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Seismic performance based-design of large earth and tailing dams

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ABSTRACT: The actual capability of predicting the seismic performance of earth structures is limited and it is important to recognize that the real application of PBSD in professional practice is still years away. However, it is important to admit that PBSD is attractive and efforts have to be done to make it closer to practitioners. In the seismic design of tailings dams there are two main factors that control the liquefaction resistance of tailings sands: density and fines content. Accordingly, test results showing the effect of these two factors are presented. In the case of large earth dams, the use of coarse materials is common because of their good mechanical behavior. However, the evaluation of their properties is difficult due to the lack of equipment to test large particles. An alternative procedure to evaluate mechanical properties of coarse soils is presented. Additionally, the long term deformations of three large dams are analyzed and an empirical expression to estimate these deformations is proposed.

1 PERFORMANCE BASED SEISMIC DESIGN

In a broad sense performance-based seismic design (PBSD) can be understood as a design criteria which goal is the achievement of specified performance targets when the structure is subjected to a defined seismic hazard. The specified performance target could be a level of displacements, level of stresses, maximum acceleration, mobilized strength, or a limit state, among others. In this respect, the limit state design can be seen as a particular case of the PBSD, where the performance target is the accomplishment of a resisting force.

The PBSD is being strongly promoted by structural engineers, probably encouraged by the heavy financial losses resulting after recent earthquakes. This comes from the fact that the main investments in building construction are the non-structural components and contents (Astrella & Whittaker, 2004). For example, in the case of office buildings, hotels and hospital structures, the investment in structural framing is only around 18, 13 and 8%, respectively; of the total cost (Miranda & Taghavi, 2003). This clearly indicates that the fundamental objective of building code provisions to guaranty structure integrity, in terms of no collapse against strong ground motion, is definitely insufficient to be considered a successful seismic behavior to the society. Accordingly, efforts are now being conducted to reduce the financial losses associated with the non-structural components and contents throughout a design that considers specific performance structural targets, such as maximum displacements, maximum accelerations, or inter-story drift, especially in those parts where the main investments are located. In this scenario, it seems naturally that building owners and insurers, among others, should be involved in making informed decisions regarding the expected performance of the structures.

It is important to recognize that the formal use of performance-based design is definitely less widespread in geotechnical engineering than in structural engineering. Nevertheless, since the 60s the earthquake geotechnical community is applying methods of analysis for predicting permanent displacements in earth structures, which is basically a performance criterion as opposed to the classical concept of limit equilibrium (Newmark, 1965; Seed 1966, Makdisi & Seed, 1977). In addition, the design of foundations placed on granular soils is normally controlled by settlements rather than bearing capacity, which is also a performance criterion. In this sense, although it is not formally stated, the performance-based design is reasonably familiar for geotechnical engineers.

Since the middle 90s, geotechnical engineers from different countries have been promoting the development and application of PBSD, following to some extend the tendency that is observed in structure seismic design. Although some efforts have been oriented to standardize and improve the use of PBSD, as it is now conceived, it is flawed in crucial elements. Our present capability of predicting the mechanical seismic performance of earth structures inherently involves an important level of uncertainty. Starting with the prediction of the seismic event, continuing with the ability of ground characterization (such as geometry, heterogeneities, properties) and ending up with the real skills to model the dynamic soil response when it goes well beyond the linear behavior. Considering these fundamental uncertainties, it is commonly suggested that PBSD should be conducted on a probabilistic basis, indicating the probability of exceeding a certain desired performance and the confidence of this probability. Unfortunately, the probabilistic approach is not accessible for most of practitioners and it does not really improve the final outcome which is the performance prediction.

Likely the most important issues behind the PBSD are the following:

– The intention of involving stakeholders (owners, insurers and regulators) in the decisions concerning the choose of target performances for a earth structure during and after seismic events, sharing in this way the decision-making process.
– The premise that seismic performance levels can be predicted analytically, so the cost associated with each level of performance can be rationally evaluated.

In spite of the benefit of involving stakeholders in the decision-making process of choosing a specific set of performance targets, it is important to be aware of the potential problems and consequences associated with this idea. To combine appropriately both, complex technical solutions and investment decision based on risk analysis, is also risky. This necessarily introduces another source of uncertainty which could be even more important that the technical uncertainties. This statement is written just when the financial crisis is striking the whole world, and it is strongly influenced by this fact.

On the other hand, the actual capability of predicting the seismic response of earth structures is a more fundamental issue. From a scientific point of view, there is a reasonable knowledge of soil and rock mechanical behavior that has been incorporated into numerical models that are basically able to reproduce a variety of laboratory test results. However, in engineering practice, the real situation is less promising, especially when the earth structures have singular geometries that need, for example, three dimensional analysis, or when several different geotechnical materials are involved, being necessary a deep geotechnical characterization of each one.

In the case of dam engineering, additionally, practitioners have to face the geotechnical characterization of rockfill materials constituted by large size particles. Normally there is a lack of available testing apparatuses for these coarse materials, therefore geotechnical properties have to be estimated to properly desing a earth dam of this type.

In addition, there are several factors that are well recognized that affect the stress-strain relationship, but they are not included in the current models used in practice. Among these factors, it is possible to indicate that the most relevant are the following:

– Stress rotations that take place during seismic loading.
– Variation of the intermediate principal stress, σ_2.
– Seismic pore water pressure generation.
– Redistribution of pore water pressure.
– Densification due to particle rearrangement.

In this context, the actual capability of predicting the seismic performance is quite limited. Consequently, it is important to recognize that the real application of the PBSD in professional practice is years away, but it is also important to admit that this design criterion is attractive and more efforts have to be done in order to improve it.

In this paper key geotechnical properties of copper tailings materials are presented, which are necessary to consider if performance based seismic design is carried out in tailings dam projects.

For the case of earth dams constructed with coarse materials, a test procedure using parallel grain size curves is proposed to estimate the mechanical properties of the coarse fills. Additionally, the variation of the deformation modulus with time obtained from the analysis of measured settlements of three Chilean dams is presented. For the application of the performance based seismic design of earth dams with coarse fills these results are considered relevant.

2 TAILINGS DEPOSITS

2.1 *General framework*

The waste products resulting from mining operations are called tailings. Typically in copper, gold and zinc mines, the extracted ore is crushed to the size of fine sand to clay from where the minerals are recovered. In the case of copper mines it is important to mention that, as an order of magnitude, around one percent in weight corresponds to the valuable minerals that are retrieved from the milled ore. Therefore, the mining operations have to manage large quantities of tailings which are around 99 times the weight of the copper, gold or zinc production. In addition, it is necessary to mention that due to the mining processes associated to the removal of the minerals, the resulting tailings are fully saturated.

In countries with a substantial mining industry, such as Australia, Canada, Chile, Chine, Peru, Poland, South Africa, and USA, among others, the design and construction of enormous tailing disposals is a crucial necessity that has been continuously imposing new geotechnical challenges. The mining industry generates everyday millions of cubic meters of waste that has to be disposed safely and inexpensively. In Chile, for instance, there are in operation tailings dams with a height of 150 meters and reservoirs with more than one thousand million tones of slimes, and there are projects under construction that will end up with tailings dams of 220 meter in high (Valenzuela et al., 1995). This trend indicates that conventional tailings dams with large dimensions in terms of height and extension are accepted solutions for disposing mining waste products, and the assessment of their mechanical stability is one of the main concerns. Additionally, in all those regions with a high seismic activity, the stability and liquefaction resistance requirements are the main issues to be analyzed and satisfied. Furthermore, because all tailings disposals will exist well after the mining operation is ended, the seismic stability has to be ensured for a large period of time after closure of the mine.

In spite of these requirements, there are several case histories associated to the total failure of tailings dams due to the occurrence of liquefaction. In general, saturated deposits of loose cohesionless soils have shown to be susceptible to liquefaction during the occurrence of earthquakes. This phenomenon has been observed in tailing dams, hydraulic fills, as well as in natural slopes of sandy soils. It is important to bear in mind that a failure of a tailings dam has catastrophic results from economical and environmental points of view, and also can be associated with human casualties. Consequently, a failure has to be avoided by all means, but at the same time it is necessary to keep in mind that over design it is just a waste of resources.

For an appropriate design of conventional tailings dams that ensures stability at a minimum cost of construction, operation and abandon, the conditions of the storage site as geomorphology, geology and seismic activity of the area have to be considered. On the other hand, the geotechnical properties of the involved tailings play a predominant role in the selection of the most convenient design. In this paper the liquefaction strength of tailings materials is discussed at the light of new experimental data considering a wide range of fines content and densities.

2.2 *Tailings disposals*

Depending on how the tailings are processed, transported, discharged and stored, the resulting tailings disposals can be divided in two different systems: thickened tailings (or paste) disposals and conventional tailings dams. There are also others procedures, for example, filtered tailings, but they are less used due to their high cost.

In the case of thickened tailings disposal system, the main goal is to create a self-supporting tailings mass, so confining dikes can be eliminated or at least minimised. To accomplish this, the water content of the initial tailings slurry is reduced as much as possible prior to discharge by mean of high-density thickeners, resulting in a tailings deposit of a gently-sloping conical shape, with typical angles between 2 to 6 percent. The concept of thickened tailings was introduced by Robinsky in the late 60's and actually used since the beginning of the 90's (Robinsky 2000; Salvas et al., 1989).

The conventional as well as the thickened tailings disposal systems have to be designed and analysed in order to guarantee the appropriate level of stability. However, the pass experience has shown that conventional tailings disposals are susceptible to undergo seismic failure due to the occurrence of liquefaction. In the case of the thickened tailings disposals there is no sufficient information about their actual seismic behaviour due to its recent widespread application and the lack of important earthquakes in any of the existing thickened tailings facilities. Nevertheless, the following three factors can be used to argue that this type of disposal is intrinsically more stable against seismic disturbances than the conventional one. First, an increment in the density of the deposited tailings at the surface tends to occur due to its natural desiccation and the associated shrinkage of the tailings mass. Second, a quite limited saturated zone can be developed at the bottom of the disposed tailings and third, the driving shear stresses are low due to the reduced slopes reached by the surface of the disposal. In spite of this reasoning, there is no much information about the static and cyclic strength of thickened tailings and in this context new experimental evidence is presented below.

The oldest procedure for tailings disposal corresponds to the conventional tailings dams with the formation of a basin through the construction of a confining perimeter with one or several dams according to the topography of the site. The dikes or dams are usually made with the sand fraction of the tailings because, in general, it is the solution that provides the lowest cost. The sandy tailings are obtained by cycloning the natural tailings, resulting a material that classifies as sandy soil with fines contents usually in the range of 10 to 30%. The saturated finer tailings (slimes) are discharged and stored into the basin that is the disposal site. According to the construction procedure, it is possible to identify three different structures of tailings dams, so-called, upstream, downstream and center-line method of construction, which are sketched in Fig. 1.

It can be observed that the upstream method of construction requires the minimum volume of coarse tailings for dikes construction, but the geotechnical properties of the slimes are involved in the overall stability of the dikes. On the other hand, the dams constructed following the downstream method

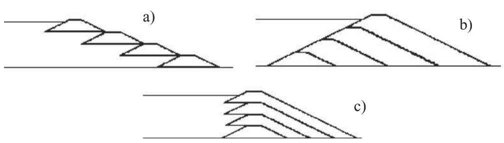

Figure 1. a) Upstream, b) downstream and c) center-line methods of construction.

produce dikes with the largest need of coarse tailings, but at the same time there are no fines tailings involved in the mechanical resistance of the resulting dam body. As can be expected, the tailings dams constructed with the upstream procedure have shown to be more vulnerable against both static and seismic failures, while the downstream dams have presented the safer behaviour. The tailings dams constructed by the center-line method have exhibited an intermediate behaviour in terms of stability. Consequently, in seismic regions the upstream dams, although attractive from an economical point of view, are avoided in favor of the downstream and center-line tailings dams. The seismic stability of these dams is basically controlled by the liquefaction resistance of the sandy tailings that constitute the dikes, which is mainly governed by the fines content and density of these sandy tailings.

Some singularities that make the conventional tailings dams different from others soil structures are the following:

- Tailings dams do not retain water, instead they contain saturated slimes, so from stability considerations, the most permeable the dam the better. Therefore, to ensure the drainage through and below the tailings dam body, it is common to build a drainage system at the bottom of the basin, in the natural ground, that passes through the base of the tailings dam. This drainage decreases the water level in the dam to a quite low position, what it is an important factor to reduce the zone with a potential occurrence of liquefaction.
- From economical considerations, only low efforts in compaction are accepted and therefore, the tailings dams tend to be in a loose to medium state of density.
- The period of construction of the embankments follows the mining operation, so the period of construction can be quite large, being possible to re adjust the original design.
- Tailings dams remain as a soil structure well after the mining operation is ended, which imply that stability must be ensured well beyond the period of the mining activity, to the so-called abandon period of the tailings dam.

These singularities have to be considered in the tailings dam design. For example, since the sand fraction of the tailings are placed with a high amount of water, a significant segregation takes place and a notable stratification is created, which has to be considered when the geotechnical properties are evaluated. However, in new projects, no information associated with the actual resulting fabric in the field is available and therefore, only reasonable estimations of properties can be done. This situation is overcame from the fact that the construction of tailings dams takes time according to the mine operation, which allows to carry out site investigations and laboratory tests to evaluate material properties and optimize the design under the light of actual data.

2.3 *Seismic failures of conventional tailings dams*

The engineering practice have registered catastrophic failures of tailings dams triggered by the dynamic forces of earthquakes, causing severe losses to the private property, important destruction of agricultural lands and in many cases loss of human lives. Most of the seismic failures of tailings dams are attributed to increase in pore water pressure and to the occurrence of liquefaction (Dobry et al., 1967; Ishihara 1984; Finn, 1980; Finn 1996). In Fig. 2 is shown one of the oldest flow failure that has been reported in a tailings dam that took place at El Teniente copper mine in Chile, following the earthquake of October 1, 1928 (Agüero, 1929). The Barahona dam of 65 m in high collapsed 3 minutes after the main shock, releasing 4 millions tons of material that flowed along the valley, killing 54 persons. The cross section of the remaining tailings after the failure is sketched in Fig. 2, where the existence of several almost horizontal terraces are observed. This configuration is attributed to the low post liquefaction strength developed by weak layers of the typical strongly horizontal stratified structure of the tailings disposed in the basin.

Later on, during the earthquake of March 28, 1965, El Cobre tailings dam located in Chile failed catastrophically and more than 2 millions tons of material flowed around 12 km in a few seconds, killing more than 200 people and destroying El Cobre town. At the time of the failure, the dam was about 33 m high and it had a downstream slope as steep as 35° to 40°, respect to the horizontal (Dobry et al., 1967). A cross section of the tailings dam before and after the failure is shown in Fig. 3, where it is possible to observe the final profile of the tailings consisting also of several terraces with 1° slope towards the valley (Dobry et al., 1967).

Another well documented seismic failure of a tailings dam took place after the earthquake of January 14, 1978, at the dikes No. 1 and 2 of Mochikoshi gold mine in Japan. The dike No. 1 collapsed around l0 seconds after the main shock, releasing 60 thousands cubic meters of slimes. The dike No. 2 failed 24 hours after the main earthquake, at the time when there was not any ground shaking and a total volume of 3 thousand

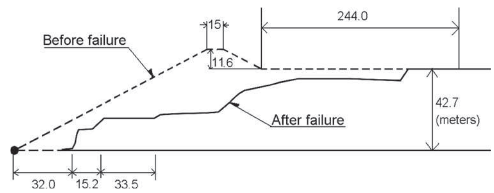

Figure 2. Failure of Barahona tailings dam, (Agüero, 1929).

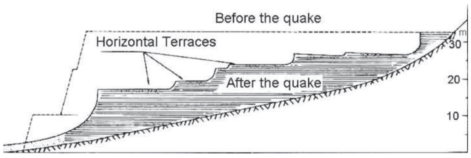

Figure 3. Failure of El Cobre tailings dam, (Dobry el al., 1967).

cubic meters of material flowed into the valley to a distance of about 240 m. The cross sections of these two dikes showing the situation before and after the failure are presented in Fig. 4 (Ishihara 1984).

Site investigation indicated that the remaining tailings in the pond adopted an average slope of 8° towards the valley. Part of the failure can be appreciated in Fig. 5.

Furthermore, during the Chilean earthquake of March 3, 1985, with a Magnitude 7.8, two tailings dams failed by liquefaction. Cerro Negro dam of 30 m in height failed and about 130 thousand tons of tailing material flowed into the valley for a distance of about 8 Km, (Castro et al., 1989). Due to this earthquake another failure occurred in Veta de Agua No. 1 dam, which at the time of the shaking had a maximum height of 15 m. According to a witness, the failure took place in the central part of the dam few seconds after the shaking had finished. The fines tailings stored in the pond moved along the El Sauce creek for about 5 km (Castro et al., 1989).

These failures, added to many others seismic failures that have occurred around the world, emphasise the importance of carrying out studies concerning the seismic response of tailings dams, with special focus on the liquefaction phenomena. However, it is also important to understand that these catastrophic failures have brought in many countries an over reaction from the community that has resulted in strong and rigid legal regulations, unnecessarily increasing the costs of tailings disposal. This situation may become more and more complicated as the tailings dams need to be larger and the construction costs grow exponentially. Therefore, studies supporting the actual liquefaction strength of tailings materials are of a paramount importance for the rational design of tailings disposals.

2.4 *Cyclic mobility and flow failure*

The term liquefaction was coined by Hazen (1920) to describe the failure of the hydraulic fill sand of Calaveras Dam on March 24th, 1918. In this failure, the up-stream toe of the under construction Calaveras dam, located near San Francisco in California, suddenly flowed moving approximately 700,000 m^3 of material for around 90 m. Apparently at the time of the failure none special disturbance was noticed, indicating that this phenomenon can occur in the absence of earthquakes.

a)

b)

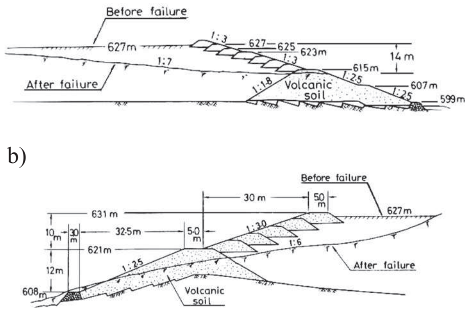

Figure 4. Failure of Mochikoshi tailings dams. (a) Dike N° 1 and (b) Dike Nª 2, (Ishihara, 1984).

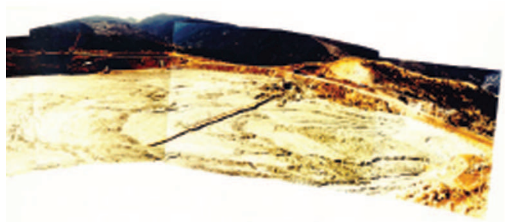

Figure 5. View of part of the Mochikoshi tailings dam failure (courtesy of Prof. K. Ishihara).

Since that failure, the term liquefaction has been used in a broad sense for describing two different phenomena that may occur in saturated cohesionless soils, which have in common a significant pore pressure build-up and large deformations of the ground. Nevertheless, to understand the actual soil behaviour it is of a great importance to distinguish between the so-called flow failure, where a sudden lost of strength takes place, and the term cyclic mobility that is essentially associated to a progressive strain softening without any lost of strength.

The term cyclic mobility was proposed by Casagrande (1975) to conceptualise the continuous development of strains that is observed during undrained cyclic loading, when the occurrence of a significant pore water pressure has been reached. Typical experimental results on loose and dense sands showing this phenomenon are presented in Fig. 6, where it is important to observe that the soil mass does not undergo any loss in strength, but important deformations are progressively developed indicating a clear degradation of stiffness.

The rate of this degradation increases after each cycle when the material is loose and it decreases when the material is dense (Ishihara 1985).

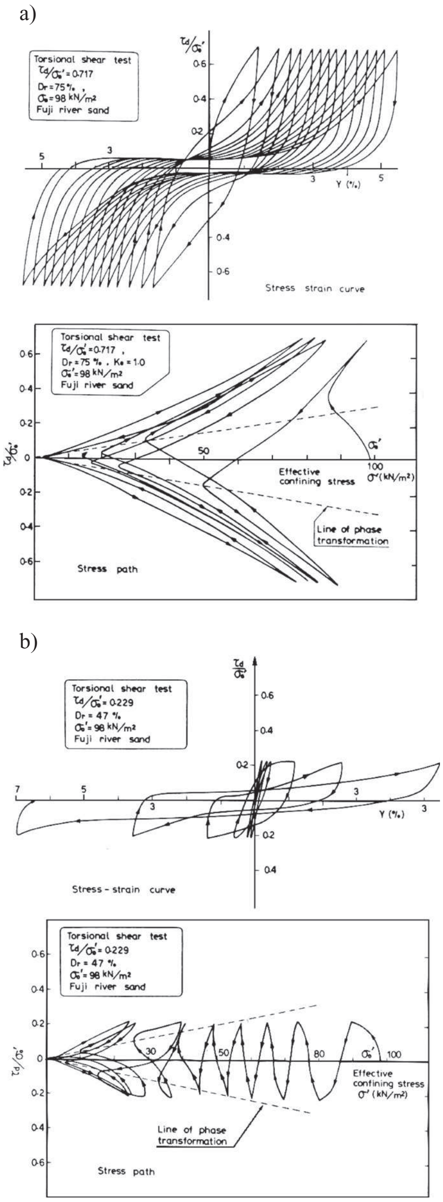

Figure 6. Typical experimental results for sands under a cyclic loading condition, (a) dense and (b) loose sand (Ishihara, 1985).

On the other hand, Casagrande (1975) proposed the term true liquefaction or flow failure for the phenomenon where a sudden loss in strength to a residual value takes place in a loose cohesionless soil (Castro, 1969). When the existing driving forces, or permanent forces, are larger than the mobilised residual strength, the failure is triggered and the soil mass deforms and flows resembling a viscous fluid. After failure has occurred, the soil mass involved in the collapse tends to reach very gentle slopes. Typically, flat angles of 1° to 8° have been observed. This failure can be triggered not only by earthquakes, but also by disturbances that are fast enough to induce an undrained response of the initially loose soil mass.

True liquefaction or flow failure is the phenomenon that has been observed in the catastrophic failures of tailings dams, causing adverse scenarios with a significant amount of soil mass flowing hundred of meters in a few minutes. Consequently, seismic analysis of tailing dams must include the evaluation of the eventual occurrence of flow failure. The condition of flow failure generates a large level of deformation where the steady state or ultimate state of the soil is reached, so the use of this concept in the evaluation of a potential flow failure is suitable.

The ultimate response of the specimen has been referred to as the steady state of deformation (Poulos, 1981). Experimental results of undrained triaxial tests performed on samples at different effective confining pressure and at the same void ratio after consolidation are shown in Fig. 7 (Verdugo, 1992; Ishihara, 1993; Verdugo et al., 1996). It can be seen that regardless the initial level of confining pressures, the same ultimate state or steady state strength is achieved.

Additionally, the effect of the stress history is shown in Fig. 8 in terms of stress strain curves on loose and dense specimens loaded monotonically and cyclically.

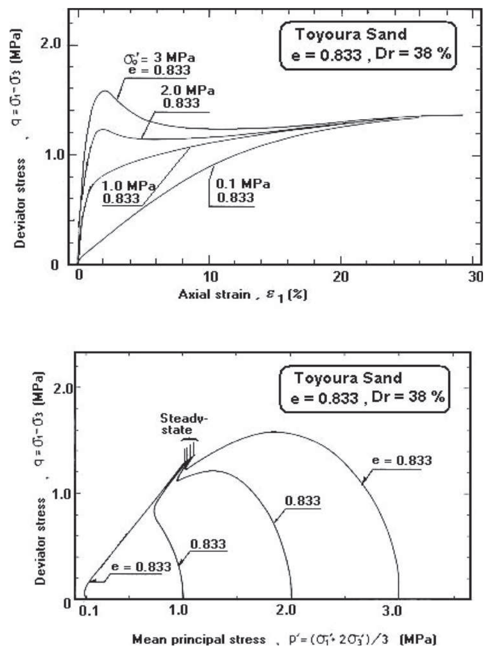

Figure 7. Steady state strength (Ishihara, 1993).

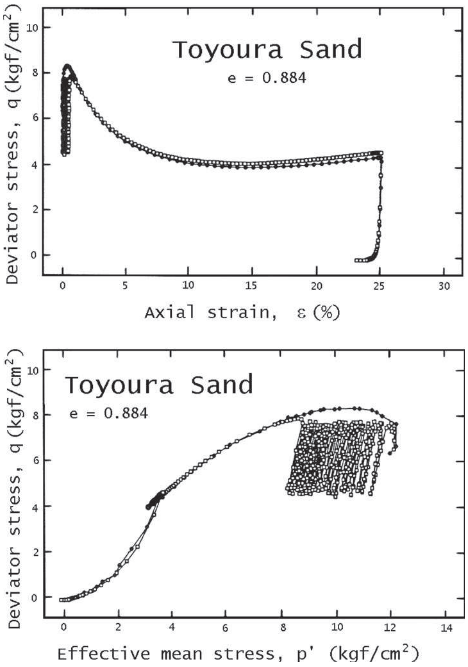

Figure 8a. Effect of stress history on loose sand (Verdugo, 1992).

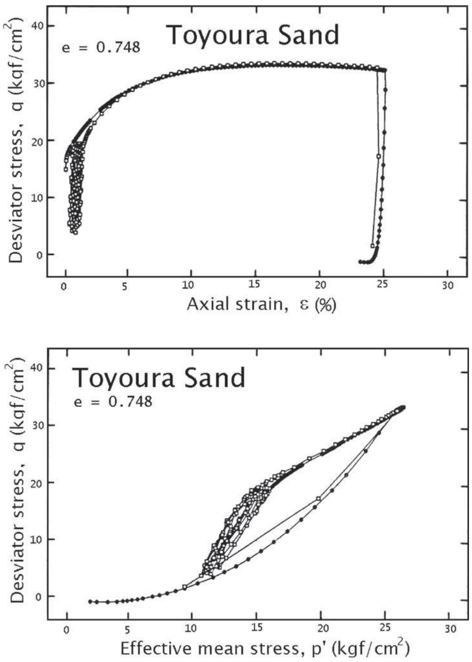

Figure 8b. Effect of stress history on dense sand (Verdugo, 1992).

As it is observed, the ultimate condition or steady state strength achieved at large deformations is independent of the previous cyclic loading indicating that the stress history does not affect the strength developed at large deformation (Verdugo, 1992). These experimental results suggest that the steady state strength is mainly a function of the void ratio. Therefore, the analysis of a flow failure basically needs to establish the level of static shear stresses and the undrained strength, which would be only dependent on the void ratio of the soil mass. The seismic action has to be seen as a trigger of the undrained strength.

To guaranty the stability of a tailing dam, the analysis of liquefaction has to be done in terms of both cyclic mobility and flow failure, and therefore an experimental program of tests covering these two phenomena has been performed. During the operation of a conventional tailing disposal there are mainly two parameters that can be modified (intentionally or incidentally), and which are directly associated with the static as well seismic strength of the dikes; density and fines content. Accordingly, the effect of these parameters in the tailings strength has been investigated.

2.5 *Maximum and minimum void ratios*

In the evaluation of the degree of compaction, or densification, of tailing sands it is important to take into account the fines content, because the maximum and minimum densities are influenced by the fine particles. Besides, the procedure to determine the maximum density is also dependent on the fines content of the sand, being recommended the use of vibration when the fines content is less than 12 to 15%, whereas for higher fines contents the use of Proctor compaction test is used. To investigate the effect of fines on the maximum and minimum densities a comprehensive series of tests was carried out by Verdugo & Viertel (2004) on copper tailings sands retrieved from the main dike of a Chilean tailings dam. The original sample was separated in two batches: clean sand free of fines and tailings with 100% of particles passing mesh # 200 (0.074 mm). The grain size curves are shown in Fig. 9 and the particle shapes can be appreciated in the photos presented in Fig. 10.

The angularity in all the particles is readily apparent, especially in the case the finer particles. The fine-grained tailings are from the dike and they classify as non-plastic soil.

Using these two batches of tailings homogeneous mixtures of tailings with 2, 5, 10, 28, 40, 50, 60 and 100% of fines contents were prepared. The original tailings sand existing in the embankment with 18% of fines was also included as another homogeneous mixture. In all these mixtures the maximum and minimum densities were evaluated and in some of them Modified Proctor tests were also carried out. Placing the soil

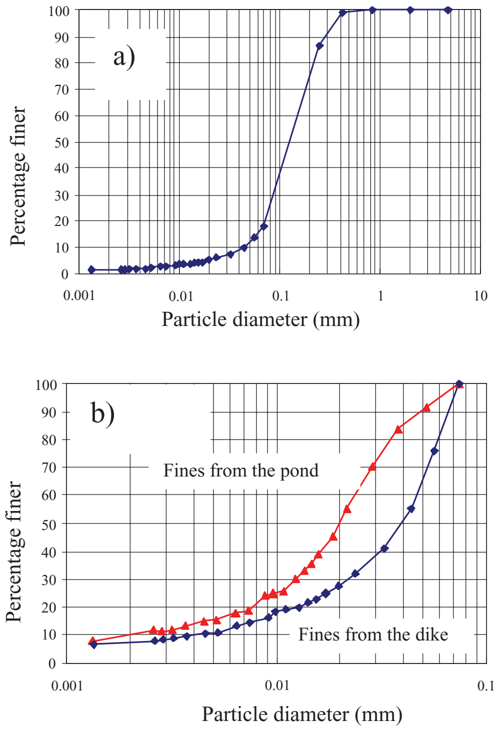

Figure 9. Grain size distribution curves. a) Sand fraction, b) fine-grained tailings.

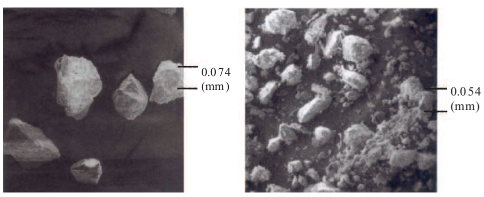

Figure 10. Particle shape of sand fraction and fines from the dike.

in a container using a paper funnel of conical shape the minimum densities were determined (Verdugo et al. 1996). The results of these tests are shown in Fig. 11 and summarized in Table No. 1.

It is interesting to observe that up to fines contents in the range of 60 to 70%, the maximum dry densities obtained by vibration are slightly higher than the values obtained by the Modified Proctor. Therefore, in the case of non-plastic fines the concept of relative density holds valid well above 15% of fines content, confirming previous results reported by Verdugo (1997).

A study related to the characteristics of maximum and minimum void ratios of natural sands reported by Cubrinovski et al. (2002) indicates the existence of a good correlation between these two indices.

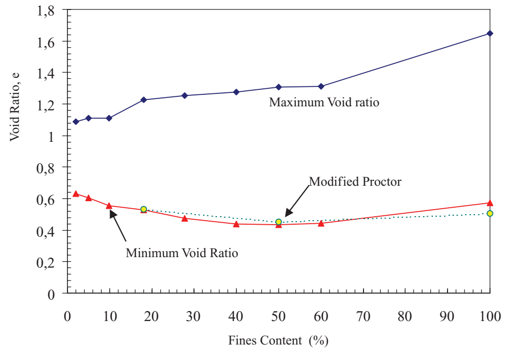

Figure 11. Maximum and minimum void ratios as function of fines content.

The maximum and minimum void ratios obtained in the present investigation are plotted in Fig. 12 together with the proposed correlations established by Cubrinovski et al., (2002). It can be seen that tailings sandy soils develop lower minimum void ratios or higher maximum void ratio than natural sands. According to the presented data, it is possible to point out that, tailings sands containing non-plastic fines achieve particular values of maximum and minimum void ratios that differ from the values reported by natural sandy soils. It is important to remark that these non-plastic fines are constituted mainly by particles with sizes in the range associated with silty soils. However, they have been created artificially by crushing fresh rocks, and therefore, in terms of behaviour they are closer to a fine sand than to a silty soil.

2.6 *Cyclic resistance ratio*

The experimental evidence indicates that mechanical properties of silty sands are largely controlled by the amount of fines and by the plasticity of these fines (Verdugo & Viertel 2004; Polito & Martin, 2001; Ni et al., 2004). For cooper tailings, Troncoso & Verdugo (1985) studied the effect of fines content on the cyclic strength of tailings sands testing reconstituted samples compacted at the same initial void ratio. Test results associated with the number of cycles required to 100% of pore water pressure build-up are shown in Fig. 13, evidencing the degradation of cyclic resistance exhibited by the tailing sands with the presence of low plastic tailing fines. It is important to mention that these results have been obtained for samples compacted at the same initial void ratio, which means that each series of soil batch has different relative density, or degree of compaction.

Ishihara et al. (1980) reported the results of a series of cyclic triaxial tests conducted on different tailings sands at different void ratios. The test results are summarized in Fig. 14, where it is possible to observe that

Table 1. Maximum and minimum dry densities.

Fines Content (%)	γ_{min} (t/m^3) (Cone paper)	γ_{max} (t/m^3) (vibration)	γ_{max} (t/m^3) (M. Proctor)	ω_{opt} (%) (M. Proctor)
2	1.298	1.659		
5	1.284	1.689		
10	1.283	1.744		
18	1.216	1.773	1.767	14.0
28	1.203	1.839		
40	1.191	1.882		
50	1.174	1.887	1.868	11.7
60	1.171	1.877		
100	1.023	1.724	1.802	14.0

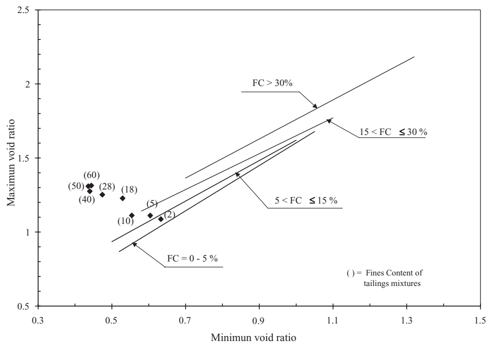

Figure 12. Maximum and minimum void ratios of tailings and natural sands.

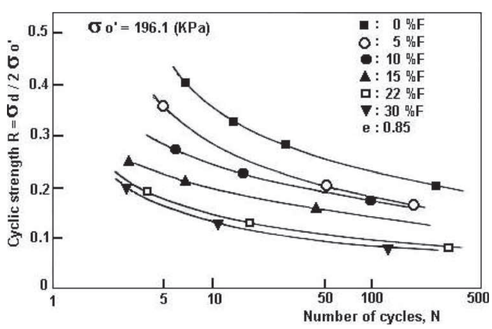

Figure 13. Effect of fines on the cyclic strength (Troncoso & Verdugo, 1985).

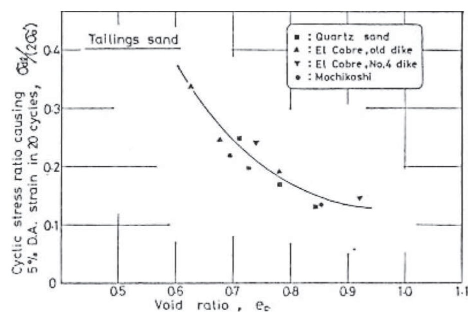

Figure 14. Variation of cyclic strength with void ratio (Ishihara et al., 1980).

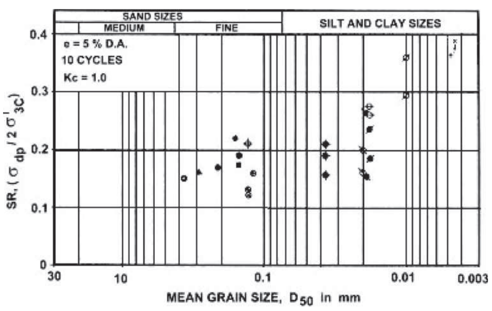

Figure 15. Variation of cyclic stress ratio with grain size in Tailings (Garga, 1984).

the cyclic strength decreases consistently with increase in the void ratio, confirming the effect of density.

Test results obtained in "undisturbed" samples of tailings with different particle sizes were compiled by Garga et al., (1984). The cyclic stress ratio to cause 5% double amplitude strain in ten cycles is shown in Fig. 15. For sand-sized materials, these data have been normalized to 50% relative density, but for the fine-grained materials no adjustment has been made. According to these data, the normalized cyclic strength of the sand-sized materials falls within a relatively narrow range, and it increases for the clayey tailings.

Using the same tailings described in section 2.5, a comprehensive series of cyclic triaxial tests on compacted samples was carried out by Verdugo & Viertel (2004), some which results are presented in Figs. 16, in terms of cyclic stress ratio and number of cycles associated with 100% of pore water pressure build-up for the tailings mixture of 18% fines content.

The general trend observed in each of these curves follows what has been reported in the literature for other sandy soils. For practical purposes, the cyclic strength associated with 20 cycles of loading, R_{20}, can be considered an appropriate value for representing the cyclic strength of each curve. Accordingly, R_{20} is plotted in Fig. 17 for each fines content and relative density.

It is observed that in the range of fines content that has been studied, the cyclic strength consistently decreases as the fines content increases, independently of the degree of densification.

Furthermore, for fines contents of 2 and 10%, the cyclic strength as a function of relative density follows a similar trend. There is a sharp increase in the cyclic strength from approximately a relative density of 50%. While for fines contents of 28%, the cyclic strength increases monotonically with the relative density. On the other hand, the mixture of 18% of fines shows an intermediate behavior with a rather pronounced increase in strength approximately from a relative density of 80%. The existence of a threshold value of relative density above which the cyclic strength increases drastically has been reported by Tatsuoka et al. (1982) for Toyoura sand samples tested on cyclic torsional simple shear tests. Considering the cyclic strength associated with a 15% of shear strain in double amplitude achieved in 20 cycles, Tatsuoka et al. (1982) reported for Toyoura sand a threshold value of relative density around 83%. This value is quite close to the threshold relative density exhibited by the tailing mixture of 18% of fines content. It is also interesting to indicate that the tailings mixtures with less fine-grained material present a cyclic strength higher than Toyoura sand in the complete range of relative density. These results are suggesting that copper tailings materials with low amount of non-plastic fines develop a high cyclic strength, probably associated with the considerable angularity and hardness of the particles.

Although it has been shown that relative density is a suitable parameter when non-plastic fines are involved, the previous results are presented again, but using just the sample void ratios, which is a straightforward representation of the effect of density on these experimental results as shown Fig. 18.

It can be seen that similar conclusions can drawn, confirming that in the wide range of non-plastic fines content that has been used, the cyclic strength consistently increases as the void ratio decreases, and as the fines content increases the cyclic strength decreases.

These results suggest that the mixtures containing non-plastic fines are always affected by the presence of these fines, regardless of how small is the amount of fines. A possible explanation is related to the actual location of the fines in a rather homogeneous mixture. It is possible to hypothesize that part of the fines will cover the larger sand particles and therefore, some of the contacts between sand grains would be contaminated with fines, which would affect to some extend the resulting overall mechanical response. Obviously, a very small amount of fines would affect only few contacts and the resulting effect would be small. On the other hand, when the amount of fines is large enough, so that the voids are completely filled with fines, the contacts

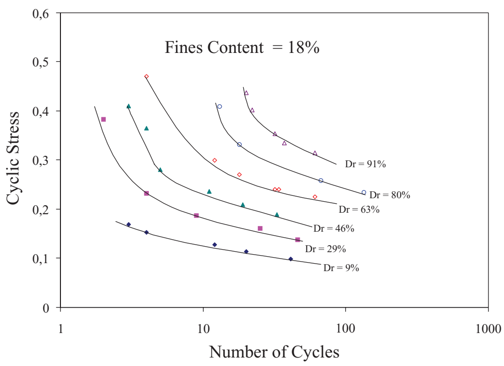

Figure 16. Cyclic strength for different relative densities. Mixture of 18% of fines content.

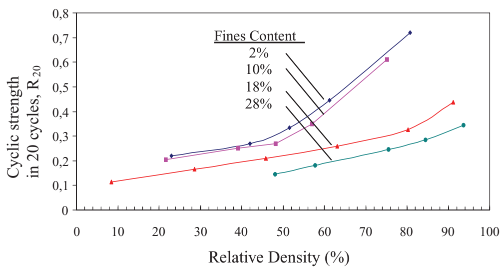

Figure 17. Cyclic strength as a function of relative density for different fines content.

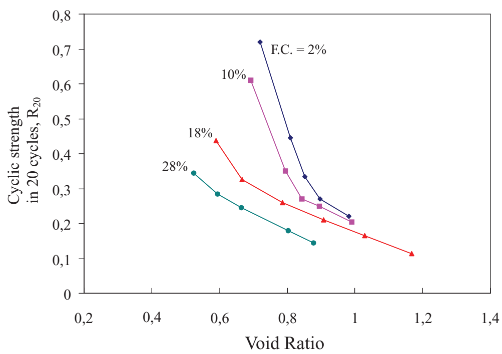

Figure 18. Cyclic strength as a function of void ratio for different fines contents.

between particles will take place mainly throughout the fines matrix, and the overall behavior would be controlled by the fine fraction. Hence, it is possible to indicate that the concept of sand skeleton usually used to model the effect of fines is inappropriate because always a part of the existing fines will contaminate some of the contacts between larger particles, which will definitely alter the mechanical response.

These experimental results resume the effects of density and fines content on the cyclic strength of tailing sands, which has a tremendous practical application on the design and operation of a conventional tailing dam. First, the seismic analysis permits to establish the cyclic strength that ensure the required stability, then using the above-presented results all the possible combinations of fines content and relative density that satisfy the needed cyclic strength can be obtained. Therefore, during the tailing dam construction it is possible to have a flexible design playing with the requested density according to the fines content produced by the cycloning process.

2.7 *Undrained steady state strength*

Series of CIU triaxial tests were carried out on the same tailings sand described above. The steady state lines defined by these tests are shown in the e-p' plane in Fig. 19. It can be seen that as the fines content increases the location of the steady state lines move down, suggesting an increase in the compressibility of the mixture. Nevertheless, the slope of the steady state lines is maintained unaffected by the presence of fines in the mixtures.

On the other hand, all the data fall in a rather unique straight line in the q-p' plane that is associated to an angle of internal friction of 35°, which means that the frictional resistance of these soils is not affected by the finer fraction. Therefore, it is possible to conclude that the non-plastic fines affect the general structure of the tailing sandy mixtures, at least, in the range of 2 to 30%, making them more contractive as the fines content increases.

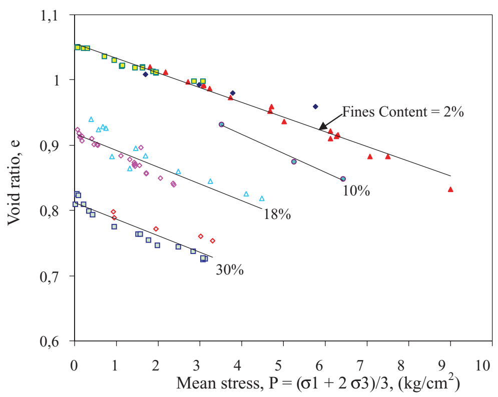

Figure 19. Steady state lines for mixtures of different fines content.

2.8 *Monotonic drained strength*

During monotonic drained loading conditions, the mobilized strength of a cohesionless material can be well characterized by the angle of internal friction. For cohesionless copper tailings, at the same initial value of void ratio, e = 0.90, the variation of the angle of internal friction with fines content is presented in Fig. 20 (Troncoso & Verdugo, 1985).

It is interesting to notice that for the range of confining pressure used, 1 to 5 kg/cm^2, the angle of internal friction mobilized at the peak failure is rather high, suggesting that tailing sands can develop high frictional resistance due to the angularity and hardness of the particles. Similar results have been reported by Pettibone et al. (1971) and others researchers. Furthermore, it can be observed that keeping the same initial void ratio, the angle of internal friction decreases as the fines content increases, suggesting that the lower the fines content the higher the strength.

2.9 *Shear modulus and damping ratio*

Provided that there is no generation of pore water pressure during seismic disturbances (dense sands, non-saturated sand), an alternative to model the seismic response of tailings dams is using the Equivalent Linear Method. In this condition both the shear modulus degradation curve and the variation of the damping ratio with the level of deformations are needed.

Test results of these parameters obtained in copper tailings sands using resonant column have been reported by Rojas et al. (1985) and they are presented in Figs. 21 and 22. It can be seen that in tailings sands the degradation of the shear modulus with the level of strain is less pronounced than the one reported for

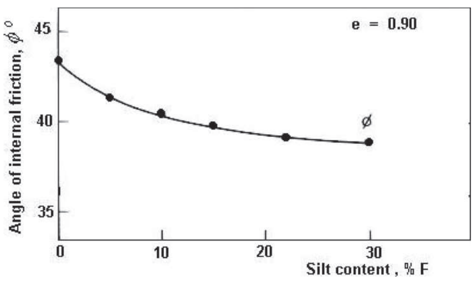

Figure 20. Angle of friction and fines content (Troncoso et al., 1985).

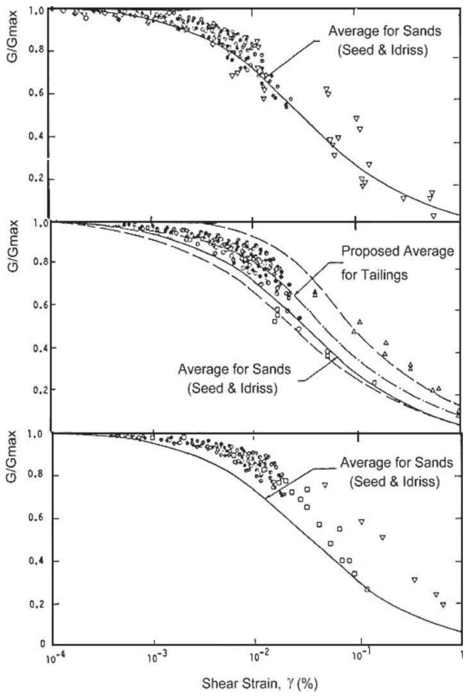

Figure 21. Degradation of shear modulus (Rojas et al., 1985).

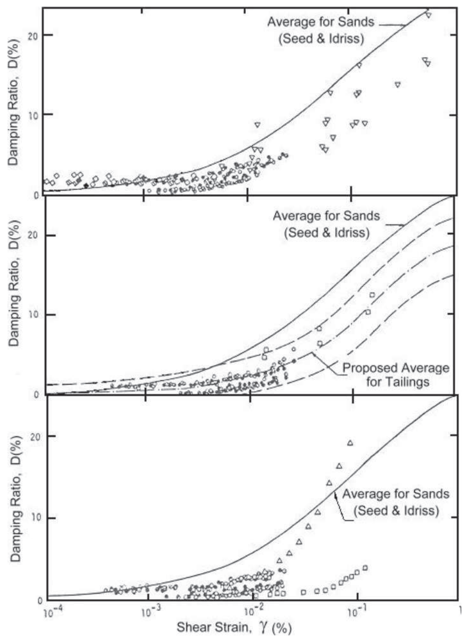

Figure 22. Damping ratio (Rojas et al., 1985).

natural sandy soils and the damping ratio achieves less values than natural sandy soils.

On the other hand, the effect of fines content on the degradation of the shear modulus has been reported by Troncoso & Verdugo (1985) and shown in Fig. 23. It is noticeable that, for any level of strain, shear modulus decreases as the fines content increases. Therefore, higher stiffness can be expected in cycloned tailings sands as the fines content decreases.

2.10 *Effect of initial fabric*

During the genesis of any soil deposit, the sedimentation and placement of soil particles is affected by the gravity force, which generates a preferential particle orientation that makes anisotropic soil structures. Casagrande et al. (1944) named this initial anisotropy caused by the geological process of deposition Inherent Anisotropy. Depending upon the environmental conditions existing during the sedimentation process, the inherent anisotropy may affect significantly the soil response. This situation is particularly important in hydraulic fills as the case of tailing dams, where there is not only a preferential orientation of particles, but also a segregation that results in a heterogeneous structure. Hence it is strongly recommended the evaluation of geotechnical properties using samples with the actual structure generated in the field.

Nevertheless, in saturated tailing sands, the field operation to retrieve "undisturbed" samples for being tested in the laboratory is complicated and expensive. Furthermore, at the beginning of the projects, when the tailing dams are not yet constructed, "undisturbed" samples are not available. Consequently, the alternative of testing reconstituted specimens compacted at the same density expected in the field it is always attractive, but it can not be ignored that the initial fabric or structure may have an important effect on the soil parameters, and therefore efforts has to be done in order to reproduce the expected actual soil structure.

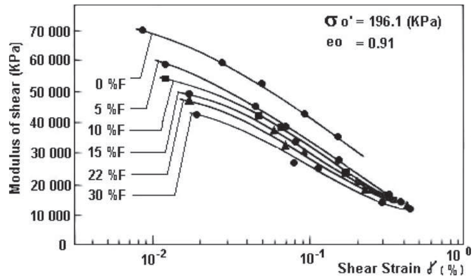

Figure 23. Shear modulus and fines content, (Troncoso & Verdugo, 1985).

Regarding the undrained steady state strength developed at large strains, there is experimental evidence showing that this parameter is independent of the initial inherent anisotropy or initial particle arrangements. However, it has been also shown that the steady state strength of non-homogeneous samples is strongly dependent on the initial configuration of particles, which suggests that even pretty large deformations are not able to erase the initial heterogeneity (Verdugo, 1992; Verdugo et al., 1995). Hence, it is proposed to divide the initial arrangement of soil particle in two groups as indicated in Fig. 24.

Firstly, it is possible to identify those homogeneous initial arrangements of soil particles that can be completely broken down at large deformation, and therefore can mobilize a unique steady state line. In the second group are those particle configurations of heterogeneous distribution of grains that can not be fully erased by large deformations, independently of how large the strains are. Tailing sand deposits are in this second group.

Triaxial tests data obtained from both reconstituted and "undisturbed" samples of tailing sands have been reported by Castro et al. (1989), and shown in Fig. 25. It is readily apparent that there is a significant difference between the undrained strength of "undisturbed" and reconstituted samples.

These experimental results confirm that an initial heterogeneous structure is not erased at large deformation and it develops a different undrained steady state strength respect to the one reached by the homogeneous soil mass. Then, for the geotechnical characterization of tailing deposits the use of "undisturbed" samples is strongly recommended.

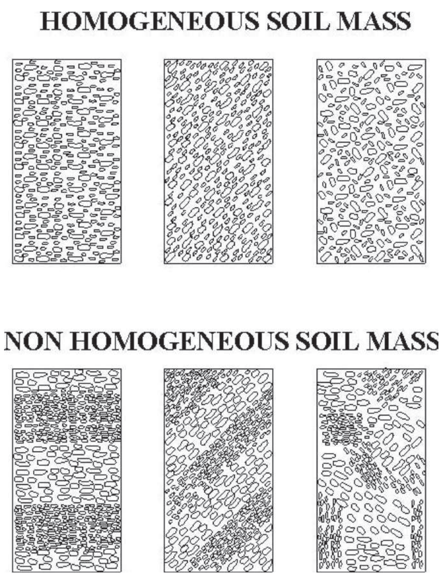

Figure 24. Proposed division of initial arrangement of soil particle.

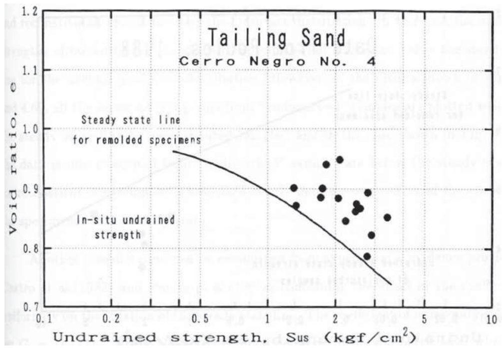

Figure 25. Undrained steady state strength from reconstituted and "undisturbed" samples (Castro et al.,1989).

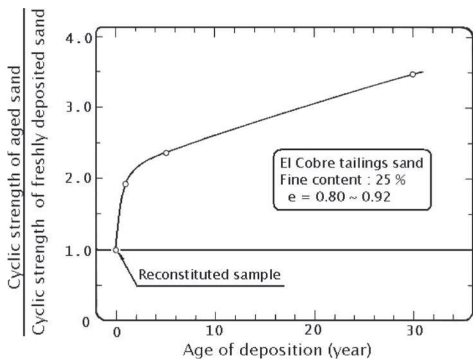

Figure 26. Aging effect on the cyclic strength of tailing sands (Troncoso et al., 1988).

2.11 *Liquefaction and aging*

It has been generally recognized that the liquefaction resistance (cyclic mobility) tends to increase with the age of the deposit, what can be associated to the development of light cementation or some welding at points of grain contact. To study the effect of the time of deposition in the cyclic strength of tailing sands, series of cyclic triaxial tests have been performed on "undisturbed" samples retrieved from an old tailing dam at different depth, which basically means different age of the samples. In addition, tests on fresh samples reconstituted in the laboratory were carried out (Troncoso et al., 1988). The test results are summarized in Fig. 26, indicating that the cyclic stress ratio required to produce a state of softening with 5% double amplitude strain tends to increase by a factor of 3.5, 2.4 and 2 for the samples of 30, 5 and 1 years of sustained deposition, respectively.

Therefore, it is strongly recommended to estimate the effect of aging for stability analysis during the abandon period. This type of study can be done when the tailings dam has been in operation for several years, so it is possible to retrieve samples at different depths, which are associated to different years of deposition. Testing this batch of "undisturbed" samples, it is possible to establish the variation of the cyclic strength with the age of deposition, which allows an estimation of the improvement of the cyclic resistance with time.

3 LARGE EARTH DAMS

3.1 *Rockfill and gravel-fill dams*

Because of the intrinsic exceptional geotechnical properties of coarse materials, they are normally used in the construction of large earth dams. These materials are used in Concrete Face Rockfill Dam (CFRD) and Concrete Face Gravel-fill Dam (CFGD). These types of dams have increased in number throughout the world mainly because of the following two reasons: modern CFRD is a high quality dam type from all technical standpoints and the CFRD is often the lowest-cost dam type when the material is readily available at site (Sherard & Cooke,1987).

However, coarse materials as rockfill, cobbles and gravel always present difficulties in the evaluation of their properties, commonly due to the lack of sufficiently large equipment to test large size particles. Hence in rockfill and gravel-fill dam projects, the available information related to mechanical properties of the coarse material of the fill is quite limited. Additionally, an important aspect to bear in mind is associated with the post construction deformations, which might affect the concrete face.

Therefore, evaluation of mechanical properties and long term deformations are two important issues that have to be faced on the design of a rockfill and gravel-fill dams. These topics are addressed in the following sections.

3.2 *Evaluation of mechanical properties of coarse soils*

Different methods to evaluate mechanical properties of coarse soils have been proposed, which involve testing of "equivalent" soil samples, free of oversized particles. The matrix model method, the parallel gradation method and the scalping and replacement method are the most commonly used methods. In the matrix model, the original coarse soil is divided in two parts: oversized particles and matrix material. The definition of oversize is arbitrary, and it is related to the maximum particle size that can be tested in the available equipment. It is assumed in this method that the oversized particles are in a "floating" state, meaning that these particles have little or no contact between them. The matrix material to be tested is compacted to a density that has to be estimated, corresponding to the actual density of the soil matrix in the field (Siddiqi et al., 1987; 1991; Fragaszy et al., 1990; 1992). Therefore, the use if this procedure is limited by the validity of the assumption that the oversized particles are "floating" and the accuracy of the procedure to estimate the actual density of the soil matrix in the field.

In the scalping-replacement method, all those particles that are considered oversized with respect to the available testing equipment are scalped and replaced with an equal weight of a smaller particle range (Donaghe & Torrey, 1979). This procedure changes drastically the original soil gradation and, although some experimental data have shown promising results, there is no real evidence to support the equivalence between the original soil and the artificially created batch of soil scalped and replaced.

In the parallel gradation method, the oversized particles are scalped and a new batch of soil is prepared using the original material, which has a grain size distribution curve parallel (in the common semi log scale) to that of the original sample (Lowe, 1964; Marachi et al., 1972; Verdugo et al., 2003; Varadajan et al., 2003). The main advantage of this procedure is that the soil gradation is maintained. However, depending upon the particular characteristics of each soil, the mineralogy and hardness of grains, particle shape, and particle roughness, may be different and function of the particle size (Al-Hussaini, 1983; Cho et al., 2005; Lee et al., 1967; Santamarina et al., 2003 & 2004). In granular soils where these factors are similar for all particle sizes, the parallel gradation method can be seen as an attractive alternative.

Verdugo & De La Hoz (2006) reported test results of gravelly soils using the parallel gradation method. The grain size distribution curves and the maximum and minimum densities of one of the material tested are presented in Fig. 27. It is interesting to observe that the maximum and minimum densities are rather similar, regardless of the mean grain size, D_{50}. All tests were performed on samples compacted to an initial relative density of 70% and in a range of confining pressure between 20 and 600 kPa.

The stress-strain curves and the volumetric strains of these batches are presented in Fig. 28.

It can be observed that both peak strength and stiffness are similar for the different batches.

The stress-strain curves present an initial linear portion that can be represented by the deformation modulus, E_{50} (stiffness associated with a stress level equal to half of the peak strength), which results are presented in Fig. 29. It is observed that the parallel gradations are able to capture the essential mechanical response of the soils, showing the same expression for E_{50}:

$$E_{50} = 175(\sigma_3)^{0.79} \quad \text{(MPa)} \tag{1}$$

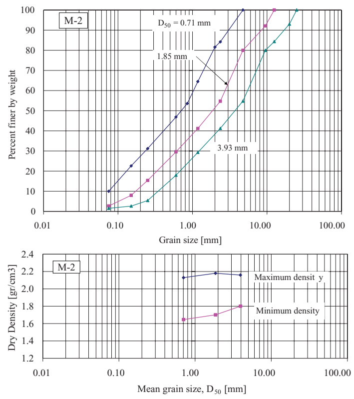

Figure 27. Grain size distribution curves and maximum and minimum densities of samples M-2.

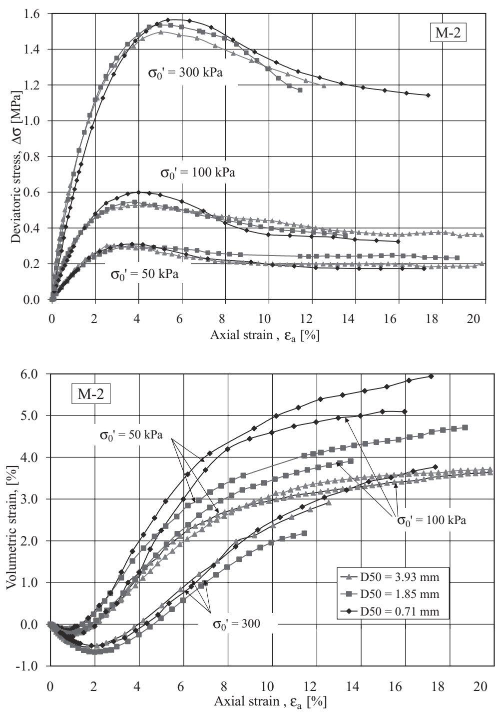

Figure 28. Stress-strain curves and volumetric strains obtained in samples M-2.

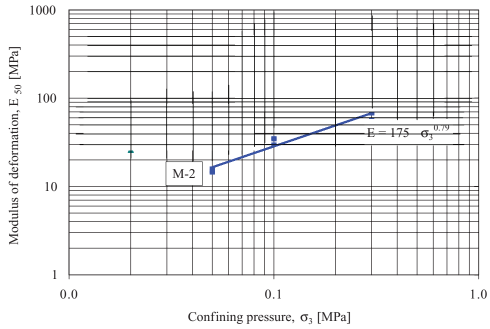

Figure 29. Deformation Modulus, E_{50} for samples M-2.

3.3 *Post-construction deformations*

A study of the behavior of the Chilean largest earth dams conducted for the Ministry of Public Work permitted the analysis of the variation of the static deformation modulus of rockfill and gravel-fill dams. In this study, long term post-construction settlements monitored on three Chilean earth dams were used to estimate the time variation of the deformation modulus these coarse materials. Basic information of the analyzed dams is indicated in Table 2.

The static dam response was modeled using a perfect elasto-plastic stress-strain relationship, implemented in the computer code FLAC. The evaluation of the deformation modulus was performed by a try and error process until the calculated and observed dam settlements matched.

The main body of Cogoti dam was finished in 1938 and the vertical deformations have been monitored since that time. The dam was finally completed in 1940.

This dam was constructed with blasted rock without compaction. In its first 15 meters, rockfill with a maximum size of 1.5 meters were just dumped in the dam site by gravity. In the following raises, rockfill with a maximum size of 1.3 meters was placed by mechanical means and it was slightly compacted by the construction procedure associated to the traffic

Table 2. Monitored chilean dams.

Dam	Cogoti	Conchi	Santa Juana
Completion year	1940	1975	1995
Foundation type	Rock	Rock	Rock fluvial
Dam type	CFRD	CFRD	CFGD
Height (m)	82.7	66.0	113.4
Crest length (m)	160	200	390
Upstream Slope (H:V)	1.45:1	1.5:1	1.5:1
Downstream Slope (H:V)	1.5:1	1.5:1	1.6:1

Figure 30. General view of Cogoti dam.

of trucks. Consequently this dam is a good example of a coarse material dam on a very loose state of compaction. A general view of Cogoti dam is shown in Fig. 30.

Conchi dam, completed in 1975, was constructed with rockfill of a maximum size of 0.65 m. Great effort to compact the rockfill was applied. The available information indicates that the compacted fill reached a degree of compaction associated with a relative density greater than 90%. Fig. 31 presents a general view of Conchi dam.

Santa Juana dam, completed in 1995, was constructed with rockfill and gravely particles with maximum sizes of 1 and 0.65 m in the upstream and downstream supporting shoulders, respectively. Compaction was also applied to the fill. A general view of Santa Juana dam is shown in Fig. 32.

The settlements along the crest of Cogoti dam for different years are shown in Fig. 33. It is interesting to observe that the maximum settlements do not take place at the location of the maximum height of the dam, but they systematically occur above the point where a change in the slope of the bedrock exists.

The maximum static vertical deformations measured at different time and for the three dams are shown in Fig. 34. It can be observed, that Cogotí dam presents the greatest settlements compared to the others two dams. According to the study, this can be attributed to the uncompacted fill of Cogotí dam.

Figure 31a. General view of Conchi dam.

Figure 31b. General view of Conchi dam.

Figure 32. General view of Santa Juana dam.

In the numerical analysis of these settlements, an elasto-plastic constitutive law with the Mohr-Coulomb failure criterion was selected. This model was considered to be a reasonable approximation of the mechanical behavior of these dams in view of the fact that the analyzed dams have developed a mechanical response that is far from failure. A constant Poisson's ratio equal to 0.3 was assumed for all the cases.

The computed values of the deformation modulus at different time after completion the dams were normalized by the computed deformation modulus at 1 year of dam completion, E_1. The resulting variation of

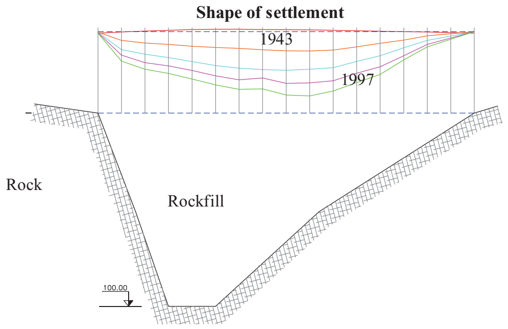

Figure 33. Vertical settlements along the crest of Cogoti Dam.

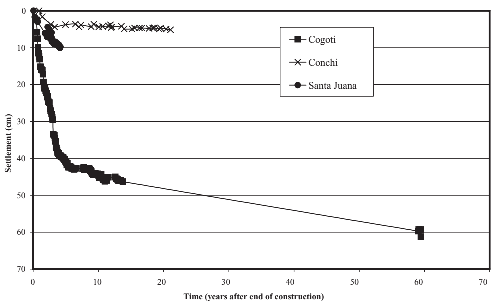

Figure 34. Development of the static maximum vertical deformation.

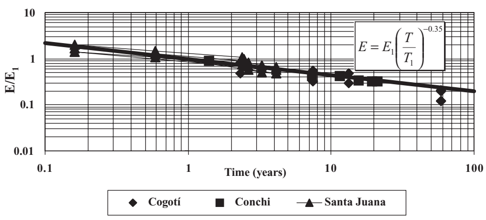

Figure 35. Normalized deformation modulus as a function of time.

the normalized deformation modulus (E/E_1) with the number of years after completion is shown in Fig. 35. These results indicated that the time effect on the static deformation modulus can be expressed as follows:

$$E = E_1 (t/t_1)^{-0.35} \tag{2}$$

Where, t represents time in years after completion and t_1 is 1 year after the end of construction. This deformation modulus is associated to the total accumulated settlements. Therefore, the use of this expression is related to the long term settlement evaluation.

4 SUMMARY

The performance-based seismic design is strongly encouraged by structural engineers that have observed heavy financial losses in recent earthquakes. Structures designed according to current codes performed well in terms of life safety, but financial losses have been surprisingly high. This comes from the fact that the main investments in building construction are made in the non-structure components and contents. Therefore, it is evident that the fundamental issue of building code provisions to guaranty structures integrity, in terms of no collapse against strong ground motion, is definitely insufficient to be considered a successful seismic behavior to the society. In this context the performance-based seismic design would help to reduce the financial losses associated with the non-structure components and contents.

Formally, the use of PBSD is less common in geotechnical engineering. However, the earthquake geotechnical community is quite familiar in predicting permanent displacements of earth structures, which basically mean a criterion of performance as opposed to the classical concept of limit equilibrium.

One important issue of PBSD is associated with the intention of involving stakeholders (owners, insurers and regulators) in the decisions concerning the choose of target performances of an earth structure during and after seismic events, sharing in this way the decision-making process. In spite of the benefits, it is important to take into account that the results of combining both complex technical solutions and investment decision are at least risky.

The second important issue of PBSD is related to the premise that seismic performance levels can be predicted analytically, permitting that the cost associated with each level can be rationally evaluated. From a scientific point of view, there is a reasonable knowledge of soil and rock mechanics behavior that has been incorporated into numerical models. However, in engineering practice the real situation is less promising, especially when three dimensional analysis is needed. Additionally, in the case of dam engineering, practitioners have to face the geotechnical characterization of rockfill materials, but, normally there is a lack of available testing apparatuses for these coarse soils, which means that in the design, geotechnical properties have to be estimated.

In this context, the actual capability of predicting the seismic performance is quite limited. Consequently, it is important to recognize that the real application of the PBSD in professional practice is years away, but it is also important to admit that this design criterion is attractive and efforts have to be done in order to make it closer to practitioners.

In the PBSD of tailings dams, the liquefaction resistance of the tailings sands play an important role, and this resistance is affected by density and the fines content. Therefore, theses effects have to be understood. On the hand, the PBSD of concrete face rockfill dams needs the mechanical properties of the coarse fill used in these dams. However, sufficiently large machines to test these materials are not usually available, so alternative procedures to obtain the required properties are needed. As can be expected, material properties are an important issue in the application of the PBSD and their evaluation and understanding is strongly needed.

In the design of tailings dams there are two important issues that control the liquefaction strength of the tailings; density and fines contents. Accordingly, results of a comprehensive study carried out on different mixtures of sand and fines compacted at different densities have been presented.

The maximum density obtained by the Modified Proctor test is only slightly lower than the maximum density achieved by vibration, even for a 50% of fines. Hence, it is possible to indicate that the concept of relative density holds valid for this tailings sand even with 50% fines or more.

The experimental results show that the cyclic strength decreases as the fines content increases. Mixtures with 2 and 10% of fines content present a sharp increase in cyclic strength from a relative density around 50%, while mixtures with 18 and 28% of fines content present only a gradual increase of cyclic strength with relative density.

A rather unique steady state line in the q-p′ plane was obtained for all the mixtures, indicating a constant residual angle of internal friction of 35°, regardless the fines content.

Fines content affect the position of the steady state lines in the e-p' plane. The higher the fines content the lower the position of the steady state line. However, the fines do not affect the slope of the steady state lines.

The amount of non-plastic fines used in this study affect the soil structure making it more contractive, and therefore, more sensitive to liquefaction.

From the seismic analysis, the required cyclic strength can be established and from the presented results, the possible combinations of fines content and relative density that satisfy the required cyclic strength can be obtained. Therefore, during the tailing dam construction it is possible to have a flexible design playing with the requested density according to the fines content produced by the cycloning process.

In the case of large earth dam, these days the most common adopted design corresponds to Concrete Face Rockfill Dam and Concrete Face Gravelfill Dam. These types of dams have increased in number throughout the world mainly because of the good behavior from all technical standpoints and also due to the lower cost. Coarse materials as rockfill, cobbles and gravel always present difficulties in the evaluation of their properties, commonly due to the lack of available equipment to test large particles. An alternative procedure to evaluate the mechanical properties is the parallel gradation method. The oversized particles are scalped and a new batch of soil is prepared using the original material, which has a grain size distribution curve parallel (in the common semi log scale) to that of the original sample. The main advantage of this procedure is that the soil gradation is maintained.

Experimental results indicate that the parallel gradation method provides a quite reasonable procedure to evaluate the geomechanical response of coarse granular materials.

Another issue of Concrete Face Rockfill and Gravelfill dams is associated with the long term deformation which might affect the concrete face. Using a simple elasto-plastic stress-strain relationship, the measurements of three Chilean dams were reproduced and the deformation modulus computed. The relationship that reproduced the calculated values of the deformation modulus can be expressed in the form: $E = E_1\ (t/t_1)^b$, where E_1 (deformation modulus associated to the settlement of the dam at 1 year after construction) depends upon the dam material, and the parameter b has shown to be a constant value for rockfill dams; $b = -0.35$.

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