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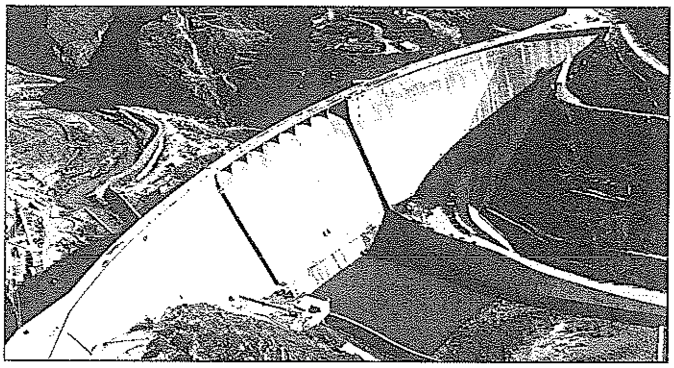

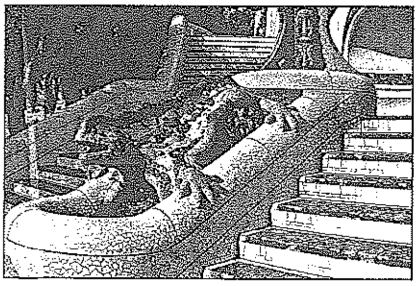

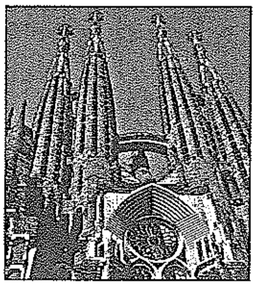

SPECIAL ISSUE: ICOLD - BARCELONA 2006
PREVIEW: HIDROENERGIA 2006

Barcelona 2006
ICOLD-CIGB SPANCOLD

Volume Thirteen, Issue 3, 2006

DAMS AND HYDRO IN SPAIN ~ ENVIRONMENTAL ASPECTS
SAFETY ~ SPILLWAYS ~ MINI HYDRO

THE INTERNATIONAL JOURNAL ON
HYDROPOWER
&DAMS

3. News 28. Bid information

ICOLD 2006 – BARCELONA

36. ICOLD and the XXI Century challenges - C.B. Viotti
37. Welcome to ICOLD Barcelona 2006 - L. Berga
38. Overview of the Meetings in Barcelona
39. Preview of the ICOLD Congress

DAMS AND HYDROPOWER IN SPAIN

45. The role of dams in Spain
L. Berga and J.C. de Cea

53. The historical development of Spanish dams
M. Arenillas

58. Current issues in Spanish dam engineering
M. Alonso Franco and A. Soriano

64. Spain's new Yesa dam
F. Molina, A. Nuñez, R. Lafuente, J.C. de Cea, J.M. Villarroel and J.L. Martinez

68. The role of hydro and future pumped-storage plans in Spain
A. Gil and J.M. Buil

72. Dam safety management in Spain
J. Yagüe Córdova and J.C. de Cea

76. Evaluation of the behaviour and safety of the new Tous rockfill dam
I. Escuder Bueno, J.L. Utrillas Serrano and J. Fleitz

82. Dams and the environment: a Spanish perspective
F. Hijos

86. Spanish dam engineering worldwide
J. Polimón López, F. Abadia Anadon, E. Bofill de la Cierva, and A. Capote del Villar

94. Hydropower & Dams in Spain and Portugal (Tables)

DAMS AND SUSTAINABLE DEVELOPMENT

99. The role of dams in the XXI century
F. Lempérière

SPILLWAYS

110. Dam safety remediation in the Snowy Mountains
B. Jones, S. Lacroix and B. Hakin

114. Design of the spillway for Campos Novos, Brazil
A.P. de Sagi Sachi, H. Ferraz Nunes, J. Ayres de Campos and S. Medeiros Eyzano

HYDRO RESERVOIRS

118. Lake Victoria: a predictably fluctuating resource
P.J. Mason

121. Applying uncertainty parameters to the HTCOM model for multipurpose reservoirs
R. Ardakanian and A. Karimi

ENVIRONMENT

127. The dinosaur and the eco-warrior: the best time for environmental assessment
J.-P. Chabal

130. Recent developments in the design of fish-friendly turbines
D. Upadhyay, J. O'Nians, O. Paish and A. Turnpenny

HIDROENERGIA / HYDRO IN THE UK

135. Hidroenergia 2006: Welcome messages
B. Pelikan (ESHA) and D. Williams (BHA)

138. Renewable Obligation breathes new life into historic UK hydro plants
S. Cryer

140. Refurbishment and upgrading reduces noise levels at Auchtertyre
J. Cox

141. Glendoe moves ahead in Scotland
N. Sandilands and M. Seaton

144. Small Hydro Updates

GLOBAL WATER ISSUES

148. Local actions for a global challenge: First report from the 4th World Water Forum

159. Green Power

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Evaluation of the behaviour and safety of the new Tous rockfill dam

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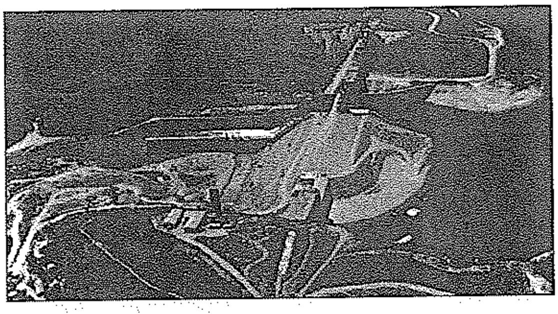
Evaluation of the behaviour and static and dynamic safety of New Tous dam, a rockfill dam with a central clay core on the Jucar river, Spain, was one of the subjects analysed (as Theme C) at the 8th International Benchmark Workshop on Numerical Analysis of Dams, which took place in Wuhan, China, in October 2005. The Workshop was organized by ICOLD's Committee on Computational Aspects of Analysis and Design of Dams (see *H&D* Issue 1, 2006). The scope of the evaluation and numerical solutions obtained by the authors are presented in this article.

Records obtained from dam instrumentation are crucial both for the interpretation of structural behaviour and for the ability to assess safety. However, because of the uncertainties involved in the process of installing such instruments, the methods used to collect readings, and the nature of the instruments and their maintenance, measurements may not be as reliable as expected.

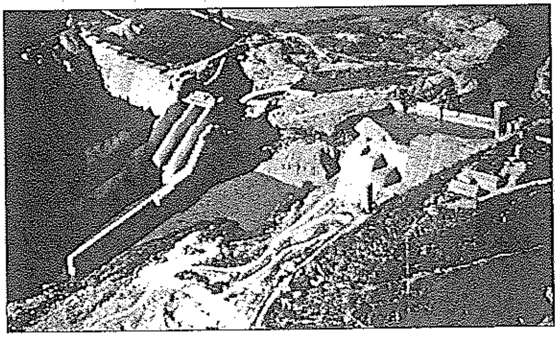
In fact, measurements can be misunderstood, causing dam engineers to make wrong decisions and, in some cases, to develop some degree of scepticism about their importance.

Numerical modelling can be a helpful tool, but also adds some additional uncertainties if it is not carefully and rigorously used: construction data should be examined, constitutive models properly chosen, and so on.

In summary, a combination of good knowledge of the instruments, appropriate maintenance of instrumentation and reading procedures, a realistic data management programme and implementation of numerical models are important and must be undertaken carefully in view of all the uncertainties involved. In addition, the behaviour of the dam itself is a source (sometimes even the main one) of uncertainty.



View of the old Tous dam.



The dam site after the failure.

In this context, the role of benchmarks, which allow a comparison of results to be made, based on the use of constitutive models, computer codes, and so on, can be very helpful for an improved understanding and assessment of the uncertainties involved in problems such as the one presented here.

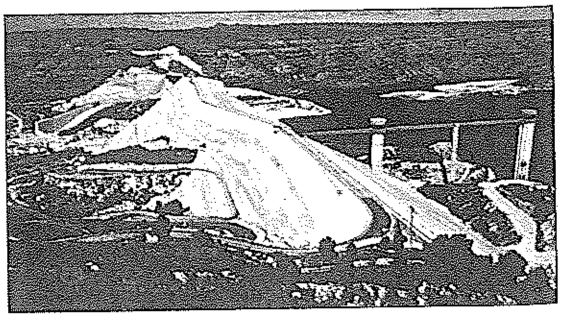
1. Scope of the work

The case study presented here relates to New Tous dam, on the Jucar river, Spain, which was the subject of Theme C at the 8th International Benchmark Workshop on Numerical Analysis of Dams, in Wuhan, China in October 2005, organized by the ICOLD Committee on Computational Aspects of Analysis and Design of Dams.

New Tous dam is a 140 m-high rockfill dam with a clay core, the geometrical definition of which was uniquely designed to accommodate old concrete blocks in the new clay core of the dam. It was constructed between 1991 and 1996. The original dam at the same site was a composite structure (with a central rockfill section and concrete at the abutments); it failed as a result of overtopping in 1982.

The problem set during the Workshop included the simulation of behaviour during construction, estimation of the movements caused by creep and wetting under certain load conditions, calculation of static safety factors and a simulation of the dynamic response for a 10 000 year earthquake. All these aspects are covered in this article; numerical results are provided and some actual measurements are also given where these are available.

The three photographs show: (a) the old dam; (b) the site immediately after the failure; and, (c) the appearance of the new dam.



The new Tous dam.

2. Section studied, properties and load conditions

The cross-section which was analysed is shown in Fig. 1, together with the location of the four different materials. (An Autocad file with the exact coordinates was available for Workshop participants.)

The upstream slope is 1V:1.85H and the downstream slope is 1V:1.5H between berms 5 m wide. The shape of the calculation grid was not fixed, and had to be chosen by participants.

As can be seen in Fig. 1, square and slightly rectangular shapes with an average height of 5 m were used for the calculation grid (and successfully tested either for stress-strain, static or dynamic stability analysis).

The main properties assumed for the materials [Utrillas, 1996[1]], are listed in Table 1.

Other required parameters, such as bulk and shear static modulus, creep constant, and so on, could be estimated and calibrated according to the behaviour patterns shown in Figs. 3, 5 and 6 (which appear later in the text). However, any approximate and consistent value for those and other parameters (that is, Raileigh dumping for dynamic analysis) was expected to lead to similar results and, in any case, would make the final discussion more interesting.

The water level has been recorded on a daily basis since 1996. Maximum and minimum reservoir levels per year are given in Table 2.

Finally, although no real significant earthquake has been recorded at the site, the 10 000 year design earthquake is given (Fig. 2) to estimate the potential response of the dam.

3. Reproduction of behaviour during construction

This behaviour analysis included the simulation of the sequence of loading by locating different layers and making use of the hyperbolic stress-strain model corrected by Duncan *et al* in 1980. The model characteristics and the routines adopted for calculations have been published by Escuder [Escuder, Andreu and Rechea, 2005[2]].

Settlement distribution recorded at the end of construction (Fig. 3) was given to Workshop participants to check their grid, model and parameters. The calculated settlement is provided in Fig. 4.

Table 3 includes the values of the best fitted parameters of quarried material (rockfill) and clay, as the foundation was consider to be much stiffer thus not affecting the constructional behaviour. Also, the influence of the filter was not taken into account.

As mentioned, constructional settlement distribution can be seen in Fig. 3. Results obtained by the numerical model programmed and executed in FLAC 2D [Itasca, 1994] are almost identical to those shown in Fig. 4.

4. Estimation of post-construction behaviour

4.1 Seepage and settlement pattern

The following data were given to Workshop participants to provide a basic idea of the seepage pattern. In particular, pore pressure records are expressed in total head (metres above sea level, obtained as location height of the piezometer plus the water column above it).

The first of the numbers associated with each pizometer (aaa/bbb/ccc) is the total head recorded by

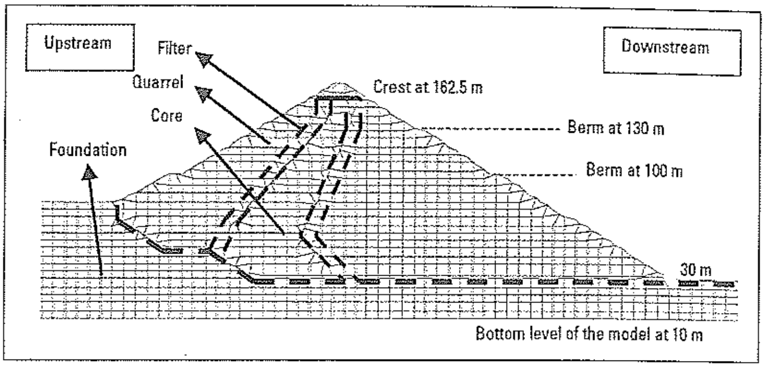


Fig. 1. Cross section of the New Tous dam.

Table 1: Static and dynamic basic properties of the materials

Material	Dry density (kN/m³)	Porosity	C (kN/m²)	ϕ (Degrees)	Dyn. Shear modules (Pa)
Core	20	0.08	100	28	$1.9X10^8$
Rockfill	23	0.25	0	45	$5.2X10^8$
Filter	19	0.2	0	30	
Foundation rock	26		....	....	10^9

Table 2: Maximum and minimum water level per year (el. m)

Year	Max. reservoir level (meters above sea level)	Min. reservoir level (meters above sea level)
1996-1997	(very stable around 80 m)	(very stable around 80 m)
1998	91.41	80.39
1999	103.05	82.41
2000	90.13	81.1
2001	96.73	82.16
2002	99.37	80.05
2003	95.87	79.81

Table 3: Hyperbolic parameters fitted for constructional behaviour

Parameter	Rockfill	Core	Units
K_b (bulk modulus number)	600	200	*
K (modulus number)	1200	400	*
m_d (bulk modulus exponent)	0.2	0.2	*
n_d (exponent for stress-dependent modulus)	0.4	0.45	*
R_f (failure ratio)	0.7	0.7	*
ϕ_0 (angle of internal friction at latm, 101.325 kPa, of confining stress	40	28	Degrees
$\Delta\phi$ (reduction factor for ϕ)	7	0	*
C (cohesion)	0	100000	Pa
Pa (atmospheric pressure)	101 325		Pa

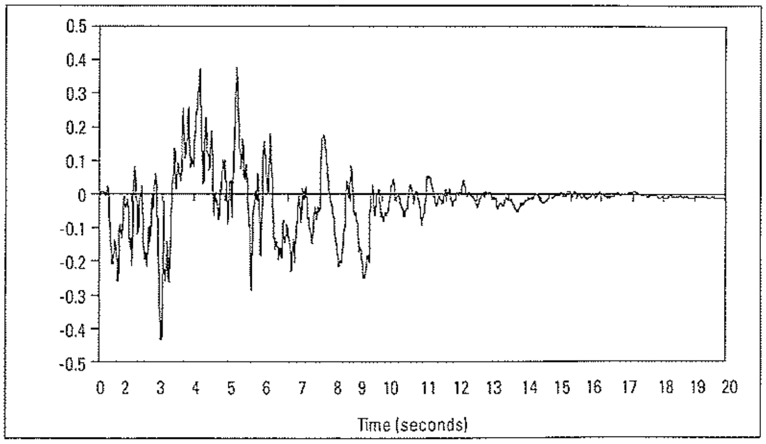


Fig. 2. Velocity time history (T=10 000 years).

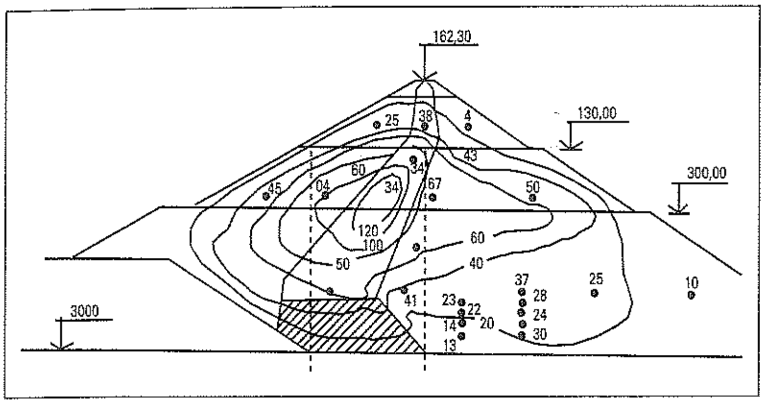


Fig. 3. Settlement in centimetres during construction.

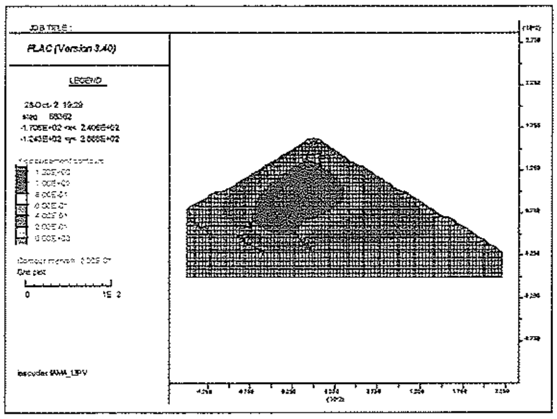


Fig. 4. Constructional settlement distribution by numerical modelling.

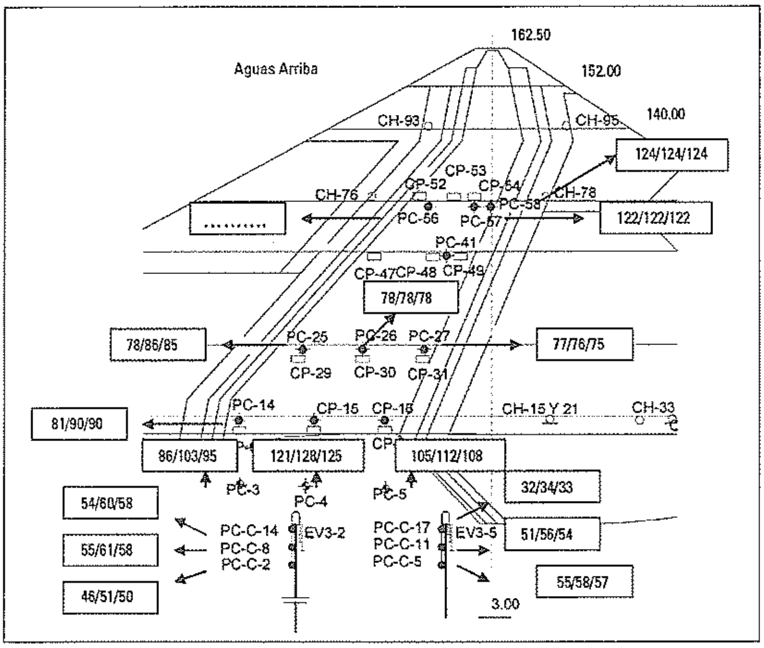


Fig. 5. Total head (m) above sea level registered by piezometers for three water levels (January, July and August 2004). PC-C = Piezometer at foundation; PC = Piezometer at core; CH = Settlement cell; CP = Total pressure cell; and, EV = Extensometer.

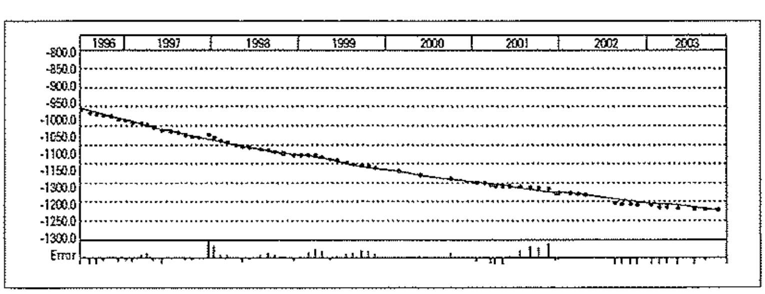


Fig. 6. Settlement records at Cell CH-76 from 1996 to 2003.

January 2004, for a reservoir water level at el. 88.467. The second value (aaa/**bbb**/ccc) was recorded by July 2004 with a reservoir water level at el. 106.11, and the third number (aaa/bbb/**ccc**) was read by August 2004 with a reservoir water level at el. 92. Fig. 5 shows the three values collected for several piezometers.

In addition, settlement measurements in milimetres recorded by hydraulic cell number 76 (CH-76 in Fig. 5), were given graphically as shown in Fig. 6.

4.2 Problem statement and solution

The problem consisted of estimating the incremental movements between January 2004 and July and August 2004, which would occur in some points of the cross section analysed.

Real values as recorded by topography and settlement cell number 76 are given in Table 4.

The indicative sign criteria are:

(a) vertical movements are positive upwards and negative downwards; and,

(b) horizontal movement are positive downstream and negative upstream

4.3 Analysis of results and numerical modelling capabilities

Three main phenomena that any numerical simulation has to incorporate in one way or another are creep, wetting and mechanical effects:

- Creep occurs from the construction stage, but only prevails after the end of the works. It decreases with time and typically becomes almost negligible within the first ten years of the dam's life.
- Wetting occurs when the reservoir level rises and typically increases the upstream rockfill settlement during the first impounding (collapse compression).
- Mechanical effects are related to the water pressure acting on the core (causing downstream movements) and upstream foundation (causing non-uniform settlement). In addition, buoyant uplift acts on upstream shell rockfill.

One of the results of these phenomena is that complex movements during reservoir filling are observed at many dams. Some of the existing models to reproduce them are the visco-elastic methodology proposed by the Central Board of Irrigation and Power of India [Escuder, 2001[3]] to simulate creep, and the Nobari methodology to reproduce wetting, updated by Escuder *et al* [Escuder, Andreu and Rechea, 2005[2]; Escuder, 2001[4]].

In this particular case, as could be partly expected from previous data, recorded movements were very small:

- Concerning creep, previous results of cell CH-76 showed almost complete stabilization.
- Concerning wetting, only 2.61 m of the upstream rockfill were wetted the first time (the maximum historical level was el. 103.5 while the maximum reached during the period analysed was el. 106.11).
- Concerning mechanical aspects, the increase in water pressure was not insignificant, but it was acting in a very rigid part of the dam (still very wide at el. 106.11).

Nevertheless, it is still very difficult to estimate what happened in terms of assessing the particular contribution of these three phenomena to the behaviour. The fact is that no horizontal movement of more than 3 mm has been measured.

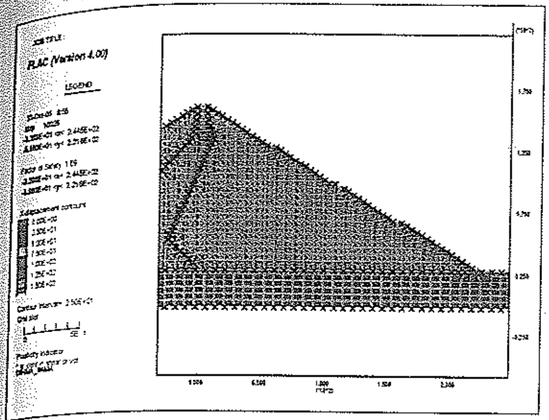


Fig. 7. Factor of safety (full reservoir).

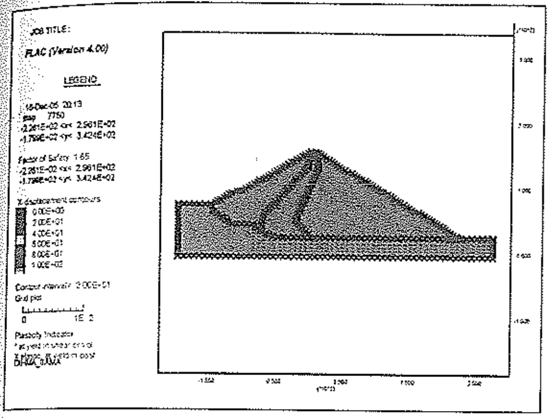


Fig. 8. Factor of safety (rapid drawdown).

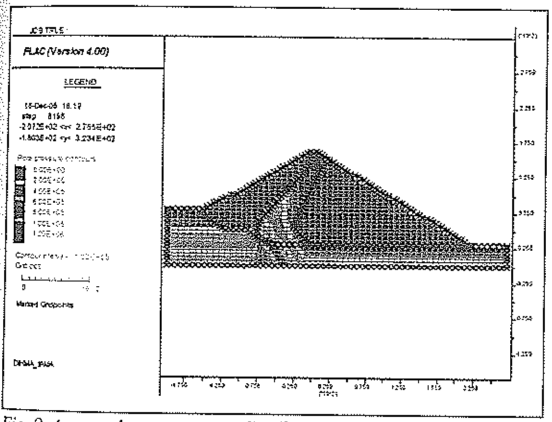


Fig. 9. Assumed pore pressure distribution for rapid drawdown (Pa).

With respect to vertical movements, only creep seems to be playing a role (the 66 mm increase in settlement measured at the crest), as the upstream shoulder does not reflect any increase in settlement.

In any case, unless the reservoir is significantly raised, any model or calculation can be validated [Escuder *et al*, 2005[5]].

5. Calculation of safety factors

Factors of safety obtained for shear static failure under full reservoir hypothesis (reservoir at el. 130) and rapid drawdown from such a level to el. 75 were estimated as 1.69 and 1.64, respectively.

In general, the plastic behaviour of zones closed to the borders is a critical issue to achieve realistic data. If plastic parameters for these areas are not reviewed carefully (typically an increase in friction angle may be adopted, which is widely accepted for granular materials under low confining stresses), calculated shear factors of safety might be significantly lower than those given by limit equilibrium analysis.

Table 4: Incremental (from Jan. 2004) vertical and horizontal displacements (mm)

Date	Water level	CA-9	CO-9	CC-17	NC-18	CB-10	CB-11	NB-7	CH-76
Jan. 04	87.46	0	0	0	0	0	0	0	0
July 04	106.11	0	1	3	-66	0	-1	-3	-11
Aug.04	92	1	0.5	0	-68	0	0	-3	-13

CA-9: inc. vert. movement at upstream berm (el. 100). CO-9: inc. horiz. movement at upstream berm (el. 100). CC-17 = inc. horiz. movement at crest. NC-18: inc. vert. movement at crest. CB-10: inc. horiz. movement at downstream berm (el. 130). CB-11: inc. vert. movement at downstream berm (el. 130). NB-7: inc. vert. movement at downstream berm (el. 100). CH-76: inc. vert. movement at CH-76 settlement cell location.

Nevertheless, when the factor of safety is obtained by a continuous and proportional degradation of the resistant parameters of the constitutive models (Mohr Coulomb in this case), if the rockfill planar type of failure is more restrictive than any other type of failure surface, the factor of safety will be lower than results obtained by limit equilibrium analysis.

In this particular case, for the first problem, the planar type of failure of the downstream rockfill prevails, as shown in Fig. 7, thus movements related to such failure mode would start at the upper part of the downstream slope. The factor of safety is almost equivalent to the ratio between the friction angle tangent and the average downstream slope.

Concerning the rapid drawdown case (Fig. 8), a very conservative pore pressure distribution has been assumed according to the known seepage pattern ,as shown in Fig. 9. In fact, a complete saturated state has been adopted for the core, and no pressure dissipation was found either in the core or in the foundation beneath the core.

It is important to note that, even under such a hypothesis, if the rapid drawdown is limited to a minimum reservoir level of el. 75, the failure of the downstream shoulder is still dominant and the factor of safety remains practically the same.

6. Simulation of dynamic behaviour for a 10 000 year earthquake

Ideally, a comprehensive model for any type of fill would account for all physical effects which occur

Fig. 10. Shear wave propagation through the model.

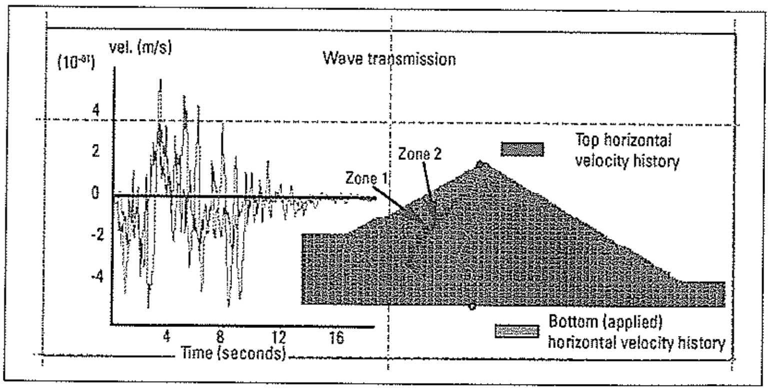


Table 5: Irrecoverable movements at various locations (in metres)

Earthquake	Water level	CA-9	CO-9	CC-17	NC-18	CB-10	CB-11	NB-7	CH-76
10000 years	130	-0.057	-1.68	-0.305	-0.118	0.663	-0.52	-0.354	-1.207

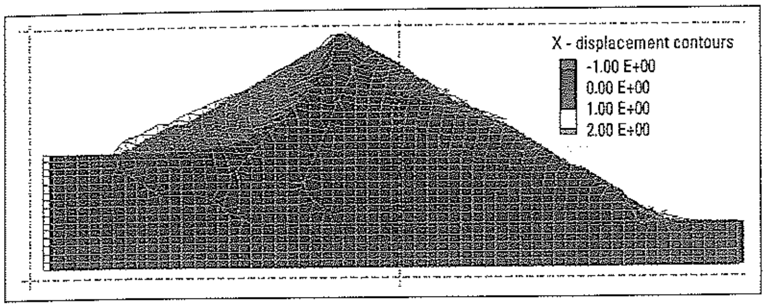


Fig. 11.Irrecoverable horizontal displacements distribution (in metres).

during cyclic loading, such as energy dissipation, volume changes and stiffness degradation. However, as such an ideal model does not exist, a compromise to account for some important aspects must be made.

For instance, in this case [Escuder and Utrillas, 2005[6]], coupling between liquid and solid phases was addressed in a simplified way, using the empirical expression proposed by Byrne, making a parallel computation of the increase in pore pressure related to the volume change, and adding such pore pressure excess to the elastic-plastic constitutive model (yielding criteria are always defined in terms of effective stresses).

The results make it possible to assess positively the seismic safety of the dam as vertical deformations are not enough to threaten the dam with overtopping and the structure has remained stable despite the fact that large deformations occurred during the earthquake simulation.

Fig. 10 shows the translation of the shear wave through the model, given in terms of horizontal velocity at the bottom and top of the model.

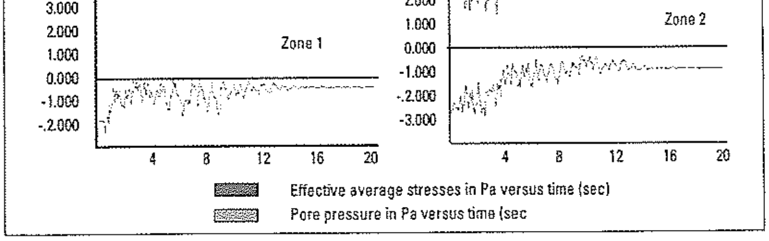
The irrecoverable horizontal movements distribution (Fig. 11) and the pore pressure development at two zones of the upstream filter (Fig. 12) show how downstream shoulder movements are higher, despite the fact that liquefaction occurs at upstream filter.

In fact, such liquefaction only occurs in the lower part of the upstream filter, where it is highly confined, so this phenomenon does not significantly affect the irrecoverable deformation pattern of the dam after the 10 000 year earthquake.

7. Summary and conclusions

Despite the fact that no comparison could be made during the Benchmark Workshop, the resulting report as summarized in this article provides available measurements and numerical solutions obtained by the authors. In any case, the role of benchmarks which allow comparisons to be made of results using different constitutive models, compute codes, and so on can be very relevant for a better understanding and assessment of the uncertainties involved in problems such as the one presented here. With this in mind, any future contribution should be addressed to the formulator of the problem (the first author). ◊

Fig. 12. Pore pressure and effective stress during the earthquake for two zones of the upstream filter (see Fig. 11).



Acknowledgements

Calculation routines have been reviewed and improved as result of a research project entitled "Analisis de la influencia de la disminucion de riesgos de rotura de presas sobre el incremento de riesgos de la insatisfaccion de las demandas en sistemas de recursos hidricos" sponsored by the Spanish Ministry of Science and Technology (30 per cent) and FEDER funds of the European Union (70 per cent).

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