INTERNATIONAL SOCIETY FOR
SOIL MECHANICS AND
GEOTECHNICAL ENGINEERING

This paper was downloaded from the Online Library of the International Society for Soil Mechanics and Geotechnical Engineering (ISSMGE). The library is available here:

https://www.issmge.org/publications/online-library

This is an open-access database that archives thousands of papers published under the Auspices of the ISSMGE and maintained by the Innovation and Development Committee of ISSMGE.

Proceedings of the 16th International Conference on Soil Mechanics and Geotechnical Engineering
© 2005–2006 Millpress Science Publishers/IOS Press.
Published with Open Access under the Creative Commons BY-NC Licence by IOS Press.
doi:10.3233/978-1-61499-656-9-1849

Potrerillos Project: Diaphragm wall design

Project Potrerillos: La conception de la paroi moulée

Arnaldo M. Barchiesi
GCPP - Toso Hnos. y Asociados, Av.España 1537- 5500 Mendoza Argentina
tosohnos@infovia.com.ar

Philippe Cazalis & Alain J. Carrère
GCPP – Coyne et Bellier , 9, Allée des Barbanniers - 92632 Gennevilliers CEDEX – France
philippe.cazalis@coyne-et-bellier.fr - alain.carrere@coyne-et-bellier.fr

ABSTRACT

Recently the Potrerillos Project`s works, at the Mendoza river into the same name province in Argentina, were concluded. The CFRD dam is allowing the filling of a 450hm^3 reservoir that is provided with a morning glory spillway, a diversion bottom and half – bottom outlet tunnel, as well as an adduction tunnel to two arranged in cascade power stations.

With the purpose to provide the waterthightness of the Mendoza River`s alluvial valley upstream to the dam, the design has disposed a diaphragm wall, whose conception includes the combination of distinct materials and constructive principles, whit a maximum deep greater than 60m. Special interest have the applied work – efficiency valuation procedure and its monitoring system.

This work presents the most relevant aspects corresponding to the diaphragm wall design evolution, until the definitive solution was achieved, as well as the work – efficiency valuation procedure.

RÉSUMÉ

Les travaux du Projet de Potrerillos sur la rivière Mendoza, dans la province d'Argentine du même nom, viennent de se terminer récemment. Le barrage de type CFRD créant une retenue de 450 hm3, dispose d'un évacuateur de crue en forme de corolle, d'un tunnel de dérivation converti en vidange de fond et de demi fond ainsi que d'un tunnel d'amenée alimentant deux centrales hydroélectriques en cascade.

Dans le but d'obtenir l'étanchéité de la fondation alluviale profonde de la rivière Mendoza, en pied amont du barrage, il a été prévu la réalisation d'une paroi moulée d'une profondeur maximale supérieure à 60m et dont la conception inclut la combinaison de différents matériaux et de procédés constructifs distincts. On portera un intérêt particulier au procédé d'évaluation de l'efficacité de la paroi en terme d'étanchéité ainsi que son dispositif d'auscultation.

Cette publication présente les aspects principaux de l'évolution de la conception de la Paroi Moulée, la solution définitive adoptée, et le principe d'évaluation de l'efficacité de la Paroi en terme d'étanchéité.

1 INTRODUCTION

The Potrerillos Project into the Mendoza River in Argentina will generate an average 770GWh/y with a global cost of U$S268M. The 120m heigh dam is the mean work of the Project. It is possible to see a Project`s general description in (Carrère et al, 2000).

In 1995 the Province of Mendoza bided the Project's concession for 25 years, awarding it to the "Consorcio de Empresas Mendocinas para Potrerillos" (CEMPP SA) integrated by the firms JCCC SA and IMPSA, that constructed the works. The subcontract corresponding to the diaphragm wall was in charged to the associated companies CIMARG (Soletange & Bachy from Argentina) and Pilotes Trevi SA.

The 1995 bid proposal included a preliminary definitive design entrusted to Coyne et Bellier (from Paris), Geotécnica Consultores (from Santiago de Chile) and Toso Hnos. y Asociados (from Mendoza).

After the approval of the Preliminary Definitive Design, the same engineers developed the Definitive Design (1998), the Executive Design (1999 – 2001) and a technical assistance to the work's construction (1999 – 2002).

Height values of seismic peak ground accelerations in rock (pga) were considered: 1.02g.

The Project's works are located into the "Cacheuta granite stock", whose hard, reddish, superficially low weathered, moderately jointed and faulted rock, configures a favorable geological surroundings.

Dense or very dense alluvial deposits constitute the riverbed, with predominance of sandy gravels and blocks, and the local presence of finer materials (sands, silts and clays). Within a deep greater than 25 or 30m, those finer materials are disposed as a collection of deposits with strong lateral changes and strong thickness variations (1 to 8m).

Because of eroded greater than 100m thickness alluvial deposits over the actual riverbed, the pack of foundation's materials, are strongly overconsolidated.

The great maximum thickness of those deposits (>60m), forced to dispose a diaphragm wall -located upper stream to the dam- as a barrier to water flow crossing the foundation.

2 THE DAM AND THE DIAPHRAGM WALL

Potrerillos dam, whose maximum height reaches 120m, was constructed with alluvial materials, similar to those constituting the foundation.

A classic arrangement for the dam section was adopted, organizing materials by increasing permeability from upstream to downstream side: The concrete face slab, by means of a transition (2B material), rests upon an controlled permeability embankment (3A material with k~1E-4 ÷ 1E-5m/s) resting in turn upon the grosser 3B material. Between those two sectors, a high permeability drain was designed (4A with k~1E-2m/s), to collect water flows and to reduce pore pressures.

The top of the diaphragm wall connects with the concrete face slab by means of the "foot-slab" (a horizontal plinth) composed by two articulated segments, provided with impermeable cupper seal-joints. This high sensibility sector, is covered by an impervious fill (material 1) consistent in a fine un-cohesive and un-compacted material.

The arrangement of the concrete face slab and plinth is the conventional one.

The watertightness is completed by means of a grout curtain that penetrates into the rock mass located under the plinths and the diaphragm wall.

3 ALTERNATIVES ANALYSIS

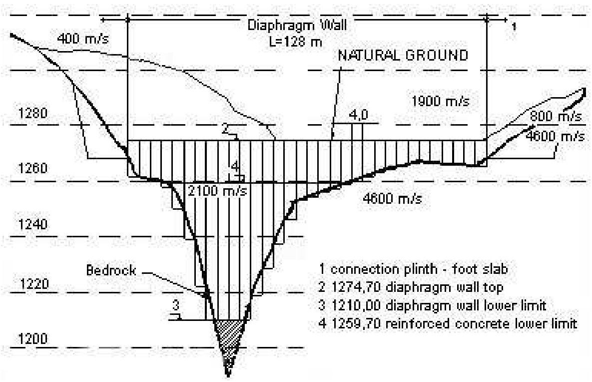
3.1 *Objectives*

In order to fulfill his function of strongly reduce the water flows across the foundation, the diaphragm wall, will must fulfill other important conditions: i) proper relation between the work and the others watertightness devices: foot – slab, concrete face slab and plinths, and with the rock mass; ii) sufficient mechanical characteristics in order to resist deformations, moments and shears that could be imposed to the work; iii) stable materials and construction without unnecessary complexities.

3.2 *Precedents*

The construction of a diaphragm wall isn't a simple task. The difficulties are related with the selection of a suitable constructive methodology, the equipments costs, the high capacity required to technicians and operators and with quality and results controls.

The Definitive Design foresaw a conventional concrete diaphragm wall with an upper 15m deep reinforced zone, penetrating 0.5m into the rock in order to assure the waterthigtness (see Fig. 1).



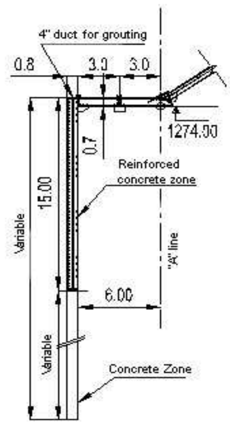


Fig.1: Diaphragm wall's outline, proposed into the Definitive Design.

The deep ancient riverbed with very steep slopes, gave rise a problem referred to the feasibility and foreseeing of the constructive processes related to fit the wall in rock. This aspect required a second analysis during the Executive Design that took into account the technical proposals of the specialized contractors.

In the following, two studied possibilities (with hydro- mill and with jet grouting) are discussed, and the selection of the second one is justified.

3.3 *Alternative with Hydro mill*

The hydro mill goes forward because of his two cutting wheels rotating in opposite sense. The very steep slopes of the rock would generate an important resistant force into the cutting wheel that would inevitably tend to deviate the hydro mill, fasting it from the rock and its verticality, without giving guarantees of correct sealing with the rock. By this reason the procedure was leaved out.

3.4 *Adopted Alternative: With Jet Grouting*

In order to achieve the fitting of the wall into the rock, the advance of each panel excavation whit a total of the order of 6,5m width in three stages (of 2,7m; 1,1m and 2,7m) still the tool reaches the steep rock in one of its extremes, was foreseeing, (Fig. 2). So, alluvial triangular windows would be shaped under each panel that would be later waterproofed. The width of these windows resulted from the with of the clam plus the intermediate strip (aprox. $2.7+1.1 \cong 3.8$m).

Since their proved efficiency, the triangular window's treatment was stipulated with jet grouting, using guide tubes leaved in each panel before putting the concrete in place. Across of them, drilling still jet grouting penetrates 0.5m in rock was specified, achieving an excellent contact between rock and alluvial and making a true "in situ concrete".

Because vertical joints represent a weak zone for waterthigtness, the sealing between contiguous panels was taken seriously, preferring to make them as large as possible. Steel CWS joints were adopted to conform a key till 25m deep (the 25m limit became from negatives experiences related to deeper CWS joints).

In deeper than 25m sectors, it was decided to use plastic concrete that, during excavating each panel allows a suitable overlap within the precedent panel.

The presented solution joins positive experiences from the designer, the contractor and the subcontractor, confirming the key points from the Executive Design (see Álvarez et al, 1982; Chanez et al, 1965; Coste et al, 1998; IWPDC, sep. 1999).

4 SPECIAL DISPOSALS

The Executive Design defined the basic constructive process of the work that was completed with detail and execution engineering realized by the contractor.

The construction started with "guide beams" in the top of the work that were demolished when it was finished.

In both margins the bedrock is hard steep and the alluvial deposits are less than 25m thickness.

There, the "classic" method with spoon clam and chisel in presence of bentonithic mud and conventional concrete in alternate panels was used. Hence the main conditions were: i) 1m thickness conventional concrete; ii) minimum 0,5m fit in rock; iii) approximately 6.5m with successive panels; iv) 300kg - cement/m^3 conventional concrete for panels; v) watertightness between panels by means of key – joints; vi) top beam and upper 15m reinforced concrete steel bars. Those disposals extend into the upper 15m of the whole work.

Furthermore the previous description, there are three particular situations: a) right margin with 7m deep aluvial: conventional opencast excavated formwork concrete wall with fit in rock by means of the same excavation procedure. b) Left margin with jet grouting because of the strong steep bedrock. c) In the event of main difficulties, the excavation deep was limited to 70m.

The demolition of dubious concrete because presence of bentonitic mud into the upper stretch (minimum 0.5m), was indicated. Two alternating jet-grouting lines with distances between centers of 0.7m, were foreseeing. A 50m deep in rock grout curtain was disposed. A complete monitoring system including 93 piezometers, 29 joints opening measurement devices, 23 well's clinometers into the wall, etc. allows a detailed observation of work's responses.

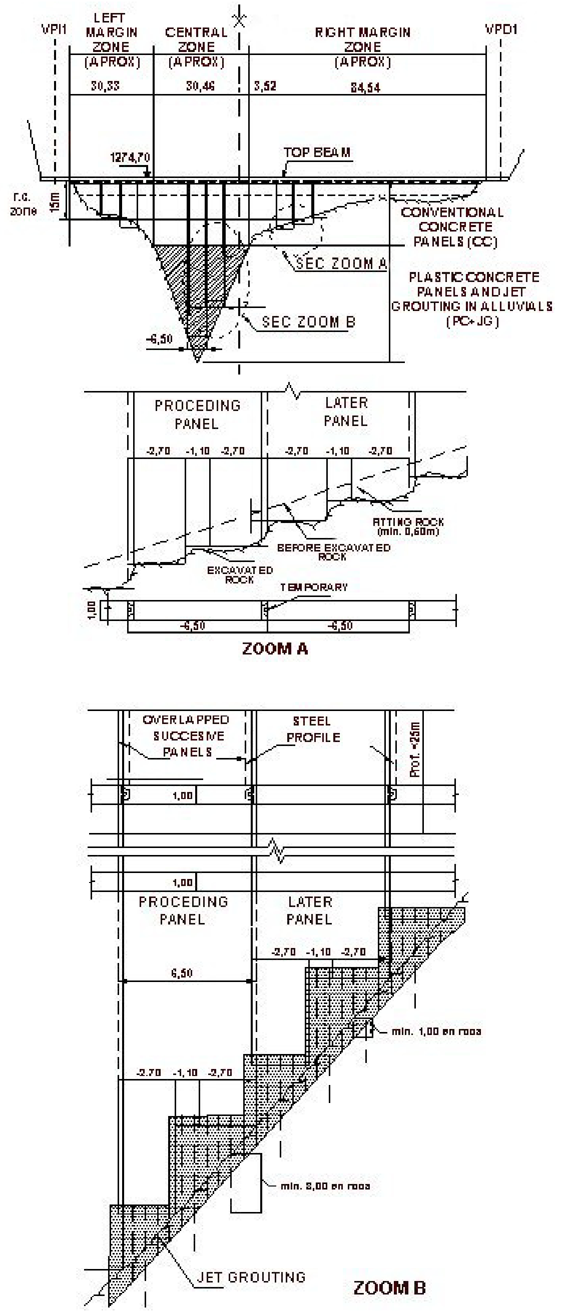


Fig. 2: Outline of the definitive diaphragm wall

5 MECHANICAL RESPONSE

5.1 *General Description – Structural Analysis*

The diaphragm wall is subjected to small deformations imposed by the alluvial deposits that contain and confine it.

The aim of its mechanical response appreciation was the estimation of the order of magnitude in joint's displacements between the work and the foot-slab. Structural analysis took into account the inherent limitations of numerical models. An elastic 2D linear finite element model was applied, whose main characteristics were:

- Two sections were studied: ancient bedrock and margins.
- The constructive sequence was represented: The foot-slab is constructed after the dam gravel fills and the diaphragm wall, and the diaphragm wall is constructed later than the majority of the gravel fills situated under the top beam.
- The articulation joint between the diaphragm wall and the foot-slab was considered. The (negative) effect to stiffen this joint was studied.
- Fasten articulation joints between the work and rigid rock foundation were represented.
- The following materials were modeled with the uniform parameters referred:
-

Parameter	Dam fill	Alluvial foundation	Concrete	
			Plastic	Conventional
Elastic modulus E [MPa]	500	500	500	27500
Poisson coeff. ν	0,25	0,25	0,15	0,15
Unit weight [kN/m^3]	22,0	22,5 (12,5)	24,0 (14,0)	24,0 (14,0)

- Water pressures against the moist face of the diaphragm wall, foot-slab and concrete slab. Downstream water level as less as possible.

Results are summarized in the following table. Cases Ia (stiff joint) and Ib (diaphragm wall – foot-slab articulation) are indicated. Displacements are indicated in mm; stresses in MPa (tensions are negative). Displacements are total: difference between final and initial positions.

Case	Ia	Ib
Dam construction		
Foundation settlement	150	150
Diaphragm wall head vertical settlement	5	5
Horizontal movement diaphragm wall head	-14	-14
Comp.(tension) max. conventional concrete	0,6 (-0,4)	4,0 (-2,8)
Comp.(tension) máx. plastic concrete	0,7 (0)	0,7 (0)
Reservoir filling		
Crest dam settlement	8	8
Concrete slab settlement	115	116
Foundation settlement	50	50
Vertical movement diaphragm wall head	19	19
Horizontal movement diaphragm wall head	83	81
Compression (tension) conventional concrete	12,5 (-5)	6 (0)
Compression (tension) max. plastic concrete	1,5 (0)	1,5 (0)
Compression (tension) max. in foot-slab	5,0 (-1.5)	4,5 (-1.4)

5.2 *Structural Analysis Consequences*

From the calculated stresses into plastic concrete, a 28 days compression resistance of 1,5MPa was specified, corresponding to 2,0MPa at reservoir filling starting.

It was considered that conventional concrete will stand small stresses and its strength isn't a critical point: ∫8MPa (except near upper section and near the contact with rock in both margins).

Steel reinforcements in the wall's head and in the foot-slab were designed verifying the convenience to divide the last one in two articulated segments (Fig. 1).

5.3 *Earthquake-resistant Response*

By means of a Newmark analysis, it was established that for a pga = 0.48g (greater than a pga = 0.34g corresponding to operating basis earthquake OBE), it doesn't occur irreversible displacements nor damages.

By the maximum design earthquake (MDE) with a pga = 1.02g, displacements in the order of 2dm were calculated, partially demanding the cinematic capacity of the foot-slab and joints without affecting its global watertightness. The auto-sealing fines recharge will can significantly attenuate eventual damages. Because of it, the diaphragm wall and foot-slab's stability is guaranteed still MDE level, and even in this case, the consequences would be limited.

It is probable that after extreme earthquakes (MDE) it should be necessary to carry out repairs and to take steps that restrict reservoir operation.

6 WORK EFFICIENCY

6.1 *Precedents*

Both design engineers and the contractor, emphasized the importance to carry out severe and systematic materials and construction's controls, in order to diminish leakage and to assure good results. Furthermore the piezometric surface was continuously observed to detect possible problems.

Experiences in similar works, indicates the relevance to develop controls that allow to verify the global efficiency and to perform the necessary repairs before starting the reservoir filling. In order of this, it was proposed to apply a similar methodology to that applied at Notre-Dame-de-Cummiers dam (Chanez et al 1995).

6.2 *Global Efficiency Criterion*

In order to define an efficiency criterion, it must be considerer that, because of concrete's permeability and constructive imperfections, the diaphragm wall isn't an absolutely waterthightness barrier.

The efficiency criterion adopted, assumes as a maximum aim to maintain the internal drains of the dam immediately above the top flow surface associated with the highest normal reservoir level (el.1377.3m): If when the reservoir reaches its highest normal water level the top flow surface locates just down internal drains, then it is assumed that the diaphragm wall reaches a 100% global efficiency, with bigger or lower values if the surface locates down or above the drains. Thus the drains are available for eventual leakages across the concrete slab or others.

6.3 *Efficiency Expression*

Efficiency can be quantified by means of the outline in fig. 3, with the following basic hypothesis: i) highest water level equal to highest normal reservoir level (el.1377.3m) because floods are short duration; ii) homogeneous alluvial foundation with k_a as permeability factor and validity of Darcy's law; iii) homogeneous diaphragm wall with e as tightness, k_p as permeability factor and validity of Darcy's law; iv) water table's average slope defined by means of piezometers P_A and P_P located at a distance L each one to the other; v) alluvial cross section A sealed by the diaphragm wall equal to control section by the line of piezometers P_A; vi) water table level upstream the diaphragm wall measured into piezometers P_E.

Before the diaphragm wall construction, P_E and P_A levels are almost the same. After construction, P_A level drops and P_E level is sustained by pumping at least 2.5m down the work surface in order to keep an equivalent charge of bentonithic mud, assuring in this way, the excavation stability.

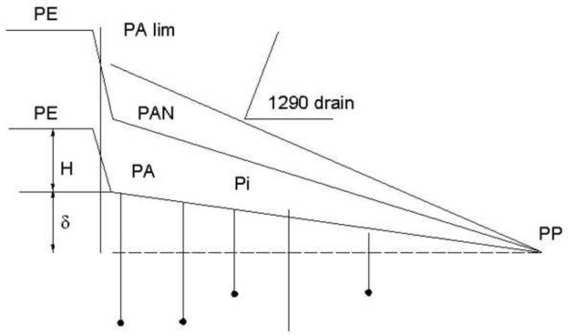


Fig. 3: Global efficiency evaluation

Calling δ to the medium difference between P_A and P_P levels and H to the medium difference between P_E and P_A levels, by means of a geometric analysis, it is possible to establish that the 100% global efficiency condition amounts to the condition $\delta/H=0.37$. It was decided to increase the requirements upon the work especially during the starting stages of the first reservoir's filling, adopting as design criterion the relation $\delta/H=0.33<0.37$. Those conditions are the basis to quantify the global efficiency of the diaphragm wall and associated section of the grout curtain in different important stages: end of construction, during the reservoir's filling and during operation.

7 CONCLUSIONS

The construction of the diaphragm wall with conventional concrete panels still 25m deep and with plastic concrete into greeter deeps with partial fitting in rock and jet grouting treatment wherein it isn't possible to achieve the fit into the rock, was the procedure that fulfilled the greater advantages for the work.

The disposition of bending, shear and fissure control steel reinforcement bars as so on the disposal of the top beam into the most stressed upper conventional concrete sector, was a very suitable design measure, especially considering the exceptional earthquake assumed accelerations.

The global efficiency quantification is a suitable procedure in order to take an indication of the work's efficiency from the start of the reservoir's filling.

ACKNOWLEDGMENTS

The Authors wish to express his gratefulness to CEMPPSA for authorize and promote the publication of this work. They are also grateful to the subcontractor responsible for work's execution for his provided experiences trough his valuable opinions and suggestions.

REFERENCES

Álvarez, Larenas, Bernal, Marín 1982. Characteristics of the Plastic Concrete of the Diaphragm wall of Convento Viejo Dam. *XIV Congrés des Grands Barrages*, Río de Janeiro.

Carrère, Cazalis, Barchiesi 2000. Proyecto Potrerillos: Presentación General y Descripción Particular de la Presa. *XV Congreso Argentino de Mecánica de Suelos e Ingeniería Geotécnica*, Buenos Aires.

Chanez, Puyo, Bonazzi 1965. La prévision et la mesure de l'étanchement réalisé dans une fondation alluvionnaire. *Annales ITBTP (Notre-Dame-de-Commiers), France.*

Coste, Post, Tardieu 1998. Le Barrage du Verney. *Travaux N°564.*

Intl. Water Power & Dam Constr.. Sept.1999. Design and Construction of Chile´s Puclaro Dam.