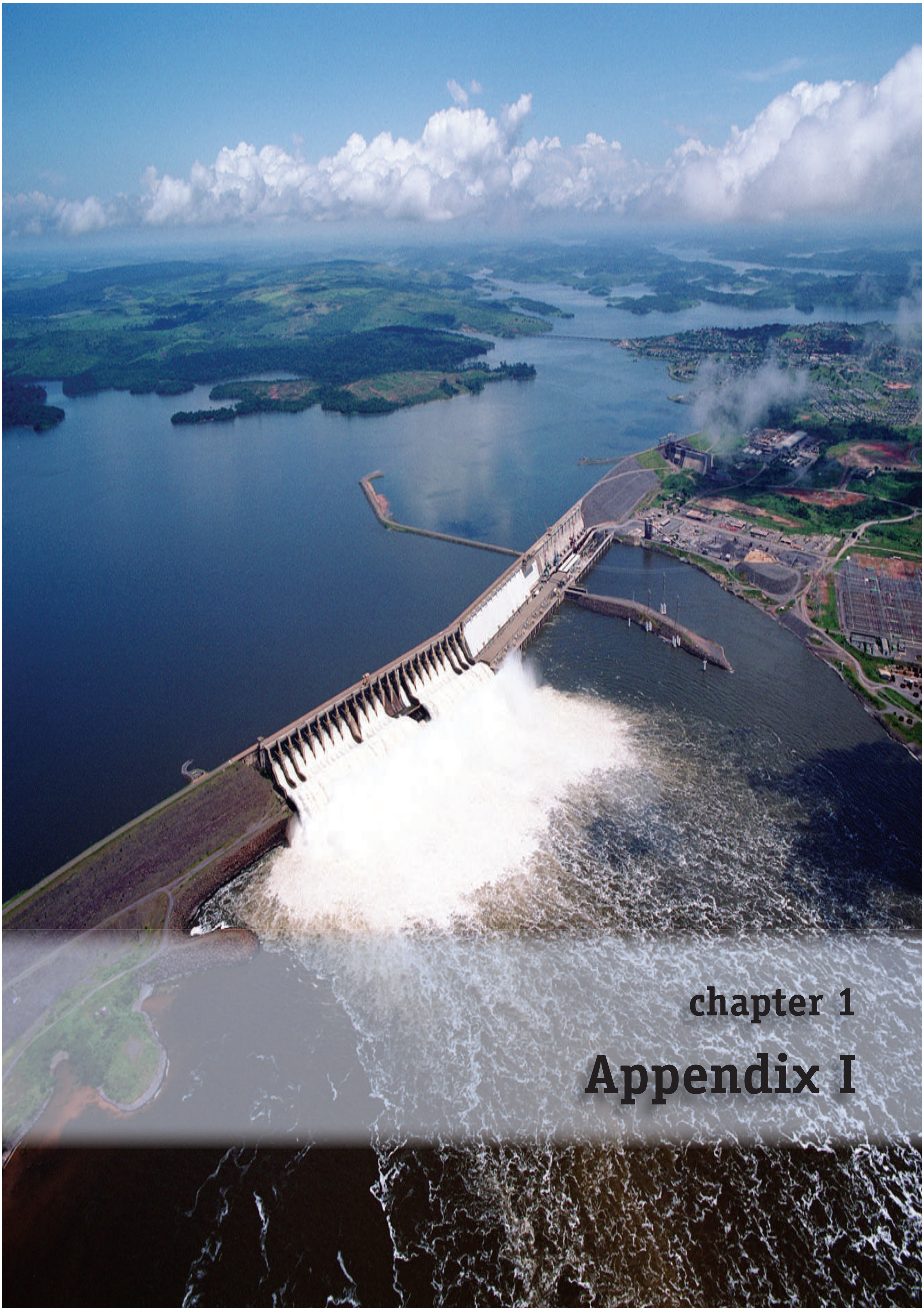
| 3.11.1 Cost/energy benefit index |
| 3.11.2 Negative socioenvironmental index |
| 3.11.3 Selection of cascades |
| 3.11.4 Characterization of cascades selected |



chapter 1

# Appendix I

# APPENDIX I

# APPENDIX I
# OVERVIEW OF HYDROELECTRIC POWER IN BRAZIL AND AROUND THE WORLD AND THE INSTITUTIONAL CONTEXT FOR HYDROPOWER INVENTORY STUDIES IN BRAZIL

## 1 Introduction

Water is a resource that is essential for human life. It is the responsibility of the Union to manage its water resources, ensuring the people of the nation the benefits deriving from human and animal consumption, food and energy production and other uses.

Using hydraulic energy for generating electricity is one potential use of water. Other uses, such as human consumption and food production, can in some situations provide greater benefits to society. Therefore, whenever the use of water resources for generating energy is being considered, the multiple water uses should be taken into account.

Over the years, Brazil has harnessed its hydroelectric potential, making it self-sufficient in electricity by making use of this low-cost renewable source of energy and developing national technology.

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

This is a topic that is currently legislated and regulated by several government entities and involves several different agents.

This Appendix to Chapter 1 of the Manual for Hydropower Inventory Studies presents an overview of the issues concerning hydroelectric power and its potential from a Brazilian and international perspective, as well as the institutional context in which Inventory Studies are undertaken in Brazil. It also explains certain key issues involved in the relationship between institutions and the legal processes and procedures needed for hydropower generation at this stage of planning.

## 2 Hydroelectric Power Around the World

### 2.1 Introduction

This Manual provides guidelines for studies that are undertaken in a context where long-term strategies for the country's development are still being examined. Evidently, there are other energy sources that compete with hydropower. It is therefore inevitable and even helpful to analyze hydropower generation from a geopolitical, strategic and global standpoint, in a context where social and environmental issues are gaining in importance and decisiveness.

Expanding energy generation means addressing an insoluble paradox in society: people want more energy for development and comfort, but they also question how it is produced, and particularly how it affects the environment and impinges on the other potential uses of natural resources. Good planning, transparency and democratic decision-making processes are the only way to resolve such conflicts. In this sense, Hydropower Inventory Studies are especially important, because it is at this stage and with this foresight that one can get a real picture of the potentialities and the impacts of hydropower projects on a river or river basin.

The world has reached a point when two major challenges must be faced:

- doubts as to how long it will be before the world's oil reserves are depleted; and
- changes to the environment on a global scale.

In this context, renewable energy has started to warrant greater attention worldwide. Indeed, the state of affairs is such that even nuclear energy, which has been severely criticized in recent years, is starting to be included in the energy mix, as it does not contribute to global warming. Conservation policies and the increased efficiency of equipment and energy users will certainly be important in shaping public policy for the future. So now more than ever the qualities of different energy sources should be analyzed from the perspective of promoting energy efficiency and preserving natural resources.

Obviously, every kind of energy production affects the environment one way or another and to a greater or lesser extent, since all involve transforming natural resources. Even so-called renewable energy sources can cause problems.[1]

Evidently, choices cannot be made purely by going for the option with the least impact, but rather by weighing up the positive and negative impacts in each case. In making this choice, the impact on regional development brought about by an energy project is an important factor, since the use of certain energy sources may affect one particular area of land, but its benefit is felt in other areas which are often distant from the place where the energy is generated.

In this sense, a power plant could be seen as the overlapping of one economic reality with another, normally exacerbating pre-existing differences.

In 2004, the International Hydropower Association published its *Sustainability Guidelines* with the aim of giving recommendations on how to take account of social and environmental issues in conjunction with the economic aspects of hydropower projects, the latter of which have been covered formally in this Manual since its 1997 edition. It is therefore an important tool towards ecological efficiency, which is based on three pillars:

- reducing the consumption of natural resources;
- reducing the impact on nature;
- enhancing the benefits of projects, taking into account multiple uses of the resources.

Focusing on the decision-making process and the criteria for making comparisons between different energy options, the IHA presents key criteria in this assessment:

1 For instance, wind energy, which is a clean form of electricity generation, occupies huge tracts of land, causes noise pollution and can pose a threat to birdlife. Solar energy does not cause pollution at the operating stage, but it uses solar panels which are manufactured using hazardous materials such as arsenic, cadmium and inert silicone. The burning of non-edible feedstocks (biomass), while it does absorb $CO_2$ at the crop growing stage, pollutes the air with particulate matter. The growing of non-edible feedstocks also occupies large areas of land that could otherwise be used to grow crops for human consumption.

- to promote energy efficiency on the demand side, viewing this as the equivalent of increasing energy production;
- to analyze the options for expending energy generation in view of the following aspects:
  - the availability of the resource, in view of the depletion of some primary sources;
  - energy payback ratio;
  - projective life of facilities;
  - technology efficiency and state-of-the-art;
  - multiple use benefits;
  - job creation and benefits for local communities;
  - impacts of carbon emissions;
  - area affected;
  - waste produced.

With these criteria in mind, hydropower schemes should avoid affecting vulnerable groups in society and focus on:

- upgrading existing projects;
- projects with multiple-use benefits;
- river basins that have already been developed;
- projects that minimize the area flooded per unit of energy produced;
- projects that prevent or minimize population displacement;
- projects that cause the least impacts on threatened species;
- projects that bring benefits to local communities, including in downstream areas.

## 2.2 The Issue of Dams Around the World

### 2.2.1 Size and Number of Dams

Table 2.2.1.01 shows the places where dams have been built around the world, independent of their function or size. Of course, simply counting them gives a distorted picture, because although Brazil has just 1% of the total number of dams, it produces almost 12% of the world's hydroelectric power. Even so, it is interesting to see that over 75% of all dams are in just four countries, showing that not all the problems involving reservoirs are connected to energy production.

The World Commission on Dams issued a report in 2000 (WCD 2000) in which they set out some recommendations based on a broad-based analysis of the experience of using dams of all kinds around the world. One of their recommendations is reproduced below:

*"Debate and controversy initially focused on specific dams and their local impacts. Gradually these locally driven conflicts evolved into a global debate about the costs and benefits of dams."*

The International Commission on Large Dams (ICOLD) defines a large dam as being one that is at least 15 meters high (counting from the lowest point of the foundations). Dams that are between 5 m and 15 m high and have a reservoir with the capacity to store over 3 million $m^3$ are also classified as large-scale. Based on this definition, there are today over 45,000 large dams around the world.

Half of the world's large dams were built exclusively for irrigation purposes, and it is estimated that dams boost the world's food production by 12%-16%. Also, in at least 75 countries, large dams have been built for flood control, and in many countries dams are the largest individual projects, measured in terms of their investments.

In the past, the provision of drinking water, energy generation, irrigation or flood control were normally seen as enough to justify the massive investments needed to build dams. Other benefits often went hand in hand, such as greater economic prosperity for the region thanks to multiple harvests, electrification of rural areas, and the expansion of basic and social infrastructure, such as roads and schools. These benefits were taken for granted. When they were compared with the construction and operating costs – from both an economic and financial perspective – the benefits seemed to make dam building easily the most competitive option.

However, projects of this kind have come under increasing scrutiny in recent years, and greater demands have started to be imposed on planned projects.

Table 2.2.1.01 – Number of dams around the world, per country (1994)

| | |
|---|---|
| China | 46% |
| USA | 14% |
| India | 9% |
| Japan | 6% |
| Spain | 3% |
| Other countries | 23% |
| Others | 16% |
| Canada | 2% |
| South Korea | 2% |
| Turkey | 1% |
| Brazil | 1% |
| France | 1% |
| Total | 100% |

Source: World Resources Institute – Eathtrends Environmental information
http://earthtrends.wri.org/maps_spatial/index.php?theme=2

The distribution of the world's 306 major dams[2] is shown in the map in figure 2.2.1.01, based on data from the World Research Institute (www.eathtrends.org). The same study shows that there are rivers that are fragmented by hundreds of dams measuring 15 m or more and thousands of small dams (<15m high). According to the study, there are 40,000 dams measuring 15 m or more in height and up to 800,000 of a smaller scale.

Dams are one way of analyzing how much rivers have been modified, which happens when they are impounded. Of the 106 river basins from around the world analyzed in the report, 46% have been modified by at least one major dam (there are 14 large dams in the Paraná river basin alone). In 1994, 56 new major dams were being planned or under construction.

The list of 56 projects for new dams is incomplete as few countries will divulge information of this nature. The dams in question are built in just five river basins: 11 in the Yangtze, seven each in the Tigris and Euphrates river basins, six in the Ganges river basin, and four each in the Hwang He and Paraná river basins.

From this information, it would appear that the only river basins with more than ten large dams are in Brazil, north-west America and west Canada.

2 In this case, a "major dam" is defined as being at least 150 m high or having a volume greater than 15 million m$^3$, or a generating capacity greater than 1000 MW. This is different from the ICOLD definition.

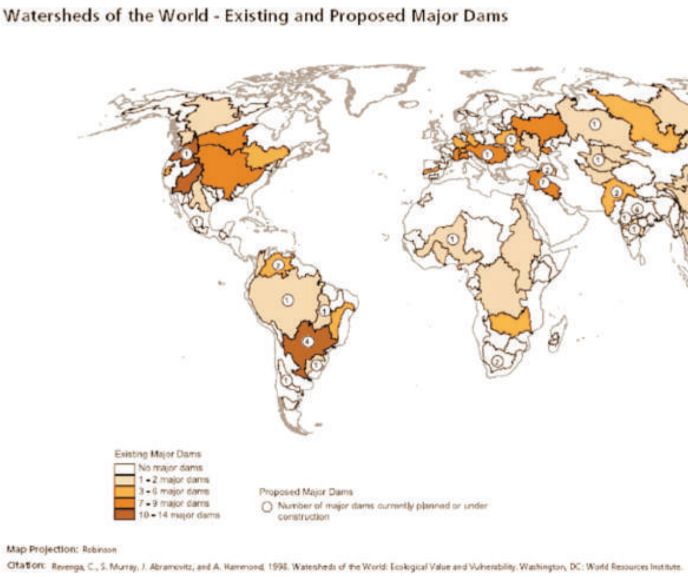



Figure 2.2.1.01 – Distribution of 306 major dams[3] around the world

### 2.2.2 The Debate over Impacts and Benefits

The International Rivers Association, an NGO devoted to "*defending the rights of communities that depend on rivers*", published Twelve Reasons to Exclude Large Hydro from Renewables Initiatives, which clearly exemplifies the strength of the lobby against large-scale dam projects. According to this organization:

1) Large hydro does not have the poverty reduction benefits of decentralized renewables do not reduce poverty.
2) Including large hydro in renewables initiatives would crowd out funds for new renewables.
3) Promoters of large hydro regularly underestimate costs and exaggerate benefits.
4) Large hydro will increase vulnerability to climate change.
5) There is no technology transfer benefit from large hydro.
6) Large hydro projects have major negative social and ecological impacts.
7) Efforts to mitigate the impacts of large hydro typically fail.
8) Most large hydro developers and funders oppose measures to prevent the construction of destructive projects.
9) Large reservoirs can emit significant amounts of greenhouse gases.
10) Large hydro is slow, lumpy, inflexible and getting more expensive.
11) Many countries are already overdependent on hydropower.
12) Large hydro reservoirs are often rendered non-renewable by sedimentation.

In an analysis of the subject, the World Bank presents counter-examples in a bid to show that this is a false dichotomy. Table 2.2.2.01 shows the data from a study that correlates reservoir area to installed capacity.

3 In this case, a "major dam" is defined as being at least 150 m high or having a volume greater than 15 million m$^3$, or a generating capacity greater than 1000 MW. This is different from the ICOLD definition.

Table 2.2.2.01 – Average Size of Reservoir per Capacity (1995)

| Size of the Plants (MW) | Number of Plants | Size of the Reservoir(ha/MW) |
|---|---|---|
| 3,000 to 18,200 | 19 | 32 |
| 2,000 to 2,999 | 16 | 40 |
| 1,000 to 1,999 | 36 | 36 |
| 500 to 999 | 25 | 80 |
| 250 to 499 | 37 | 69 |
| 100 to 249 | 33 | 96 |
| 2 to 99 | 33 | 249 |

Source: Goodland, Robert (1995), How to Distinguish Better Hydros from Worse: the Environmental Sustainability Challenge for the Hydro Industry, The World Bank.

Both positions look at the issue from a broad perspective. Evidently, each case has its differences and size is not the only issue at stake. Indeed, the "big vs. small" discussion is far from drawing to an end, with respected experts taking a stand on both sides of the divide and increasing attention being given to the subject in important documents from the energy industry.

Whatever the merit of the question, there is clearly a strong lobby against large-scale projects. Below we reproduce a short section of the report that illustrates the concern and the conflicts identified by the World Commission on Dams.

*"The enormous investments and widespread impacts of large dams have seen conflicts flare up over the siting and impacts of large dams - both those in place and those on the drawing board, making large dams one of the most hotly contested issues in sustainable development today.*

*Proponents point to the social and economic development demands that dams are intended to meet, such as irrigation, electricity, flood control and water supply. Opponents point to the adverse impacts of dams, such as debt burden, cost overruns, displacement and impoverishment of people, destruction of important ecosystems and fishery resources, and the inequitable sharing of costs and benefits." (WCD, 2000:6)*[4]

The report recommends a number of stances organized as shown below:

National governments can:

- Require a review of existing procedures and regulations concerning large dam projects;
- Adopt the practice of time-bound licenses for all dams, whether public or privately owned;
- Establish an independent, multi-stakeholder committee to address the unresolved legacy of past dams.

National and international NGOs can:

- Monitor compliance with agreements and assist any aggrieved party to seek resolution of outstanding disagreements or to seek recourse;
- Actively assist in identifying the relevant stakeholders for water and energy projects using the rights-and-risks approach.

Affected people's organizations can:

- Identify unresolved social and environmental impacts and convince the relevant authorities to take effective steps to address them;
- Develop support networks and partnerships to strengthen technical and legal capacity for needs and options assessment processes.

4 WCD, 2000. Dams and Development: A New Framework for Decision-Making – The Report of the World Commission on Dams – An Overview. Available in http://www.rivernet.org/general/wcd/wcd_overview_english.pdf.

Professional associations and agencies can:

- Develop processes for certifying compliance with WCD guidelines;
- Extend national and international databases, such as the ICOLD World Register of Dams, to include social and environmental parameters.

The private sector can:

- Develop and adopt voluntary codes of conduct, management systems and certification procedures for best ensuring and demonstrating compliance with the Commission's guidelines, including, for example, through the ISO 14001[5] management system standard;
- Abide by the provisions of the antibribery convention of the Organisation for Economic Co-operation and Development;
- Adopt integrity pacts for all contracts and procurement, as developed by Transparency International.

Bilateral aid agencies and multilateral development banks can:

- Ensure that any dam options for which financing is approved emerge from an agreed process of ranking alternatives and respect the Commission's guidelines;
- Accelerate the shift from project- to sector-based finance, especially through increasing financial and technical support for effective, transparent, and participatory needs and options assessment, and the financing of non-structural alternatives;
- Review the portfolio of past projects to identify those that may have underperformed or present unresolved issues.

However, despite this huge effort and however complex the procedures that need to be analyzed, when it is a matter of comparing different energy sources, the methods used do not seem entirely satisfactory when it comes to hydropower generation. This sense of non-adaptation, which is so controversial, arises from the fact that unlike most of the other energy sources, the generation of electricity by hydropower plants could even be seen as a byproduct of a series of other benefits provided by a project that could be classified as being of an entirely different nature in the region in question. While a thermoelectric power station is no more nor less than a facility to generate power, a hydroelectric power plant can have many other vocations, some of which could even be more important. For instance, how can one compare a dam built for flood control and which has also been designed to produce energy with the energy generated by a thermoelectric plant?

## 2.3 Some International Data on Hydropower

With these issues in mind, some international data are presented below to contribute to a global geopolitical understanding of the problem. The idea is to give an overview of the situation, the part played by hydroelectricity and Brazil's role in all this. In 2004, if we consider all the primary sources of energy generation, the world produced the equivalent of 10.2 billion tons of oil (Energy Information Administration – US Department of Energy – 2005). The current mix of primary sources is shown in table 2.3.01.

5 ISO 14001 is the standard used for the certification of organizations' environmental management. Certification is not granted by the ISO, which is an international standards body, but by an accredited third-party entity. In Brazil, the National Council for Metrology, Standards and Industrial Quality, CONMETRO, established the Brazilian Compliance Evaluation System, and designated Inmetro (the Brazilian weights and measures institute) to be Brazil's official accreditation agency. Any accreditation given within the ambit of CONMETRO's Sistema Brasileiro de Avaliação da Conformidade must be carried out by an entity accredited by Inmetro. However, as the ISO 14001 is voluntary in nature, it can be certified outside the Brazilian system by entities with or without Inmetro accreditation. Whether or not the certification is given within the Brazilian system, when it is carried out by an Inmetro accredited entity, it is based on the same requirements and methods.

Table 2.3.01 – Primary Sources of Energy in the World (2003)

| Source | Proportion (%) |
|---|---|
| Crude Oil and Manufactured Gas | 38 |
| Coal | 24 |
| Natural Gas | 24 |
| Hydropower | 7 |
| Nuclear | 6 |
| Other | 1 |

Source: Table 11.1 World Primary Energy Production by Source, 1970-2003
Energy Information Agency – US Dept. of Energy

However, it is important to consider just electricity generation if the weight of the different forms of production is to be correctly evaluated.

Table 2.3.02 – Sources of Electricity in the World (2003)

| Source | Proportion (%) |
|---|---|
| Oil | 6.9 |
| Coal | 39.9 |
| Natural Gas | 19.3 |
| Hydropower | 16.3 |
| Nuclear | 15.7 |
| Other | 1.9 |

Source: Electricity in World in 2003 – International Energy Agency Statistics
http://www.iea.org/Textbase/stats/

It is important to bear in mind that hydroelectric power accounts for around 16% of all energy produced in a world where oil, natural gas and coal still rule supreme. In this sense, the recent hike in oil prices, partly because of the depletion of known reserves, allied with the precarious state of the global environment, can now be seen as increasingly important variables in any strategic investigation of energy in the world.

Of all renewable sources of energy, water power is still the most promising, as it can generate large quantities of electricity with good economies of scale. The ten largest producers of electricity in the world are listed in table 2.3.03.

Table 2.3.03 – Ten Largest Producers of Electricity in the World

| Country | TWh | % of total |
|---|---|---|
| USA | 4,150 | 23.8 |
| China | 2,187 | 12.5 |
| Japan | 1,110 | 6.4 |
| Russia | 931 | 5.3 |
| India | 651 | 3.7 |
| Germany | 607 | 3.5 |
| France | 572 | 3.3 |
| Canada | 568 | 3.3 |
| UK | 400 | 2.3 |
| Brazil | 386 | 2.2 |
| Other countries | 11,561 | 33.8 |

The ten largest producers of hydroelectric power are shown in table 2.3.04, with Canada, China, Brazil and the USA occupying the top of the table.

Table 2.3.04 – Largest Producers of Hydroelectric Power (and proportion of hydropower to the energy mix) (2001)

| Country | TWh | % of total |
|---|---|---|
| Canada | 344 | 12.0 |
| China | 334 | 11.7 |
| Brazil | 326 | 11.4 |
| USA | 269 | 9.4 |
| Russia | 180 | 6.3 |
| Norway | 111 | 3.9 |
| Japan | 102 | 3.6 |
| India | 86 | 3.0 |
| Venezuela | 72 | 2.5 |
| France | 67 | 2.3 |
| Others | 1,890 | 35.1% |

Source: WEC Member Committees, 2000/2001; Hydropower & Dams World Atlas 2001, supplement to The International Journal on Hydropower & Dams, Aqua Media International; Energy Statistics Yearbook 1997, United Nations; national and international

Another aspect that is worth observing is the 'productivity' of hydroelectric systems around the world. As Table 2.3.05 shows, it is not every system that can obtain capacity factors greater than 50%. In many countries, hydro plants are only used to meet peak demand or do not have enough reserve capacity to regulate their production. This last aspect is of the utmost importance and sets Brazil's generation system apart from its peers. Indeed, of all the large-scale systems around the world, only the Canadian one has a similar reserve capacity to Brazil's. This is an important factor when analyzing the feasibility of new projects. If viewed from the perspective of the growing need to strike a balance between social and environmental impacts and energy benefits, Brazil's system is particularly efficient.

Table 2.3.05 – Leading Countries and the Capacity Factors of the Hydroelectricity Systems (1999)

| Country | Capacity in Operation (MW) | Generation in 1999 (TWh) | Capacity Factor(%) |
|---|---|---|---|
| Canada | 66,954 | 341 | 58 |
| Brazil | 57,517 | 286 | 57 |
| Venezuela | 13,165 | 61 | 53 |
| Norway | 27,528 | 122 | 51 |
| Sweden | 16,192 | 71 | 50 |
| USA | 79,511 | 319 | 46 |
| India | 22,083 | 82 | 43 |
| Russia | 44,000 | 161 | 42 |
| Austria | 11,647 | 42 | 41 |
| Mexico | 9,390 | 32 | 39 |
| Turkey | 10,820 | 35 | 37 |
| China | 65,000 | 204 | 36 |
| Japan | 27,229 | 84 | 35 |
| France | 25,335 | 77 | 35 |
| Italy | 16,546 | 47 | 32 |
| Switzerland | 13,230 | 37 | 32 |
| Spain | 15,580 | 28 | 21 |

Source: WEC Member Committees, 2000/2001; Hydropower & Dams World Atlas 2001, supplement to The International Journal on Hydropower & Dams, Aqua Media International;

When it comes to different countries' potential for expanding their hydropower generation, Brazil is one of the countries with the greatest potential as it has by far the largest amount of water resources, as we can see in table 2.3.06.

Table 2.3.06 – Total Water Resources per Country (2003)

| Country | Water resources entirely in national territory (km3/year) | Water resources that originate outside national territory (km3/year) | Total Resources | % of total |
|---|---|---|---|---|
| Brazil | 5,418.0 | 2,815.0 | 8,233.0 | 19 |
| Russia | 4,312.7 | 194.6 | 4,507.3 | 10 |
| Canada | 2,850.0 | 52.0 | 2,902.0 | 7 |
| Indonesia | 2,838.0 | 0.0 | 2,838.0 | 6 |
| Mainland China | 2,812.4 | 17.2 | 2,829.6 | 6 |
| USA | 2,000.0 | 71.0 | 2,071.0 | 5 |
| Peru | 1,616.0 | 297.0 | 1,913.0 | 4 |
| India | 1,260.5 | 636.1 | 1,896.6 | 4 |
| Congo | 900.0 | 383.0 | 1,283.0 | 3 |
| Venezuela | 722.5 | 510.7 | 1,233.2 | 3 |
| Top 10 | 24,730.1 | 4,976.6 | 29,706.7 | 57 |
| World | 43,764.0 | | 43,764.0 | 100 |

Source: FOOD AND AGRICULTURE ORGANIZATION OF THE UNITED NATIONS – Review of World Water Resources by Country, Rome, 2003
Internal renewable water resources is that part of the water resources (surface water and groundwater) generated from endogenous precipitation.
External water resources as the part of a country's renewable water resources that enter from upstream countries through rivers (external surface water) or aquifers (external groundwater resources).

Table 2.3.07 sets out an international appraisal of the potential expansion of hydropower generation capacity. However, it should be noted that there is much room for error in the estimates not only for Brazil but for other countries in view of the increasing restrictions on large-scale and even medium-scale projects. Even so, Brazil is still an important player, even in this scenario of growing constraints on the construction of new hydropower projects.

Table 2.3.07 – Estimate of Hydroelectric Potential around the World (2000/2001)

| Country | Theoretical Capacity (TWh/year) | Technically Available (TWh/ year) | Economically Feasible (TWh/ year) | % of world total |
|---|---|---|---|---|
| China | 5,920 | 1,920 | 1,260 | 13 |
| Russia | 2,800 | 1,670 | 852 | 12 |
| Brazil | 3,040 | 1,488 | 811 | 10 |
| Canada | 1,289 | 951 | 523 | 7 |
| Congo | 1,397 | 774 | 419 | 5 |
| USA | 4,485 | 529 | 376 | 4 |
| Tajikistan | 527 | 264 | 264 | 2 |
| Ethiopia | 650 | 260 | 260 | 2 |
| Peru | 1,578 | 260 | 260 | 2 |
| Norway | 600 | 200 | 180 | 1 |
| Nepal | 727 | 158 | 147 | 1 |

Source: WEC Member Committees, 2000/2001; Hydropower & Dams World Atlas 2001, supplement to The International Journal on Hydropower & Dams, Aqua Media International.

## 2.4 Competitive Aspects of Hydropower

Despite its many problems, hydropower has several advantages which are often understated. These include:

- replacement or postponement of the need for generating electricity from fossil fuels, with corresponding benefits for air quality;
- provision of a very reliable service using technology which has been tried and tested for over a century, with low operating costs, high energy efficiency and a long working life;

- totally renewable and the only source in this category that can produce energy on a large scale;
- low greenhouse gas emissions[6] if compared to fossil fuels;
- any negative impacts are only felt in its area of influence;
- building a hydropower plant can, given its size, present the opportunity to meet regional development needs, such as flood control, river transportation, irrigation, etc.;
- although investments are intensive for a limited period, they can be made using domestic technology, skills and materials, contributing to a country's strategic independence;
- part of the investments are catalysts for other sectors of the economy, creating direct and indirect jobs;
- as an operationally flexible energy source, it can support large energy distribution networks;
- particularly good for providing ancillary services for the electricity grid, such as spinning and non-spinning reserves, frequency regulation and response, voltage control and stability.

The issue of future energy supply is more than ever of international concern. Increasingly, countries are becoming aware of the limitations inherent to exploiting the planet's natural resources, especially given the inevitable depletion of reserves of oil, which is still the main primary source of energy, and the environmental impacts associated with this and other forms of energy production.

Some international initiatives are starting to be taken in response to this scenario. The Clean Development Mechanism, established as part of the Kyoto Protocol, provides an incentive for companies from developed nations to invest in optional emission reduction projects in developing nations. The Kyoto Protocol requires all CDM projects to be validated and inspected/certified by a "designated operational entity"; i.e. subject to an independent evaluation.

This all indicates that the world is turning its attention to energy projects, especially in developing countries. It also means that energy options will be scrutinized according to far more complex criteria, without the "linearity" traditionally seen in financial and economic assessments.

In this context, inventory studies of the hydroelectric potential of Brazil's river basins gain greater importance, as they address future scenarios and can pinpoint actions that can be taken in advance to minimize impacts or even make new hydropower projects feasible.

Around the world, great concern is being expressed about the impacts of dams and reservoirs, and there is a clear dividing line between projects according to their scale.

While hydropower is clearly not a panacea for the world's energy problems, it is certainly part of the solution to the dual issue of energy production and socioenvironmental impacts. As Brazil still has such great and as yet untapped water resources, it can plan out its future energy needs and the expansion of its electricity system on the back of hydropower. Hydropower inventory studies are a key part of this process.

6 Some authors propose that large reservoirs that flood forests in tropical ecosystems are responsible for high $CO_2$ emissions. Recent studies of the Tucuri reservoir show that in the worst case scenario, the lake emits 213 g $CO_2$ per kWh generated. This is five times lower than $CO_2$ emissions by coal-fired power stations. See Hydropower and the World's Energy Future – International Hydropower Association – International Commission on Large Dams - International Energy Agency – Nov 2005.

# 3 Hydropower in Brazil

## 3.1 Brazil's Hydroelectic Potential and the Feasibilty of its Being Harnessed

The drafting and future use of this Manual for Hydropower Inventory Studies is part of a broad review of criteria, methods and even legislation pertaining to hydropower generation in Brazil. Indeed, the scenario of potential future projects is very likely to change substantially after this procedure. Therefore, what is set out here is merely a portrayal of what is available from previous studies and will probably be subject to review.

The information available in the SIPOT[7] database maintained by Eletrobrás gives a full assessment of Brazilian territory. The data set out below can be found on the Eletrobrás website and may not represent the latest assessment of Brazil's hydroelectric potential[8].

The sites earmarked for hydropower projects and registered in the SIPOT system are classified into three groups according to the stage their studies have reached, as shown below.

- Estimated Potential (sections of river, SR, and individual dam sites, In).

The hydroelectric potential of sections of river is estimated from office-based studies of sections of river (head available at multiple dam sites). The potential for individual dam sites is also estimated from office-based studies undertaken when deciding on specific sites to be harnessed. The existence of this potential in Brazil's different regions (based on data from the SIPOT system) is shown in table 3.1.01 below.

Table 3.1.01 – Estimated Potential per Region (MW)

| Region | SR | In | SR + In |
|---|---|---|---|
| North | 16,034.76 | 37,288.03 | 53,322.79 |
| Northeast | 267.6 | 874.78 | 1,142.38 |
| Southeast | 2,373.30 | 2,858.10 | 5,231.40 |
| Central-West | 7,545.61 | 8,607.53 | 16,153.14 |
| South | 2,020.72 | 2,602.69 | 4,623.41 |
| Total | 28,241.99 | 52,231.13 | 80,473.12 |

- Potential under Study (Inventory (I), Feasibility (F) and Basic Design (BD)).

Potential classified under "Inventory" is the outcome of Inventory Studies of different river basins. The sites classified under "Feasibility" are the ones whose overall conception is being examined with a view to verifying their technical and economic feasibility. The sites at the "Basic Design" stage are already being detailed for public tender. The existence of hydroelectric potential at these different stages in the different regions of Brazil is shown in table 3.1.02 below.

Table 3.1.02 – Potential under Study per Region (MW)

| Region | I | F | BD | Deactivated (D) | I+F+BD-D |
|---|---|---|---|---|---|
| North | 17,275.59 | 28,744.60 | 1,327.23 | 2.34 | 47,345.08 |
| Northeast | 6,593.64 | 7,050.50 | 406.16 | 0.8 | 14,049.50 |
| Southeast | 10,236.03 | 3,974.45 | 1,753.02 | 2.67 | 15,960.83 |
| Central-West | 9,535.40 | 1,501.75 | 2,286.72 | 2.33 | 13,321.54 |
| South | 9,758.32 | 4,676.58 | 2,826.36 | 0 | 17,261.26 |
| Total | 53,398.98 | 45,947.88 | 8,599.49 | 8.14 | 107,938.21 |

7 SIPOT – Information System on Brazil's Hydroelectric Potential, Eletrobrás. sipot@eletrobras.com.

8 There is actually some difference between the SIPOT data and the data on installed capacity provided by ANEEL, as Eletrobrás is no longer responsible for storing and updating these data and keeping them consistent with other sources. Despite this, it is still the best source for revealing the percentages of projects at different stages of development.

## Plants Under Construction (C) and In Operation (O)

Hydroelectric potential classified as being "under construction" relates to plants that have already started being constructed, while "in operation" relates to those with at least one unit already in service. The existence of projects in the different regions of Brazil is shown in table 3.1.03.

Table 3.1.03 – Energy Potential under Construction and In Operation per Region (MW)

| Region | C | O | Total |
|---|---|---|---|
| North | 3,109.50 | 7,229.85 | 10,339.35 |
| Northeast | 25 | 10,783.25 | 10,808.25 |
| Southeast | 1,313.38 | 22,109.10 | 23,422.48 |
| Central-West | 642.8 | 9,006.89 | 9,649.69 |
| South | 2,725.77 | 18,631.10 | 21,356.87 |
| Total | 7,816.45 | 67,760.19 | 75,576.64 |

The sum of these three categories is shown in table 3.1.04.

Table 3.1.04 – Total Potential per Region (MW)

| Region | Total |
|---|---|
| North | 111,011.90 |
| Northeast | 26,001.73 |
| Southeast | 44,620.05 |
| Central-West | 39,129.03 |
| South | 43,241.54 |
| Total | 264,004.25 |

It is possible to use data from Eletrobrás to further break down some of these categories according to the size of the projects. The projects whose basic design is being prepared are detailed in table 3.1.05.[9]

Table 3.1.05 – Capacity of projects at Basic Design (BD) stage

| Capacity Range | Total Capacity (MW) | % | No. of Plants | % |
|---|---|---|---|---|
| P > 1000 | 2,820 | 32.0 | 2 | 0.8 |
| 500< P < 1000 | 700 | 7.9 | 1 | 0.4 |
| 200< P < 500 | 1,072 | 12.2 | 4 | 1.6 |
| 100< P <200 | 549 | 6.2 | 4 | 1.6 |
| 30< P <100 | 1,051 | 11.9 | 27 | 10.6 |
| 0< P <30 | 2,625 | 29.8 | 217 | 85.1 |
| | 8,817 | | 255 | |

A breakdown of the power available from plants at the Feasibility stage[10] is shown in table 3.1.06.

Table 3.1.06 – Capacity of Projects at Feasibility (F) stage

| Capacity Range | Total Capacity (MW) | % | No. of Plants | % |
|---|---|---|---|---|
| P > 1000 | 30,903 | 80.3 | 11 | 15.9 |
| 500< P < 1000 | 1,352 | 3.5 | 2 | 2.9 |
| 200< P < 500 | 2,956 | 7.7 | 9 | 13.0 |
| 100< P <200 | 2,011 | 5.2 | 14 | 20.3 |
| 30< P <100 | 1,150 | 3.0 | 21 | 30.4 |
| 0< P <30 | 91 | 0.2 | 12 | 17.4 |
| | 38,462 | | 69 | |

9 There is a slight difference between the total for this table and the total for table 3.2, as the data from different versions of SIPOT are not consistent.

10 There is a slight difference between the total for this table and the total for table 3.2, as the data from different versions of SIPOT are not consistent.

A breakdown of the power available from projects that are at the Inventory Studies stage is shown in table 3.1.07.

Table 3.1.07 – Capacity of Projects at Inventory Studies (I) stage

| Capacity Range | Total Capacity (MW) | % | No. of Plants | % |
|---|---|---|---|---|
| P > 1000 | 20,270 | 33 | 8 | 1 |
| 500< P < 1000 | 9,268 | 15 | 13 | 1 |
| 200< P < 500 | 6,521 | 11 | 22 | 2 |
| 100< P <200 | 7,566 | 12 | 54 | 4 |
| 30< P <100 | 8,850 | 14 | 161 | 12 |
| 0< P <30 | 9,537 | 15 | 1,136 | 81 |
| | 62,012 | | 1,394 | |

It is also interesting to look at how the hydroelectric potential being investigated at the Inventory Studies, Basic Design and Feasibility stages is distributed across the country's river basins. This is in keeping with CNRH Resolution 32 of October 15th 2009, by which a new National Hydrography Division was established.

Table 3.1.08 – Breakdown of projects at Inventory Studies, Basic Design and Feasibility stages per river basin

| | Amazon | Tocantins | East Atlantic | São Francisco | Southeast Atlantic | Paraná | Uruguay | South Atlantic |
|---|---|---|---|---|---|---|---|---|
| S | 30% | 10% | 2% | 17% | 14% | 16% | 8% | 4% |
| F | 49% | 16% | 1% | 13% | 3% | 7% | 7% | 4% |
| BP | 21% | 3% | 0% | 1% | 18% | 35% | 13% | 8% |

These percentages reveal some important facts:

- Around 30% of the potential evaluated from the SIPOT (264 GW) is in operation or under construction (~ 75 GW).
- Around 30% is just estimated (~ 80GW).
- Around 40% is under study (107 GW).
- Of the proportion under study, 8% is at the Basic Design stage, 43% is having Feasibility Studies done and the remaining 49% is the object of Inventory Studies.
- The regional breakdown of the projects at the Basic Design stage is as follows: 27% in the central-west region; 33% in the south, 20% in the southeast, 15% in the north and 5% in the northeast.
- Of all the projects at the Basic Design stage, just two of the total of 255 projects account for 30% of the power (~ 9 GW). Around 85% of all the projects are for plants with an installed capacity of less than 30 MW.
- Of the projects at the Feasibility stage (~39 GW), 63% are in the north, 10% are in the south, 9% are in the southeast, 3% are in the central-west and 15% are in the northeast. Around 80% are for large-scale projects whose installed capacity would be over 1 GW. Just 0.2% of the total energy would come from plants with an installed capacity of less than 30 MW.
- The majority of the projects at the Feasibility stage are in the Amazon and Tocantins river basins.
- 33% of the projects at the Inventory Studies stage are in the north, 19% are in the southeast, 18% in the south, 12% in the northeast and 18% are in the central-west. 96% of these projects are for plants with less than 200 MW, while 81% would be under 30 MW.

It seems clear that geographically speaking, the most important untapped resources are in the north and central-west regions of the country. The longer-term trend in terms of scale seems to be towards medium-sized plants.

## 3.2 Background on Hydropower Generation

The experiences that marked the beginning of Brazil's electricity industry were in the areas of lighting and public transportation. It was in 1879 that indoor electric lighting was inaugurated at Dom Pedro II railroad station (Central do Brasil) in Rio de Janeiro. Two years later the first street lighting in the city was installed along a section of gardens in Campo da Aclamação, now Praça da República, using dynamos run by locomotives. In the same year, at the opening of the Industrial Exhibition, electricity was used to light the Transportation Ministry building at Largo do Paço (now Praça XV), also in Rio de Janeiro. In 1883, the first electricity generation plant came into operation, with 52 kW installed capacity, in Campos (Rio de Janeiro state). It was a thermoelectric unit fired by steam produced from timber and provided energy for 39 light bulbs. This was the first ever public lighting service in South America. Meanwhile, the first use of electricity as traction for public transportation came in 1883 in Niterói, with Brazil's first ever electric streetcars[11].

However, it was probably the opening of Marmelos power plant[12] in 1889 in Juiz de Fora, Minas Gerais state, which really marked the beginning of Brazil's electricity industry. This was followed by several other important projects, including Monjolinho and Piracicaba in 1893, Corumbataí in 1900, Fontes over Ribeirão das Lajes in 1908, and the famous Delmiro Gouveia plant on São Francisco river in 1913.

However, Brazil's slow-growing system of almost continental proportions only started to gain more structure after the creation of CHESF in 1945 and CEMIG in 1946. With these companies came the beginning of a period of large-scale, long-term, consistent state intervention in the electricity sector. The following decades spawned a wealth of major projects, including Três Marias in 1962 and Furnas in 1963. It was also in 1963 that the renowned project for Canambra was begun. The scale and depth of the studies undertaken in the 1960s were a turning point in Brazil's electricity industry.

- They were the first comprehensive hydropower inventories, leading in some cases to feasibility studies;
- they were decisive in the option for large dams; and
- they contributed to the training of hundreds of skilled workers, who went on to join the ranks of the planning entities of the country's leading electricity companies and private consultancies.

All this was decisive in the development of large dams, which started to be included in Brazil's power generation expansion plans, including Plan 90, Plan 95, and Plans 2000, 2010 and 2015, published in 1974, 1979, 1982, 1988 and 1994, respectively.

Centrais Elétricas Brasileiras S.A. (Eletrobrás) was officially incorporated on June 11th 1962 at a ceremony held by Conselho Nacional de Águas e Energia Elétrica (Cnaee) at Palácio Laranjeiras in Rio de Janeiro, with the presence of the Brazilian President João Goulart (1961-1964). The investment portfolio and the administration of the Federal Electrification Fund was taken from the Brazilian Development Bank (Banco Nacional de Desenvolvimento Econômico, BNDE) and put under Eletrobrás's control[13].

As part of the reorganization of the sector, hundreds of small companies were grouped together or taken over by state-owned utilities. For instance in 1966 the São Paulo government merged 11 state-owned companies into one: Centrais Elétricas de São Paulo (CESP).

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11 O SETOR ELÉTRICO Faria, Gomes, Abarca and Fernandes. http://www.bndes.gov.br/conhecimento/livro_setorial/setorial14.pdf

12 Built in the Paraibuna sub-basin, Juiz de Fora, Minas Gerais state, and owned by Companhia Mineira de Eletricidade, this was the first hydroelectric power plant to supply energy for street lighting. Marmelos, with 250 kW installed capacity, was developed by Max Nothman & Co. and used equipment supplied by Westinghouse.

13 Memória da Eletricidade (www.eletrobras.gov.br)

More important yet, the interdependence of the systems means that entities representing the industry had to be created, such as the Coordinating Committee for Interconnected Operations (Comitê Coordenador para Operação Interligada, CCOI), formed in 1969 by generation and distribution companies from the southeast under the technical guidance of Eletrobrás and the supervision of the National Department of Water and Electricity (Departamento Nacional de Águas e Energia Elétrica, DNAEE). Two years later, another committee was formed by companies from the south of Brazil.

The system grew with the development of new projects in the southeast, starting with the Tietê and Paraíba do Sul river basins. Until 1950, most power plants were near the coast in São Paulo, Rio de Janeiro and Minas Gerais states. Little by little, this picture changed, with new facilities being built in São Paulo, Minas Gerais, Mato Grosso do Sul and Goiás. This growing distance from load centers was resolved by introducing cascades of hydropower projects to maximize the water resources of a given river, and thanks to the hydrological diversity that emerged as different river basins were harnessed.

The biggest step in the development of Brazil's electricity system was the building of large-scale plants in the 1960s and 70s, especially in the Paraná river basin. With their large reservoirs, these projects brought long-lasting advantages not previously seen in hydro-dependent electricity systems. Using a carefully dimensioned transmission system, it was possible to provide a reasonable level of assurance, since by interconnecting the reserves, it became feasible to share them throughout the system, effectively "regulating" all the power plants.

### 3.3 Hydropower System Today, Prospects for Growth, and the Role of the Transmission System in Interconnecting River Basins

In 2006 Brazil had 1,588 projects in operation totaling 96,340,783 kW installed capacity.

Table 3.3.01 – Brazil's Installed Capacity

| Type of Project | Quantity | Contracted Capacity (kW) | Actual Capacity (kW) | % |
|---|---|---|---|---|
| Hydro (< 1 MW) | 196 | 104,655 | 104,208 | 0.11 |
| Wind Power | 14 | 189,250 | 186,850 | 0.19 |
| Small Hydro | 269 | 1,457,551 | 1,415,863 | 1.47 |
| Solar | 1 | 20 | 20 | 0 |
| Hydroelectric | 156 | 73,348,695 | 71,820,411 | 74.55 |
| Thermoelectric | 950 | 23,950,514 | 20,806,431 | 21.60 |
| Thermonuclear | 2 | 2,007,000 | 2,007,000 | 2.08 |
| Total | 1,588 | 101,057,685 | 96,340,783 | 100 |

Source: ANEEL

In other words, around three quarters of Brazil's installed capacity comes from hydroelectric power. However, what makes this system even more unique is the fact that all the plants that are linked up to the grid share the same assurance thanks to the fact that Brazil has a higher reserve capacity than any other hydro-based system in the world.

This is one of the factors that justified the expansion of the transmission system, not just to meet demand but also to provide a wider range of dispatch options by improving the system's interconnectivity. This architecture required the adoption of an operationally centralized system, where several agents sell not their own production but a portion of energy from the whole system as if they were shares in a company. This flexibility combined with centralized operations means that large chunks of energy can be "transferred" from one basin to another, providing a gain of almost 25% over local energy sources.

Although this reserve capacity is declining as a result of the fact that no new large reservoirs have been added, it will still be the largest capacity in the world for a long time and is certainly a unique feature of Brazil's system. The importance of this to Hydropower Inventory Studies is that it affects how the feasibility of new projects is analyzed.

Even if social and environmental demands, which are likely to grow in importance as time goes by, are such that they limit the reserve capacity of new hydro projects, these may still not be made unfeasible, as even with limited reserve capacity, they will be able to draw on the "virtual" reserve of the interconnected system.

### 3.4 Effect Of Social and Environmental Issues on Hydro Generation In Brazil

Many of the hydropower plants in operation today were planned and built under a quite different political, institutional and developmental context than what prevails today. The decision-making process that is inherent to the introduction of new hydropower projects has developed greatly, not only by allowing greater participation and transparency, but also by factoring in issues concerning the distribution of costs and benefits. There is also increasing awareness on the part of Brazilian society about social and environmental issues, which is reflected in a rigorous, comprehensive, new legal framework aimed at assuring social and environmental sustainability and the conservation of water resources. It covers a broad range of aspects, including inspection activities and the defense of the environment and minorities, for which it provides a special public prosecution service, as the public prosecution service (Ministério Público Federal) plays a major part in the process of developing new hydropower projects.

Alongside the establishment of this legal framework, a critical analysis of social and environmental experiences has led Brazil's electricity sector to expend its efforts not only to adapt to the new legal context, but above all to adopt a new attitude from the study and design stage through to the building and operation of projects. Since the early 1990s clear guidelines organized into different levels of regulation have been developed to incorporate social and environmental variables at all stages of the decision-making process.

With the introduction of the principles of socioenvironmental feasibility, regional involvement and a more broad-based decision-making process in the second Master Plan for the Environment (II Plano Diretor de Meio Ambiente, II PDMA), published in 1991, new guidelines were drafted for the introduction of hydropower plants and manuals were drafted that set out the methods and procedures to be used in conjunction with engineering and environmental considerations.

Until the mid 1980s, generation projects had been prioritized in sector expansion plans almost exclusively as a function of the unit cost of the energy they would produce (in US$/MWh), without any systematic method for taking into account the measurable environmental costs, not to mention the non-quantifiable environmental variables. Based on the principles set out in the II PDMA, new efforts were made to incorporate socioenvironmental costs into the overall costs of hydropower projects, including estimates of such costs in Hydropower Inventory Studies. At the subsequent stages of developing these projects – the Feasibility Studies, Basic Design and Executive Design – these costs are included in greater detail so the costs of environmental compensation and mitigation can be accounted for with greater accuracy.

At the end of the 1990s, ANEEL and Eletrobrás republished their set of manuals providing guidelines for the studies required for Inventory Studies, Feasibility Studies, Basic Designs, transmission systems and projects for small hydros. All these documents provide guidelines for contracting and undertaking the studies from the perspective of integrated planning and including in the decision-making process all institutional, legal, energy, economic and socioenvironmental aspects of relevance.[14]

Alongside the development of the legislation and regulatory framework for the electricity sector, the system for planning and implementing projects is determined by the actions of private agents, inspection agents and government entities. Figure 3.4.01 shows the main stages of their interaction in this process.

14 The manuals produced between 1995 and 1998 are available at www.aneel.gov.br and www.eletrobras.gov.br.

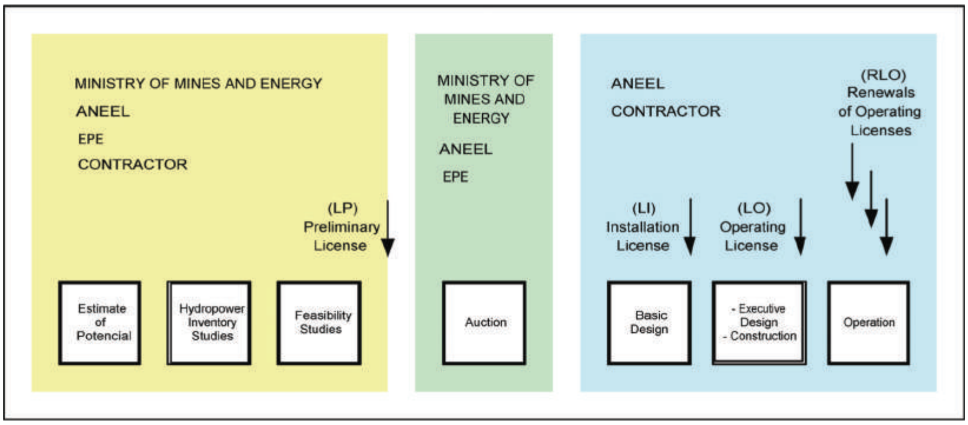



Figure 3.4.01 – Stages in the development of new hydropower projects

In the early years of the 21st century, hydroelectricity entities around the world and a new international commission published new guidelines for the development of hydropower projects that adopted the principles of sustainable development.[15] Methodologies and tools were also developed for socioenvironmental studies, such as georeferenced information systems and environmental zoning, which are fundamental for the integrated planning of river basins. It would not be wrong to say that Brazil's electricity sector entered the 21st century with the instruments required for it to address socioenvironmental issues adequately during the planning, building and operating of its projects.

However, even though hydropower projects use a clean, renewable source of energy, improving the quality of life of the population by the production of electricity, it would be wrong to ignore the major impacts caused by some hydroelectric power plants when it comes to the sustainability of ecosystems and local communities.

One way of getting a general idea of how these impacts are changing is to calculate the area flooded and the number of people affected as a ratio to the power generated, providing two broad indicators of the performance a hydropower plant.

In December 1989 there were 60 hydro plants operating in Brazil with an installed capacity of over 30 MW, summing 52,225 MW. Their reservoirs occupied 23,847 $km^2$, or 0.28% of national territory, and their ratio was 0.46 $km^2$/MW installed (assuming their final capacity). The 60 reservoirs for energy generation represented 11% of the total of 516 dams in the country (IIPDMA, vol. I, p.75).

In 2005, there were 116 hydroelectric power plants with over 30 MW in operation in Brazil, providing a total installed capacity of 71,000 MW and occupying 36,847.64 $km^2$. The areas of the reservoirs in use represented around 0.4% of national territory. Jointly, their ratio of area flooded to installed capacity was 0.52 $km^2$/MW.

According to the forecasts in the Ten-Year Plan for 2006-2015, which analyzed 46 out of 83 new reservoirs, land covering 5,862.21 $km^2$ would be flooded for these plants, raising the total amount of land flooded by 79% in comparison the 1989 figure.

The ratio between the flooded area and the final installed capacity remained practically unchanged between 1989 and 2005, staying at around 0.46 $km^2$/MW.

15 The aims of the report by the World Commission on Dams were: to examine what contribution the construction of large dams makes to development and assess alternatives for water use and energy generation; to develop internationally acceptable criteria, guidelines and standards for the planning, design, assessment, construction, operation, supervision and deactivation of dams. WORLD COMMISSION ON DAMS, 2000 – *Dams and Development: A New Framework for Decision-Making*. Earthscan Publication Ltd, London.

It is harder to gather data on the number of people affected, so in order to gain an idea of how this has changed over the years the information on the number of people affected by power plants with over 100 MW between 1992 and 2002 was taken. In this ten-year period, 17 hydropower plants came into service mostly in the southeast, northeast, central-west and south. Their combined installed capacity was 15,647 MW and they flooded 6,990 km² in total, affecting a total of 20,912 households (83,650 people). It is expected that the power plants included in the ten-year plan for 2005-2016 will affect some 73,300 people.

If we bear in mind the extra 30,963 MW generated in the period, the favorable hydrographic network and the low population density in Brazil, it is reasonable to state that the expansion of the country's hydroelectric power system has taken place in conjunction with improved indicators, especially if one bears in mind that the present rates already encompass the large reservoirs that were built many years ago.

## 4 Hydropower Projects in The Context of the Electricity Sector's Institutional Framework

Brazil's electricity sector was remodeled on March 15th 2004 by Act 10.848, which establishes how electricity is to be sold between the different industry agents and by these agents to end users within the context of the National Interconnected System (Sistema Interligado Nacional, SIN). By the new law, energy could now be sold on the regulated market (Ambiente de Contratação Regulada, ACR) or the free market (Ambiente de Contratação Livre, ACL).

The regulated market (ACR) is the market segment where electricity is bought and sold by brokers and distribution agents, preceded by tenders, while the free market (ACL) is the market where electricity can be bought and sold under the terms of freely-negotiated contracts.

The purchase and sale of energy in the regulated market takes place using bilateral contracts called Contracts for the Sale of Energy in the Regulated Market (Contratos de Comercialização de Energia no Ambiente Regulado, CCEAR), which must be signed by the generation companies and each of the distribution companies.

The tenders are different depending on whether the energy comes from an existing or new generation facility (something akin to the concepts of "old" and "new" energy). The law defines new generation plants as those that have no concession, permit or authorization, or are part of an existing venture that is being expanded and will result in an increase in installed capacity, by the beginning of the bidding process. There was one circumstance in which existing generation projects or expansion projects could take part in "new" energy bids by 2007. They had to fulfill all the following requirements: a) have obtained a concession or authorization by the date on which the act was passed; b) not have entered commercial service before January 1st 2000; and c) not have their energy contracted by the data the act was passed.

Customers of Independent Energy Producers (Produtor Independente de Energia Elétrica, PIE) may be electricity utilities operating in the regulated market. They may also be free-choice consumers, consumers of electricity and steam from independent cogeneration plants, groups of electricity consumers, independent of the voltage or load, according to the conditions previously agreed with the local distribution utility. Finally, they are also any consumer which demonstrates to the concession-granting authority that its local utility has not provided energy by 180 days from submission of the request. In the first and last two cases, the price of the electricity sold is subject to the general criteria set by the concession-granting authority.

One of the ways of selling energy to small hydros is to take part in PROINFA, the Alternative Energy Program, which was created on April 26th 2002 by Act 10.438. Originally, the starting date for the supply of energy under PROINFA contracts was December 30th 2006, but Act 11.075 of 2004 extended this date to December 30th 2008.

Act 9.427 (1996) provides for another way of buying and selling energy. It states that all hydropower plants with 30,000 kW or less, characterized as small hydros, can sell their electricity to a consumer or set of consumers that form a market, whether formally constituted as such or not, whose load is 500 kW or greater, independent of the grace periods defined in article 15 of Act 9.074 of 1995 (which defines a free consumer as one that consumes 10,000 kW or more and receives its energy supply at 69 kV or higher), with the possibility of the supply being supplemented by generation projects using the sources quoted in the act, with a view to assuring the supply of energy but limiting this proportion to 49% of the mean energy they produce. When the consumer or group of consumers are in isolated systems, the limit for the load drops to 50 kW.

Another way of selling energy is by distributed generation, which in the case of a hydropower project, and according to Decree 5.163 of 2004, is the output of electricity from concession-holders, permit-holders or authorization-holders connected directly to the buyer's electricity distribution system, whose plant has an installed capacity of 30 MW or less.

Before electricity can be bought from distributed generation projects, it must be made public by the distribution agent in order to assure publicity, transparency and equity of access to all stakeholders.

Additionally, there are a number of items that apply to the issue of benefits and charges.

Decree 2003 of 1996 gives independent generators and autogenerators free access to the transmission and distribution systems run by electricity utilities, for which they reimburse the transmission costs involved, which are calculated using the formula set by the regulatory agency of the concession-granting authority.

Act 9.427 of 1996 provides for and ANEEL stipulates (see regulation 77, 2004) a 50% reduction to be applied to tariffs for the use of electricity transmission and distribution systems on energy produced and consumed from hydroelectric power plants of an installed capacity of 30,000 kW or less.

Acts 10.438 (2002) and 10.848 (2004) gave power stations built in isolated electricity systems that replace energy generated by thermal plants fired by oil products the right to benefit from the Conta de Consumo de Combustível (CCC), an account to cover the fuel expenses of thermal plants, until 2002. This applies to independent generation or autogeneration hydropower plants between 1,000 kW and 30,000 kW, provided they are classified as small hydros, and the associated transmission and/or distribution system.

The following also have the right to benefit from the CCC in isolated systems:

1) autogenerators – for the portion of energy sold to distribution utilities;
2) independent generators – in the portion of energy sold by energy utilities to electricity consumers, independent of voltage or load, by the terms previously agreed with the local distribution utility or with any consumer that proves to ANEEL that its local utility cannot assure it a supply within 180 days of the date the request was made. Additionally, when the plants are classified as small hydros, it relates to the portion of energy sold to a consumer of group of consumers that form a market, whether or not it is formally constituted as such, whose load is 50 kW or over, provided that serving their requirements implies in reducing CCC expenses.

Act 10.848 of 2004 also states that the subrogation of hydropower projects with over 30 MW installed capacity and that have already received a concession, to be built entirely in an isolated system to replace thermal generation fired by oil products, is limited to 75% of the value of the project and until the quantity of the subrogated project reaches a total of 120 MW mean power, and they will be able to sell the energy generated to electricity utilities.

Additionally, independent generators and autogenerators are subject to the following financial charges: financial compensation for the states and municipalities and any federal entities that directly administrate water resources for the water used for electricity generation; electricity service inspection charge; and monthly charges for the Fuel Consumption Account (Conta de Consumo de Combustíveis, CCC).

## 5 Institutional Organization

The National Council for Energy Policy (Conselho Nacional de Política Energética, CNPE) was established on August 6th 1997 by Act 9.478. Its task is to propose national policies and measures related to energy to the President of the Republic. Its members are: the Minister of Mines and Energy (chair), Minister of Science and Technology, Minister of Planning, Budget and Management, Minister of Finance, Minister of the Environment, Minister of Development, Industry and Foreign Trade, President's Chief of Staff, one representative from each state and the federal district, one Brazilian citizen who is a specialist in energy, and one representative from a Brazilian university who is a specialist in energy.

The Ministry of Mines and Energy (Ministério de Minas e Energia, MME) was established on July 22nd 1960 by Act 3.782, extinguished in 1990 by Act 8.028 and re-established in 1992 by Act 8.422. It covers the following subjects: a) geology, mineral resources and energy resources; b) generation of hydroelectric energy; c) mining and metallurgy; and d) oil, fuels and electricity, including nuclear energy. The MME is also responsible for: a) rural energy projects, agricultural energy products, including countryside electrification, when this is covered by funds from the National Electricity Grid; and b) oversee the balance of electricity supply and demand in Brazil from a structural and industry perspective.

The Electricity Industry Monitoring Committee (Comitê de Monitoramento do Setor Elétrico, CMSE) was established in 2004 by Act 10.848 and constituted by Decree 5.175 of 2004 with the purpose of overseeing and assessing the continuity and security of electricity supplies across Brazil. It is chaired by the Minister for Mines and Energy and has four representatives from the Ministry of Mines and Energy and representatives from ANEEL, ANP, CCEE, EPE and ONS.

The National Electricity Agency (Agência Nacional de Energia Elétrica, ANEEL) is an autonomous state entity linked to the Ministry of Mines and Energy that was created on December 26th 1996 by Act 9.427. It is the electricity sector's regulatory agency and has the following powers: a) to regulate electricity services and provide ongoing inspections of the rendering of said services; b) to arbitrate conflicts of interest between agents from the electricity sector and between these and consumers; c) to implement federal government policies and guidelines for the exploitation of electricity and harnessing of water resources for energy generation; and d) to hold public tenders to contract utilities for the generation, transmission and distribution of electricity and to grant contracts for hydroelectric power plants.

The Energy Research Company (Empresa de Pesquisa Energética, EPE) was created on March 15th 2004 by Act 10.847 and operates under the auspices of the Ministry of Mines and Energy. It undertakes studies and research required for the planning of the energy sector in the areas of electricity, fossil fuels, renewable energy sources and energy efficiency.

The National Operator of the Electricity System (Operador Nacional do Sistema Elétrico, ONS) was created on August 26th 1998 by Act 9.648. It is a private, not-for-profit entity which is responsible for coordinating and controlling the operation of electricity generation and transmission facilities that make up the National Interconnected System (Sistema Interligado Nacional, SIN). It is inspected and regulated by ANEEL.

The North Region Technical Group (Grupo Técnico Operacional da Região Norte, GTON) was created on November 29th 1990 by Ministry of Infrastructure directive 895. It is responsible for planning isolated systems in the north region and overseeing their operation. Among other tasks, it is responsible for preparing the Operating Plan and Monthly Program for the Operation of Isolated Systems.

Câmara de Comercialização de Energia Elétrica (CCEE) was created on March 15th 2004 by Act 10.848 to replace the wholesale energy market (Mercado Atacadista de Energia Elétrica, MAE). It is a private, not-for-profit entity that is regulated and inspected by ANEEL. Its task is to enable the sale of electricity in the National Interconnected System (SIN). The tasks undertaken by CCEE include holding auctions for the purchase and sale of electricity, when this is delegated by ANEEL; keeping a record of all energy purchase and sale contracts signed within the regulated market; keeping a record of the amounts of energy sold under contracts signed in the free market; measuring and recording data on electricity purchase and sale operations; and calculating the Preço de Liquidação de Diferenças (PLD) for the different sub-markets.

Eletrobrás (Centrais Elétricas Brasileiras) was created in 1962 to carry out studies and projects for the construction and operation of power plants, power lines and substations, and to provide support for the government's strategic programs. Its main functions are to provide supplementary funding for the expansion of the electricity industry, to function as a holding of state-owned companies, to administrate industry charges and funds, to sell energy from Itaipu and from alternative sources of energy covered by Proinfa, and to coordinate GTON.

Generation Agents:

- Electricity Autogenerator – individual, company or joint venture of companies that are granted a concession or authorization to generate electricity for their own use.
- Public Service Utility (concession- or permit-holder) – company or joint venture of companies that holds a contract to render public electricity services on the federal ambit, granted by the concession-granting authority by means of public tender, regulated by Act 8.987 of February 13th 1995.
- Independent Electricity Generator (Produtor Independente de Energia Elétrica, PIE) – Company or joint venture of companies which are granted a concession or authorization from the competent authority to generate electricity, all or part of which they sell on their own account and at their own risk.

Transmission Agents:

- Transmission Companies – companies responsible for providing the equipment and facilities needed to convey electricity from power plants to centers of consumption.

Distribution Agents:

- Distribution Utilities – The electricity distribution market comprises 64 state-owned and private utilities that operate around the country. They serve around 47 million consumers, of which 85% are residential consumers from over 99% of Brazil's municipalities.

Energy Brokers:

- Broker – This is a company established especially to buy and sell electricity to utilities or consumers that have free choice of energy supplier. The activities of brokers are regulated by Decree 5.163 of July 30th 2004.

The National Environment System (Sistema Nacional do Meio Ambiente, SISNAMA) was created on August 31st 1981 by Act 6.938. It is made up of federal and state entities and by foundations established by the government that are responsible for protecting and improving the quality of the environment. It has the following structure:

- supreme body: government board;
- advisory body: National Council for the Environment (Conselho Nacional do Meio Ambiente, CONAMA);
- central body: Ministry for the Environment;
- executive body: Brazilian Institute for the Environment and Natural Renewable Resources (Instituto Brasileiro do Meio Ambiente e dos Recursos Naturais Renováveis, IBAMA);
- section bodies: state entities responsible for conducting programs and projects and for controlling and inspecting activities that may harm the environment; and
- local bodies: municipal entities responsible for controlling and inspecting these activities in their respective jurisdictions.

The National Council for Water Resources (Conselho Nacional de Recursos Hídricos, CNRH) is an entity that draws up rules for mediation between different water users, and is therefore one of the most important entities for implementing water resource management in Brazil. As it coordinates the integration of Brazil's public policies, it is known by society for conducting transparent dialog in decision-making processes in the field of water resource legislation. Its powers include:

- analyzing proposals to change legislation that will impact on water resources;
- setting complementary guidelines for the implementation of the National Policy for Water Resources;
- promoting the coordination of water resource planning with national, regional and state-wide plans and users;
- arbitrating conflicts over water resources;
- deliberating on projects for hydropower plants whose impacts will reach out beyond the states in which they will be built;
- approving proposals for new river basin committees;
- setting general criteria for authorizing the right to use water resources and for charging for their use; and
- approving the National Plan for Water Resources and overseeing its execution.

The Ministry of the Environment (MMA) is a public federal administration entity which has the following powers: a) national policy for the environment and water resources; b) policy for the preservation, conservation and sustainable use of ecosystems, biodiversity and woodland; c) proposals for social and economic strategies, mechanisms and instruments for improving the quality of the environment and the sustainable use of natural resources; d) policies for integrating the environment and production; e) environmental policies and programs for the Amazon region; and f) ecological/environmental zoning.

The National Water Agency (Agência Nacional de Águas, ANA) was created on July 17th 2000 by Act 9.984 as an autonomous state entity connected to the Ministry of the Environment but with administrative and financial autonomy. It is responsible for providing the technical conditions for the Water Bill to be implemented; promoting decentralized, participative management in conjunction with the entities that make up the National Water Resource Management System; implementing the management instruments provided for in Act 9.433 of 1997, including permits to use water resources; charging for water use and inspecting its use; and pursuing solutions to the country's two major problems: prolonged droughts, especially in the northeast, and river pollution.

CONAMA is the advisory body for the national environment system (SISNAMA) and was created on August 31st 1981 by Act 6.938 with the purpose of assisting, studying and proposing to the

Government Board guidelines for government policies for the environment and water resources, and to give opinions on standards compatible with ensuring the ecological balanced of the environment and a healthy quality of life.

IBAMA is an autonomous state entity with administrative and financial autonomy that was created on February 22nd 1989 by Act 7.735 by merging four environmental entities: the Department of the Environment (Secretaria do Meio Ambiente, SEMA), the Rubber Department (Superintendência da Borracha, SUDHEVEA), the Fishing Department (Superintendência da Pesca, SUDEPE) and the Brazilian Forest Institute (Instituto Brasileiro de Desenvolvimento Florestal, IBDF). It is connected to the Ministry of the Environment and has the following tasks: to put into practice national environmental policies for the preservation, conservation and sustainable use of environmental resources and their inspection and oversight; and to carry out further activities on the behalf of the Union in compliance with current legislation and guidelines provided by the Ministry of the Environment.

The state environmental agencies are responsible for issuing environmental licenses for plants on state rivers.

The National Development Bank (Banco Nacional de Desenvolvimento Econômico e Social, BNDES) was created on June 20th 1952 by Act 1.628 as an autonomous state entity, then redefined as a private federal company with its own equity on June 21st 1971 by Act 5.662. It reports to the Ministry of Development, Industry and Foreign Trade and its main purpose is to provide support for projects that contribute to the country's development.

# 6 Legislation Concerning Hydropower Developments

## 6.1 General Information

Act 5.655 of May 20th 1971 covers the legal remuneration for investments made by electricity utilities.

The Federal Constitution of 1988 states that all hydroelectric potential and all rivers in more than one state are owned by the Union, which has the right to directly exploit it for generating electricity or do so indirectly by means of authorization, concession or permit, in conjunction with the states where the hydropower potential is located.

Act 8.631 of March 4th 1993 contains provisions on the setting of tariffs for public electricity services and revokes the previous practice of setting fixed rates.

In 1995, Act 8.987 was passed that brought a major change to the way public services were to be contracted. It regulates article 175 of the Federal Constitution of 1988, which states that contracts for public services and works and permits for public services must be the object of public tenders.

There is one provision of particular note, which states that the winning bidder is responsible for reimbursing the expenses incurred in carrying out the studies, investigations, surveys, designs, works and other expenses or investments involved in the contract that were of use for the bid and were undertaken by the Concession-granting authority or on its authorization, as set forth in the notice of tender. This effectively gives parties interested in taking part in public tenders access to these documents.

Act 9.074 (1995) was passed in response to article 1, single paragraph, of Act 8.987, passed in the same year, chapter 2 of which addresses electricity services, and sets out by what terms all concessions, permits and authorizations for exploiting electricity services and facilities and plants to harness water courses are to be given, extended or granted.

This law sets out what projects can only be contracted after a public bidding process, in which case the concession-granting authority must define the “optimal project”. This is the overall energy potential of a plant from the cascade scheme selected for a river basin that is provided by the best dam axis, general physical layout, operating water levels, reservoir and installed capacity.

The National Electricity Agency (Agência Nacional de Energia Elétrica, ANEEL) was created on December 26th 1996 by Act 9.427 with the purpose of regulating and overseeing the generation, transmission, distribution and sale of electricity in compliance with government policies and guidelines. One of its powers is to hold public tenders for contracts for electricity generation, transmission or distribution services and to grant contracts for hydropower plants, as well as signing and managing contracts or permits for the rendering of public electricity services, public utility contracts, the issuing of authorizations, and the direct inspection or indirect inspection by means of agreements with state entities of contracts for the rendering of electricity services.

This document was amended by Acts 9.648 (05/27/98) and 10.438 (04/26/02), which establish rules for the granting of authorization for hydropower projects of over 1,000 kW and up to 30,000 kW for independent generation or autogeneration that are classified as small hydros.

ANEEL took over the technical archives, equity, duties, powers and revenues of DNAEE. The network of river gauging stations, technical archives and hydrology activities required for hydroelectric projects continue to be the responsibility of the Ministry of Mines and Energy under the temporary administration of ANEEL, as a member of the National Water Resource Management System.

On July 18th 2000, Act 9.984 was published in the federal publication *Diário Oficial da União* creating the National Water Agency (Agência Nacional de Águas, ANA) as an autonomous state entity with administrative and financial autonomy and linked to the Ministry of the Environment. Its remit is to implement the National Policy for Water Resources, making up part of the National Water Resource Management System. It is also responsible for granting the right to use water resources from bodies of water controlled by the Union by means of authorizations. It also sets and oversees the conditions for the operation of reservoirs with the purpose of assuring the multiple uses of the water resources and, in the case of reservoirs for hydropower generation, in conjunction with the ONS.

There is therefore some common ground in the remits of ANEEL and ANA. They should work together to obtain the declaration of available water resources for bidding processes for concessions or authorizations to harness the energy potential of bodies of water under Union control. However, when the energy potential is in water bodies controlled by states or the Federal District, the declaration of available water resources must be obtained from the state entity water resource management entity.

In 2003 a new bill was proposed for organizing Brazil’s electricity sector, whose objective is to ensure that energy demands are fulfilled reliably, profitably and in an economically sustainable manner.

This bill culminated in Act 10.848, passed on March 15th 2004, which pertains to the sale of electricity and amends other laws, including Acts 9.074/95 and 9.427/96.

Act 10.848 states that the sale of electricity among agents that hold concessions, permits or authorizations for electricity services or facilities, and between these agents and their consumers within the National Interconnected System (SIN), should be contracted within the regulated or free markets, and that the distribution agents in this system should make sure they meet the entire demand of their markets by taking out regulated contracts with the winners of bidding processes. These tenders should include procedures for contracting energy from existing generation facilities, new generation facilities and alternative sources.

The new law states that electricity generation concessions granted prior to December 11th 2003 will have up to 35 years to pay off their investments, counting from the date the contract was signed, with the right to extend this term for a further 20 years on the discretion of the concession-granting authority, which is an express amendment of the provisions of Act 9.074 (1995).

By the provisions of this law, distribution agents are prevented from undertaking electricity generation or transmission activities or from selling electricity to free-choice consumers, unless this be under the same conditions as those established for captive consumers. This restriction does not apply to distribution utilities that provide services for isolated electricity systems or whose market is limited to 500 GWh/year or less. It also amends Act 9.427/96, by which ANEEL was created, stripping this agency of its power to coordinate activities with states and the federal district, the harnessing of the hydroelectric potential of water courses and bringing these activities into line with the national policy for water resources, the signing of contracts granting concessions or permits for electricity utilities, the granting of authorizations, the definition of optimal projects, and the termination of contracts, all of which are returned to the Ministry of Mines and Energy (concession-granting authority).

This law was regulated on July 30th 2004 by Decree 5.163, which stipulates which of the agents that will take part in the new sector model are the winning agents, which are understood as being the holders of concessions, permits or authorizations issued by the competent authority for the generation, importation or sale of electricity.

The winning bidders for energy from new plants will be eligible to pay a fee in exchange for concessions for the generation of electricity as a public service, or concessions for the use of public assets if they are autogenerators or independent generators. New generation facilities are those which had no kind of concession, authorization or permit, nor were part of an existing project that was being expanded to increase its installed capacity, by the beginning of the bidding process.

When generation projects win that are classified as small hydros, as per ANEEL Resolution 652 of December 9th 2003 or as thermoelectric generation plants, they are granted an authorization. When electricity is being imported, these authorizations should include the introduction of the associated transmission systems and provide for unrestricted access to these systems within their technical boundaries by means of the payment of a charge to be approved by ANEEL.

Another law, Act 10.847, was passed together with Act 10.848, by which Empresa de Pesquisa Energética (EPE) was created, replacing CCPE as the entity to provide support for the Ministry of Mines and Energy's planning activities. EPE provides the research services and studies required for the planning of the energy sector, and is under the auspices of the Ministry of Mines and Energy. It is responsible for conducting studies to determine the "optimal projects" for hydropower projects, social impact studies, technical, economic and socioenvironmental feasibility studies for electricity projects and projects involving renewable resources, obtaining the preliminary environmental license and the declaration of available water resources needed for public tenders involving electricity generation and transmission projects, and conducting the studies needed for drawing up short-, medium- and long-term electricity generation and transmission expansion plans.

### 6.2 Contracts for Electricity Sector Projects

All public utility concessions, whether or not they are preceded by public works, must be put to public tender according to the terms of the specific legislation and in accordance with the principles of legality, morality, publicity, equity and objective judgment of criteria, and according to the terms of the notice of tender (art 14 of Act 8.987 of February 13 1995).

The notice of tender must be prepared by the concession-granting authority in compliance with the general criteria and standards contained in the legislation covering public tenders and contracts (art. 18 of Act 8.987 of 1995). In particular, in the case of public services preceded by public works, it must contain the details of the works, including the elements of the basic design needed to fully characterize said work, as well as the guarantees required for this specific part of the contract, according to each case and limited to the value of the work (item 15, art. 18, Act 8.987, 1995, plus the revision in Act 9.648 of May 27th 1999).

According to article 4 of Act 9.074 (July 7th 1995), all concessions, permits and authorizations for the rendering of electricity services and exploitation of electricity facilities and plants that harness water courses must be contracted out, extended or authorized according to the terms of this law and Act 8.987 and any other pertaining to the matter.

All public tenders held for contracts of this nature should be in compliance with the provisions of the following acts: 8.987 (February 13th 1995), 9.074 (July 7th 1995), 9.427 (December 26th 1996) and, as a general standard, 8.666, (June 21st 1993) and article 23 of Act 9.427 (1996).

The public tenders for the exploitation of hydroelectric energy must take the form of a competition or an auction and the concessions will be granted upon payment of a fee (Art. 24, Act 9.427, 1996).

No hydroelectric project may be subject to a bidding process without the "optimal project" having been defined by the concession-granting authority. The winning bidder may be responsible for developing the basic and executive designs (item 2, article 5, Act 9.074, 1995).

An "optimal project" is understood as being all the energy to be generated by a project in the final selected cascade option for a river basin, defined by its general layout, with the best dam axis, general physical layout, operating water levels, reservoir and installed capacity, as set out in article 5 of Act 9074, 1995

ANEEL should be notified for registration purposes whenever feasibility studies[16], pre-projects or projects for hydroelectric power plants are being developed, although this does not grant any preferential treatment in obtaining the concession for rendering a public service or use of a public asset (art. 28, Act 9.427, 1996).

Hydroelectric facilities of 1,000 kW or less and thermoelectric power plants of 5,000 kW or less do not require a concession or authorization, although the concession-granting authority's regulatory agency and inspection unit should be notified of their existence for registration purposes (art. 5, Decree 2.003 of September 10th 1996).

Hydroelectric facilities of over 1,000 kW and up to 30,000 kW for independent generation or autogeneration should first be authorized by ANEEL, provided they are classified as a small hydro (item 1, Art. 26, Act 9.427 of 1996, revised in Art. 4, Act 9.648 of 1998).

ANEEL resolution 394 of December 4, 1998 establishes that small hydros are hydroelectric power plants with an installed capacity of over 1 MW and no more than 30 MW, with a total reservoir area of up to 3.0 $km^2$ (art. 2).

The area of the reservoir is set by the water elevation associated with the 100-year flood flow (Single Paragraph of Art. 2, ANEEL resolution 394 of 1998).

Projects that exceed the maximum reservoir area may, depending on regional characteristics, also be defined as small hydros upon special consideration by the ANEEL board based on a technical report which sets out the related economic and socioenvironmental aspects, among others (Art 3, ANEEL resolution 394 of 1998).

Authorization for hydroelectric facilities of over 1 MW and no more than 30 MW, as set out in paragraph 8 of this section, is granted after the basic design is approved by ANEEL (art 2, ANEEL Resolution 395 of December 4th, 1998).

16 The overall design of a given plant from the best cascade scheme selected in the survey of hydroelectric potential is defined at the feasibility stage, with a view to achieving optimal technical, economic and environmental performance and the best evaluation of associated costs and benefits.

According to the terms of ANEEL Directive 173 of May 7th 1999, authorization to exploit a hydroelectric power plant with a capacity of over 1 MW and no more than 30 MW for independent generation or autogeneration can only be granted upon the submission of the basic design and proof of receipt of the documents submitted for environmental licensing purposes by the relevant environmental management entity.

When an authorization is issued by ANEEL according to the terms of this directive, the start of construction works are still dependent on the submission of the Installation License and approval of the Basic Design. The Basic Design can be approved before the authorization is given, provided a Preliminary License has been submitted.

Those projects that fall outside the category of small hydros, as defined in ANEEL resolution 394 of 1998, as listed in paragraphs 10 to 20 of this item, are subject to concession via a bidding process (single paragraph, art. 2, ANEEL resolution 395 of 1998).

Those parties interested in applying for a concession to run hydroelectric power plants of a capacity greater than 30 MW or those that fall outside the category of small hydro as set forth in paragraph 14 of this text should submit their Feasibility Studies or Basic Designs to ANEEL, requesting the project's inclusion in the public tender program (art. 3 of ANEEL resolution 395 of 1998).

The general procedures for registering the Feasibility Studies and Basic Designs are set forth in ANEEL resolution 395 of December 4th 1998 (articles 4-12).

The subjects that must be covered in the Feasibility Studies and Basic Designs include work in conjunction with environmental and water resource management entities with a view to defining the optimal project, the assurance of multiple water uses, and the obtainment of the relevant environmental license(s).

The general procedures for the choice of the Feasibility Studies and Basic Design to be put to tender are established in ANEEL resolution 395 of December 4th 1998 (art. 13 and 14).

As noted in paragraph 8 of this section, the final Feasibility Study report may form the technical basis for the bidding process for hydroelectricity generation projects.

The overall design of a given project from the best cascade option selected in the Inventory Studies is defined at the Feasibility stage, with a view to achieving optimal technical, economic and environmental performance and the best evaluation of associated costs and benefits.

The Basic Design should incorporate the main items set forth in the notice of tender for the project and those that are defined in the Feasibility Studies in order to assure the energy generation as originally estimated. These items are:

- maximum upstream water level;
- downstream water level;
- minimum capacity; and
- geographic coordinates.

The building of thermoelectric power plants of over 5,000 kW for autogeneration or independent generation is subject to authorization (see Art. 4, Decree 2.003 of September 10th 1996).

Those hydropower plants with a capacity of 1,000 kW or less and thermoelectric plants of 5,000 kW or less do not require a concession or authorization, but the concession-granting authority's regulatory and inspection entities must be notified of their existence for registration purposes (art. 5, Decree 2.003, September 10th 1996).

All transmission facilities that are part of the Basic Network of Interconnected Electric Systems[17] must be included in a bidding process, and will work as integral facilities of these systems. Their operating rules will be defined by an agent under federal control so as to assure the optimization of existing or future electricity resources (item 1, art. 17, Act 9.074, 1995).

Those transmission facilities within the ambit of a distribution utility may be considered by the concession-granting authority as making up part of the distribution contract already granted (item 2, article 17, Act 9.074, 1995).

Those transmission facilities of interest only to generation plants may be considered an integral part of their respective concessions, permits or authorizations (item 2, article 17, Act 9.074, 1995, and textual review in act 9.648 of 1998).

Imports and exports of electricity require authorization from ANEEL, as does the implementation of the respective transmission systems.

### 6.3 Resolutions Relating to Hydropower Inventory Studies

Hydropower Inventory Studies of river basins are defined as the planning stage when the hydroelectric potential of a river basin is quantified by studying different cascade options and deciding on an optimal project. An optimal project is understood as being all the energy to be generated by a project in the final selected cascade option for a river basin, defined by its general layout, with the best dam axis, general physical layout, operating water levels, reservoir and installed capacity, as set out in article 5 of Act 9074, 1995.

Inventory Studies may be undertaken by any of the stakeholders upon their registration with ANEEL. The general procedures for registering and approving these studies are set down in ANEEL resolution 393 of December 4th 1998.

ANEEL resolution 393 also recognizes the right to the reimbursement of costs incurred and recognized by ANEEL should any of the projects identified in the Inventory Studies be included in a public tender for electricity generation contracts.

The procedures for presenting expenses incurred in developing studies or designs for hydropower projects and the regulation of the remuneration of these sums are set forth in DNAEE Directive 40 of February 26th 1997.

ANEEL resolution 398 of September 21st 2001 sets out the general requirements for the submission of studies and the specific conditions and criteria for analyzing and comparing Inventory Studies, with a view to selecting amongst competing studies.

### 6.4 Financial Compensation and Royalties

The Federal Constitution of 1988 states that Brazilian states, the federal district, municipalities, and entities that are administrated directly by the Union have the right to a share of the profits from activities involving the exploration of oil, natural gas and water resources for the purposes of generating electricity, and of other mineral resources within Brazilian territory, the continental shelf, territorial waters or the exclusive economic zone, or to financial compensation for such activities.

Act 7.990 of December 28th 1989 provides financial compensation for the use of water resources for electricity generation. The amount currently paid by contract- or authorization-holders for such activities is 6.75% of the value of the energy produced, which is calculated by multiplying the energy generated by a reference tariff. When this legislation was passed, plants with a nameplate capacity of

17 The Basic Network Interconnected Electric Systems will consist of all the substations and transmission lines at voltages of 230 kV or more that are part of electricity service concessions that have been granted by the competent authority.

10 MW or less were exempted from this payment. This exemption has been extended to plants of a capacity of 30 MW or less, provided they are classified as small hydros.

Of the total of 6.75%, 6% goes to the municipalities in which the dams are built and the states that host the reservoirs. This amount is broken down as follows: 45% for states, 45% for municipalities and 10% for the Union, of which 3% goes to the Ministry of the Environment, 3% to the Ministry of Mines and Energy and 4% goes to the National Fund for Scientific and Technological Development, which is administrated by the Ministry of Science and Technology.

In the case of hydropower plants that benefit from upstream reservoirs, the extra energy is calculated as being generated by these regulation reservoirs, and ANEEL is responsible for making the evaluation necessary to determine the proportion of financial compensation due to the states, federal district and municipalities affected by these reservoirs.

In the case of Itaipu, the percentages owed to the entities directly administrated by the union, the states and the municipalities directly affected by the plant are unchanged, but the other percentages established above are based on 85% of the royalties owed by Itaipu Binacional ao Brasil, as per Annex C, item III of the Itaipu Treaty, signed on March 26th 1973 between Brazil and Paraguay, and its related documents, while 15% goes to the states and municipalities with reservoirs upstream from the power plant, which contribute to the amount of energy it produces.

The remaining 0.75% goes to the Ministry for the Environment for the implementation of the National Policy for Water Resources and the National Water Resource Management System. The portion of the 6% that the Ministry for the Environment receives should be used for implementing the National Policy for Water Resources, the National Water Resource Management System and for managing the country's network of water gauging stations.

## 6.5 National Policy for Water Resources

The management of water resources should always provide for multiple water uses based on the guiding principles of the National Policy for Water Resources. The harnessing of hydroelectric power is one such use of water resources.

The concern for multiple water uses is stressed in Decree 24.643, better known as the "Water Code", which establishes that in all hydroelectric plants the "requirements of general interests" must be assured, which are:

- consumption and other needs of riverside communities;
- public health;
- shipping;
- irrigation;
- protection against flooding;
- conservation and free circulation of fish;
- flow maintenance.

However, the conditions for exploiting water for other uses are limited by the provisions of the decree that state that water and energy reserves for public services (Union, states or municipalities) should not deprive a hydropower plant of more than 30% of the energy at its disposal. The Water Code further states that utilities lose the rights granted to them in their concessions if they repeatedly use flows greater than those to which they were entitled, whenever this infraction affects the quantity of water reserved for other uses.

According to Federal Constitution 1988, the lakes, rivers and any water courses in land under its control or which flow through or occupy more than one state, which form borders with other countries or extend into foreign territory or come from it, as well as the land along their banks, the river beaches and the hydroelectric potential are all the property of the Union. Meanwhile, the States own all flows, sources or stores of surface or ground water, except those owned by the Union and which arise as the result of works undertaken by the Union.

The Federal Constitution of 1988 states that it is the Union's right to exploit electricity facilities and harness the energy from water courses, either directly or through authorization, concession or permit, working in coordination with the States where these sources of energy are. The harnessing of "low capacity renewable energy potential" is exempted from the requirement to obtain an authorization or concession.

It also states that it is the right of all citizens to live in an ecologically balanced environment, which is to the benefit of all people and is essential for a healthy quality of life, the defense and preservation of which for present and future generations is the duty of the public authorities and society as a whole. As such, the public authorities are bound to carry out a number of actions, including that of "requiring, in the terms of the law, that prior environmental studies be undertaken and made public before any works or activities can be undertaken that could potentially harm the environment."

Another concern is the indigenous issue, which is highlighted in the Constitution. An example of this is the fact that it is the exclusive power of the National Congress to authorize the exploitation and harnessing of water resources and the exploration and extraction of mineral resources on indigenous lands, and grants indigenous peoples the right to occupy their traditional lands, stating that "the harnessing of water resources, including hydropower potential, and the exploration and extraction of mineral resources on indigenous lands can only be carried out upon authorization by the National Congress, after the communities affected have been heard, and assuring them a share of the profits of the mining, in abidance with the law."

In this chapter we set out the main legal instruments passed on a federal level that pertain to the planning and operating of hydropower stations, starting with the issue of financial compensation addressed in the Constitution.

### Legislation Pertaining to Water Resources

Act 9.433 of January 8th 1997 created the National Policy for Water Resources and the National Water Resource Management System.

The aim of the National Policy for Water Resources is to assure the availability of water to meet the needs of present and future generations and levels of water quality compatible with its respective uses; the efficient, integrated use of water resources, including water transportation, with a view to sustainable development; and the prevention of and defense against critical hydrological events of a natural origin or arising from the inappropriate use of natural resources.

The following are instruments of the National Policy for Water Resources:

- all plans for water resources;
- the classification of bodies of water according to their most prevalent uses;
- the granting of rights for the use of water resources;
- charging for the use of water resources;
- compensation for local authorities; and
- information database on water resources.

The harnessing of hydroelectric power is one of the water uses that is subject to the granting of rights, as are all other uses that alter the hydrological patterns, and the quantity or quality of water existing in the body of water.

These rights are granted by the competent authority of the federal, state or federal district executive, though the federal executive may also delegate to the states or the federal district the power to grant rights for the use of waters under Union control.

The granting of such rights is subject to the priorities of use established in the individual plans for water resources, and should respect the classification of the body of water in question and assure that the conditions required for water transportation, when such exists, are maintained, adding that the multiple uses of the water should also be preserved. The plans for water resources are master plans that provide guiding principles for implementing the National Policy for Water Resources and water resource management, and are prepared per individual river basin, per state and for the whole country.

The granting of such rights and the use of water resources for the purposes of generating electricity is subordinated to the National Plan for Water Resources. Until the National Plan for Water Resources is approved and regulated, the use of hydraulic potential for generating electricity will continue to be covered by the relevant sector legislation.

It is the responsibility of River basin Committees to approve the Plan for Water Resources for the river basin in question and that of the National Council for Water Resources (Conselho Nacional de Recursos Hídricos, CNRH) to oversee its execution and approve the National Plan for Water Resources and to determine any measures needed to meet its targets.

On January 30th 2006 at the 17th extraordinary meeting of the National Council for Water Resources, the National Plan for Water Resources was approved as per the terms of resolution 58 of the same date, published in *Diário Oficial da União* on March 8th 2006, comprising the following volumes:

I – Overview and State of Water Resources in Brazil;

II – Waters for the Future: Scenarios for 2020;

III - Guidelines; and

IV – National Programs and Targets.

This resolution states that the operational details of the programs and targets contained in item 4 should be coordinated by the Department of Water Resources, within the ambit of the Ministry of the Environment, and submitted for approval by the National Council for Water Resources by December 31st 2007.

One of the major guidelines of subprograms V.2 (Ensuring Compatibility and Integration between Sector Projects and Incorporation of Guidelines of Interest to the Integrated Management of Water Resources) and V (Program for Inter-Sector, Inter-Institutional and Intra-Institutional Coordination in the Management of Water Resources) is to assess ways of implementing article 52 of Act 9.433 of 1997 (pp 20 and 61 of Volume 4 – National Programs and Targets).

It is also the responsibility of the River Basin Committee to hold discussions about the issues concerning the water resources within its area of influence and to coordinate the action of the entities involved, and provide arbitration at a first administrative level for any conflicts over water resources. The National Council for Water Resources is also responsible for ensuring that water resource planning is in line with national and state-wide planning and with plans for different groups of users, and to deliberate on projects for harnessing water resources whose repercussions may extend beyond the state which hosts them.

Act 9.984 of July 17th 2000, which covers the creation of the National Water Agency (ANA), establishes that in its interaction with the electricity sector this agency has the power to:

- issue authorizations that grant the right to the use of water resources in bodies of water controlled by the Union;
- inspect the uses of water resources in bodies of water under Union control;
- define and inspect the operating conditions of reservoirs by public and private entities, with a view to assuring the multiple uses of water resources, as set forth in the water management plans for the respective river basins. The conditions for the operation of reservoirs for hydroelectric facilities will be set in coordination with the ONS;
- foster the coordination of the activities undertaken by the national network of river gauging stations in association with the public or private entities that are part of it or use it;
- organize, implement and manage the National Water Resource Information System (Sistema Nacional de Informações sobre Recursos Hídricos).

The granting of rights to use water resources under Union control, including hydropower facilities, will respect the following deadlines, counting from the date that the respective authorizations are published:

- up to two years to start implementing the project in question;
- up to six years to conclude the implementation of the project;
- up to 35 years for the term of the authorization is granted.

Additionally, in the case of concessions or authorizations for utilities or for the generation of hydroelectric energy, the deadlines are the same as those on their respective concession or authorization contracts. Meanwhile, the deadlines for beginning and concluding the implementation of projects may be extended when the size and the socioeconomic importance of the project so justify, in the view of the National Council for Water Resources, while the duration of the concession or authorization can be extended by the National Water Agency (ANA), in compliance with the priorities set in the Plans for Water Resources.

Act 9.984 of 2000 states that for public tenders for concessions or authorizations to harness the hydroelectric potential of water bodies under Union control, ANEEL should work together with ANA to first obtain a declaration of available water resources. When the water resources are under state or federal district control the declaration of available water resources is obtained from the respective water resource management entity.

The declaration of available water resources is automatically transformed by the competent authority into an authorization to use such water resources for the institution or company that receives the concession or authorization from ANEEL to harness the energy potential of the water resource in question.

According to the provisions of Act 10.847 of March 15 2005, EPE is responsible, among other things mentioned in the law, for obtaining the preliminary environmental license and the declaration of available water resources, which are needed for tenders for hydroelectricity generation and electricity transmission projects selected by the company.

Resolution 16 issued by the National Council for Water Resources (CNRH) on May 8th 2001 sets the basic guidelines for granting the right to use water resources, stating that ANEEL should obtain a declaration of available water resources before it can hold a bid for a concession or authorize the harnessing of hydroelectric potential, and this declaration will be turned into an instrument that grants the right to use said water resources. Further, CNRH Resolution 37 of March 26th 2004 sets more specific guidelines for granting the use of water resources for the construction of dams on bodies of water under state, federal district or union control. It sets the following terms:

- minimum outflow: minimum flow for the satisfactory fulfillment of the needs of multiple uses of the waters, which is instrumental in the operation of the reservoir;
- contingency plan: set of measures and procedures required to ensure that the authorized multiple uses of the water continue to be fulfilled, while observing minimum outflows;
- emergency action plan: document containing the procedures to be taken in emergency situations, as well as the flood maps indicating the maximum range of floodwaters and their respective times of arrival in the event of the rupture of the dam;
- sector declaration: declaration issued by the respective government department.

CNRH Resolution 37 states that at the start of the project planning stage, the main stakeholder should contact the competent authority to request a list of the documents and details of the technical studies required for the corresponding application for the right to exploit the water resources, and that this competent authority will define what information must be included in the technical studies at the planning, design, construction and operation stages of the project, formulating a *Termo de Referência* (official list of technical criteria) with the hydrological characteristics of the river basin, the size of the dam, the purpose of the works and the uses of the water resources. The competent authority will notify the main stakeholder of when it should submit the most important documents, such as environmental licenses, sector plans and emergency action plans for the project. It should be noted that the absence of a sector plan, when justified, should not prevent the application for the right to use water resources from being made or analyzed. In this case, the competent authority will be liable for taking any measures required for the process to advance normally.

The rules for operating the reservoirs, the emergency action plan and the contingency plan can be reassessed by the competent authority and by ANA (as far as the items within its ambit are concerned), taking into account the multiple water uses, risks arising from accidents, and critical hydrological events. The resolution also states that users should establish and maintain reservoir monitoring activities (upstream and downstream), sending the competent authority the data recorded and/or measured in the format determined by said authority.

ANA resolution 131 of March 11th 2003 covers procedures for the issue of the declaration of available water resources and the concession or authorization for the right to use water resources for projects of over 1 MW in bodies of water under Union control. This resolution lists the documents that ANEEL must send ANA to obtain the declaration, as well as setting its validity at up to 3 years, renewable for a further 3 years. It also exempts holders of concessions or authorizations issued by March 11th 2003 from applying for a new concession or authorization.

## 6.6 National Policy for the Environment

This section sets out the main legal instruments covering the environmental aspects of hydropower facilities, as well as the process for environmental licensing. Rather than a very detailed explanation of the legislation, a summary of the most important aspects is given.

### Brazil's Environmental Legislation and Hydropower Projects

Brazil's environmental legislation is formed of instruments for the control, prevention, promotion, fostering of the environment, and has taken great strides even with respect to international legislation. Brazil's legislation aims to encourage voluntary actions, provide for environmental planning and preventive actions, going beyond the limitations of corrective and mitigating actions.

Act 6.938 of 1981 introduced the National Policy for the Environment, its objectives, and the mechanisms by which it was formulated and is to be implemented. By this law, the National

Environment System (Sistema Nacional do Meio Ambiente, SISNAMA) and the Technical Federal Register of Instruments and Activities for the Defense of the Environment (Cadastro Técnico Federal de Atividades e Instrumentos de Defesa Ambiental) were established. The policy's instruments include[18]:

- environmental quality standards;
- environmental zoning;
- assessment of environmental impacts;
- licensing and review of activities that cause or could cause pollution.

By this law, the construction, installation, expansion and operation of facilities and activities that use environmental resources that are considered polluting or potentially polluting, as well as those that could in any way harm the environment, are subject to prior licensing by the relevant state entity. These requirements are within the ambit of the National Environmental System (Sistema Nacional do Meio Ambiente, SISNAMA) and IBAMA, and do not substitute the other licenses required. In the case of activities and works within a national or regional ambit that cause a major environmental impact, IBAMA is the entity responsible for issuing licenses.[19]

Environmental licensing and environmental impact assessments are essential instruments for environmental planning and prevention of damage to the environment. The environmental licensing process for those activities that affect the environment is regulated by CONAMA resolution 001 of January 23rd 1986. This resolution sets the liabilities, basic criteria and general guidelines for the use and implementation of environmental impact assessments within the scope of the National Policy for the Environment. The resolution also states that the licensing of activities that alter the environment, such as electricity generation plants over 10 MW, whatever their primary source of energy, require an Environmental Impact Assessment (EIA) and Environmental Impact Report (RIMA), which must be submitted for approval by the competent authority[20].

CONAMA resolution 006 of September 16th 1987 sets out the general rules for the environmental licensing of large-scale works, especially those in which the Union has a particular interest, with the purpose of bringing into line the concepts and language used by the different stakeholders in the process. In virtue of the importance and specific nature of the cycle of development of electricity projects, specific procedures have been designed for licensing projects in this area. From the analysis of their feasibility during the initial study phase until their construction and operation, hydropower facilities are subject to a set of studies and procedures in order to ensure that the projects are as environmentally friendly as possible.

The environmental licensing process requires the following environmental licenses at the different stages of introducing new hydroelectric power plants[21]:

- Preliminary License (Licença Prévia, LP) – granted at the project planning stage, approving its location and design, attesting to its environmental feasibility, and setting requirements and preconditions to be met at the location, installation and operation phases, in accordance with existing municipal, state and federal plans for land use;
- Installation License (Licença de Instalação, LI) – authorizes the beginning of construction in compliance with the specifications contained in the approved plans, programs and projects, including the environmental control measures and other factors;

18 Act 6938/81 – Art. 9, items 1 to 4.

19 Act 6938/81 – Art. 10.

20 CONAMA resolution 001/86 – Art. 2

21 CONAMA resolution 006/87 – Art. 4

- Operating License (Licença de Operação, LO) – authorizes the operation of the plant, upon verifying the effective compliance with the previous licenses, environmental control measures and other factors required for its operation. The granting of this license depends on compliance with all items examined and approved for the first two licenses (LP and LI).

Environmental licenses can be issued individually or successively, depending on the nature, characteristics and phase of the project or activity.

The EIA and RIMA are required before the preliminary license can be granted, and should be prepared at the preliminary planning phase. They contain the basic or essential requirements, guidelines, recommendations and limitations that must be addressed at the planning, installation and operating phases of the project. The final project should meet all the recommendations set out in the EIA/ RIMA.

The Federal Constitution of 1988 makes the public authorities responsible for "demanding, in the terms of the law, that a preliminary environmental study be undertaken and made public before a work or activity that could potentially cause major environmental damage is installed," (art. 225, item &1, VI). This consolidates the role of environmental impact studies as preventive instruments within the ambit of the National Policy for the Environment, and gives the Public Prosecution Service (Ministério Público) a central role in protecting Brazil's environment.

Another concern is the indigenous issue, which is highlighted in article 49 of the Constitution, which states that it is the exclusive power of the National Congress to authorize the exploitation and harnessing of waters on indigenous lands. In article 231, paragraph 3, in acknowledging indigenous peoples' original rights to the lands they traditionally occupy, it states that "the harnessing of water resources, including hydropower potential, and the exploration and extraction of mineral resources on indigenous lands can only be carried out upon authorization by the National Congress, after the communities affected have been heard, and assuring them a share of the profits of the mining, in abidance with the law," thereby guaranteeing these peoples' rights.

CONAMA resolution 237/97 amends resolution 001/86 in aspects pertaining to environmental licensing. The new resolution covers new topics, including a list of projects subject to environmental licensing, confirming that environmental licensing will depend on the prior issue of an EIA and RIMA for projects that could cause environmental damage, and environmental studies of a relevant nature are required for those that are unlikely to cause harm to the environment.

While amending and expanding on resolutions 001/1986 and 006/87, CONAMA resolution 237/1997 also sets out the powers of environmental entities, previously mentioned in Act 6.938/1981. It defines not only the powers of the different environmental bodies from different spheres of government, but also states that projects should be licensed by a single entity.

Article 4 of this resolution sets out the situations in which IBAMA is responsible for the environmental licensing, these being for activities and projects with a major environmental impact of a national or regional scope.

In November 2002, Resolution 15 of the National Council for Energy Policy (Conselho Nacional de Política Energética, CNPE) created a working group to propose procedures and mechanisms designed to ensure that as of 2004, all projects designed to expand electricity supply have a Preliminary Environmental License before they can be authorized or put to tender. Likewise, Act 10487/2004 authorizes the creation of EPE, giving it the power to obtain preliminary licenses and issue a declaration of available water resources needed for tenders of hydroelectric generation facilities selected by EPE.

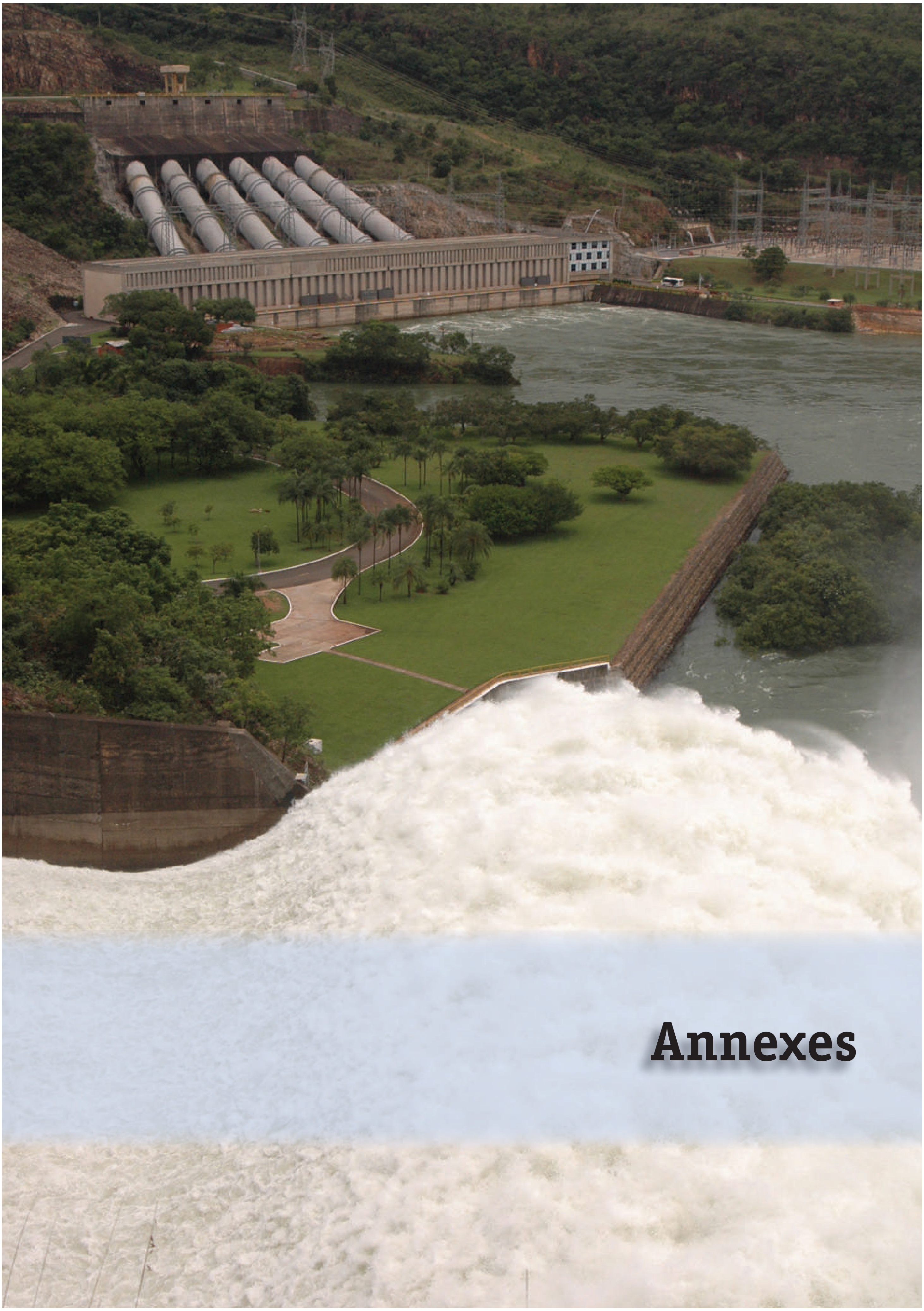
Aside from these general legal and normative instruments, when studies and projects are being developed for the electricity industry, all federal, state and municipal environmental legislation should be fulfilled, not only for environmental licensing purposes, but for the different topics relating to the physical, biota, socioeconomic and cultural factors covered in environmental studies.

Although CONAMA resolutions 006/87 and 237/97 regulate and adjust environmental licensing requirements for large-scale works, especially for electricity projects, they do so focusing on projects in isolation. However, with the passage of time and the experience gained by companies from this sector, it has been found that for hydroelectric projects, socioenvironmental studies have to be made with a broad enough reach to cover the set of projects from a single river basin, which is the scope of the Hydropower Inventory Studies, and which has a direct impact on industry planning. The first initiative in this respect was taken when the Manual for Hydropower Inventory Studies was reviewed in 1996 and 1997. The socioenvironmental aspects addressed in it were expanded and gained new depth, being given equal importance as the other topics covered (engineering and economic) in the decision-making process for selecting the projects to comprise the best cascade.

The integration of socioenvironmental aspects from the earliest stages of planning is an increasing requirement. An example of this is the analysis of the likely interferences brought about by the projects in the final selected cascade, which is now required by environmental entities and the Public Prosecution Service, resulting in the execution of Integrated Environmental Analyses (IEA) for basins with projects that already exist, are under construction or are being planned.

Methodological improvements of this nature are now common practice in the electricity sector and are increasingly being incorporated into the methodology used in Inventory Studies in a quest to refine them. However, these improvements have not yet led to any change in environmental legislation, since Inventory Studies precede the studies and design stages addressed by the legislation that pertains to the licensing process for such projects.

However, studies for hydroelectricity projects in a given river basin – which are of strategic importance to planning the supply of electricity – are gaining in importance for the environmental area. Even if they are not covered by the law, they can enhance the quality of environmental licensing processes, in that Inventory Studies include Integrated Environmental Assessments, which provide an analysis of the synergistic and cumulative effects arising from the environmental impacts caused by the hydropower projects within the basin, providing indicators, parameters and guidelines for the preparation of environmental studies that are required for the environmental licensing of each project.



# Annexes

# ANNEXES

# ANNEX A

## Graphs and Tables from the Preliminary Studies

| Name of Graph – PRELIMINARY STUDIES | Name of File |
|---|---|
| HYDRAULIC TURBINES<br>selection of type of turbine | grfA01.pdf |
| LAND DEVELOPMENTS IN THE PLANT AREA<br>unit cost per installed MW | grfA02.pdf |
| RIVER DIVERSION AND CONTROL<br>global cost of civil construction work and hydromechanical equipment | grfA03.pdf |
| EMBANKMENT DAMS (earthfill and rockfill) – COST PER METER OF LENGTH OF CREST<br>cost per meter of crest | grfA04.pdf |
| ROLLER-COMPACTED CONCRETE DAM<br>cost per meter of crest | grfA05.pdf |
| CONVENTIONAL CONCRETE DAM<br>cost per meter of crest | grfA06.pdf |
| TRANSITION AND RETAINING WALLS MADE OF CONCRETE<br>global cost | grfA07.pdf |
| CHANNEL SPILLWAY (LOW OGEE CREST)<br>CIVIL CONSTRUCTION WORK – global cost | grfA08.pdf |
| ROLLER-COMPACTED CONCRETE OVERFLOW SPILLWAY<br>(HIGH OGEE CREST) – CIVIL CONSTRUCTION<br>cost per $m^3/s$ of capacity | grfA09.pdf |
| CONVENTIONAL CONCRETE OVERFLOW SPILLWAY<br>(HIGH OGEE CREST) – CIVIL CONSTRUCTION<br>cost per $m^3/s$ of capacity | grfA10.pdf |
| SURFACE SPILLWAY<br>HYDROMECHANICAL AND HOISTING EQUIPMENT<br>unit cost | grfA11.pdf |
| INTAKE – CIVIL CONSTRUCTION<br>cost per block | grfA12.pdf |
| INTAKE – EQUIPMENT FOR KAPLAN TURBINES WITH A CONCRETE SEMI-SPIRAL CASING<br>global cost of hydromechanical and lifting equipment for the water intake | grfA13.pdf |
| INTAKE – EQUIPMENT FOR BULB TURBINES<br>global cost of hydromechanical and lifting equipment for the water intake | grfA14doc |
| INTAKE – EQUIPMENT FOR FRANCIS TURBINES OR KAPLAN TURBINES WITH A STEEL SPIRAL CASING<br>global cost of hydromechanical and lifting equipment for the water intake | grfA15.pdf |
| HEADRACE CANALS<br>cost per meter | grfA16.pdf |
| HEADRACE TUNNELS<br>cost per meter | grfA17.pdf |
| STEEL SURFACE PENSTOCKS<br>cost per meter (without valves) | grfA18.pdf |
| PRESSURE TUNNELS<br>cost per meter (without valves) | grfA19.pdf |
| FRANCIS TURBINES<br>unit cost | grfA20.pdf |
| KAPLAN TURBINES WITH A STEEL SPIRAL CASING<br>unit cost | grfA21.pdf |
| KAPLAN TURBINES WITH A CONCRETE SEMI-SPIRAL CASING<br>unit cost | grfA22.pdf |
| BULB TURBINES<br>unit cost | grfA23.pdf |

| | |
|---|---|
| VERTICAL-AXIS GENERATORS<br>unit cost | grfA24.pdf |
| CONVENTIONAL HORIZONTAL-AXIS GENERATORS<br>unit cost | grfA25.pdf |
| HORIZONTAL-AXIS BULB GENERATORS<br>unit cost | grfA26.pdf |
| TOTAL COST OF CIVIL CONSTRUCTION FOR OUTDOOR POWERHOUSE<br>global cost | grfA27.pdf |
| ROADS<br>unit costs (R$/km) | quadA01.pdf |
| RAILROADS<br>unit costs | quadA02.pdf |
| ROAD BRIDGES<br>unit costs (R$/m$^2$) | quadA03.pdf |

# ANNEX B

## Graphs and Tables from the Final Studies

| Name of Graph – FINAL STUDIES | Name of File |
|---|---|
| HYDRAULIC TURBINES<br>selection of type of turbine | grfB01.pdf |
| PELTON TURBINES<br>initial specific velocity | grfB02.pdf |
| PELTON TURBINES<br>coefficient of peripheral velocity | grfB03.pdf |
| FRANCIS TURBINES<br>initial specific velocity | grfB04.pdf |
| FRANCIS TURBINES<br>coefficient of peripheral velocity | grfB05.pdf |
| KAPLAN TURBINES<br>initial specific velocity | grfB06.pdf |
| KAPLAN TURBINES<br>coefficient of peripheral velocity | grfB07.pdf |
| BULB TURBINES<br>initial specific velocity | grfB08.pdf |
| BULB TURBINES<br>coefficient of peripheral velocity | grfB09.pdf |
| FRANCIS TURBINES<br>unit cost | grfB10.pdf |
| KAPLAN TURBINES WITH A STEEL SPIRAL CASING<br>unit cost | grfB11.pdf |
| KAPLAN TURBINES WITH A CONCRETE SEMI-SPIRAL CASING<br>unit cost | grfB12.pdf |
| BULB TURBINES<br>unit cost | grfB13.pdf |
| CONVENTIONAL HORIZONTAL-AXIS GENERATORS<br>unit cost | grfB14doc |
| HORIZONTAL-AXIS BULB GENERATORS<br>unit cost | grfB15.pdf |
| VERTICAL-AXIS GENERATORS<br>unit cost | grfB16.pdf |
| MAIN BRIDGE CRANE FOR POWERHOUSE<br>unit cost | grfB17.pdf |
| MAIN GANTRY CRANE FOR POWERHOUSE<br>unit cost | grfB18.pdf |
| LAND DEVELOPMENTS IN THE PLANT AREA<br>unit cost | grfB19.pdf |
| INSTALLATIONS AND FINAL WORKS FOR THE POWERHOUSE<br>unit cost per installed MW | grfB20.pdf |
| TAINTER GATE FOR SURFACE SPILLWAY<br>unit cost | grfB21.pdf |
| TAINTER GATE FOR BOTTOM SPILLWAY<br>unit cost | grfB22.pdf |
| FIXED-WHEEL GATE<br>unit cost | grfB23.pdf |
| SURFACE STOPLOG<br>unit cost | grfB24.pdf |
| BOTTOM STOPLOG<br>unit cost | grfB25.pdf |

| | |
|---|---|
| GANTRY CRANE FOR SPILLWAY<br>unit cost | grfB26.pdf |
| GANTRY CRANE FOR INTAKE<br>unit cost | grfB27.pdf |
| STEEL TRASH RACKS FOR INTAKE<br>unit cost | grfB28.pdf |
| BUTTERFLY VALVE<br>unit cost | grfB29.pdf |
| SPHERICAL VALVE<br>unit cost | grfB30.pdf |
| BUILDING OF CONSTRUCTION SITES AND WORKERS' CAMPS<br>total cost | grfB31.pdf |
| MAINTENANCE AND OPERATION OF CONSTRUCTION SITES AND WORKERS' CAMPS<br>total cost | grfB32.pdf |
| UNDERGROUND EXCAVATION IN ROCK<br>cost per $m^3$ | grfB33.pdf |
| ROADS<br>unit costs (R$/km) | quadB01.pdf |
| RAILROADS<br>unit costs | quadB02.pdf |
| ROAD BRIDGES<br>unit costs (R$/$m^2$) | quadB03.pdf |
| INTEREST RATES DURING CONSTRUCTION | quadB04.pdf |

# ANNEX C

## Spreadsheets for Dimensioning and Quantification

| Spreadsheets – Final Studies | Name of File |
|---|---|
| Standard Cost Estimate – Preliminary Studies | 49ope.xls |
| Standard Cost Estimate – Final Studies | 56ope.xls |
| Unit Costs | 57punit.xls |
| Powerhouse – Pelton turbines | 572p.xls |
| Powerhouse – vertical-axis Francis turbines | 572fv.xls |
| Powerhouse – horizontal-axis Francis turbines | 572fh.xls |
| Powerhouse – Kaplan turbines with a steel spiral casing | 572ka.xls |
| Powerhouse – Kaplan turbines with a concrete spiral casing | 572kc.xls |
| Powerhouse – Bulb turbines | 572b.xls |
| Cofferdam for river diversion through tunnels or galleries | 573ert1.xls |
| Cofferdam for river diversion in stages | 573ert2.xls |
| Diversion Channel | 573c.xls |
| Diversion Gallery | 573ga.xls |
| Diversion Tunnel | 573td.xls |
| Earthfill Dam | 574t.xls |
| Rockfill dam with vertical clay core | 574enav.xls |
| Rockfill dam with inclined clay core | 574enai.xls |
| Concrete-Faced Rockfill Dam | 574efc.xls |
| Conventional Concrete Gravity Dam | 574ccg.xls |
| Conventional Concrete Gravity Dam with Sluiceways | 574ccgad.xls |
| Roller-Compacted Concrete Dam | 574ccr.xls |
| Roller-Compacted Concrete Dam with Diversion Sluiceways | 574crad.xls |
| Transition and Retaining Walls | 574m.xls |
| Gated Spillway with a High Ogee Crest and Stilling Basin | 575cobd.xls |
| Gated Spillway with a High Ogee Crest and Stilling Basin and Diversion Sluiceways | 575cobda.xls |
| Gated Spillway with a High Ogee Crest and Ski Jump | 575cose.xls |
| Gated Spillway with a High Ogee Crest, Ski Jump and Diversion Sluiceways | 575cosea.xls |
| Gated Abutment Spillway with Stilling Basin | 575coenb.xls |
| Gated Abutment Spillway with Ski Jump | 575coens.xls |
| Ungated Spillway with a High Ogee Crest and Stilling Basin | 575lobd.xls |
| Ungated Spillway with a High Ogee Crest and Stilling Basin and Diversion Sluiceways | 575lobda.xls |
| Ungated Spillway with a High Ogee Crest and Ski Jump | 575lose.xls |
| Ungated Spillway with a High Ogee Crest, Ski Jump and Diversion Sluiceways | 575losea.xls |
| Ungated Abutment Spillway with Ski Jump | 575loens.xls |
| Ungated Abutment Spillway with Stilling Basin | 575loenb.xls |
| Headrace Canal | 576cn.xls |
| Gravity Intake | 576tg.xls |
| Intake Penstock | 576ca.xls |
| Surge Tank | 576ch.xls |
| Pressure Penstock with no Intake Tunnel or Surge Tank | 576cf.xls |
| Pressure Penstock with Intake Tunnel and Surge Tank | 576cfch.xls |
| Pressure Tunnel with no Intake Tunnel or Surge Tank | 576tf.xls |
| Pressure Tunnel with Intake Tunnel and Surge Tank | 576tfch.xls |

# ANNEX D

## SINV System

### SINV 6.0

### FEATURES

SINV version 6.0 has **seven functions** for **Energy Studies**:

- ***Firm Energy***: calculates the firm energy of a given cascade option and the projects it contains.
- ***Optimize Live Storage***: determines the live storage of the projects that make up a given cascade option.
- ***Energy Dimensioning***: determines the live storage, reference head and installed capacity of the projects from a group of cascades.
- ***Live Storage Replenishment***: checks whether the time taken to replenish the live storage of the reservoirs in a cascade option is more or less than 36 months counted as of the end of the critical period.
- ***Eliminate***: calculates the cost/energy benefit index (ICB) of the projects in a given cascade option, and identifies which projects' ICB is greater than the reference unit cost, so that the user can eliminate these from the cascade.
- ***Economic-Energy Assessment***: calculates the ICB of the different cascade options under study and ranks them by their ICB.
- ***Sequencing***: after the final cascade has been selected, based on its cost/energy benefit index (ICB), negative socioenvironmental index (IAn) and positive socioenvironmental index (IAp), ranks the projects from this cascade by their cost/energy benefit indexes (ICB).

The six functions used in the energy studies can be calculated by means of a ***simplified procedure*** during the ***Preliminary Studies***, or by ***simulating operations*** during the ***Final Studies***.

For the Socioenvironmental Studies, SINV version 6.0 has a function for calculating the negative socioenvironmental impacts of the cascade options (***Calculate Negative Socioenvironmental Impact***). This function has two procedures for calculating the socioenvironmental impact indexes of the cascade options, one for use in the Preliminary Studies and the other for the Final Studies. In the Final Studies, there is also a function for calculating the positive socioenvironmental impact indexes.

There are two more functions for selecting the best cascade option. One is used in the Preliminary Studies (***Preliminary Multiobjective Analysis***) and the other in the Final Studies (***Final Multiobjective Analysis***). Both functions are based on multiobjective analysis.

# ANNEX E

## Technical Specifications of Projects - Inventory Studies

| Technical Specifications of Projects – Inventory Studies |
|---|
| 1. IDENTIFICATION: |
| Name of project |
| River |
| Distance to mouth |
| Drainage area of project |
| Drainage area of river basin |
| urban infrastructure: [ ] good [ ] average [ ] poor |
| Basin |
| Sub-basin |
| latitude |
| longitude |
| State(s) |
| Municipality(ies) |
| Codename |
| Status in cascade option: [ ] included or [ ] excluded |
| Name of next plant downstream |
| Type of System |
| 2. BASIC DATA: |
| 2.1. Topography: |
| 2.1.1. Geographical maps available: |
| Entity |
| Name |
| Number |
| Scale |
| Date |
| 2.1.2. Remote sensing data available: |
| 2.1.2.1. Aerial photographs: |
| Entity |
| Performed by |
| Scale |
| Date |
| Section |
| Service |
| Photos |
| 2.1.2.2. Multispectral Images: |
| Entity |
| Performed by |
| Scale |
| Date |
| Service |

| Technical Specifications of Projects – Inventory Studies |
|---|
| 2.1.2.3. Radar Images: |
| Entity |
| Performed by |
| Scale |
| Date |
| Service |
| 2.1.3. Aerial photogrammetric maps available: |
| Entity |
| Performed by |
| Scale |
| Date |
| Contract |
| 2.1.4. Topographic maps available: |
| Entity |
| Performed by |
| Scale |
| Date |
| Contract |
| 2.1.5. Other topographic services available (polygonal, sections, leveling, etc.): |
| Entity |
| Performed by |
| Scale |
| Date |
| Contract |
| 2.2. Geology: |
| 2.2.1. Reservoir: |
| Are there any rocks or geological features that could compromise the watertightness of the reservoir? [ ] yes [ ] no |
| Describe in brief: |
| Are there slopes or rocks that could compromise the stability of the reservoir's banks? [ ] yes [ ] no |
| Describe in brief: |
| Is there any geotectonic evidence that the reservoir could be subject to natural and/or general induced seismic activity? [ ] yes [ ] no |
| Describe in brief: |
| 2.2.2. Dam Axis: |
| Mean estimated thickness of soil cover: |
| On the river bed: |
| On the right bank of the river: |
| On the left bank of the river: |

| Technical Specifications of Projects – Inventory Studies |
|---|
| On the right abutment at the height of the crest: |
| On the left abutment at the height of the crest: |
| Type of rock |
| Are there any geological features that could jeopardise the building of this kind of construction? [ ] yes [ ] no |
| Describe in brief: |
| 2.2.3. Natural building materials, availability of: |
| Clay: [ ] yes [ ] no |
| distance to borrow areas on the right-hand bank |
| distance to borrow areas on the left-hand bank |
| Sand and gravel: [ ] yes [ ] no |
| distance to deposits |
| Rock: [ ] yes [ ] no |
| distance to quarries on the right-hand bank |
| distance to quarries on the left-hand bank |
| 2.3. Hydrometeorology: |
| Climate classification: |
| 2.3.1. Temperatures: |
| maximum: |
| minimum: |
| mean monthly: |
| hottest three months: |
| coldest three months: |
| 2.3.2. Net evaporation: |
| Net evaporation |
| 2.3.3. Precipitation: |
| Gauging stations used: |
| Code |
| Name |
| lattitude |
| longitude |
| elevation |
| period of observation |
| drainage area |
| operating entity |
| months with most rainfall |
| months with least rainfall |
| Mean monthly precipitation: |
| 2.3.4. Streamflow: |
| Gauging stations used: |
| Code |
| Name |
| lattitude |
| longitude |
| elevation |
| period of observation |
| drainage area |

| Technical Specifications of Projects – Inventory Studies |
|---|
| opeating entity |
| river |
| 2.3.5. Flows and water levels: |
| Long-term mean flow |
| Period of long-term mean flow |
| Specific long-term streamflow |
| Max. mean monthly streamflow |
| Month of max. mean monthly streamflow |
| Min. mean monthly streamflow |
| Month of min. mean monthly streamflow |
| Max. observed daily streamflow |
| 10,000-year flood |
| Min. outflow: (defined by environmental and/or operating constraints) |
| Series of natural mean monthly streamflows |
| Observed water level for max. mean streamflow |
| Staff gauge zero |
| Min. mean monthly streamflow |
| Date of min. observed streamflow |
| Observed water level for min. mean streamflow |
| Staff gauge zero |
| Date of 10,000-year flood |
| Water level for 10,000-year flood |
| Staff gauge zero |
| 2.3.6. Reservoir: |
| Maximum normal water level (NAmxn) |
| Minimum normal water level (NAmin) |
| Mean water level (NAm) |
| Normal downstream water level |
| Total reservoir volume |
| Live storage |
| Sum of live storage capacities upstream |
| Maximum drawdown |
| Volume corresponding to crest of the spillway sill |
| Water level corresponding to half the live storage |
| Volume at minimum normal water level |
| Elevation of spillway sill (m) |
| Area flooded for Namax |
| Area flooded for Namin |
| Evaporation losses |
| Loss due to other uses of the waters |
| Net regulated flow |
| Gross regulated flow |
| Residence time (days) |
| 3. ENERGY PARAMETERS: |
| Maximum gross head (Hb1) |
| Maximum net head (H1) |

| Technical Specifications of Projects – Inventory Studies |
|---|
| Mean net head (H2) |
| Minimum net head (Hb1) |
| Mean flow during critical period (Qr) |
| Firm energy (Ef) [Mwmed] |
| Reference capacity factor (Fk) |
| Reference capacity (Pr) |
| Installed capacity (P) |
| Reference head (m) |
| Hydraulic losses (m) |
| Mean tailrace canal water level (m) |
| Normal downstream water level (m) |
| Points and volume/area, elevation/area and streamflow/downstream water level curves (incl. notes about any peculiarities). |
| 4. LAND, RESETTLEMENTS, RELOCATIONS AND OTHER SOCIOENVIRONMENTAL ACTIONS: |
| 4.1. Urban land and land developments affected: |
| District |
| Municipality |
| State |
| Total population |
| Population affected |
| Population affected (%) |
| Urban infrastructure: [ ] good [ ] average [ ] poor |
| Average standard of buildings: [ ] good [ ] average [ ] poor |
| 4.2. Rural land and land developments affected: |
| Municipality |
| State |
| Total area |
| Area affected |
| Area affected (%) |
| Total population |
| Population affected |
| Population affected (%) |
| crops |
| pasture |
| fields |
| forests |
| 4.3. Indigenous communities and/or other ethnic groups affected: |
| Name |
| Municipality |
| State |
| Total population |
| Population affected |
| Population affected (%) |
| Total area |
| Area affected |
| Area affected (%) |

| Technical Specifications of Projects – Inventory Studies |
|---|
| 4.4. Conservation Units and permanent preservation areas affected: |
| Name |
| Municipality |
| State |
| Total area |
| Area affected |
| Area affected (%) |
| 4.5. Other developments affected: |
| 4.6. Resettlements and relocations: |
| 4.6.1. Roads: |
| paved federally-owned roads |
| unpaved federally-owned roads |
| paved state-owned roads |
| unpaved state-owned roads |
| paved municipally-owned roads |
| unpaved municipally-owned roads |
| 4.6.2. Railroads: |
| gauge |
| length |
| 4.6.3. Bridges: |
| type |
| length |
| 4.6.4. Transmission and distribution system: |
| voltage |
| type of tower |
| length |
| 4.6.5. Communications system: |
| 4.6.6. Population: |
| urban |
| rural |
| indigenous communities and/or other ethnic groups affected |
| 4.6.7. Other: |
| airport |
| river port |
| other |
| 4.7. Other socioenvironmental actions: |
| 4.7.1. Reservoir cleaning: |
| area corresponding to drawdown |
| total area |
| area to be cleared |
| area to be cleared (%) |
| type of vegetation |
| 4.7.2. Conservation areas and permanent preservation areas created: |
| name |
| municipality |
| state |
| total area |

| Technical Specifications of Projects – Inventory Studies |
|---|
| area purchased |
| area purchased (%) |
| 5. POWERHOUSE: |
| type |
| installed capacity (P) |
| type of turbines |
| number of units (N) |
| capacity of each turbine (P1) |
| capacity of each generator (P1) |
| synchronous velocity (n) |
| diameter of rotor (D3) |
| output of turbine-generator set |
| maximum turbine discharge (Qt) |
| reference date for costs |
| currency (at the time) |
| payment code |
| currency used for costs and payment schedule |
| total cost of construction |
| payment schedule |
| 6. OPERATORS' VILLAGE: |
| expected population |
| location |
| 7. RIVER DIVERSION AND CONTROL: |
| diversion flow |
| recurrence time |
| type of scheme: [ ] through tunnels [ ] through sluiceways<br>[ ] through galleries [ ] through a canal |
| 7.1. Tunnels: |
| number of tunnels |
| location |
| Exclusively for diversion? [ ] yes [ ] no |
| form of section |
| diameter |
| length |
| maximum flow per tunnel |
| maximum velocity |
| 7.2. Galleries: |
| number of galleries |
| location |
| height |
| width |
| maximum flow per gallery |
| maximum velocity |
| 7.3. Sluiceways: |
| number of sluiceways |
| location |
| height |

| Technical Specifications of Projects – Inventory Studies |
|---|
| width |
| maximum flow per sluiceway |
| maximum velocity |
| 7.4. Canal: |
| location |
| depth |
| width |
| length |
| maximum flow |
| maximum velocity |
| 8. DAMS AND DIKES: |
| type |
| maximum height |
| length |
| mean height |
| volume |
| 9. TRANSITION AND RETAINING WALLS: |
| type |
| maximum height |
| length |
| mean height |
| volume |
| 10. SPILLWAY: |
| type |
| design flood |
| recurrence time |
| maximum height |
| length |
| mean height |
| volume |
| number of gates |
| type of gates |
| width of gates |
| height of gates |
| 11. INTAKE STRUCTURE: |
| 11.1. Water Intake: |
| type |
| maximum height |
| length |
| mean height |
| volume |
| number of intakes |
| maximum discharge per intake |
| number of gates |
| type of gate |
| width of gates |
| height of gates |

| Technical Specifications of Projects – Inventory Studies |
|---|
| 11.2. Low-Pressure Intake Conduit: |
| type: [ ] canal [ ] tunnel |
| number of tunnels |
| length |
| velocity |
| cross-section |
| maximum flow per conduit |
| 11.3. Surge Tank: |
| type |
| diameter |
| height |
| 11.4. Pressure Tunnel: |
| flow through tunnel |
| maximum velocity |
| diameter |
| length |
| length of lined section |
| volume of excavation in rock |
| length of unlined section |
| volume of excavation in soil |
| 11.5. Pressure Penstock: |
| type |
| number of penstocks |
| mean unit length |
| diameter |
| flow through penstock |
| maximum velocity |

| Technical Specifications of Projects – Inventory Studies |
|---|
| 11.6. Tailrace Canal: |
| flow |
| maximum velocity |
| volume of common excavation |
| length |
| volume of excavation in rock |
| depth |
| width |
| 11.7. Tailrace Tunnel: |
| flow |
| maximum velocity |
| volume of common excavation |
| length |
| volume of excavation in rock |
| 12. ACCESS TO CONSTRUCTION SITE: |
| 12.1.Roads: |
| type |
| length |
| 12.2.Railroads: |
| type |
| length |
| 12.3.Bridges: |
| type |
| length |
| 12.4.Airport: |
| type |
| length |

# ANNEX F

## Integrated environmental assessment: example of methodological procedure

This annex contains a summary of the methodology used for an IEA, as a suggestion. This methodology was used successfully in three IEAs conducted by EPE/Sondotécnica (2007)[1].

## Characterization of the study area

The aim of this stage is to build up a current scenario of the river basin, seeking to identify the areas with the greatest environmental sensitivity to human intervention, and attempting to foresee the main impacts associated with the building of the hydropower plants. If it is to meet these goals, the characterization must be designed to:

- identify the most significant social, environmental and economic features of the basin;
- select suitable indicators for each topic to be assessed so as to build up a general, comprehensive socioenvironmental characterization of the study area;
- obtain the necessary information by consulting databases, environmental inventory studies and the scientific literature, as well as carrying out field surveys and interviews, etc.;
- organize the above information in thematic maps, using a geographical information system (GIS), basing its use on the later stages by superimposing maps and undertaking multicriteria analyses, so that the information thus mapped out can be used in the environmental sensitivity analysis;
- analyze the spatial distribution of the information in such a way that subunits of analysis can be identified, especially for wider, more complex regions, which will make it easier to undertake the theme-based analysis or integrate the different topics, and thus to identify the areas of sensitivity;
- describe the socioenvironmental context of the river basin from a macro-regional viewpoint, including an initial identification of the main conflicts encountered and the most significant aspects to be taken into account when selecting the variables and formulating the indicators to represent the environmental sensitivity of the major topics under study in the ambit of the environmental system and its synthesis components.

The environmental characterization should seek to identify the topics that are most pertinent for assessing environmental impacts and for the study of prospective scenarios, taking into account not just the socioenvironmental aspects relative to the area under study, but whenever relevant their ramifications in the macro-regional processes with which the projects under study are most closely related.

Likewise, the conflicts identified at this stage will mainly be focused on the regional scale (the whole river basin), but can also be local or on a smaller scale if they are important in the decision-taking process, in view of national, regional or local interests.

1 Alongside these IEAs, there are others available for consultation on the EPE website which use methodologies that are slightly different from these (http://epe.gov.br/Lists/MeioAmbiente/MeioAmbiente.aspx).

At this stage, then, the following should be identified:

- the most significant socioenvironmental issues in the current situation (existing socioenvironmental processes);
- indications of the future trends of the socioenvironmental issues of relevance and their spatial distributon;
- the social agents involved in each of the socioenvironmental issues and significant conflicts;
- the spatial subunits to be analyzed;
- the socioenvironmental sensitivity indicators; and
- the mapping of these indicators such that the areas of sensitivity in the river basin can be identified.

### Technical Support

The GIS (geographical information system) architecture, which provides technical support for the methodology, should be developed as of the start of the characterization process, making it possible to systematize and analyze the whole set of data and information so that the aspects of relevance can be identified, serving as a basis for formulating the sensitivity indicators. The main stages in structuring the GIS are: (i) composition of planimetric and altimetric databases and thematic maps for the characterization; (ii) image-based mapping and adjustments; and (iii) integration of statistical information with the maps.

## Distributed Environmental Assessment

### Sensitivity Indicators

**Configuration of Environmental Sensitivity**: based on the key aspects identified in stage 1 (diagnosis/characterization), the aim is to formulate a set of environmental sensitivity indicators (ESIs) based on variables that represent the natural conditions and current state of conservation or degradation of the natural resources in the region.

These sensitivity indicators are then organized into an indicator matrix, which provides the structure for including and ranking the variables. This matrix consists of a set of progressive assessments that result in a table of references for interpreting the information produced in the environmental studies, using a system of comparative assessments that is guided by ranking and weighting criteria. The Indicator Matrix is one of the tools used in the assessment made for the purpose of producing maps and diagrams to represent the environmental conditions in the river basin.

Having obtained these initial definitions, multidisciplinary meetings should be held to weight the variables and then reclassify them on the thematic maps from the GIS database. This means that the resulting environmental sensitivity indicators can be expressed in map form, where the degrees of sensitivity can be visualized, based on the previously established parameters.

The flow chart in Figure 1 shows the main steps in assessing environmental sensitivity indicators (ESIs).

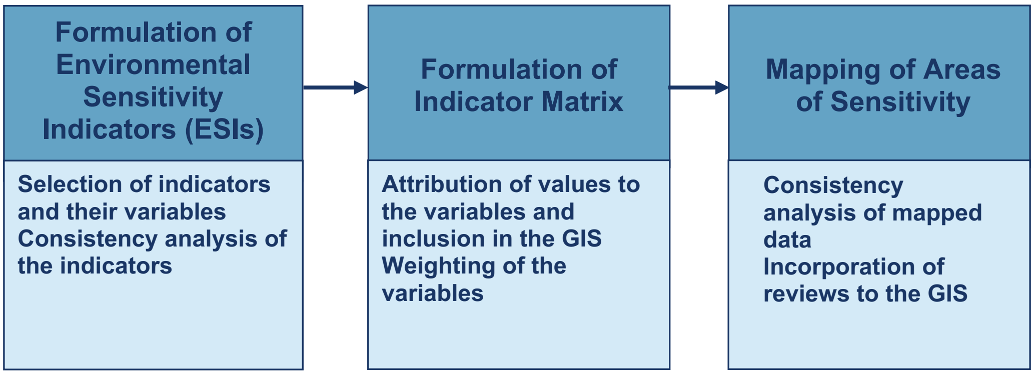



Figure 1 – Flow chart showing the activities involved in assessing environmental sensitivity indicators

Below, a step-by-step description is given of the activities involved in each of the main stages of assessing environmental sensitivity indicators.

### Selecting the Environmental Sensitivity Indicators (ESI's)

The aim of the first activity of this phase is to consolidate the set of ESI's included in the "most significant aspects" item of the characterization report for the IEA. However, the concepts of magnitude, scale and scope used in these indicators, which are specific to each topic and sub-topic, present some inconsistencies. As such, they have to be adjusted before they can be included in the system of analysis.

In order to do so, the selection of ESI's is supplemented by a consistency assessment, addressing the following points:

- the need to minimize the amount of overlapping environmental information;
- an assurance of the maximal objectivity possible in the mapping activities;
- observance of the availability of information and assessment of the consistency of indicators in terms of their representativeness;
- observance of the main interfaces with the overall objective of the work (to assess the cumulative impacts of the hydroelectric projects in the river basin);
- assessment of the possibility of representing the information spatially and of exceeding the time frame.

The final consolidation of the ESI's begins by listing them, as they are presented in the characterization, and then reducing them to the smallest possible number. Many can be grouped together and some can be excluded as a result of the assessment undertaken.

### Formulating the Indicator Matrix

Not only does the characterization provide an understanding of the general socioenvironmental issues in the river basin, but it is needed for thematic databases to be organized in a GIS environment, which provide more detailed, structured and spatially-defined information about the environmental aspects under study. This set of information serves as a basis for selecting the variables to be used in formulating the ESI's.

The Matrix is first built up by selecting the variables that make up each ESI, taking note of certain aspects, such as what kind of variable they are and what weight they have in the indicator.

Figure 2 shows a broad diagram of the Matrix of ESI's, explicating how the variables should be included in the formulation of each indicator, as well as the main steps for assessing them by attributing different weights and degrees of sensitivity.

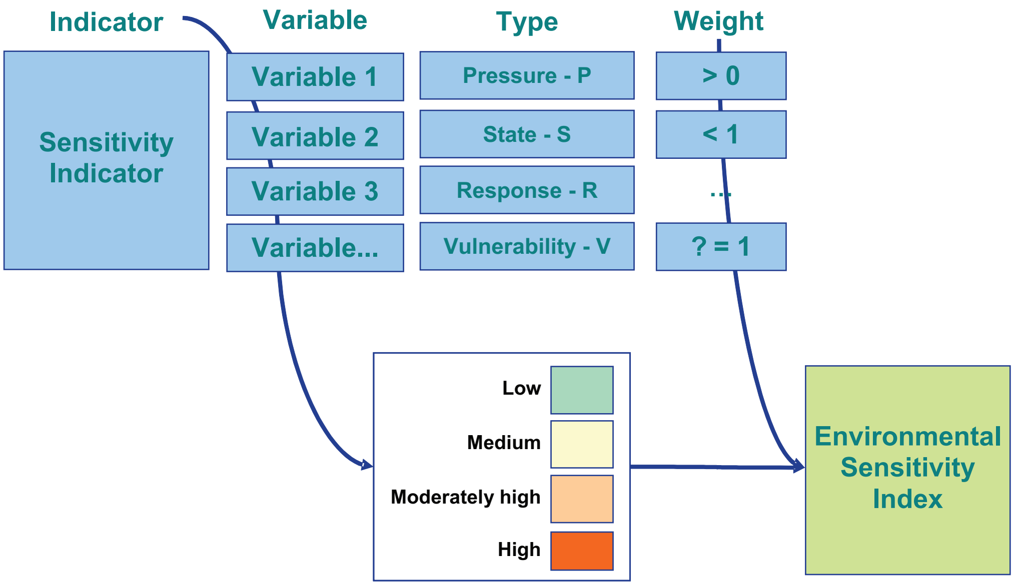



Figure 2 – Components of the Matrix of Environmental Sensitivity Indicators

### Defining Types of Variables

The development of the environmental variables should be based on the OECD (Organisation for Economic Cooperation and Development) methodology, providing control and environmental monitoring mechanisms for its member countries. The OECD system is based on environmental indicators that assess the integrity of, pressure on and society's interest in preserving the main natural resources used by man. It is founded on the pressure-state-response (PSR) model (explained below), and is used to choose the most suitable variables to be monitored and assessed when diagnosing the environmental conditions in different environments and ecosystems.

- **Indicators of pressure (P)** serve not only to identify the main natural resources used by society, but also to indicate the state of preservation and degradation of the natural habitats, while also, whenever possible, identifying the carrying capacity of the natural resources.
- **Indicators of state (S)** are associated with the natural state of the resources and identify the quality of the natural habitats. As such, they involve issues such as biological diversity, size of remaining forest patches, stocks, etc.
- **Indicators of response (R)** identify mechanisms created by society for monitoring, controlling and recuperating given resources and also, whenever possible, the efficiency of these mechanisms.

Alongside these indicators, the use of another indicator is also recommended, which is designed to identify special conditions of vulnerability in given habitats. This classification is included so that the variables can be ranked objectively, and is the main element used in classifying the weights for the variables.

- P – Pressure: levels of conservation or degradation;
- S – State: quality of natural habitats;
- R – Social Response: control mechanisms;
- V – Vulnerability: special conditions, such as indigenous areas and conservation units.

### Weighting the Variables

The process of weighting and ranking the variables is started by defining and identifying the kinds of variables used to make up the indicators. In order to define the index for each ESI, weights should be attributed numerically, with a view to converting the environmental sensitivity scale to a comparable scale.

The weights are attributed in such a way that they represent the importance of each variable with respect to the construction of the indicators, in the scale of sensitivity of the ES'Is, and ultimately which areas should be considered as being environmentally sensitive.

### Defining Degrees of Sensitivity

In order to define the degree of sensitivity of each variable, the variation scale of the information in the database for each variable must be consulted, seeking from this scale which references could determine the ranges to be used in identifying degrees of sensitivity, as shown in Figure below.

Each variable has its own scale of values, so to select the range of values that define what sensitivity parameters to use, it is necessary always to consult the references for these parameters.

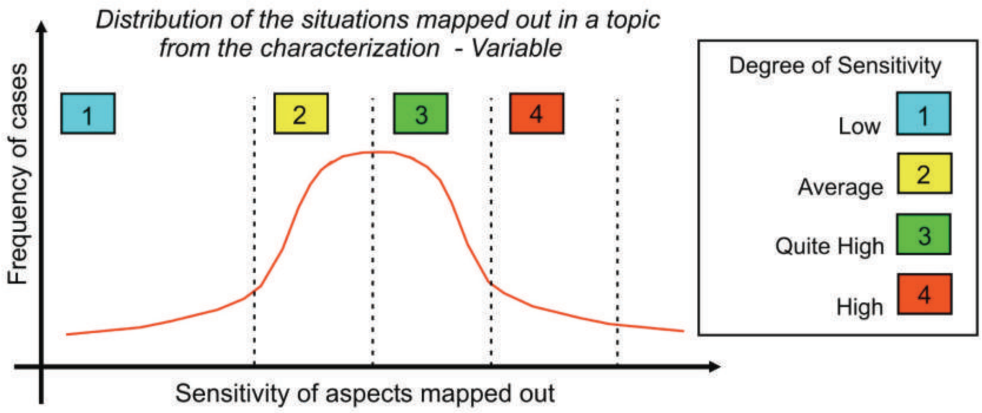



Figure 3 – Schematic representation of process for defining levels of sensitivity

The formulation of environmental sensitivity indicators offers a number of advantages that can assure the fulfillment of some of the main aims of the work. Starting with structuring a set of analysis tools, the ESI's can be used to accomplish the activities required for several of the stages of the IEA's. The main advantages of using ESI's are:

- they represent in map form the different ranges of sensitivity of the environmental aspects, using the working scales proposed, thereby preventing spatial generalizations from being made that could mask local differences and heterogeneities;
- the matrixes and maps ensure that a record is kept of the characterization information, since the map representations of the ESI's involve reclassifying the databases of thematic maps, with the information being kept accessible in the GIS throughout the assessment process;
- the areas of fragility and potentiality inside the river basin and each sub-area are identified.

### Mapping out the indicators

A GIS that permits the integration of the different elements that make up the environmental sensitivity indicators should be structured from the maps created during the characterization.

The maps of the different geographical features, alongside the database containing information about the different environmental topics, provide the basis for an integration of the components from the

DEA, especially by means of the restructuring of the databases with information about the weights and degrees, which are determinant in the integration of the attributes with the GIS platform.

Having restructured the database, the thematic maps should be cross-referenced by geographically superimposing the information they contain, where the attributes that have been weighted and with values on the scales of sensitivity can be summed, defining the presentation of the indicators with the areas of sensitivity represented in the maps.

### Integrating the indicators into topics

The sensitivity indicators are integrated under the topics proposed in the study (water resources and water ecosystems, physical environment and terrestrial ecosystems, and socioeconomics) by cross-referencing the sensitivity maps as per their identification. Table 1 gives an example from a particular river basin.

Table 1 – List of sensitivity indicators per integration topic

| INDICATOR | INTEGRATION TOPIC |
|---|---|
| Sensitivity of water quality<br>Sensitivity to conflicts over water use<br>Sensitivity of water habitats | Water Resources and Aquatic Ecosystems |
| Geological and geotechnical sensitivity<br>Sensitivity to soil erosion<br>Sensitivity of terrestrial ecosystems | Physical Environment and Terrestrial Ecosystems |
| Sensitivity to population pressures<br>Sensitivity of ways of life<br>Sensitivity to conflicts over land use (agriculture)<br>Sensitivity of natural, archaeological, historical, and cultural heritage<br>Sensitivity to alterations or destructuring of communities<br>Sensitivity to curtailment of economic activity<br>Sensitivity of social and land organization | Socioeconomic Aspects |

The maps of potentialities are not integrated, since they cover different topics.

The integration of the indicators into topics gives rise to three representations, based on the sum of the grouped ESI's. The representation based on the weighted sum of the ESI's can be understood as a spatial superimposition of the sensitivities.

The maps of each of the topics should be integrated according to different criteria, which will depend on the characteristics of the river basin.

The areas where several elements of high sensitivity occur are designated areas of sensitivity.

The socioeconomic indicator must be considered in the integrated representation of potentialities. The ranges of higher sensitivity to socioeconomic potentialities represent local features that could leverage positive impacts.

### Impact Indicators

The main information from the environmental impact assessment undertaken as part of the Preliminary Inventory Studies should be about the main characteristics of the projects analyzed, seeking to associate and dimension the potentially impacting elements of each of them. Thus, as well as listing and selecting the set of potential impacts, values should be attributed for measuring each impact for each of the projects under study. Figure 4 shows the main stages, activities and tools used in the process of assessing potential impacts.

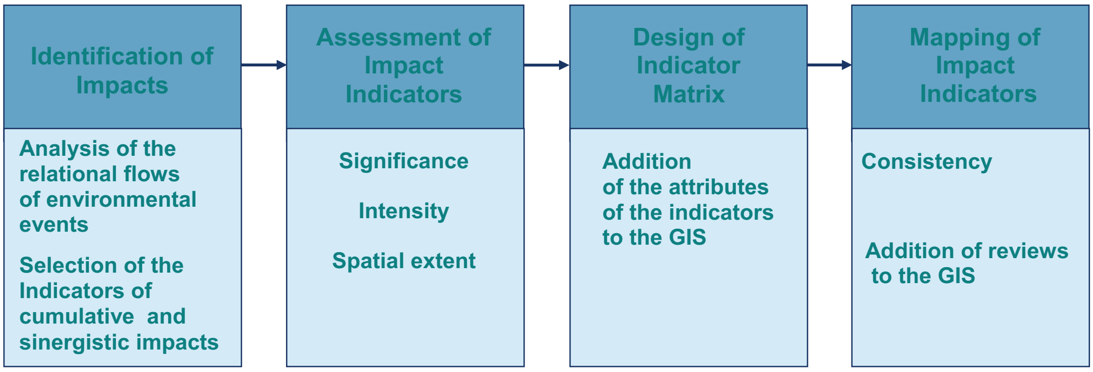



Figure 4 – Flow chart of impact analysis

The impact assessment is designed to identify which aspects from the group of impacts and projects under study could affect the intensity of manifestation of these impacts and the spatial extent of their effects. As such, the following guidelines were established:

- Differentiate the intensity according to each project: since the prospective projects in a river basin will have particular features, such as their size, the capacity of the reservoir, their regulating capacity and other aspects that differentiate them, which must be taken into account throughout the analysis;
- Rank the impacts according to their importance and significance: each impact has a differentiated state as a function of its potential to generate cumulative and synergistic effects and the nature of how it is manifested and whether it is direct or indirect, etc.;
- Establish the spatial distribution of the effects by representing them geographically in line with the spatial distribution of the resources directly associated with the impacts identified, such that the spatial interactions between the impacts can be highlighted;
- Assess the cumulative and synergistic effects between the projects, observing what additional effects could be generated by the projects in question.

Below are presented the main stages in the assessments of environmental impacts that make up the DEA.

### Identification of Environmental Impacts

The impacts associated with the hydropower projects should be inferred from an association matrix, called the Environmental Flowchart (FREA – "Fluxo Relacional de Eventos Ambientais"), which represents the relationship between each phase of development and the main impacts associated.

The idea behind formulating the flowchart is to list the potential impacts that could take place while building or operating the hydropower plants.

Interaction networks are used to identify the events responsible for the most significant environmental changes. The flowchart is an inference model that provides a framework for identifying a more comprehensive set of impacts, so that a broad context can be provided to which selection criteria can be applied for determining which impacts are most significant in the process under analysis.

### Selection of Environmental Impacts

Based on an initial set of impacts, analyses are carried out with the aim of grouping or eliminating them as a function of the characteristics of the impacts. Thus, the following steps should be taken in the selection process:

- identify the permanent or long-term impacts. Temporary impacts should be disregarded insofar as they are of little significance on the time scale used, which is 10 years at the least;

- identify the impacts with different spatial distribution within the river basin, since those which extrapolate the boundaries of the basin will not give rise to a comparative differentiation;
- identify the impacts that can be objectively distinguished, meaning that they can be measured on the working scale established for the assessment. In this way, impacts with a local reach which are neither cumulative nor synergistic are also discarded.

Figure 5 shows a schematic flow chart of the process for selecting impacts based on the methodology described in this section.

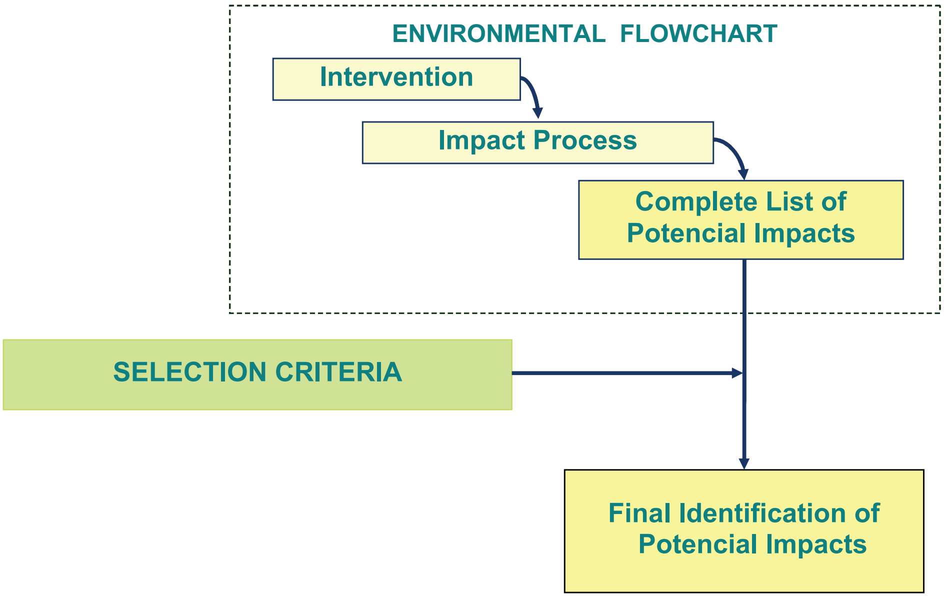



Figure 5 – Flow chart of impact selection process

### Assessment of Environmental Impacts

Depending on the information from each impact, the environmental analysis to be undertaken consists of an assessment of the significance, spatial extent and intensity of the environmental impacts related to each existing project.

The significance of a given impact is taken as being the value that expresses the manifestation of this impact on the environment. In order to define the significance of an environmental impact, its magnitude, intensity and spatial extent are taken into account; the assessment is based on the attributes of the impacts and the perceptions and experience of the professionals in the multidisciplinary team.

The magnitude of an impact is given by assessing its attributes that induce large or small and fast or slow changes to the quality of the environment in the area where they are manifested. Thus, the methodology includes an objective analysis of three attributes: whether the impact is direct or indirect; whether it is local or regional; and whether it is medium- or long-term.

The same assessment also seeks to differentiate between the impacts per project, investigating their basic criteria and thereby giving a clearer picture of the relative contribution of each of the projects to the interaction between the impacts. Next, the main characteristics used to assess the impact indicators are described, as well as how the degrees of sensitivity were established based on the set of plants in the river basin.

***Operating Regime*** – differentiates plants with regulation reservoirs from run-of-river plants.

***Residence Time*** – the mean retention time of water in the reservoir.

***In-Stream Flow*** – the mean flow released downstream after regulation.

***Regulating Capacity*** – measures retention efficiency as a percentage.

***Capacity*** – the installed capacity of the turbines.

***Flood Control*** – the capacity to contain the natural cyclical floods in the river.

***Reservoir Area*** – describes the area covered by the water surface.

***Section with Reduced Flow*** – identifies and measures the distance over which flow is reduced because of the use of a diversion channel or other similar scheme.

***Dam Height*** – describes the head at the dam.

## Preparation of indicator matrix of synergistic and cumulative effects

The indicator matrix is a set of interrelated assessment tables used to transfer the data from the different phases of assessment to the maps. It is built up from a set of information about and the main characteristics of the existing and planned plants in the river basin, and includes an assessment of significance and spatial extent, and the selection of the kind of intensity indicator to be used for each of the impacts. A flow chart of the components in the matrix is shown in Figure 6.

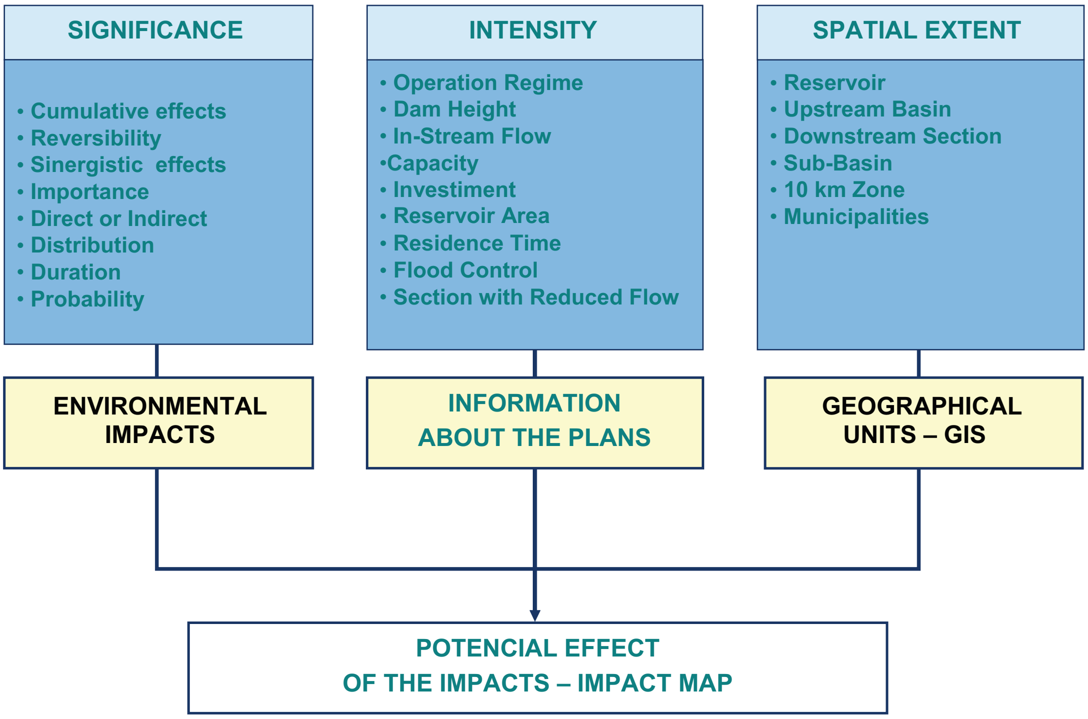



Figure 6 – Diagram of the flow of information in the Impact Matrix

Based on these definitions, the values obtained for each impact are grouped together according to the spatial extent of their effects, so that they can be included in the other maps formulated as part of the DEA.

## Mapping Impact Indicators

The impacts are represented spatially by identifying the geographical elements that could best represent the natural resources directly involved in each environmental impact. The concept of "spatial extent" was developed for this representation.

The spatial extent of each impact is a representation of the extent to which the effects of each impact are felt in space, so that the impact indicator can be included in the GIS, as can its analysis integrated with its sensitivity.

First, the possible "spatial extent" of the impacts must be established, albeit conservatively, meaning that the worst-case scenarios should be addressed.

**Reservoir**: the area of the reservoir or the land to be occupied by it;

**Upstream Basin**: the whole hydrographic network that contributes to the formation of the reservoir;

**Downstream Section**: a 10 km section around the section downstream from the dam until the backwaters in the reservoir from the next project downstream;

**Sub-Basin**: the whole sub-basin where the project is built or is to be built;

**10 km buffer around the reservoir**: a 10 km area around the reservoir, adopting the same principle that defines the buffer zones around conservation areas as defined in the Brazilian System of Conservation Areas (SNUC);

**Municipality**: municipalities under the direct influence of each project, meaning those whose land is partially flooded to form the reservoir.

The analysis of the environmental impacts of the existing hydropower plants should be prepared using the methodology described, seeking to assess how these impacts would change from the time they were first manifested until the present.

### Analysis of cumulative and synergistic effects

The assessment of cumulative and synergistic effects should be carried out by crossing the data from the environmental sensitivity mapping and the mapping of impact indicators. The geographical distribution of the two indicators will give a clear picture of the main relationships between the different impacts analyzed in the study, especially identifying the areas where there is a greater overlapping of these effects.

Two items should be checked in this spatial representation:

- **concerning cumulative effects**: defined as effects that are combined by overlapping spatially;
- **concerning synergistic effects**: obtained by constructing and assessing the interrelations between the impacts; quantified by the number of synergy correlations between impacts obtained from a matrix whose lines and columns represent the impacts identified from experience of the projects already built. The greater the number of synergies in the matrix, the greater the synergistic effect of each impact.

To undertake these analyses, some methods should be selected, such as:

a) geographical information system (GIS);

b) analysis networks (of diagrams);

c) biological/geographical analysis;

d) analysis of assimilation capacity (limits and constraints).

### Integrated Environmental Assessment

#### Prospective Scenarios

The current scenario taken from the socioenvironmental characterization of the river basin and represented by the areas of sensitivity is the reference scenario against which the prospective scenarios should be built up, with and without the projects from the best cascade. During the IEA, these scenarios project the socioeconomic development trends and prospects for environmental degradation/

conservation, crossing them against the prospects for expansion of energy supply thanks to the building of the planned hydropower plants and those already being built in the river basin.

The prospective scenarios are built up in seven stages:

- modeling for the formulation of reference socioeconomic scenarios using system dynamics models, such as STELLA (High Performance Systems, 1997);
- survey of the main scenarios for electricity generation;
- formulation of system for assessing synergistic and cumulative effects;
- comparison of areas of fragility and potentiality in the reference socioeconomic scenarios with the main synergistic and cumulative effects projected for the groups of hydropower projects planned for the river basin;
- analysis of fragility and potentiality maps created for the prospective scenario with the projects built, resulting from crossing the fragilities and potentialities with the main synergistic and cumulative effects.

**Guidelines and Recommendations**

Socioenvironmental guidelines and recommendations for each sub-area of the river basin should be drawn up for the electricity industry and other public and private entities operating in the basin, taking into consideration the use of the water resources and the land, the conservation and sustainable use of the natural resources, and the sustainability indicators and criteria.

# ANNEX G

## Format of the File Showing the Monthly Flows of the Projects

Name of project

| YEAR | Jan | Feb | Mar | Apr | May | Jun | Jul | Aug | Sep | Oct | Nov | Dec |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 1931 | | | | | | | | | | | | |
| 1932 | | | | | | | | | | | | |
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**Observation:**

A spreadsheet should be prepared for each project using the above format. The number of lines should be the same as the number of years available in the mean monthly flow series of the project.

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Front cover: Tucuruí Power Plant spillway (Eletrobrás archives)
p.17: Meandering river (Eletrobrás archive)
p.33: Construction work for Simplício hydropower plant (Marcos Pinto – Furnas archive)
p.61: Tailrace canal area, Simplício hydropower plant, powerhouse (Marcos Pinto – Furnas archive)
p.79: Itaipu power plant (Eletrobrás archive)
p.187: Porto Colômbia power plant (Eletrobrás archive)
p.595: Furnas power plant: aerial view with spillway open (Roberto Rasa – Furnas archive)
p. 607: Marimbondo power plant (Eletrobras archive)
p.619: Tucuruí power plant (Eletrobras archive)
p.659: Luiz Carlos Barreto de Carvalho power plant with spillway open (Aluísio de Souza – Furnas archive)

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