Case study of the post-earthquake behavior of a CFRD dam

Étude de cas sur le comportement post-sismique d'un barrage CFRD

Núñez E., Sfriso A.O.
University of Buenos Aires, Argentina

ABSTRACT: Potrerillos dam is a 120m high CFRD dam recently built in Mendoza, Argentina. The dam was built on top of a riverbed containing loose sand seams. Concern about the seismic stability of the dam derived in the construction of a large downstream buttress, not considered in the original design and bid. The key question that arose during the consequent claim is: was the buttress essential for the dam's stability? In the paper, a twofold analysis of the problem is performed: i) the analytical methods employed in the original discussion are briefly introduced and discussed; ii) a simple static pushover analysis is performed numerically, where the sand seams are forced to follow the full strain path through contractive to dilating to critical state. In both analyses, the thrust required to displace the dam body is compared to the hydrostatic pressure to assess on dam's stability. The results obtained using both methods are compared. The main conclusion is that the self-weight of the dam produces an increase in the undrained shear strength of the sandy seams that yields the downstream buttress not essential.

RÉSUMÉ: Le barrage de Potrerillos est un type CFRD de 120 m d'hautesse construit récemment à Mendoza, Argentine. Le barrage fut construit dans un lit de rivière qui contient des strates de sable lâche. Il y avait une inquiétude sur la stabilité sismique du barrage laquelle vient dérivée de la construction d'un contrefort en aval, qui n'était pas considérée dans le design originaire et l'offre. La question clé qui surgit pendant la réclamation conséquente c'est : est-ce que le contrefort était essentiel pour la stabilité du barrage? Dans l'essai, une analyse double du souci est proposée: i) les méthodes analytiques utilisés dans la discussion originaire sont brièvement introduits et discutés; ii) une simple analyse statique faible est réalisé numériquement, où les strates de sable sont forcés à suivre le parcours de tension complète de la contraction en passant à la dilatation jusqu'à l'état critique. Dans touts les deux analyses, la poussée nécessaire pour déplacer le corps du barrage est comparée avec la pression hydrostatique en évaluant la stabilité du barrage. Les résultats obtenus en utilisant les deux méthodes sont comparés. La conclusion principale c'est que le poids il-même du barrage entraîne un augment de la résistance au coupe et non drainée des strates de sable, lequel ne fait pas nécessaire le contrefort.

KEYWORDS: CFRD dam, seismic analysis, liquefaction, numerical analysis.

1 INTRODUCTION

Potrerillos is a 120m high CFRD dam located in Mendoza, Argentina, in a region of high seismicity (Carmona et al 2004, Zabala et al 2006) yielding a design earthquake with a magnitude M = 7.7 and a peak ground acceleration PGA = 1.02. The dam has upstream and downstream slopes of 1.5:1 and 1.8:1 and is founded on an alluvial riverbed made of dense gravels up to 70m thick. The hydraulic closure of the upstream face in the riverbed is furbished by a conventional concrete slab connected to a cast-in-situ diaphragm wall by a floating plinth.
The project has been thoroughly described elsewere (e.g. Barchiesi et al 2006, Núñez 2010a). The cross section of the dam is shown in Figure 1 (Barchiesi et al 2006).

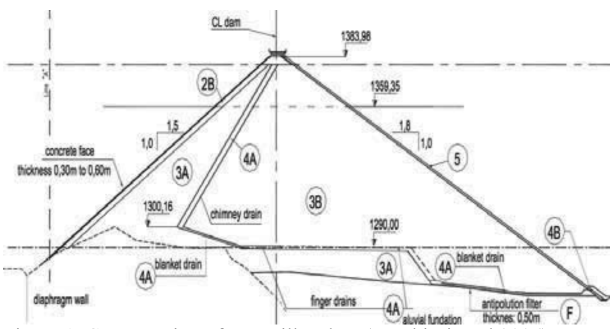

Figure 1. Cross section of Potrerillos dam (Barchiesi et al 2006).

Geotechnical campaigns performed before the construction of the dam failed to identify sand and low plasticity silt lenses, 2m to 4m thick, that are located some 30m below the mean foundation level and that were sistematically found during the execution of the diaphragm wall and the installation of piezometers. It could not be determined and is yet uncertain if these sands and silts lie in isolated pockets or form a continuous layer throughout the dam foundation; in the latter case, they might pose a significative risk of failure due to liquefaction / strength loss during and after a major earthquake.
While client engineers assumed that the layer was continuous and liquefiable, design and construction engineers did not believe the continuous layer assumption and argued that the existence of those materials was of no concern. As the construction of the dam was highly advanced and the foundation surface was no longer accessible, only a few additional SPT tests were scheduled and performed, where low plasticity silts (4 < PI < 8) and medium dense sands were found again.
Client engineers sat back on their worst case scenario of a continuous layer, assumed an undrained shear strength of 25kPa to 50kPa for the layer and concluded that the dam would not be stable during earthquake loading.
Based on this body of information, the client instructed the contractor to build a 19m high downstream buttress embankment to improve the overall seismic stability of the dam. The contractor built the buttress and started a claim that soon escalated to the Supreme Court of Justice of Mendoza. The first author was nominated as expert witness and was asked only one, yet crucial question: was the buttress indispensable for the stability of the dam?
To answer it, he performed an independent study of the case based on a thorough review of the geotechnical data and a few rigid body stability calculations. This paper summarizes such study, presents its main conclusions, and briefs the results of an alternative numerical analysis of the same problem where the risk of progressive failure is conceptually addressed.

2 BACKGROUND

2.1 *Basics of sand liquefaction analysis*

A. Casagrande (1936) introduced the concept of critical state void ratio e_c, stating that, upon continued shearing, sands reach the so called critical state where they yield at constant shear stress and volume. The shear stress attained is only a function of e_c, which is in turn a function of the confining effective stress σ_c at the criticial state (Casagrande 1975). The concept applies to drained and undrained shear: in drained shear, σ_c is constant, void ratio e changes to pair that σ_c at the critical state; in undrained shear, e is constant and, therefore, σ_c must change to pair $e = e_c$ (Casagrande 1936, 1975; Castro 1975, Castro and Poulos 1977, Poulos 1981, Poulos et al 1985, Núñez 1991).
During the undrained shear of loose sands, an intermediate "phase transformation state" exists where the shear stress is lower than the undrained shear strength at the critical state $s_{u,c}$ (Castro 1975, Ishihara 1993).
The shear stress at the phase transformation state is not a properly defined "strength" because it does not correspond to either a peak or a final stress state. However, it is noted that it remains fairly constant for a considerable strain span and is therefore highly relevant for engineering computations. It will be denoted s_u in this paper for practicity.
Depending on the relative density of the sand, three distinct behaviors are observed: a) for very loose sands, the phase transformation undrained shear strength s_u is very low and equals the critical state undrained shear strength $s_{u,c}$; b) for intermediate densities, a long plateau is observed in the stress-strain plot where s_u can be measured, but it may be considerably lower than $s_{u,c}$; and c) for dense sands a very high s_{uc} is observed and the plateau, if any, has a very short strain span (see Figure 2).

2.2 *Theoretical estimation of undrained shear strength*

For drained shear of dilating sands, it is a common practice to compute the peak friction angle ϕ as the sum of the critical state friction angle ϕ_c and a dilatancy term ψ which in turn depends on void ratio e and effective mean pressure p, or

$$\phi = \phi_c + \psi\left(p, e\right) \tag{1}$$

Critical state is reached when dilatancy vanishes, a fact that can be put by the implicit relationship

$$\psi\left(p, e_c\right) = 0 \tag{2}$$

The most widely used expression in the form of eqn. (1) is that of Bolton (1986) which can be put in the form

$$\phi = \phi_c + \Delta\phi \cdot D_r \cdot \left(Q - \ln\left(\frac{p}{p_{ref}}\right)\right) - R \tag{3}$$

where $\Delta\phi$, R, and Q are fit parameters and p_{ref} is a reference pressure.
Sfriso (2009) elaborated on the meaning of parameter Q as a crushing strength measure, suggested that it should depend on void ratio and presented the modified expression

$$\phi = \phi_c - \Delta\phi \cdot D_r \cdot \ln\left(\frac{p}{e^{-2.5} p_r\, p_{ref}}\right) - R \tag{4}$$

where p_r is a parameter. It has been demonstrated that eqn. (4) better predicts the critical state void ratio for high pressure drained shear and the critical state undrained shear strength for high pressure undrained shear (Sfriso 2009, 2010, Sfriso and Weber 2010).

2.3 *In situ estimations of shear strength*

For field conditions and earthquake loading, s_u depends both on the initial void ratio e_0 and overburden stress p_0 acting at the beginning of the quake. To estimate the in-situ void ratio, the SPT is used as the standard procedure in Argentina. A trigger is always used to release the hammer, yielding an efficiency factor of 90% employed in the energy corrections used to compute the normalizes SPT value $(N_1)_{60}$ (Leoni et al 2008).
For clean sands, correlations are frequently used to estimate various parameters after SPT results. For the undrained shear strength s_u (at the phase transformation line) Núñez has been employing the following expressions (Núñez 2010b)

$$s_u = \frac{D_r}{4.35 - 3.9 D_r} p_0' \tag{5}$$

or, in terms of the SPT blowcount

$$s_u = \frac{\left(N_1\right)_{60}}{100 - 0.8\left(N_1\right)_{60}} p_0' \tag{6}$$

3 THE POTRERILLOS CASE STUDY

3.1 *Estimation of shear strength*

The silt and sand layers / pockets laying under the foundations of Potrerillos dam have a wide range of fines content, thickness and densities, but can be roughly split into cohesive and non-cohesive materials.
Cohesive materials were of no concern from the point of view of cyclic liquefaction but could form a weak layer, thus reducing the sliding stability of the dam. the undrained static shear strength was estimated using the common expression

$$s_u = 0.22 \cdot p_0' \tag{7}$$

For non-plastic silts and sands, the focus was directed to the estimation of their undrained shear strength. The limited in-situ values showed that $(N_1)_{60}$ ranged from 20 to 100 for the non-plastic lenses, with an absolute minimum of 18 (Barchiesi et al 2006). A worst case value $(N_1)_{60}$ = 16 was adopted (Núñez 2010a, 2012) and introduced in eqn. (6) to yield

$$s_u = \frac{16}{100 - 0.8 \cdot 16} p_0' = 0.183 \cdot p_0' \tag{8}$$

Comparison of eqn. (7) and eqn. (8) shows that the non-plastic materials are weaker and therefore critical for the stability of the dam. It must be noted that, in both cases, the vertical effective stress p_0 must include the effect of the self-weight of the dam. The favourable effect of the densification of the loose silt layers under the dam dead loading was disregarded for simplicity.

3.2 *Sliding stability*

Focusing only in the post-earthquake sliding stability of the dam, simple rigid-body computations can be performed, assuming thath the dam will slide along an horizontal plane and that sliding resistance is provided only by the shear strength at the plane of potential sliding and passive earth pressure of the foundation material at the downstream toe of the dam.
The self-weight of the dam W_d, the weight of the foundation soils above the potential sliding plane W_f and the vertical component of the water loading U_v were computed to yield:

$$\begin{aligned} W_d &= 550\, MN/m \\ W_f &= 150\, MN/m \\ U_V &= 100\, MN/m \end{aligned} \tag{9}$$

The total shear strength at the plane of potential sliding was computed using eqn. (8) to be

$$S_u = 0.183 \cdot \left(W_d + W_f + U_v\right) = 144\, MN/m \tag{10}$$

The horizontal forces are the hydrostatic loading U_h, the active earth pressure acting on the upstream vertical face of the sliding block E_A and the passive earth pressure acting on the downstream face E_P. They were computed to be:

$$U_h = 100\, MN/m \\ E_P - E_A = 15\, MN/m \tag{11}$$

The factor of safety against block sliding is therefore

$$FS = \frac{S_u}{U_h + E_A - E_P} = \frac{144\, MN/m}{100\, MN/m - 15\, MN/m} > 1.6 \tag{12}$$

Therefore, the dam was stable without the buttress. To stress the importance of this conclusions - based on crude estimations and simple analyses - a back-calculation can be performed. Failure might occurr if the shear strength in the sliding plane were 85*MN/m*. After eqn. (10), this would imply a s_u/p ratio 0.108, which inserted in eqn. (6) would in turn mean that the dam is founded on a continuous, horizontal layer with $(N_1)_{60} = 10$.

4 NUMERICAL ANALYSIS

4.1 *Hypotheses*

A simple 2D pushover analysis of the dam was performed numerically using Plaxis. The model geometry and mesh reproduce Figure 1; material parameters were estimated based on published information only (e.g., Barchiesi et al 2006, Carmona et al 2004) and are described below.

A continuous horizontal layer 4m thick was assumed to be located 30m below the foundation level of the dam, running throughout the model. Construction stages were not simulated; the dam was activated in one drained loading step. For the pushover analy-ses, all materials were assumed drained except for the sand layer, where undrained behavior was enforced numerically. Due to its permeability and thickness, it was assumed that any cohesive layer would complete primary consolidation during the construction of the dam.

4.2 *Description of the constitutive model*

The model focused on the progressive sliding deformations of the weak layer; only this material was modelled accurately using a constitutive model developed by Sfriso for the monotonic shear of sands (Sfriso and Weber 2010).

The model is designed to simulate the behavior of sands in the whole stress and strain range of engineering interest, using state independent material parameters. The formulation is based on effective stresses, pressure dependent hyperelasticity, non-associative elasto-plasticity, an isotropic hardening law and Rowe’s stress-dilatancy theory (Sfriso and Weber 2010).

4.3 *Material parameters and loading*

Shear modulus was computed using the Hardin and Drnevich (1972) expression

$$G = c_s \cdot \frac{(c_e - e)^2}{1+e}\left(\frac{p}{100kPa}\right)^m 100kPa \tag{13}$$

where $c_s = 840$, $c_e = 2.17$, $m = 0.50$ and p is mean pressure.

A Matsuoka-Nakai yield function was employed, calibrated for triaxial compression with the friction angle obtained using eqn. (4), where $\Delta\varphi = 3°$, $\varphi_c = 31°$, $p_r = 55$, and $R = 2°$.

A major issue is the estimation of relative density. A back-calculation of eqn. (5) to yield a $s_u/p = 0.18$, as used in the rigid body calculations, would imply D_r = 46%, a somewhat high value to expect undrained yielding (Sfriso 2010). Therefore, numerical analyses were run adoptng D_r in the range 30% - 45%. The resulting stress-strain and p-q plots for undrained triaxial compression are shown in Figure 2 for the D_r = 15%, 25% and 50% and for a confining pressure of 100kPa.

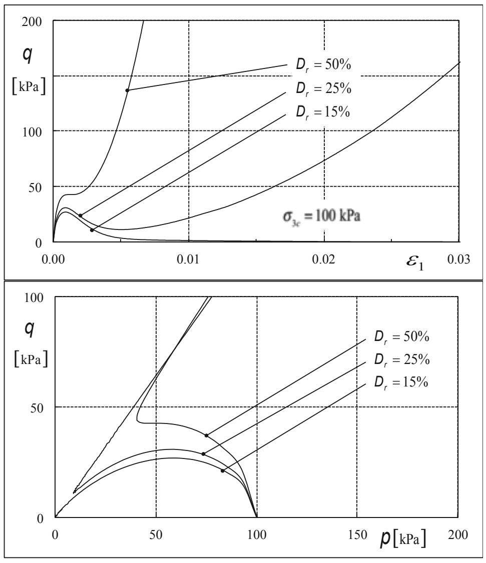

Figure 2. p-q plot of the simulated undrained compression of the weak layers underlying the foundation of Potrerillos dam.

The HSM model available in Plaxis (Brinkreve et al 2006) was employed for the remaining foundation and dam. Only the elastic parameters of the alluvium were of real interest for the analysis of progressive failure. However, due to the formulation of the HSM model, strength parameters do impact on the elastic response, and therefore all parameters required realistic values to yield a reliable prediction. The adopted values were E_{50}^{ref} = 80MPa, E_{oed}^{ref} = 90MPa, E_{ur}^{ref} = 200MPa, ν = 0.20, c = 1 kPa, $\varphi = 45°$, $\psi = 10°$, OCR = 1.0. For the definitions of these parameters see Brinkgreve et al (2006).

4.4 *Factor of safety*

The numerical computation of factor of safety is routinely performed in Plaxis using the "phi-c reduction" concept (Brinkreve et al 2006). For user defined soil models, Plaxis does not support the use of the "phi-c reduction" scheme and therefore an alternative definition must be attempted.

In this contribution, two "factors of safety" are defined: i) FS_1: the minimum factor augmenting the horizontal water load U_h and yielding non-convergence of the field equations of the boundary value problem; and ii) FS_2: the factor augmenting U_h to yield a permanent displacement of 1m at the base of the concrete wall. The vertical component U_v remained unchanged during both analyses. The results are shown in Table 1.

Table 1. Factors of safety computed for various relative densities.

Relative density	30%	35%	40%	45%
FS_1	1.39	2.45	-	-
FS_2	1.16	1.35	1.64	1.88

The large differences that can be observed between the two factors of safety is due to the fact that the constitutive model goes beyond the phase transformation plateau into the dilating phase, as shown in Figure 2. FS_1 is not informed for D_r = 40% and 45% because the failure mode was sliding at the foundation level of the dam instead of deep sliding along the sand layer.

It must be noted that the all available test results consistently show that D_r > 45% throughout the dam foundation.

4.5 *Progressive failure*

The risk of progressive failure was estimated for D_r = 30% and D_r = 35%. It was assumed that the indicator of progressive failure is the proportion of the sand layer yielding beyond the phase transformation plateau (where s_u is defined) and reaching the critical state (where $s_{u,c}$ is defined).
Using this concept, if $s_{u,c}$ is only reached at few points within the sand layer, the soil mass is yet far from it's critical state and the risk of progressive failure is nil, while if $s_{u,c}$ is reached at a significant proportion of the dam base, progressive failure requires a detailed analysis. The ratio of the length at the critical state L_{cr} compared to the total length of the foundation (440m) is shown in Table 2.

Table 2. Factors of safety computed for various relative densities.

Relative density	30%	35%
L_{cr} [m]	55	25
L_{cr} / length of base	14%	6%

The analysis could not be run for D_r = 40% and 45% because the failure modes do not include sliding through the undrained sand layer. The results shown in Table 2 demonstrate that the risk of progressive failure is negligible and that a detailed analysis is not necessary.

5 SUMMARY AND CONCLUSIONS

The case study of the port-earthquake behavior of Potrerillos CFRD dam has been presented. The dam and the existence of weak materials embedded in the alluvial foundation soils were briefly described. Those weak materials were not encountered during the design investigation phase but found during the construction of the diaphragm wall and piezometers. Client engineers feared that the dam would not be stable under earthquake loading and ordered the construction of a downstream buttress embankment to improve it's sliding stability. The contractors obeyed but started a claim which escalated to the Supreme Court of Justice in Mendoza.
Núñez was appointed as one of the expert witnesses by the Court. He evaluated the sliding stability of the dam using simple rigid-body procedures and demonstrated that the dam was stable without the buttress, ending the judicial discussion. Numerical computations performed after the incident proved that progressive failure - a subject that could not be addressed using simple assumptions - did not change the conclusions of the analyses.
Three well known lessons were confirmed after this incident: i) at design stage, the use of the best available geotechnical investigation techniques is crucial for the success of large projects; ii) where weak non-plastic materials are found, all available efforts must be made to get a full, comprehensive and reliable characterization of their mechanical properties; and iii) simple yet sound procedures for analysis suffice to support an expert opinion; more elaborate approaches can be employed to enrich the knowledge, but do not substitute the need for a basic comprehension of the problem.

6 REFERENCES

Barchiesi, A., Toso, J., Lara, A, and Carrere, A. 2006. La fundación y el diseño sismorresistente de la presa Potrerillos. IV Argentinan Conference on Dams, Posadas, Argentina, T_A6.
Bolton, M. 1986. *The strength and dilatancy of sands*. Geotechnique, Vol. 36, No. 1, pp. 65-78.
Brinkgreve, R., Broere, W., Waterman, D. 2006. Plaxis Users Manual. ISBN-10: 90-76016-02-X
Casagrande, A. 1936. Characteristics of cohesionless soils affecting the stability of slopes and earth fills. Journal Boston Society Civil Engineers, Vol. 23, No. 1, pp. 13-32.
Casagrande, A. 1975. Liquefaction and cyclic deformation of sands - a critical review. V Pan American Conference Soil Mechanics Foundation Engineering, Buenos Aires, Argentina, Vol. 5, pp. 1-35.
Castro, G. 1975. Liquefaction and cyclic mobility of saturated sands. Journal Geotechnical Engineering, ASCE, Vol. 101, No. GT6, pp. 551-569 .
Castro, G., Poulos, S. 1977. Factors a4ecting liquefaction and cyclic mobility. Journal Geotechnical Engineering, ASCE, Vol. 103, No. GT6, pp. 501-516.
Carmona, J., Carmona Palau, J., Palau, R. 2004. The evaluation of the permanent seismic crest displacement of the CFGD Potrerillos Dam in Argentine by means of a tridimensional limit state analysis. XIII World Conference on Earthquake Engineering, Vancouver, Canada. Paper 487. CD-ROM.
Hardin, B., Drnevich, V. 1972. Shear modulus and damping in soils: Measurement and parameter effects. Journal of the Soil Mechanics and Foundation Division ASCE 98(SM6):603-624.
Ishihara, K. 1993. Liquefaction and flow failure during earthquakes. The 33rd Rankine Lecture. Geotechnique, Vol. 43, No. 2, pp. 349- 415.
Leoni, A., Guaraglia, D., Bielecki, A. Cánchero, J. 2008. Medición de la energia entregada por el impacto del pisón, en la ejecución de los ensayos de SPT que se ejecutan en la República Argentina. XIX CAMSIG, La Plata, Argentina.
Núñez, E. 1991. Propiedades mecánicas de materiales granulares incoherentes. Academia Nacional Ciencias Exactas, Físicas y Naturales, Buenos Aires, Argentina, Vol. 46, pp. 71-89.
Núñez, E. 2010. Dos grandes Presas CFRD en Zonas Sísmicas. XX CAMSIG , Mendoza , Argentina, CD-ROM.
Núñez, E. 2010. Resistencia no drenada y licuación en arenas. XX CAMSIG , Mendoza , Argentina, CD-ROM.
Núñez, E. 2012. Presa Potrerillos, Diez años de discusión. XXI CAMSIG , Rosario, Argentina, CD-ROM.
Poulos, S. 1981. The steady state of deformation. Journal Geotechincal Engineering Division, ASCE, Vol. 107, No. 5, pp. 553-562.
Poulos, S., Castro, G., France, T. (1985). Liquefaction evaluation procedure. Journal Geotechnical Engineering Division ASCE,111(6).
Sfriso, A. 2009. The friction angle and dilatancy of sands. XVII ICSMGE, Alejandría, Egipto, pp. 433-435.
Sfriso, A., and Weber, G. 2010. Formulation and validation of a constitutive model for sands in monotonic shear. Acta Geotechnica, 10.1007/s11440-010-0127-y.
Sfriso, A. 2010. Estimación de la resistencia al corte no drenada de arenas. XX CAMSIG, Mendoza, Argentina, CD-ROM.
Verdugo, R., and Ishihara, K. 1996. The steady state of sandy soils. Soils and Foundations, Vol. 36, No. 2, pp :81-91.
Zabala, F., Oldecop, L., Rodari, R. Zandarín, M. 2006. Presas de materiales sueltos en zonas sísmicas. IV Argentinan Conference on Dams, Posadas, Argentina, T_A2.