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Geotechnical aspects of the closure of a tailings dam

Aspects géotechniques de la clôture d'un barrage des résidus miniers

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ABSTRACT

The paper describes the geotechnical aspects of the closure of a tailings dam in the North of Spain. The dam has an initial earth embankment, 19 m high, constructed in 1983, and raised upstream in two further steps (1987 and 1993) to a final height of 27 m. It has been closed in 2003. The foundation ground is formed by Cretaceous sandstone and marl, covered by colluvial and residual soils, and also by old mine dumps.

In the paper, the geotechnical investigations made for the initial project, for the two raisings and for the closure are described. They include borings, dynamic and static penetration and CPTU tests. The stability analyses are also presented, as well as the evolution of soil strength during the elapsed 20 years. The behaviour of the dam has been satisfactory.

RÉSUMÉ

La communication décrit les aspects géotechniques de la clôture d'un barrage de résidus miniers au Nord de l'Espagne. Le barrage est composé d'une digue initiale 19 m haut, construit en 1983, avec deux accroissements (1987 et 1993) vers l'amont, jusqu'une hauteur finale de 27 mètres. Il a été fermé en 2003. Le terrain de fondation est composé par des grès et des marnes du Crétacée, et des sédiments quaternaires et des stériles de mine anciennes.

On décrit les reconnaissances géotechniques pour le projet initial, pour les deux accroissements et pour la clôture. On a fait des forages, des pénétromètres statiques et dynamiques et CPTU. Les analyses de stabilité sont aussi présentées, en tenant compte de l'évolution de la résistance du sol avec le temps de consolidation. Le comportement du barrage a été satisfaisant.

1 INTRODUCTION

Tailings dams are a very peculiar kind of structure. The design must be active along the whole life of the structure: initial construction, working period and, finally, closure. This paper describes the geotechnical aspects of a particular case.

2 DESCRIPTION

The dam is a part of a zinc mine in Cantabria (Spain), which is being closed after two centuries of works. The dam was constructed across a wide valley, with gentle slopes about 4(H):1(V). The valley discharges to a pond, which in turn goes to a nearby river.

The main dam (Figure 1) is about 300 m long. The elevation of the foundation lowest point is +83. The final elevation of the crest is +110, i.e., with a maximum height of 27 m. Its construction started in 1983, its height was increased in 1987 and 1993, and it was finally abandoned in 2003.

The bedrock is formed by cretaceous limestone, sandstone and marl, with a weathered thickness of 2 to 5 m. However, the left part of the valley was occupied by old mine refuse from open pit excavations in the period 1860-1920. These deposits form three horizontal platforms, at elevations +83, +91 and +103. As a result, the valley axis was displaced about 150 m to the right and raised to +83 from its original elevation of +60.

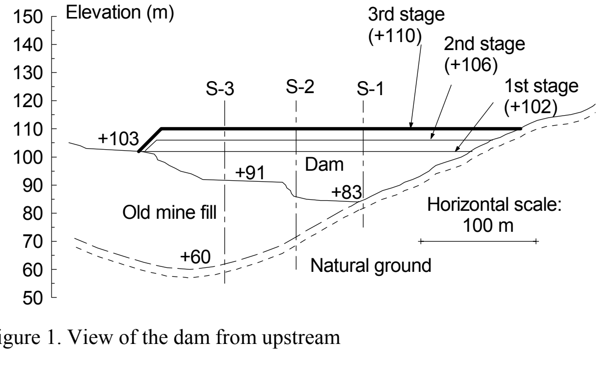

Figure 1. View of the dam from upstream

The profiles shown in Figure 1 are representative of the three typical dam conditions. S-1: maximum dam height on natural ground; S-3: maximum thickness of the mine fill; S-2: an intermediate situation, with dam height near the maximum, and a significant thickness of mine deposits. This latter S-2 section was considered critical for most aspects.

The initial dam (Figure 2) was constructed in 1983 until elev. +102, using natural colluvial soil from the nearby areas, compacted at maximum Proctor density. The slopes were 3(H):2(V). A toe drain was installed downstream in the central portion of the dam to avoid water flow at the downstream slope.

The tailings were deposited from the dam crest. A sand beach 50 m wide was formed immediately behind the upstream slope, on which light equipment could circulate. The toe drain only gave some discharge in the very initial months of dam operation. The downstream slope was always dry.

In 1987 the dam was raised upstream to elev. +106, also using compacted natural soil. A layer of gravel was extended on the tailings adjacent to the dam in order to provide a support as uniform as possible for the new dam extension. As an additional safety measure, a berm was added at the downstream toe in the

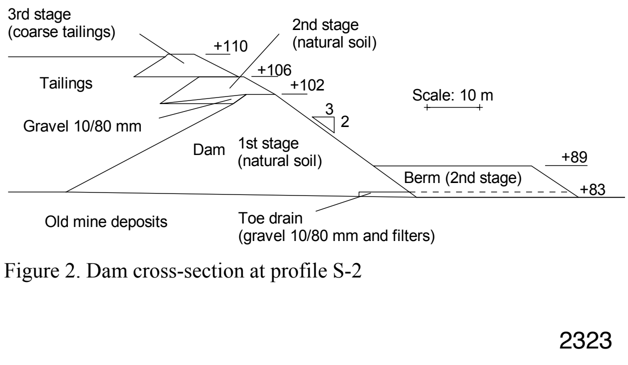

Figure 2. Dam cross-section at profile S-2

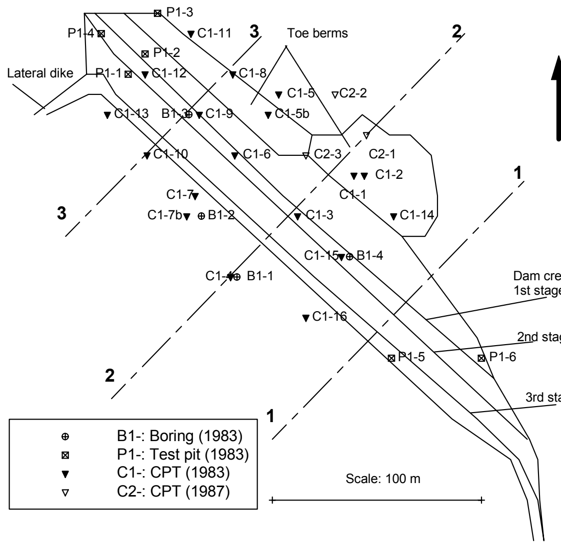

Figure 3. Site investigations for design, stages 1 and 2 (1983-87)

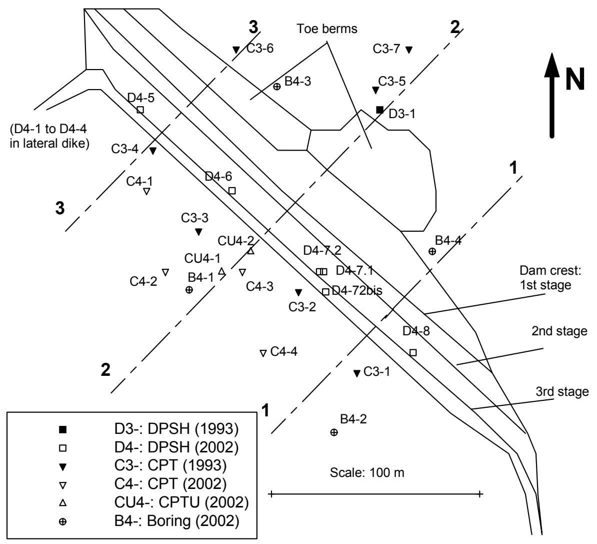

Figure 4. Site investigations for stage 3 and closure (1993-2002)

central and left portions, where the dam was founded on old mine deposits. In the central part (section S-2) it was 6 m high and 30 m wide, and 3 m high and 12 m wide in section S-1 (see cross-section in Figure 2 and plan view in Figures 3 and 4.

A lateral dike, 2 m high, at right angle with the main dam, was constructed at elev. +103 in the left margin to reduce land occupation, using the coarser fraction of the tailings.

In 1993 the dam and the lateral dike were heightened again to elev. +110. Coarse tailings were used, given the good behaviour shown in the previous stage. An average slope 2(H):1(V) was used for the upper part.

3 GROUND CONDITIONS

3.1 *Site investigations*

Given the nature of the foundation ground, particularly the granular layers of the old mine deposits, the characterization of soil strength was based mainly on static penetration tests. Some exploration borings and test pits were made for identification of soil types. There was a campaign for the initial design (1983) and additional investigations for each of the rising stages (1987 and 1993), and also for the closure (2002). Figure 3 shows the layout of exploration points for the first two campaigns and Figure 4 for the last two ones.

The initial campaign consisted in 18 static cone penetration tests (CPT), 4 rotary borings with 23 undisturbed samples and 28 SPT tests, and 6 test pits for identification of materials for the dam. The 1987 study was focused to the characterization of the downstream zone for fixing the toe berm dimensions.

In the 1993 campaign, besides a further investigation of the downstream area (three new CPT and one dynamic penetration test), the aim was the evaluation of the strength of the tailings, which had been sedimented for ten years. Four CPT tests were done on the tailings, upstream of the dam crest. This aspect was further analysed in the study of 2002, for the closure of the dam. CPT and CPTU tests were used, together with dynamic penetration (DPSH) for the dam itself.

3.2 *Old mine deposits*

The nature of the old mine deposits varies from clayey silt to silty clay, with a sand content between 15% and 35%, and PI of 10-30%. There is a desiccated crust, 2-3 m thick. Below it, the undrained shear strength of the clayey levels from unconfined compression tests is in the range 20-40 kPa. These values were considered on the low side due to partial drainage. Triaxial C-U tests gave values of s_u in the range 40-70 kPa, and a drained shear strength defined by a cohesion intercept of 6 to 17 kPa and a friction angle of 35º-38º.

The SPT resistance increases with depth, between N=2-12 in the shallow part, and N=20 at a depth of 25 m. The CPT tests show a tip resistance, q_c, of 2-3 MPa in the desiccated crust, decreasing to 1.0-1.5 MPa at a depth of 5-6 m, and increasing again up to 2 MPa at a depth of 20 m (Figure 5). These values correspond to undrained shear strength of 50-70 kPa. A harder layer is systematically found at a depth of 10 m.

The oedometric tests gave C_c=0.25-0.35, C_s=0.03-0.04 and a coefficient of consolidation, c_v, of (1.5-6.0)×10^{-3} cm²/s. This value was confirmed by dissipation (holding) tests in piezocones (CPTU) in the last campaign. The method of Teh and Houlsby (1991) with I_r=50 and a correction factor of 0.15 (Robertson et al., 1992), led to values of (0.85-7.5)×10^{-3} cm²/s. These values are relatively high, indicating a rapid dissipation of excess pore pressures in the foundation ground.

3.3 *Tailings*

CPT and CPTU tests were used in the 1993 and 2002 campaigns for characterisation of the tailings. The friction ratio was around 0.5% (always below 1%), indicating a granular nature. This was confirmed by the CPTU tests, in which no

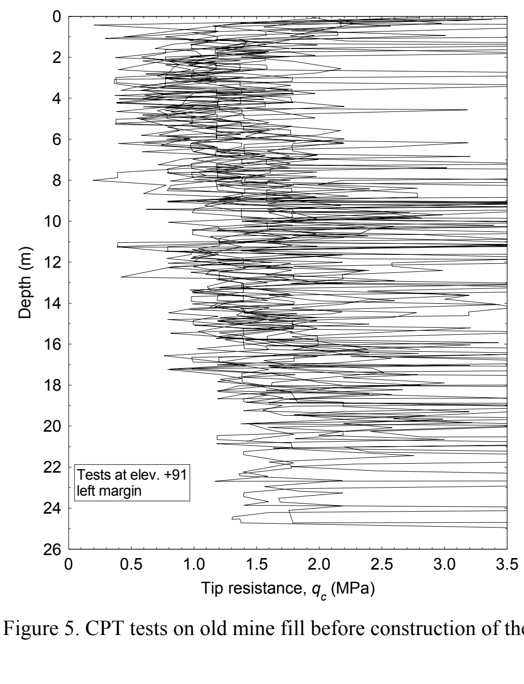

Figure 5. CPT tests on old mine fill before construction of the dam

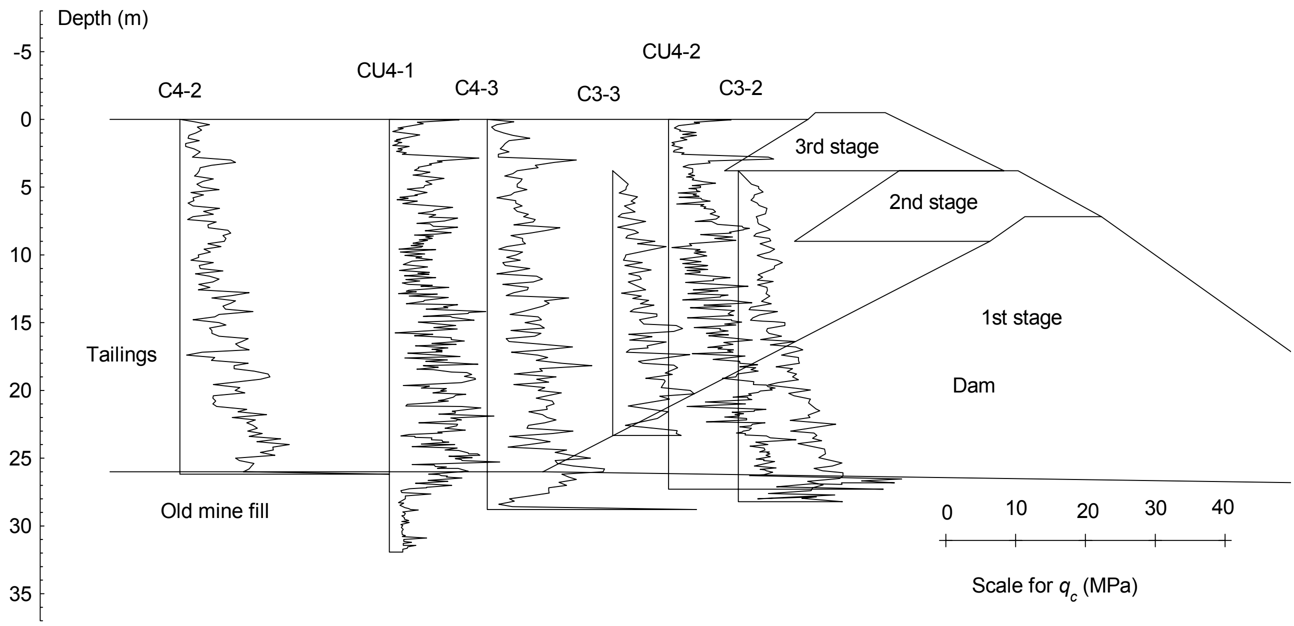

Figure 6. Tip resistance in CPT in tailings. Section 2-2.

excess pore pressure was registered during the driving. The depth of the water level was 20 to 26 m at a distance of 10 to 30 m from the dam.

In Figure 6 the CPT tests made in section 2-2 are shown. A clearly linear increase with depth is found, typical of a sandy material, with values of 3-5 MPa at 10 m and 6-10 MPa at 20 m. This implies a relative density of 40-50% and a friction angle of 30º-36º, based on usual correlations.

Some of the tests in Figure 6 (CU4-1, C3-2 and C4-3) also indicate clearly the transition to the old mine fill of the foundation. The strength of this material below the crust is clearly lower than of the tailings, about 2-4 MPa.

4 DESIGN CONSIDERATIONS

4.1 *Stability analyses*

Based on the above considerations, for the old mine fill an undrained shear strength s_u, of 50 kPa, was adopted, together with c=6 kPa and ϕ=35º. The same drained parameters were taken for the dam body.

The rate of rising of the dam and tailings was slow, about 2-3 meters per year. Given the coefficient of consolidation obtained for the less pervious layers (10^{-3}-10^{-2} cm²/s), this was considered as a drained process.

An exception was made for the first 8 m of dam rising (before reaching the platform at elevation +91). As can be seen in Figure 1, the lower part of the valley is relatively narrow, and hence the plan area of the reservoir for this stage is small. Therefore, a rapid rate of increase of dam height was anticipated

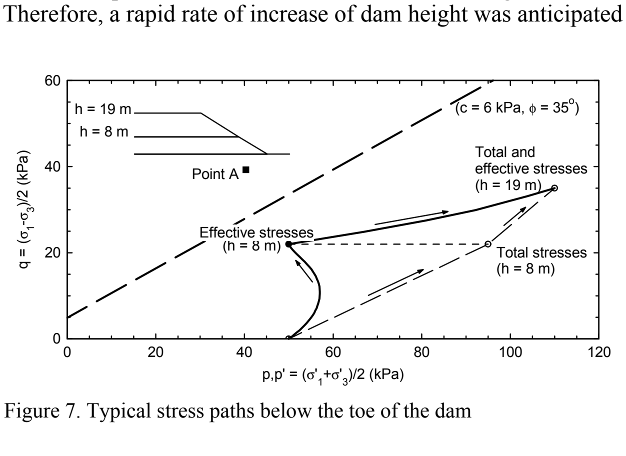

Figure 7. Typical stress paths below the toe of the dam

(in fact, the first 8 m were built up in less than one year). That led to the assumption of undrained loading for this part of the process. Figure 7 shows a typical stress path for a point below the dam toe. As the dam is heightened upstream, the influence of the first meters is greater, and so the short term conditions for a height of 8 m are critical. Further rising to 19 m produces smaller stress increments at the toe. The plotted effective stress path represents the combination of these stress increments and gradual consolidation during this phase. This is valid only if the initial dam is constructed in horizontal layers. If the downstream method were used, then the toe would be gradually displaced downstream, being always the most loaded area.

On these bases, the stability analyses were run using the modified Bishop method. The obtained safety factor was 1.74 for the initial undrained phase (8 m), and 1.50 for the drained situation for a 19 m dam.

The influence of the 2nd and 3rd stages was small, reducing the safety factor to 1.45. However, as an additional safety measure, a stabilising berm was established at the toe of the dam. Its influence was not considered in the analysis.

4.2 *Evolution of soil strength*

Some of the site investigations of the 1993 campaign were intentionally performed in the same location of some old tests, in order to study the evolution of the soil strength due to the consolidation under the tailings weight.

Table 1 shows the relevant points. For each point, the two tests (before and after) are identified (refer to Figures 3 and 4), together with the initial and final elevation of the ground surface. For each test, the values of q_c are given by its minimum and average. The gain in tip resistance is normalised by the increase in overburden pressure due to tailings. If the soil was soft clay, this ratio had to be of the order of 2.0 to 4.0, whilst for clean sand it must be in the range 10-30. The values shown in the last column of Table 1 corresponding to the minimum resistances fall into the range for clays, and the average values are closer to the range for sands. This is consistent with the type of soil as described above.

Figure 8 shows two examples of the above, corresponding to 4th and 6th of cases included in Table 1. The difference of behaviour is noticeable.

Table 1. Evolution of strength of old mine deposits 1983-1993

Point	Elev. (m)	q_c (MPa)(1)	$\Delta\sigma_v$ (MPa)(2)	$\Delta q_c/\Delta\sigma_v$
C1-10	+91.4	0.5-1.5		
C3-4	+106.9	1.5-3.0	0.26	4.0 – 6.0
C1-4	+82.5	1.2-1.8		
C3-3	+106.3	> 2.0	0.41	> 2.0
C1-7b	+92.0	0.5-1.5		
C3-3	+106.3	> 2.0	0.24	> 5.6
C1-16	+83.9	0.7-1.2		
C3-1	+106.7	2.5-5.0	0.39	4.6 - 9.7
C1-16	+83.9	0.7-1.2		
C3-2	+106.8	2.8-6.0	0.39	5.4 - 12.3
C1-8	+91.4	0.5-1.5		
C3-6	+91.5	0.5-1.5	-	-
C2-1	+83.0	1.0-1.2		
C3-5	+82.0	0.7-1.0	-	-

(1) minimum-average (2) Unit weight of tailings 17.0 kN/m³

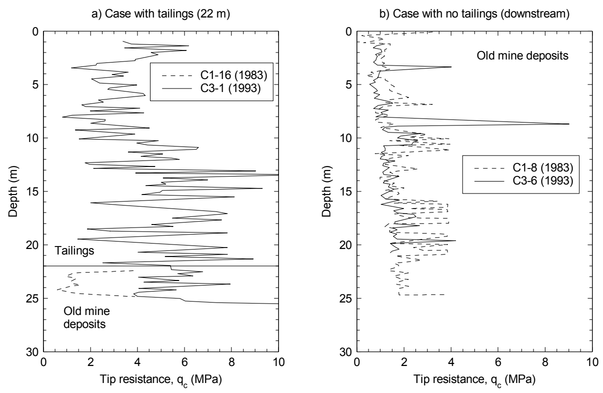

Figure 8. Example of cases with and without tailings surcharge

5 DESIGN FOR THE CLOSURE

5.1 *Stability analysis*

A new stability analysis was performed for the closure. The strength parameters for the dam used in the previous analyses were kept, not considering the gain in strength described in the foregoing section. For the tailings, c=0 and ϕ=30° were taken, based on the new CPT and CPTU tests. The water level in the tailings was consistently found at a depth of 20-22 m up to a distance of 30 m upstream from the dam, in the borings and CPTU tests.

The design situation were: a) normal, with the above conditions (obtained safety factor, F_t=2.08 for sliding affecting to the tailings and F_d=1.41 for shallow instability of the dam downstream face); b) accidental, with a rise of the water level in the tailings to +95 (F_t=2.05, F_d=1.41), and c) extreme, with general flooding of the reservoir (F_t=1.60, F_d=1.41).

5.2 *Reach of failure*

The Spanish Code for tailings dams (M.I.E. 2000) requires that regardless the dam safety factor, an analysis must be presented of the extension of the area affected by a hypothetical failure.

In this case, the downstream area extends from the toe (elev. + 83) to an artificial pond (elev. +80, at 60 m from the dam), then to a small lake (elev. +55, at 360 m) and finally to the river (elev. +22, at 1100 m).

The area is relatively open, limited laterally by higher zones, and not occupied by housing or facilities. Given these favourable conditions, the evaluation of the possible reach of a failure within this area was done with simple analyses.

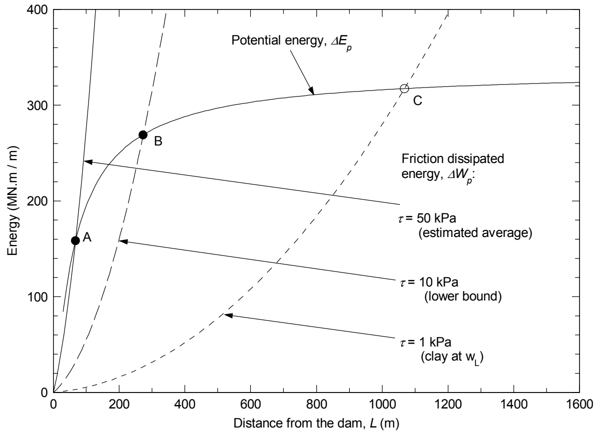

Figure 9. Energy balance, according to Hungr (1995)

First, the correlations presented by Corominas (1996) were used. For a V-shaped failure affecting a length of 100 m of the dam crest and 60 m upstream (V=25000 m³), and considering the natural ground profile as a flat area with no obstacles, the horizontal travel distance results in the range from 80 to 120 m.

A second analysis was made using the simplified method proposed by Hungr (1995), based on an energy balance of the original and final geometry of the sliding mass and the work dissipated in friction along the sliding surface. The results are shown in Figure 9. The main controlling parameter is the average shear strength of the tailings, τ. Given the sandy nature of the tailings (ϕ=30º), for an effective unit weight of 8-10 kN/m³ and an average height of the sliding tongue of 10 m, τ can be estimated about 50 kPa. Pastor et al. (2002) have used values of 20-30 kPa for finite element analyses in similar materials. So, a value of 10 kPa has been considered as a safe lower bound for the present case. In Figure 9, point A is the result for τ=50 kPa (estimated average) and point B for τ=10 kPa (lower bound). A final range is obtained for L of 100-200 m, in agreement with the first analysis. For comparison, a third curve has been added for a clay at liquid limit (τ=1 kPa), giving a much longer distance.

6 CONCLUSIONS

The described case shows the influence of geotechnical aspects on tailings dams, from initial design, along working life and closure. The evolution of foundation soil shear strength due to consolidation under the tailings has been put into evidence in the site investigations.

REFERENCES

Corominas, J. 1996. The angle of reach as a mobility index for small and large landslides. *Canadian Geotechnical Journal* 33:2,260-271.

Hungr, O. 1995. A model for the run out analysis of rapid flow slides, debris flows and avalanches. *Canadian Geotechnical Journal* 32:4,610-623.

Ministerio de Industria y Energía (M.I.E). 2000. *Instrucción Técnica Complementaria 08.02.01 "Depósitos de lodos en procesos de tratamiento de industrias extractivas",* Chapter VIII of 'Reglamento General de Normas Básicas de Seguridad Minera'. O.M. 8528/00. B.O.E. 09/05/00 [in Spanish].

Pastor, M., Quecedo, M., Fernández, J.A., Herrores, M.I., González, E. and Mira, P. 2002. Modelling tailings dams and mine waste dumps failures. *Géotechnique* 52:8,579-591.

Robertson, P.K., Sully, J.P., Woeller, D.J., Lunne, T., Powell, J.M. and Gillespie, D.J. 1992. Estimating coefficient of consolidation from piezocone tests. *Canadian Geotechnical Journal*, 29:4,539-550.

Teh, C.I. and Houlsby, G.T. 1991. An analytical study of the cone penetration test in clay. *Géotechnique* 41:1,17-34