PROGRESSIVE FAILURE OF A TAILINGS DAM: PRE-FAILURE PHASE

Joaquín Martí, Francisco Riera and Francisco Martínez

<principia@principia.es

Principia S.A.

Velázquez 94, 28006 Madrid, Spain

Abstract

The paper deals with some specific modelling aspects of the forensic investigations conducted in relation with the failure of a tailings dam. The specific aspects covered here involve the gradual construction of the dam and filling of the tailings pond; the very slow consolidation of its thick, low permeability foundation; and the process of progressive failure allowed by the brittle behaviour of the overconsolidated foundation materials.

The problem is solved in effective stresses, coupling the structural and hydraulic phenomena. A Mohr-Coulomb elastoplastic behaviour with non-associative flow is used to represent the mechanical behaviour of the materials. Isotropic softening, different for the cohesive and frictional parts of the strength, is introduced via field variables to represent the degradation.

The construction process is modelled in discrete phases, each of which includes the gradual increasing of gravity loads. ABAQUS/Standard allowed tracing the evolving consolidation of the foundation and the progressive development of the failure. Excellent matching was observed with the timing and the somewhat unusual geometry of the failure experienced.

It should be mentioned that the onset of failure triggered liquefaction of the tailings behind the dam, a process which entails a large sudden increase in the forces applied and hence a considerable acceleration of the dam displacements. Hence, at that time, a migration to ABAQUS/Explicit was carried out; the latter is the object of a companion paper (Martí et al, 2001).

Introduction

In the case studied here, the failure of a tailings dam resulted in the uncontrolled release of several million cubic metres of tailings and tailings water. This led to the contamination of rivers and land surfaces, caused interruption of the mining operations and required adopting a series of costly mitigation and remediation measures.

Very detailed forensic investigations have been carried out, of which only a small part is summarised in this and a companion paper (Martí el al, 2001). The object of the work conducted was to investigate the failure of the facility and to determine its causes. In this task, both ABAQUS/Standard (HKS, 1998a) and ABAQUS/Explicit (HKS, 1998b) were extensively employed.

Based on the knowledge available about the materials involved and the sequence of construction, computer simulations allow reproducing the complete history of events. Hence, it becomes possible to benchmark the computer results against the wealth of information that always becomes accessible after a failure of this type. Clearly, the greater the amount of information, the smaller the possibility that an incorrect explanation might provide a good match. Also, “what if” questions can be addressed: in particular, it is of interest to know how the evolution and final outcome might have been affected by design modifications or changes in material properties.

Description of the problem

The ground cross-section is relatively simple. The upper 4 m are constituted by permeable granular deposits of alluvial origin. The underlying clays have an approximate thickness of 70 m; their very low permeability, combined with their thickness and uniformity, leads to very slow rates for the dissipation of any excess pore pressures, as generated by the construction of the dam and filling of the pond. These clays are heavily overconsolidated and rich in carbonates, two factors which ally to provide very brittle characteristics to the strength. Finally, below the clays, the base rock hosts a slightly artesian aquifer.

The main body of the dam was formed by schist rock. A low permeability clay, placed on the upstream slope, provided the necessary isolation. This isolation was completed underground with a cement-bentonite wall though the permeable alluvium.

The stored tailings can be considered a fine granular material with a high bulk density (3 Mg/m^3). In the pond they were covered by about 1-2 m of water.

The construction was gradual, proceeding at the pace demanded by the mining operations; in this case, construction had been progressing for about 20 years. For modelling purposes, the construction sequence was discretized into 5 phases (Fig. 1); within each phase, the weight of the materials added was activated linearly over time. At the time of failure, the dam was 27 m high and the level of tailings and water behind it was 26 m.

Materials and initial conditions

From a mechanical point of view, all the materials have been considered to have an elastoplastic response, with a shear strength that is related to the level of the effective compressive stress being exerted on the sheared surface. With the exception of the foundation clays, the strength developed is considered to remain unaffected by deformations. For these clays, though, straining past the peak strength decreases the available strength towards a residual value, with different rates for its cohesive and frictional components (Fig. 2).

The strain softening of the foundation clays poses numerical problems of a rather complex nature. Indeed, we are still lacking well established procedures to introduce this behaviour in a way that is objective with respect to the mesh and type of elements adopted in the calculations. In spite of this, it was decided to include the strain softening in the material model. The material parameters would need to be altered if the mesh were changed; this does not invalidate the results, but requires special care in their evaluation.

The key parameters in the clays are their consolidation and strength properties. Although ranges of variation have been used for all of them, the coefficient of consolidation is on the order 0.001 cm^2/s; peak effective cohesions are 65 kPa, decreasing to nil over strains of a few per thousand, and peak effective friction angles are 24º, decreasing to 12º over strains of a few percent. The strain levels mentioned are applicable only to the mesh used in the calculations.

Because of overconsolidation, the initial state of stress in the ground was taken as hydrostatic, with total stresses increasing with depth as implied by gravity. Initial pore water pressures corresponded to the steady state solution between the underlying, slightly artesian aquifer and the watertable, located some 4 m below the surface. Once construction starts, the stresses in all materials are allowed to evolve as demanded by the growing gravity loads and controlled by their constitutive laws, coupled to the flow of interstitial water.

Modelling approach

ABAQUS/Standard was used to analyse the coupled mechanical-hydraulic problem. The mesh used to represent the global domain of interest can be seen in Fig. 3; the various materials are colour coded in the figure. The elements used were 2D solid elements type CPE4P.

The actual history of dam heights is presented in Fig. 4, together with the equivalent history activated in the calculations; the latter employs the linear increase in gravity loads used for representing the gradual process of construction.

The upper surface of the model remains free of any loads during the analysis. The two vertical side boundaries allow the material to move freely in the direction parallel to the boundary, but impede movements in the normal direction. At the bottom of the model, all movements are considered to remain negligible. The only mechanical loading introduced is that arising from the action of gravity on the masses of the various materials.

For implementing the isotropic softening of the foundation clays, use was made of the USDFLD routine. A field variable was introduced and assigned the value of the accumulated plastic strain. Appropriate expressions were given to calculate cohesion and friction angle as a function of the field variable, since the friction angle cannot be made to depend directly on plastic strain.

All materials undergoing water flow have been assumed to follow Darcy's law. No hydraulic calculations were activated in materials where flow seemed to be of little relevance; they include the base rock below the clays, the alluvium downstream from the cement-bentonite wall, the dam body and the core and filter material. In essence, the upstream face of the dam and the cement-bentonite barrier have been taken as impermeable. This is not exactly correct, but is sufficiently realistic from the viewpoint of the forces exerted on the dam.

The initial watertable was located at the base of the alluvium. At the bottom of the clays, the aquifer maintains the water pressure. Initially, a linear distribution of water pressure exists between these two depths. Downstream from the cement-bentonite barrier, the watertable remains permanently at the base of the alluvium. Within the pond, though, pore pressures have been allowed to evolve freely. The only condition imposed is that the watertable in that area be located at the outer surface of the tailings. Pore pressures in the natural ground are also allowed to evolve freely.

No flow has generally been allowed across the vertical side boundaries of the model. They are considered to be far enough for conditions to be horizontally uniform at that location. However, the alluvium is fairly permeable and its comunications with the outside would allow its pressures to remain hydrostatic. Thus, the upstream vertical boundary of the alluvium is considered to be always in a state of hydrostatic balance with the position of the watertable above it.

The construction is followed step by step. The process of time integration is carried out along the construction phases described earlier.

Results and discussion

The application of the loads corresponding to successive phases of construction results in increases of pore pressures in the clays: these pressures, gradually but very slowly, dissipate and produce a corresponding increase of effective stresses. As an example, Fig. 5 presents the distribution of pore pressures at the end of phase 3, when the dam reaches 20 m. From the viewpoint of shear stresses, though, there is no delay; these are activated as soon as the growing dam and pond generate the loads.

Fig. 6 shows the plastic strains in year 20 for one of the cases studied. It can be seen that the failure surface is already well developed: only the area under central upstream region of the dam remains intact. The dam is very close to failure and residual strengths are the only ones that can be mobilised over a large portion of the eventual failure surface. Also, the concept of progressive failure is clearly indicated: with a small increment in dam height, the forces exerted by the tailings will complete the progressively extending failure surface, where only residual strengths remain. When this happens, the failure will be essentially planar and traverse the clays some 12-14 m below the contact with the alluvium.

It is interesting to note the role played by the two key factors which made the failure possible: the slow rate of pore pressure dissipation and the brittleness of the foundation materials. There is no space here to present sensitivity analyses, but some key ideas will be offered.

In most cases, the pore pressures induced by a construction process that takes 20 years will have sufficient time to dissipate. However, in this case, the low value of the coefficient of consolidation, together with the large uniform thickness, imply that hundreds of years must elapse before a substantial dissipation occurs. Until then, the weight of the dam does not contribute towards the sliding strength, which is consequently much smaller than its long-term value; it remains essentially unmodified from its pre-construction value.

Brittleness plays two roles. On the one hand, it allows developing a failure surface with an average mobilised strength which is well bellow its peak value. This is because, as soon as the peak strength is reached at a point, this strength decreases, shedding to other regions the loading that cannot be carried locally any longer. At the time when the last regions in a kinematically viable failure surface reach their peak strength, a large portion of the failure surface will be mobilising only residual or intermediate strengths.

The second key aspect played by brittleness is that it influences the geometry of the failure surface. Although there is no space here for very detailed explanations, traditional failure surfaces, as developed with a more ductile foundation, would be approximately shaped as circles, descending some 25 m inside the foundation clays (i.e. about 30 m below the ground surface). Here, the failure surface observed was a horizontal plane, some 12 m inside the clays. Fig. 7 shows the role played by brittleness in this respect. Brittleness, measured in this case as the plastic strain over which the cohesive part of the strength goes to residual, is plotted against the depth reached by the predicted failure surface, based on the result of many calculations. The more brittle behaviours are indeed the ones which predict failure surfaces consistent with the observations. Incidentally, all the conditions represented in this curve are consistent with the timing of the failure: the combination strength-brittleness is the precise one to trigger collapse at the right time. But only the more brittle alternatives are consistent with the observed geometry.

Finally, the first movements of the dam, triggered by the failure, caused liquefaction of the loose structure of tailings behind it. This results in the loss of any strength that the tailings might have been contributing, together with a large sudden increase (about 65%) in the horizontal forces exerted on the dam. The dam, already unstable, accelerates with over 0.1g. The characteristics of this problem are unsuitable for treatment with ABAQUS/Standard and require more space than allowed by the present paper. The migration to ABAQUS/Explicit and the development of, and results from, the post-liquefaction analyses are described in a companion paper (Martí et al, 2001).

Conclusions

A large number of analyses was carried out in order to investigate the causes of the failure experienced in a tailings pond. As a result of this work, some conclusions may be of interest beyond the specific scope of the project:

a) The strain softening behaviour, required for representing the brittleness of the foundation clays, was successfully introduced via field variables. Softening was isotropic but different ranges were used to degrade the two parameters (cohesion and friction) of a Mohr-Coulomb criterion. The results were stable, consistent and matched physical reality.

b) Additional work is needed, not just in ABAQUS but generally in the scientific community, to make strain softening calculations more robust and objective with respect to the mesh and type of elements employed. This work is likely to require theoretical advances which are currently unavailable.

c) Brittleness may play several key roles in problems like the one studied here. One is fairly obvious: the average mobilised strength at failure will be comprised between the peak and residual values. Others may be less obvious, such as the influence exerted on the geometry of the failure surface; the geometry of the present failure could never be reproduced using a more ductile model, whatever the value of the strength associated to it.

d) Generally, an excellent match of reality was achieved in this case. This includes not just the timing and geometry of the failure surface, but also data from other sources (pore pressures, inclinometer data, etc) which could not be discussed in the paper for reasons of space.

References

Martí, J., Riera, F. and San Vicente, J.L., (2001) “Progressive Failure of a Tailings Dam: Post-Failure Phase”, ABAQUS User’s Conference, Maastricht, Netherlands.

HKS - Hibbitt, Karlsson & Sorensen (1998a) ”ABAQUS/Standard User’s Manual”, version 5.8, Pawtucket, Rhode Island.

HKS - Hibbitt, Karlsson & Sorensen (1998b) “ABAQUS/Explicit User’s Manual”, version 5.8, Pawtucket, Rhode Island.

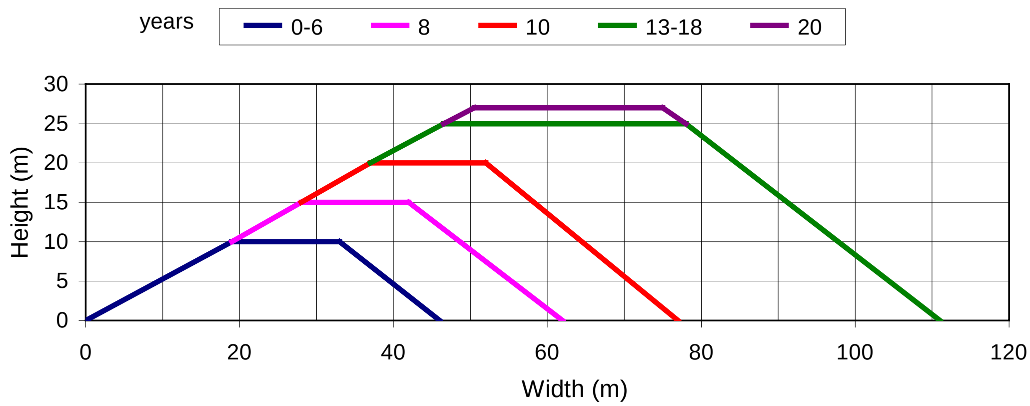

Fig. 1 Cross-sections of the dam in different construction phases

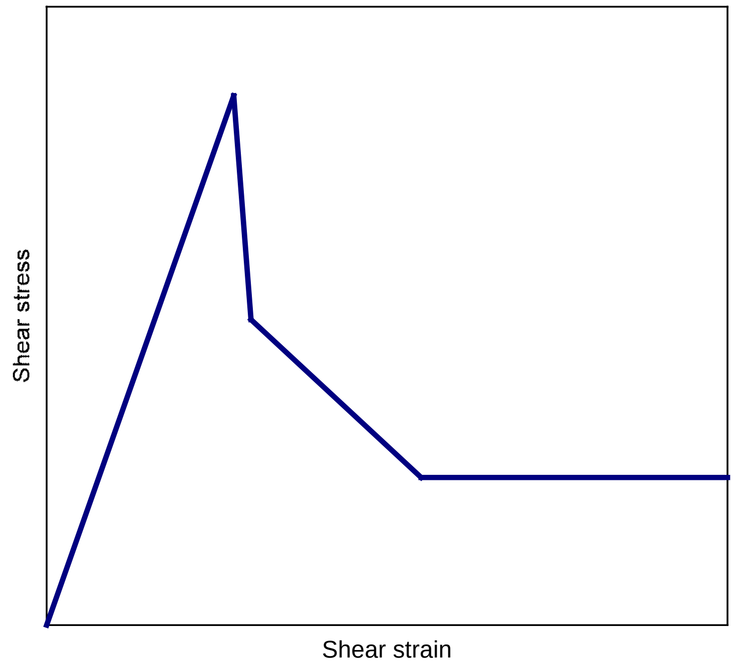

Fig. 2 Schematic stress-strain curve for the clays

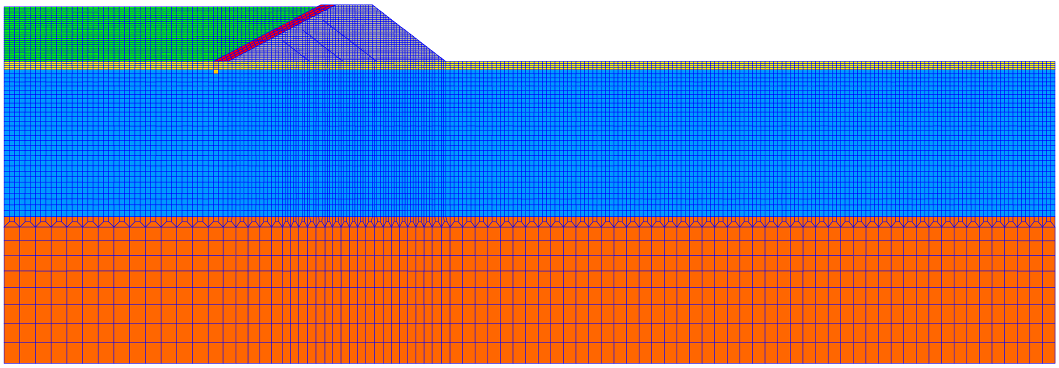

Fig. 3 Mesh used for the calculations

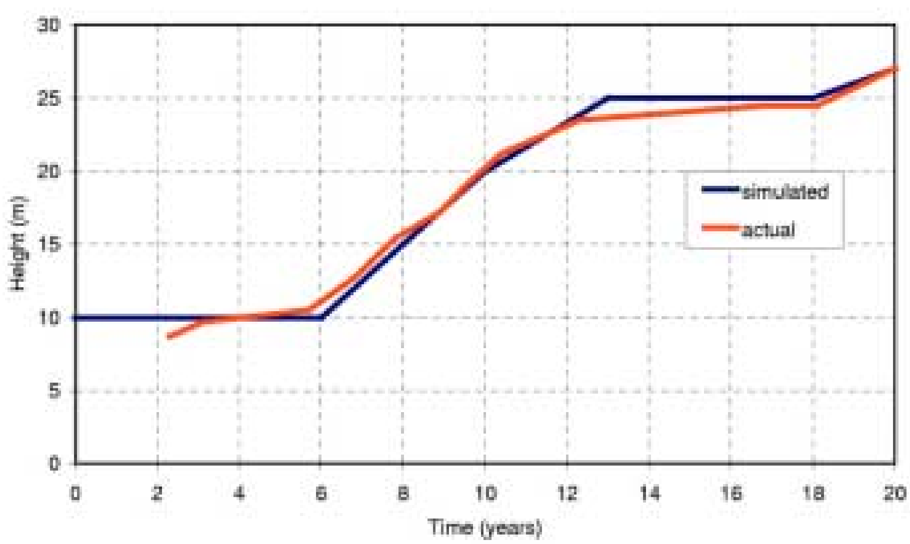

Fig. 4 Evolution of dam height for gravity purposes

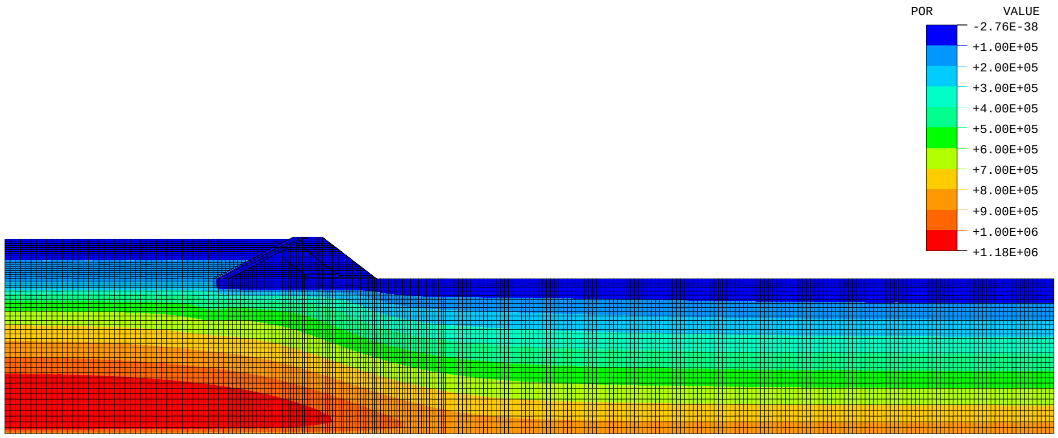

Fig. 5 Pore pressures after phase 3 (at 20 m)

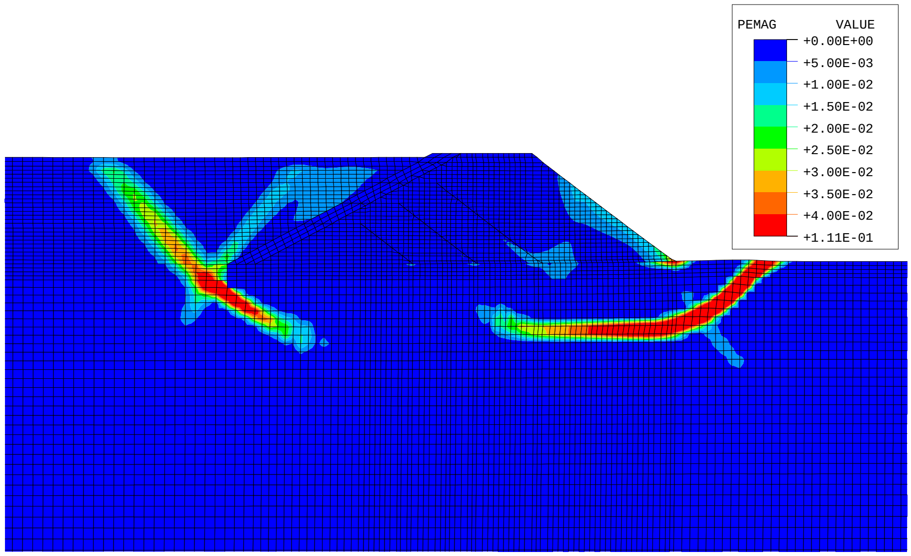

Fig. 6 Plastic strains at 27 m height

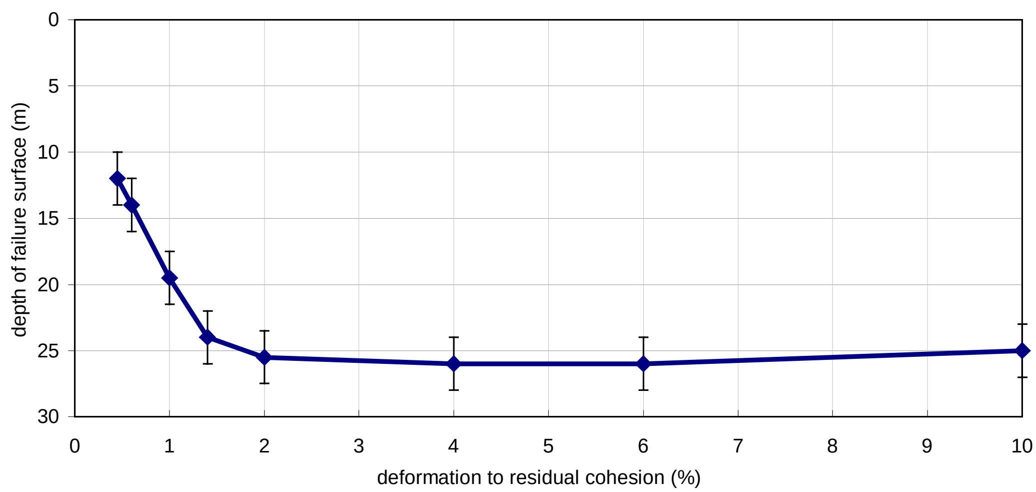

Fig. 7 Depth of failure surface as a function of brittleness