Zeitschrift: IABSE structures = Constructions AIPC = IVBH Bauwerke

Band: 7 (1983)

Heft: C-25: The Itaipu Dam: Design and construction features

Artikel: Hollow gravity and buttress dams

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DOI: https://doi.org/10.5169/seals-18269

6. Hollow Gravity and Buttress Dams

6.1 Shape of Hollow Gravity Blocks

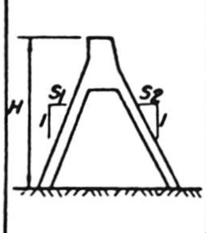
There are two sub-types of hollow gravity dams. In one type a large longitudinal opening or cavity is formed inside a conventional solid gravity dam. The second group consists of several variations of the buttress dam. For Itaipu, the double-buttress cell type was adopted. Each hollow gravity block is designed as a monolithic cell, consisting of an upstream head supported by two buttress stems and enclosed by a downstream face slab. Each block is a cantilever, elastically fixed to the foundations and structurally independent of the adjoining blocks. Adjoining blocks abut against each other at the upstream head, at the downstream face slab and in the upper portion. But the blocks are separated by transverse contraction joints.

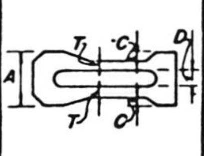
At the time, the highest hollow gravity dam in the world was the José Maria de Oriol (Alcantara) Dam in Spain, which is 130 m high and was completed in 1969. Since the Itaipu Dam represented an increase of 46 per cent in height over the present record, its design was analysed not only by conventional methods, but also by two and three dimensional finite element methods (FEM) and by structural model tests. In addition, comparative studies of the design and performance of existing hollow gravity dams were made to determine the various structural details of the Itaipu Dam.

The more important dimensional data for Itaipu as well as for some existing hollow gravity dams are shown in Fig. 11. The width of each Itaipu block is 34 m, which was determinded by the spacing of the power generating units located at the toe of the dam. It is 55 per cent more than the 22 m width of the blocks in the Alcantara, Ancipa and other dams. On the other hand, the upstream and downstream slopes of the Itaipu blocks are flatter than most existing hollow gravity dams.

The Hollowness Index, defined as the ratio of the net volume of concrete to the gross volume of the solid cross-section, for the various dams is also shown in Fig. 11. The typical Itaipu block is about average in massiveness as compared to the others. Of the 18 hollow gravity blocks of the Itaipu Dam, 16 have a power intake located on the top and a 10.5 m diameter penstock supported on the downstream face. The cross-section and plan of a block with a power intake are shown in Fig. 12. The remaining 2 blocks, which are without a power intake are similar in configuration to the blocks with power intakes for the lower 50 per cent of the height with the downstream face slab eliminated in the upper part.

The large power intakes through the upper part of the blocks, considerably reduce the weight and stabilizing force. This was partly compensated by the flatter slopes of the upstream and downstream faces and consequent greater base width of the Itaipu blocks, as compared to other dams.



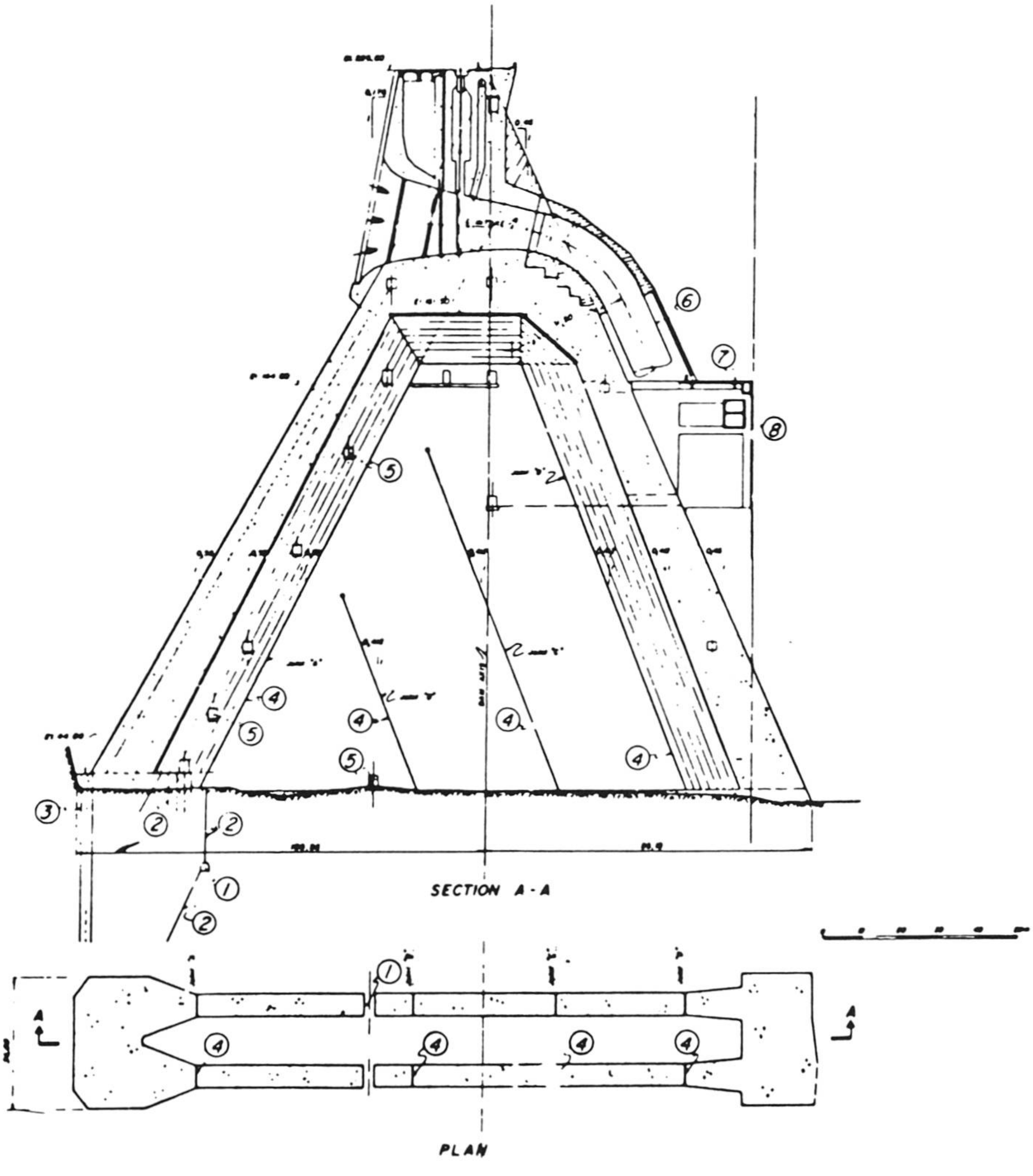


DAM	COUNTRY	YEAR COMPLETED	MAX. HEIGHT H (m)	A (m)	S_1	S_2	C (m)	D (m)	T (m)	HOLLOWNESS INDEX
ITAIPU	BRAZIL PARAGUAY	1982	190	34,0	0,58	0,46	4,8	12,4	6,0	0,57
ALCANTARA	SPAIN	1969	130	22,0	0,45	0,45	2,5	7,0	5,0	0,61
ANCIPA	ITALY	1952	112	22,0	0,45	0,45	3,5	7,0	4,0	0,55
MALGA BISSINA	ITALY	1957	87	22,0	0,45	0,45	2,85	7,0	4,65	0,60
LATIYAN	IRAN	1967	87	28,0	0,45	0,48	5,38	5,75	5,75	0,56
DIXENCE	SWITZERLAND	1937	81	26,0	0,04	0,81	4,00	8,00	5,00	0,60
SABBIONE	ITALY	1952	66	22,0	0,45	0,45	3,0	7,0	4,50	0,61
PANTANO D'AVIO	ITALY	1956	65	22,0	0,45	0,45	2,88	7,0	4,62	0,61
BAU MUGGERIS	(SARDINIA) ITALY	1949	63	22,0	0,45	0,45	3,5	7,0	4,0	0,57
TRONA	ITALY	1942	58	24,0	0,05	0,64	3,7	7,4	4,6	0,62
INGA	ZAIRE	1977	48	18,0	0,55	0,40	1,75	7,5	3,5	0,65

Fig. 11 Comparison of hollow gravity dams

The shape of the upstream head of the hollow gravity block, which will withstand the reservoir pressure, was refined by a series of two-dimensional FEM studies, which considered the influence of the following variables on the stresses in the massive head.

- Chamfers to trim the upstream corners, which affect the resultant water pressure.
- Location of waterstops in the transverse joints, which determine the amount of balancing lateral hydrostatic pressure on the head.
- Net minimum thickness at the crown, and downstream slope of the head.
- Thickness of the buttresses supporting the head.



1. *Foundation Drainage Gallery*
2. *Drains*
3. *Grout Curtain*
4. *Contraction Joints*
5. *Access Gallery*
6. *Penstock*

Fig. 12 Hollow Gravity Dam
Typ. Highest Block with Power Intake Section and Plan

6.2 Longitudinal Contraction Joints in Hollow Gravity Blocks

For large hollow gravity dam blocks, concrete cannot be placed in continuous lifts without either steel reinforcement or contraction joints. The configuration of a hollow gravity block in plan is complex for continuous placement of concrete, and for the Itaipu Dam, the volume of concrete in each pour is so large that cold joints could not be avoided. An additional factor, discussed in greater detail later under temperature control, is that the uneven cooling of concrete in the various parts of the block, causes temperature gradients between the more massive head and the slender buttress stems. This, in turn, causes high tensile stresses and cracking, starting near the foundation where the restraint is maximum.

The provision of longitudinal contraction joints is a practical solution for avoiding cracking in hollow gravity dams and at the same time facilitating their construction.

In the system adopted, the contraction joints are parallel to the upstream and downstream faces of the dam.

The lower portion of the highest block consists of 8 monoliths with 6 contraction joints. This joint arrangement has the following practical advantages:

- Concrete can be placed in as many as 4 monoliths per block at the same time.
- Layout of monoliths is compatible with the parallel cableways used for concrete placement.
- Partially complete blocks are stable against water pressure to the top of the complete upstream head.
- The largest concrete pour (upstream head, 2,200 m^3, in 2.5 m lift) can be placed by one cableway in about 24 hours.

These advantages were proven during construction, permitting placement of an average of 135,000 m^3 of concrete per month in the hollow gravity dam during 1979-81.

To provide sufficient shearing resistance between the monoliths, two types of unreinforced concrete keys were formed along the longitudinal contraction joints: (1) trapezoidal in the joint parallel to the upstream face and (2) triangular in other joints. The longitudinal contraction joints will be grouted before the reservoir pressure is applied against the dam to obtain partial or full monolithic action of the hollow gravity block. The degree of monolithic action attained would depend on the opening of the joint at the time of grouting and the penetration and coverage of the grout.

Hollow gravity dam

6.3 Transverse Contraction Joints

The transverse contraction joints between the upstream heads of the hollow gravity and buttress blocks are not keyed. This would permit independent movement between adjacent blocks in a direction perpendicualr to the axis. As shown in Fig. 13, the transverse joints between hollow gravity blocks have 3 polyvinylchloride seals. A formed drain to handle percolating water, and carry it through the galleries to sumps and pumping plants is provided between the two upstream seals. The joint space between the two downstream seals can be grouted at a later date if it is considered beneficial or necessary.

Concrete with maximum aggregate size of 38 mm was used adjoining the transverse contraction joint and the waterstops to ensure water tightness and better consolidation of concrete in this critical area. The upstream seals were located 2.5 m downstream of the upstream face to let the lateral water pressure in the open joint counterbalance the tensile stresses which might occur in the massive head. In order to relieve the pore pressure in the concrete, a series of 200 mm diameter drains are formed in the head at 3 m spacing. These drain into the galleries provided at various elevations from which the seepage water can flow by gravity into a collector system.

The watertightness of the transverse contraction joints was tested during July-September 1981, by capping the exits of the drains near the bottom and filling them with water to various levels up to El. 140. In this manner some of the joints were subjected to hydrostatic heads of as much as 100 m. Any leakage detected through construction joints or pockets in the concrete itself was sealed with epoxy mortar on both the upstream and downstream faces.

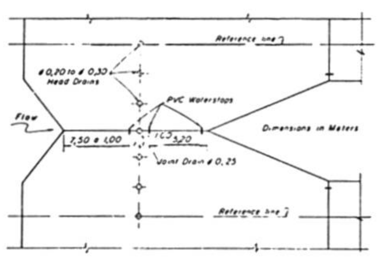


Fig. 13 Transverse contraction joint

6.4 Concrete Characteristics

The specified strengths of concrete in the various parts of a typical hollow gravity dam block and the heights of lifts are shown in Fig. 14. The standard lift for concrete placement was 2.5 m.

The predominant class of concrete used in the dam was A140f, which means a minimum compressive strength of 140 kg/cm² at 365 days age and using aggregate with a maximum size of 15 cm. To obtain low permeability and a higher tensile strength, concrete in the upstream head was made from smaller (7.5 cm) aggregate and with a higher cement content.

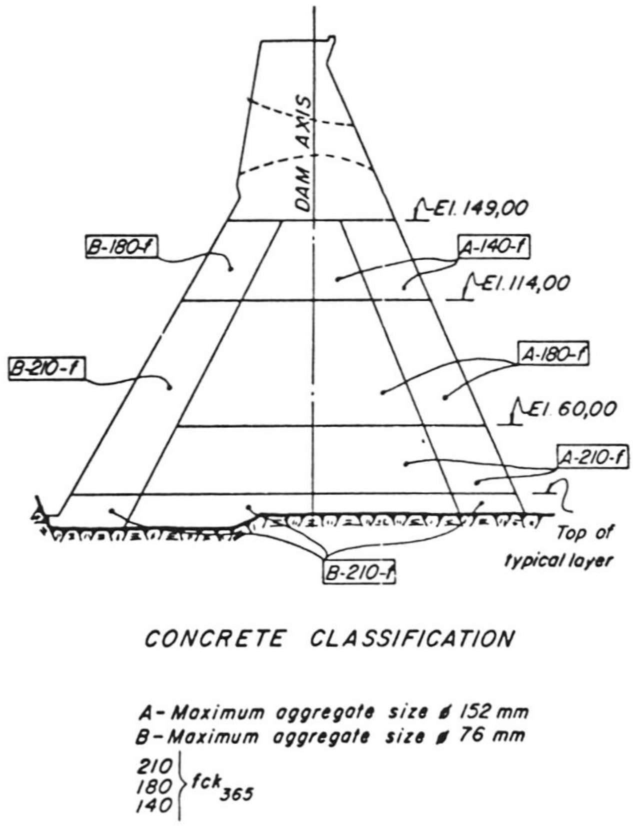


Fig. 14 Hollow gravity dam

6.5 Stress and Stability Analyses; Criteria

The loading conditions used for the stress and stability analyses of the buttress and hollow gravity dams were divided into 4 classes:

- ***Normal Loading Conditions*** (NLC): all load combinations during normal project operation and routine maintenance for average hydrological conditions.
- ***Exceptional Loading Conditions*** (ELC): all statistically possible but infrequent cases during project operation and major maintenance.
- ***Ultimate Loading Conditions*** (ULC): highly improbable combination of events (gross overloads, catastrophic floods, major equipment malfunction and accidental loads) during construction or operation.
- ***Construction Loading Conditions*** (CLC): temporary loads applied during construction, such as construction equipment loading, loading during erection of permanent equipment, or any temporary condition on an incomplete monolith or block.

Although the project region is considered to be aseismic or of very low seismic activity, horizontal and vertical seismic accelerations of 0.05 g were included in NLC and 0.08 g in ELC in the most unfavorable directions.

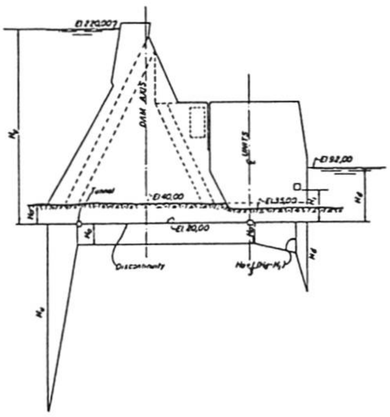


Fig. 15 Uplift criteria

The hydraulic uplift pressure distribution assumed along the foundation contact of the highest blocks of the main dam and the adjacent powerhouse for loading conditions NLC, ELC and ULC is as follows. (Fig. 15):

- Upstream of the dam and downstream of the powerhouse as far as the grout curtains, uplift pressure equals the upstream and downstream head of water (H_u and H_d) respectively down to the rock discontinuity (El. 20).
- Between the upstream and downstream longitudinal underground drainage tunnels (El. 20), the uplift pressure H_o is constant and equals the distance between ground surface and the rock discontinuity elevations (El. 35 to 40 and El. 20).
- Between the grout curtains and the drainage tunnel upstream, there is a linear uplift pressure variation (from H_u to H_o).
- At the downstream drainage curtain the uplift equals $H_o + 0.33.(H_d - H_o)$ and from there the uplift pressure has a linear variation increasing to H_d at the downstream grout curtain and decreasing to H_o at the downstream drainage tunnels.
- Both underground tunnels will normally be pumped and subsequently $H_o = o$ approximately. For safety reasons however and because of possible damage to the pumps, the above indicated uplift value H_o has been maintained.

The factor of safety against shear sliding was defined as follows:

$$FS = \frac{\dfrac{\Sigma N.\tan \phi}{: Y_\phi} + \dfrac{C.A}{Y_c}}{\Sigma H}$$

where:

ΣN = Sum of vertical forces.
ΣH = Sum of horizontal forces.
C = Cohesion or unit shear strength.
A = Effective area of section considered.
ϕ = Angle of frictional resistance.
Y_ϕ = Partial safety factor pertaining to friction.
Y_c = Partial safety factor pertaining to shear strength.

Table VII-I shows the minimum permissible factors of safety against shear sliding which were used in the analyses for the Itaipu Dam for the various loading conditions.

TABLE VII-I

Allowable partial safety factors affecting cohesion and coefficient of friction

	Normal Loading Conditions	Exceptional Loading Conditions	Ultimate Loading Conditions
γ_c	4.0 3.0 (*)	3.0	1.5
γ_ϕ	2.0 1.5 (*)	1.4	1.1

*In case of adequate knowledge and good quality of the foundation.

Maximum allowable stresses in the dam for the various classes of concrete used were determined by the field laboratory of Itaipu Binacional. Table VII-II shows the various classes of mass concrete used in the Hollow Gravity Dam, the specified minimum compressive strength and the allowable compressive and tensile stresses.

TABLE VII-II

Allowable compressive and tensile stresses of the mass concrete used in the hollow gravity dam

Concrete Type f_{ck} (kg/cm²)	Allowable Compressive Stress (kg/cm²)	Allowable Tensile Stress (kg/cm²)
365		
210	70	7.0
180	60	6.0
140	46	4.6

Considering the size of the Itaipu concrete dams, in addition to the conventional methods of flexural and sliding analyses, mathematical models using the finite element technique (FEM) and physical models were extensively employed.