PRELIMINARY DESIGN OF A DAM IN BEÇA RIVER

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SUMMARY

The purpose of this work is to present the design, on a preliminary level, of a dam located in river Beça, including the design of all appurtenant hydraulic structures and the stability analysis of the dam.

With the purpose of estimating the design flood, with a return period of 1000 years, according to the current legislation, a hydrologic study was developed. For that purpose, a precipitation-runoff model, the SCS' HUD model, was used. The hydrometric records of the nearby gauging stations were also considered to evaluate the design flood.

For the design of the spillway, a WES ogee crest was considered and several geometries were studied. The solution adopted establishes a compromise between the storage capacity of the reservoir and the flood routing through the reservoir.

The river diversion was designed for an average flow that is exceeded, in average, 30 days per year. The diversion is made through a channel that crosses the dam body and the cofferdams. On the upstream section of the river diversion, an entrance transition structure was considered. An orifice left on the dam body was also considered to discharge the flow resultant from upstream cofferdam overtopping.

The bottom outlet and the water intake conduit were designed through the dam body. The diameter of the bottom outlet and the Howell-Bunger valve were calculated to empty the reservoir within six weeks. The water intake conduit was designed for a flow that represents the water demand downstream.

The dam stability analysis comprehended the sliding stability analysis, the overturning stability and the stress analysis, in sections of the main sections of the dam. All the analysis made included different loading and reservoir level conditions.

Keywords: hydrological study, spillway, river diversion, bottom outlet, water intake, stability analysis.

OBJECTIVE

The main purpose of this work is to present the design, on a preliminary level, of a concrete dam located in river Beça, near Boticas city.

Several appurtenant hydraulic structures were studied: spillway, river diversion, bottom outlet and water intake.

A stability analysis of the dam was also made, comprehending sliding stability, overturning stability and stress analysis. All the analysis made included different loading and reservoir level conditions.

The main characteristics of the appurtenant hydraulic structures studied and the results of the stability analysis are summarized on this abstract.

HYDROLOGICAL STUDY

In order to design the appurtenant hydraulic structures, the design flood needs to be determined. To accomplished that, a hydrologic study was developed.

The flood hydrograph and the peak discharge were determined for return period of 1000 years, according to the current legislation.

The hydrograph and the peak discharge were determined through the precipitation-runoff model developed by *Soil Conservation Service*, called HUD model.

To apply this model, the design precipitation was determined taking into account the precipitation data obtain from meteorological stations near the dam watershed, the formulation developed by Brandão and Hipólito (1997) and the intensity-duration-frequency curves studied by Brandão *et al* (2001).

Based on the design precipitation, the hyetographs were defined, according to the procedure presented by Portela (2006a and 2006b). The peak discharge affluent to the reservoir was estimated based on the hyetographs applying the SCS's HUD model.

The peak discharge was also determined through statistical analysys of maximum annual discharges obtained from nearby hydrometric stations, and compared with the values obtained from precipitation.

Table 1 presents the main results obtained from the hydrological study.

Table 1 – Main results from hydrological study

Watershed area (km^2)	110.46
Time of concentration (h)	6.00
Return period (years)	1000
Design precipitation (mm)	98.8
Peak discharge (m^3/s)	265.8

RESERVOIR AND DAM – GENERAL CHARACTERISTICS

To define the return period of the hydrograph and the corresponding peak discharge of the design flood, the dam axis was chosen and the dam height was determined.

The chosen dam axis was defined taking the topography characteristics of the construction site into account and in order to obtain the maximum water storage capacity in the reservoir. Thus, the crest of the dam was established at elevation 725.00 m, resulting a dam height of 28.50 m.

The dam body is made of blocks of concrete with 15.00 m width. The downstream slope is 1.00:0.80 (V:H) and the upstream slope is vertical.

In terms of the reservoir, the accumulated volume curve was multiplied by a factor of 4 to assure the damping capacity of the peak discharge to the reservoir.

Table 2 and 3 presents the main characteristics of the dam body and the reservoir.

Table 2 - Main characteristics of dam body

Type	Gravity / Concrete
Crest elevation (m)	725.00
Crest length (m)	103.30
Crest width (m)	3.50
Height (m)	28.50

Table 3 - Main characteristics of dam reservoir

Normal storage capacity (m^3)	172 x 1000
Maximum storage capacity (m^3)	434 x 1000
Normal Water Level (NWL) (m)	719.50
Maximum Flood Level (MFL) (m)	724.00
Minimum Operating Level (MOL) (m)	712.50

SPILLWAY

Due to the type of dam, the spillway is located over the dam body and comprises a WES weir and an energy dissipation structure.

During the design of the spillway, several solutions were studied in terms of crest length, design head and crest elevation. The routing of the peak discharge was also taken into account during the design of the WES weir.

Table 4 presents the main characteristics adopted for the WES weir.

Table 4 - Main characteristics of the WES weir

Length (m)	37.50
Effective length (m)	35.00
Max head (m)	4.50
Design head (m)	3.30
Discharge coefficient	0.52
Elevation (m)	719.50
Affluent flow (m^3/s)	265.8
Effluent flow (m^3/s)	176.1

The solution adopted for the WES ogee crest is the one that ensures the best compromise between the storage capacity of the reservoir and the decrease of the peak outflow.

The energy dissipation structure is located at the toe of the spillway, which design was based on the methodology presented by Pinheiro (2006a) and the methodology presented by CHS (-). Table 5 presents the main characteristics adopted for the energy dissipation structure.

Table 5 - Main characteristics of the energy dissipation structure

Radius (m)	2.00
Minimum submergence (m)	4.40
Maximum submergence (m)	5.80
Adopted submergence (m)	5.00
Elevation (m)	700.10
Water level downstream (m)	705.10

The geometry of the energy dissipation structure is shown in figure 1.

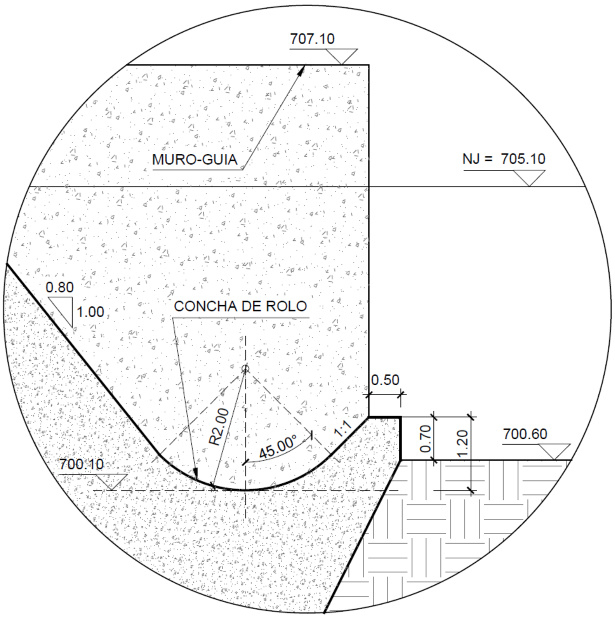

Figure 1 - Energy dissipation structure

To ensure an adequate hydraulic operation of the entire spillway, excavations of the river bed downstream and upstream of the dam body are required, according the drawings shown in figures 2 and 3. The main characteristics of the spillway are also presented on the figures.

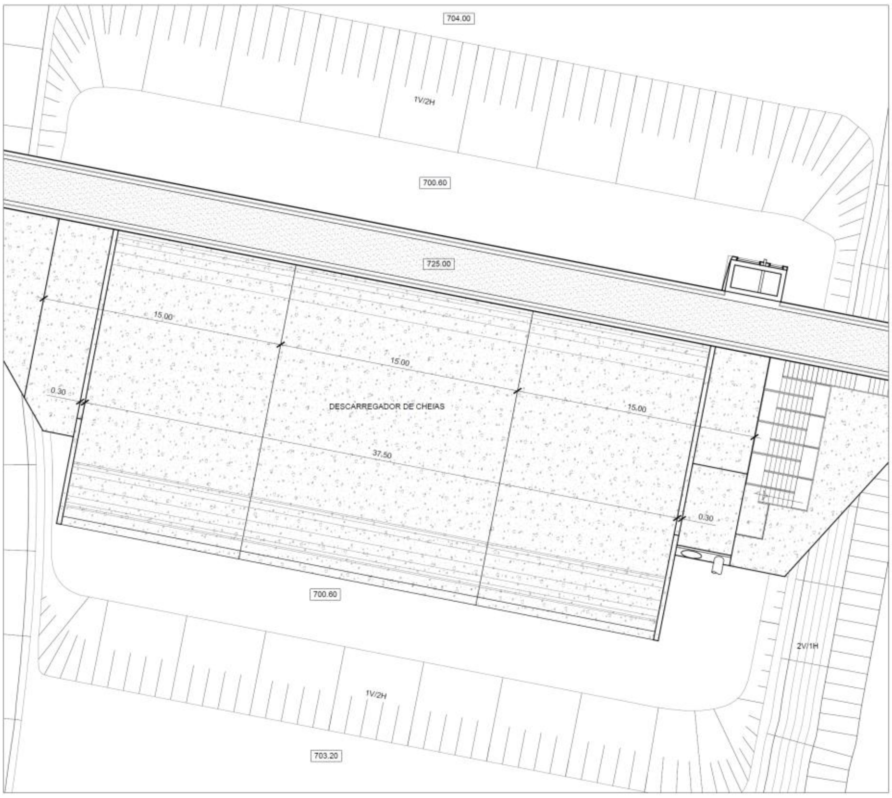

Figure 2 – Spillway. Plan

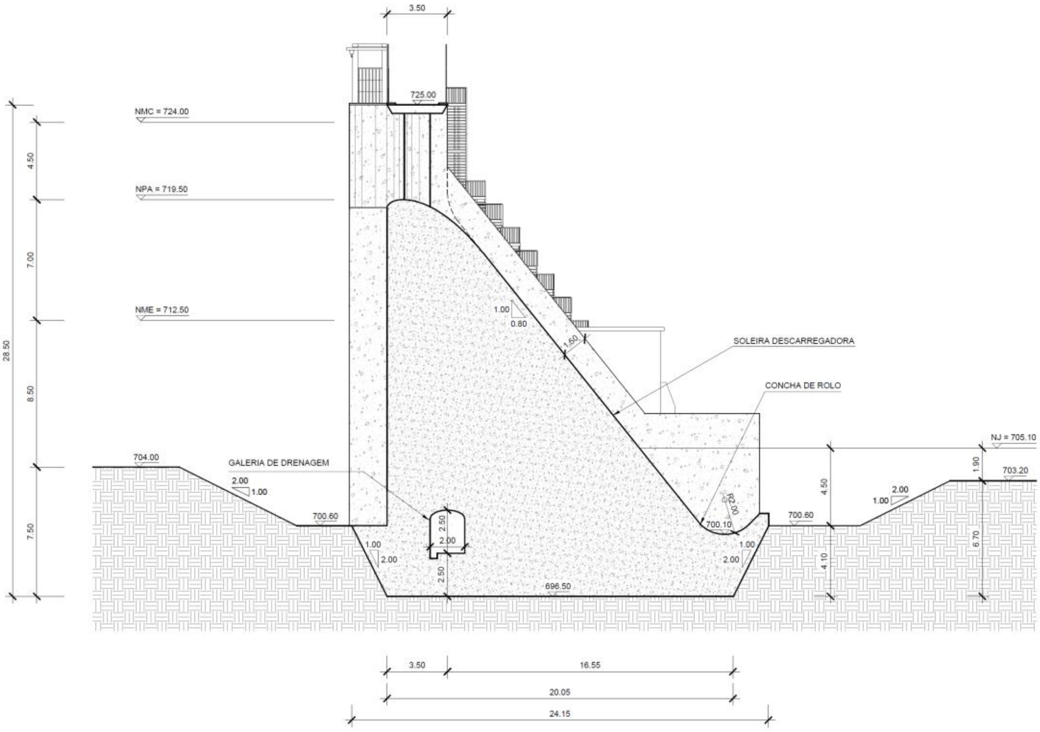

Figure 3 – Spillway. Longitudinal section

RIVER DIVERSION

The river diversion was designed for an average flow that is exceeded, in average, 30 days per year and according to the methodology presented by Pinheiro (2002). The diversion is made through a channel that crosses the dam body and the cofferdams located upstream and downstream of the construction site.

A transition structure between the reservoir and the channel, in which the width changes from 6.00 to 2.00 m along 6.00 m, was foreseen. The walls of the transition structure have an hydrodynamic shape, as showed in figure 4, that guarantees a transition between the subcritical flow in the reservoir to the supercritical flow in the channel without flow separation. The groundsill of the transition structure is located at elevation 705.05 m.

The upstream cofferdam crest, at elevation 707.50 m, was defined according to the maximum water level expected in the reservoir created upstream of the cofferdam – 706.70 m. The upstream cofferdam presents 1.00:2.00 (V:H) slopes.

The diversion channel starts at the end of the transition structure and end at the restitution zone, 100.00m downstream. The cross section of the channel has 2.00 m width and 2.00 m height, as shown in figure 5. The downstream section of the diversion channel was established at elevation 703.05 m to ensure that the hydraulic jump occurs on the river and not inside the channel.

Once the riverbed was considered to be composed of solid rock, the energy dissipation device located downstream of the diversion channel was not considered. The restitution of the discharged flow is made directly into the river.

However, the downstream cofferdam is constructed near the downstream section of the diversion channel to prevent the reflux of the flow into the construction zone. The downstream cofferdam crest was established at elevation 705.20 m and 1.00:2.00 (V:H) slopes are adopted.

An orifice left on the dam body was also considered to discharge the flow resultant from upstream cofferdam overtopping. The orifice has 1.50 m width and 2.00 m height and is located in one of the side blocks of the dam body. To evaluate the discharge capacity of the river diversion in the scenario of overtopping of the cofferdams, the stage discharge equation was determined.

Tables 6 and 7 include the main characteristics of the river diversion structures.

Table 6 - Main characteristics of the river diversion

Design discharge (m^3/s)	6.7
Water level on reservoir (m)	706.70
Water level downstream (m)	703.70
Upstream cofferdam elevation (m)	707.50
Upstream cofferdam height (m)	2.70
Downstream cofferdam elevation (m)	705.20
Downstream cofferdam height (m)	2.00

Table 7 - Main characteristics of the river diversion structures

Channel upstream elevation (m)	705.05
Channel downstream elevation (m)	703.05
Channel length (m)	100.00
Channel width (m)	2.00
Channel height (m)	2.00
Channel slope (m/m)	0.02
Orifice elevation (m)	702.50
Orifice width (m)	1.50
Orifice height (m)	2.00

The main characteristics of the river diversion are also presented on figures 4 to 8.

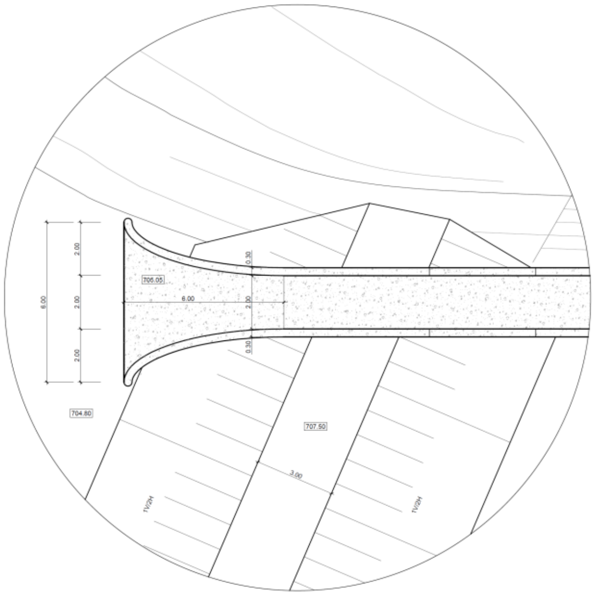

Figure 4 – River diversion. Upstream transition structure. Plan.

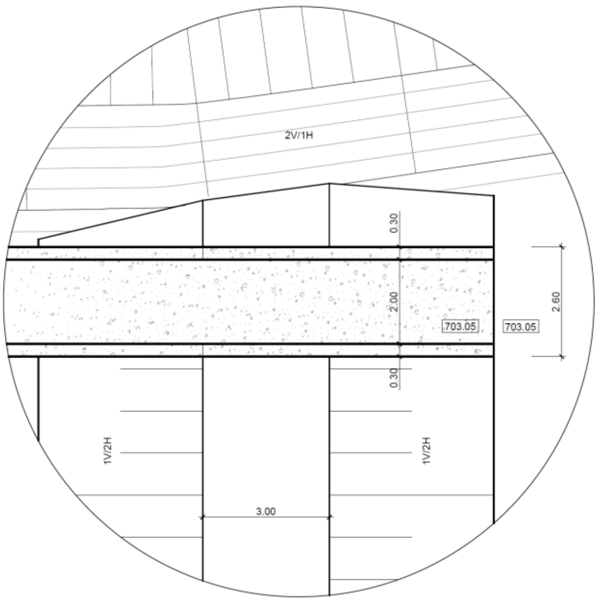

Figure 5 – River diversion. Downstream restitution zone. Plan.

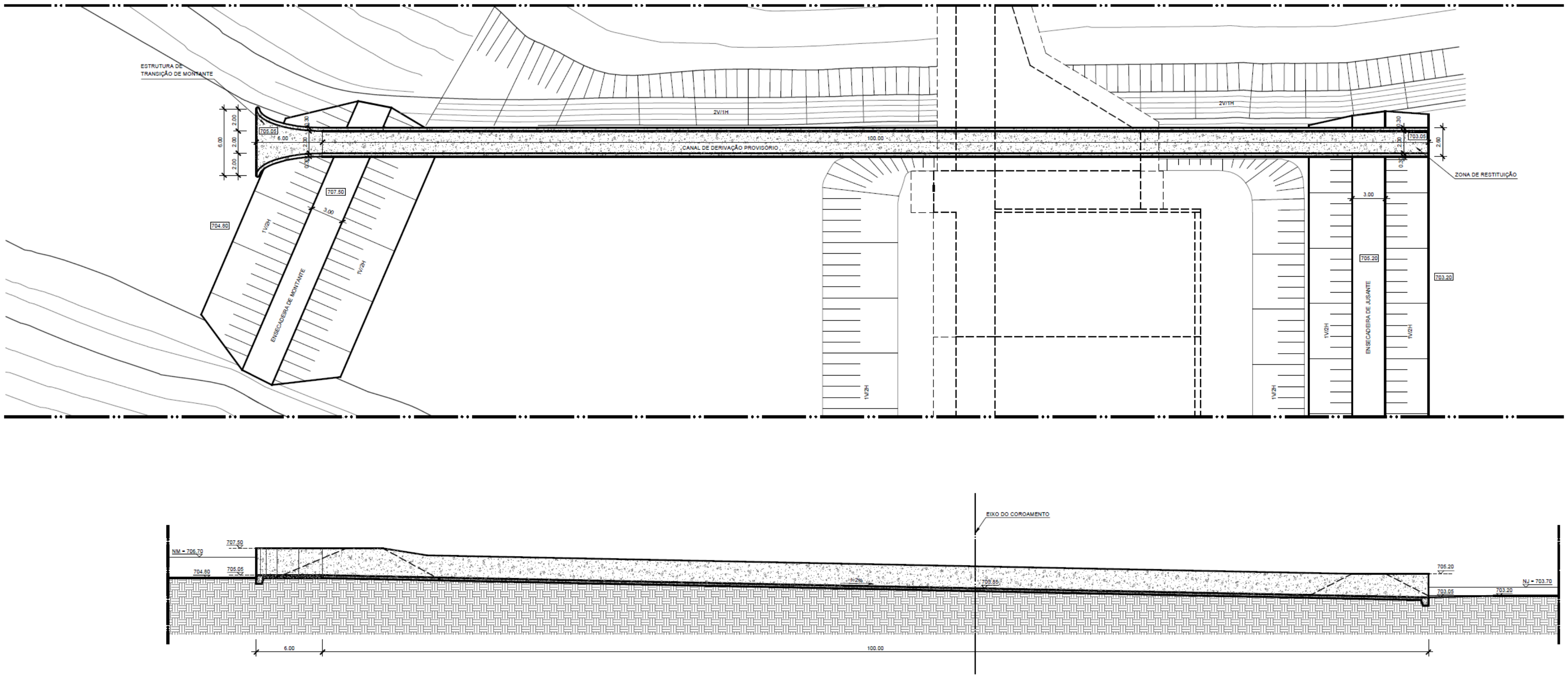

Figure 6 - River diversion. Plan and Longitudinal section.

Figure 7 - River diversion; Orifice on dam body. Plan.

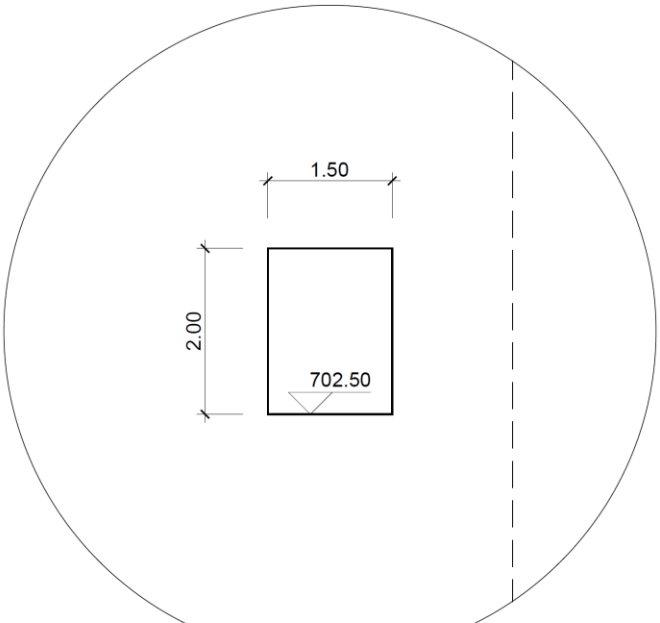

Figure 8 - River diversion. Orifice on dam body

BOTOM OUTLET

The bottom outlet was designed to cross the dam body. The cross section of the bottom outlet and the Howell Bunger valve were calculated to empty the reservoir within six weeks.

The bottom outlet is composed by several elements such as the upstream trashrack, the transition, the steel pipe, the upstream gate, the reduction cone and the Howell Bunger valve.

The upstream trashrackwas designed according to the methodology presented by Pinheiro (2006b) and has 1.50 m width and 2.60 m height. The entrance of the bottom outlet was established at elevation 709.00 m due to the required minimum submersion of 3.15 m and the minimum operation level defined of 712.50 m.

After the entrance, with hydrodynamic shape, there is a transition from the rectangular section to the circular section of the steel pipe.

The steel pipe establishes the connection between the entrance of the bottom outlet and the Howell Bunger valve. In order to empty the reservoir in six weeks, the conduct has 12.00 m length and a diameter of 1.00 m.

A plane gate with 1.25 m width and 1.25 m height, is located upstream of the Howell Bunger valve. The gate can shut down the hydraulic circuit to allow maintenance operations. An air pipe with a diameter of 0.30 m is located downstream of the gate to avoid the formation of vortexes.

The Howell Bunger has a diameter of 0.65 m in order to empty the reservoir in six weeks. The axis of the valve is at elevation 707.80 m. The valve provides a maximum discharge of 3.8 m^3/s with an average velocity in the conduit of 4.85 m/s. The reservoir elevation at the end of the emptying will be 713.00 m.

The entire bottom outlet is controlled through the operating building, which is accessible through stairs. Table 8 presents the main characteristics of the bottom outlet.

Table 8 - Main characteristics of bottom outlet elements

Upstream trashrack width (m)	1.50
Upstream trashrack height (m)	2.60
Steel pipe diameter (m)	1.00
Steel pipe length (m)	12.00
Plan gate width (m)	1.25
Plan gate height (m)	1.25
Air conduct diameter (m)	0.30
Howell Bunger valve diameter (m)	0.65
Elevation of bottom outlet entrance (m)	709.00
Elevation of Howell Bunger valve axis (m)	707.80
Maximum discharge (m^3/s)	3.8
Maximum flow velocity (m/s)	4.85
Final water surface elevation (m)	713.00

The main characteristics of the bottom outlet are also shown on figures 9 to 12.

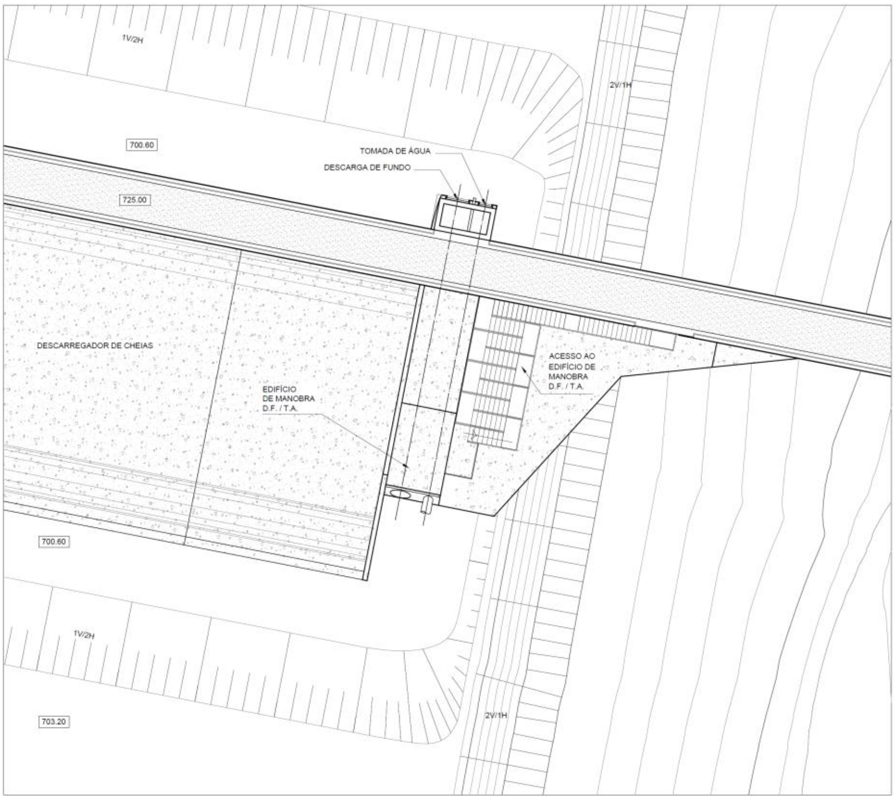

Figure 9 – Bottom outlet. Plan

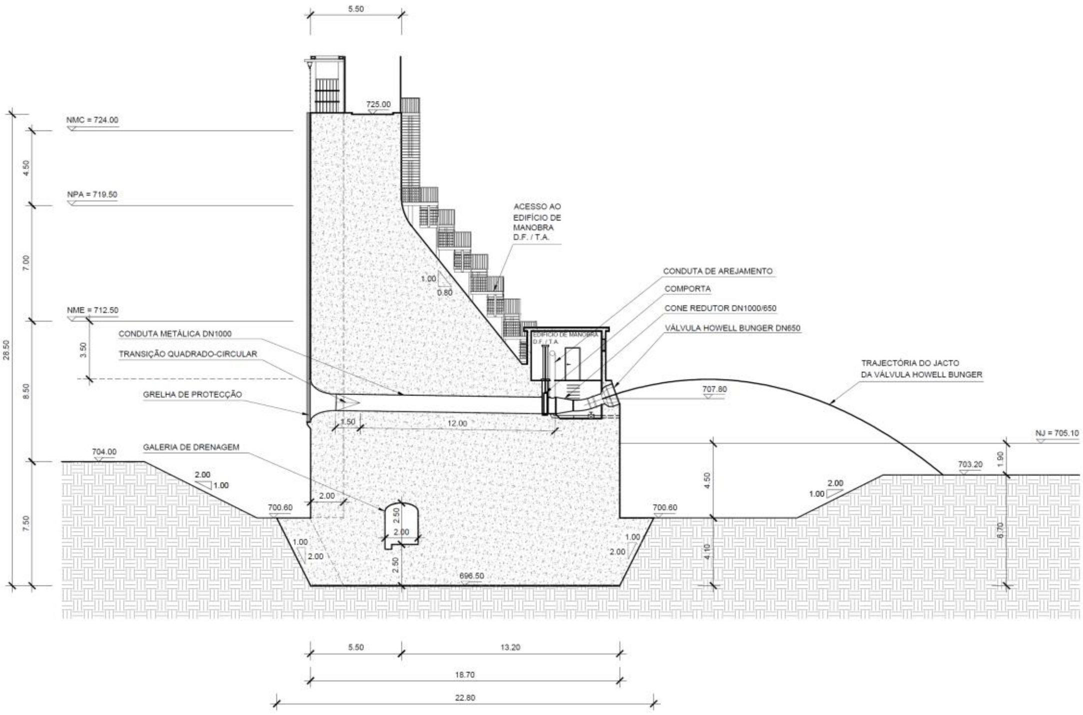

Figure 10 - Bottom outlet. Longitudinal section

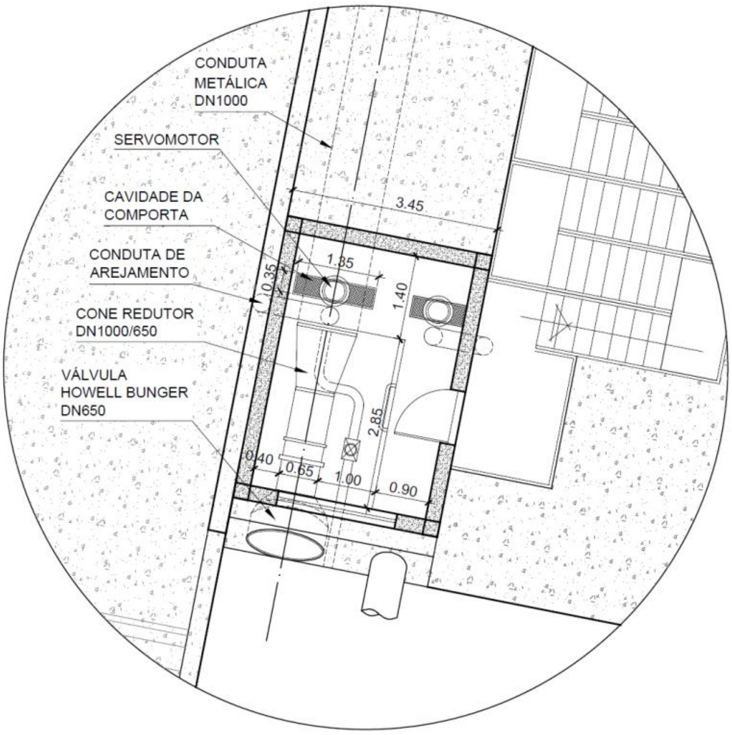

Figure 11 - Bottom outlet; operating building; plan

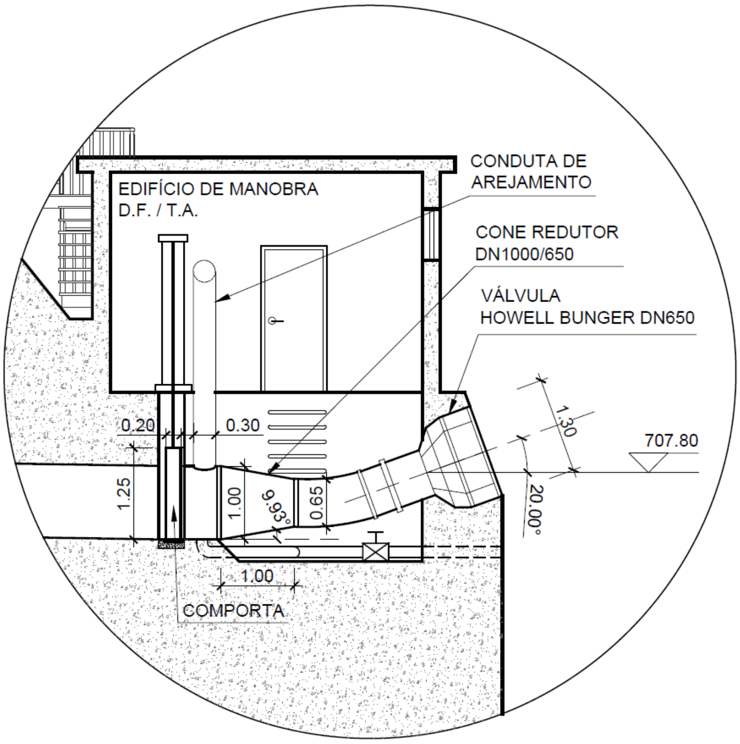

Figure 12 - Bottom outlet; operating building; longitudinal section

WATER INTAKE

The water intake conduit was designed for 1.0 m^3/s, assuming that this discharge represents the water demand downstream.

As the bottom outlet, the water intake is composed by several elements such as the upstream trashrack, the section transition, the steel pipe and the plan gate.

The upstream trashrack was designed once again according to the methodology presented by Pinheiro (2006b) and has 0.80 m width and 1.20 m height. The entrance of the bottom outlet was established at elevation 709.00 m due to the minimum submersion needed of 1.80 m and the minimum operation level defined of 712.50 m. This way the entrances of the two conducts are aligned from the top.

Once again, after the entrance with hydrodynamic shape there is a change from the rectangular section to the circular section of the steel pipe. The steel pipe has 16.85m length and a diameter of 0.60 m due to the design flow considered and assuming a flow velocity on the interior of 4.00 m/s.

The circuit of the water intake also has a plan gate with 0.80 m width and 0.80 m height. The gate can shut down the hydraulic circuit to allow maintenance operations. Once again, an air conduct with a diameter of 0.30 m is located downstream of the plan gate to avoid the formation of vortexes.

The entire water intake is also controlled through the operating building. Table 9 resumes the main characteristics of the water intake elements.

Table 9 - Main characteristics of bottom outlet elements

Upstream trashrack width (m)	0.80
Upstream trashrack height (m)	1.20
Steel pipe diameter (m)	0.60
Steel pipe length (m)	16.85
Plan gate width (m)	0.80
Plan gate height (m)	0.80
Air conduct diameter (m)	0.30
Elevation of water intake entrance (m)	709.00
Design flow (m^3/s)	1.0
Design flow velocity (m/s)	4.00

The main characteristics of the water intake outlet can also be consulted on the figure 9 and on figures 13 and 14.

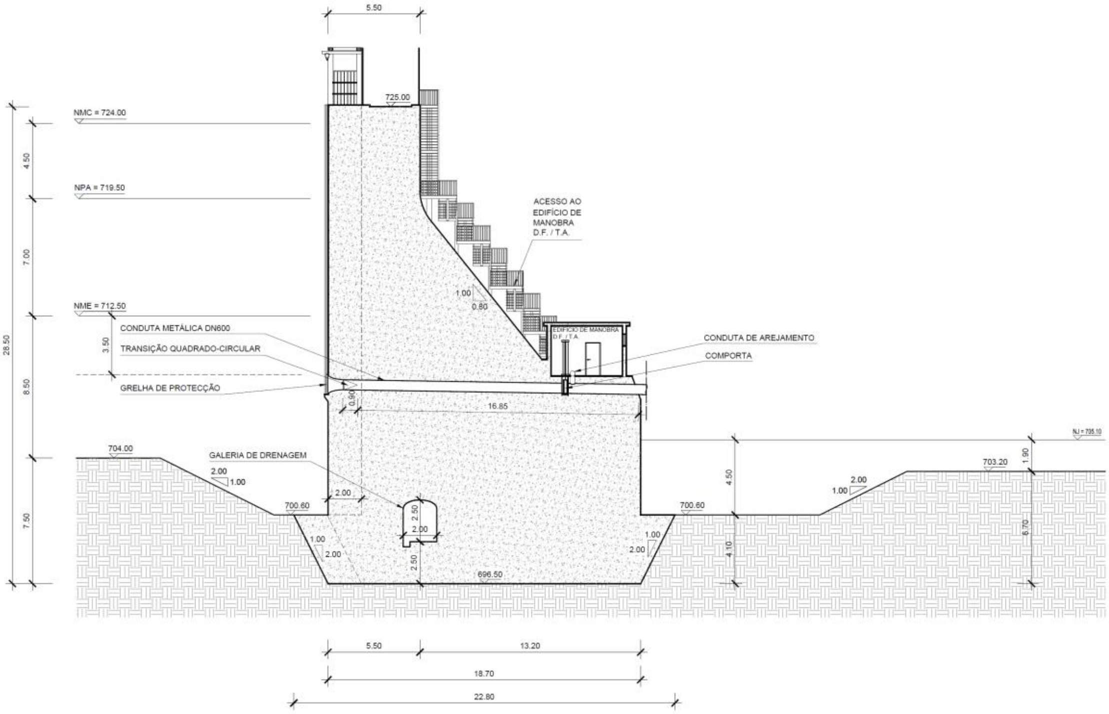

Figure 13 – Water intake; longitudinal section

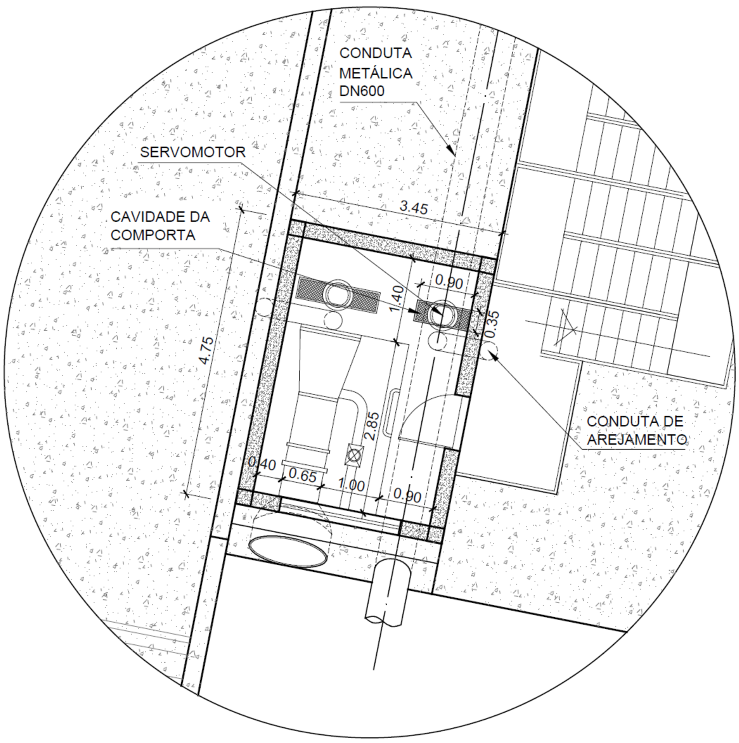

Figure 14 – Water intake; operating building; plant

DAM STABILITY ANALYSIS

The dam stability analysis consists on a sliding stability analysis, an overturning stability analysis and a stress analysis, in sections of the dam that were considered main sections.

All the analysis made include different loading and reservoir level conditions, according to the methodology presented by Batista e Farinha (2011), based on Portuguese legislation. The sections that were analyzed represent the main sections of the dam body - figures 15 and 16.

It should be noted that the sections analyzed have some simplifications relatively to the real blocks that are constructed. Those simplifications were adopted in order to simplify the calculation process and do not affect the truth of the analysis made.

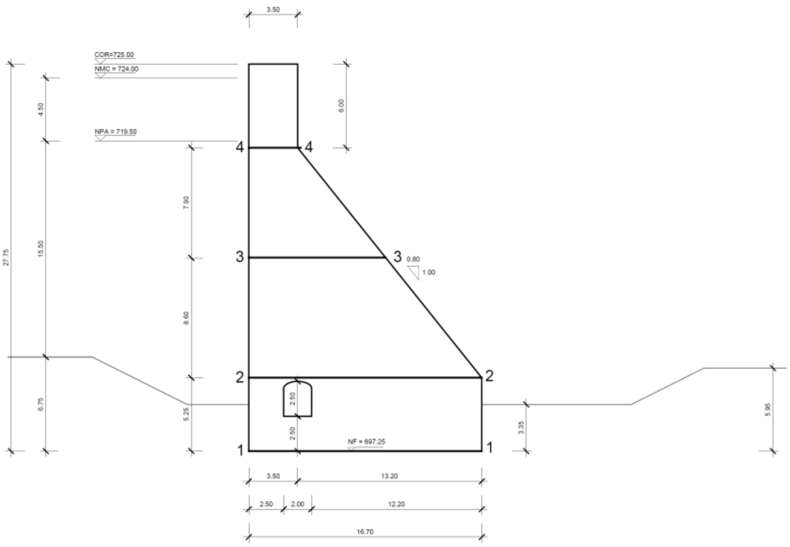

Figure 15 – Sections considered on stability analysis; section 1

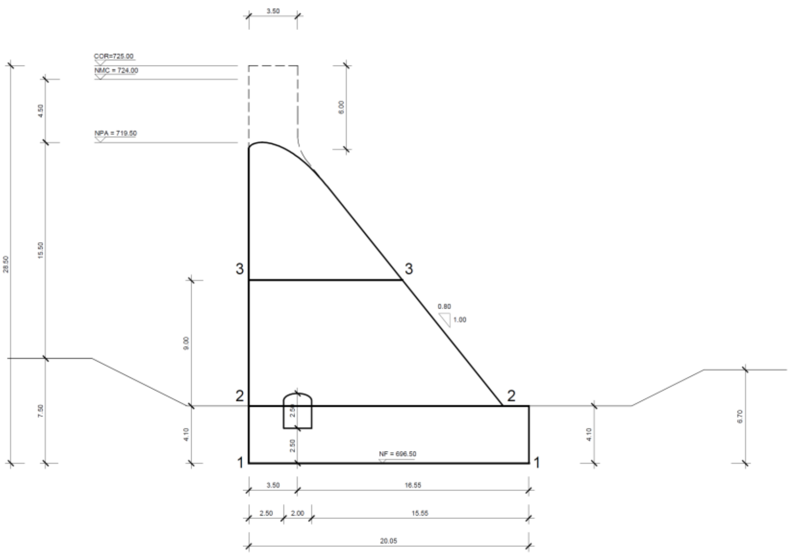

Figure 16 – Sections considered on stability analysis; section 2

The results obtained for the different scenarios analyzed are resumed on tables 10 and 11.

Table 10 – Results of the stability analysis; section 1

		Usual Scenario 1	Usual Scenario 2	Extreme Scenario 1	Extreme Scenario 2
Sliding stability	(kN/m)	1881.21	1473.94	420.07	1253.62
Overturning stability	(kNm/m)	38236.07	34218.90	17243.60	30390.38
Stress analysis at foundation	σ_{max} (kPa)	490.93	565.51	723.26	625.48
	σ_{min} (kPa)	165.76	91.18	0.00	14.11
Stress analysis at section 2 (KPa)	σ_{max} (kPa)	388.77	345.50	398.33	295.10
	σ_{min} (kPa)	150.44	188.23	140.88	232.26
Stress analysis at section 3 (kPa)	σ_{max} (kPa)	321.75	293.23	199.67	260.01
	σ_{min} (kPa)	37.86	62.73	159.94	91.69
Stress analysis section 4 (kPa)	σ_{max} (kPa)	144.10	167.84	144.10	195.50
	σ_{min} (kPa)	143.90	117.23	143.90	86.16

Table 11 - Results of the stability analysis; section 2

		Usual Scenario 1	Usual Scenario 2	Extreme Scenario 1	Extreme Scenario 2
Sliding stability	(kN/m)	1714.05	1290.62	225.43	1178.02
Overturning stability	(kNm/m)	48627.45	44552.08	25086.09	41365.16
Stress analysis at foundation	σ_{max} (kPa)	302.05	415.71	418.54	447.51
	σ_{min} (kPa)	245.23	124.83	0.00	85.17
Stress analysis at section 2 (KPa)	σ_{max} (kPa)	308.05	268.37	362.60	294.06
	σ_{min} (kPa)	219.77	254.08	165.21	222.15
Stress analysis at section 3 (kPa)	σ_{max} (kPa)	194.13	175.22	212.51	153.42
	σ_{min} (kPa)	103.16	118.95	84.78	137.34