Dams in Spain

COLEGIO DE INGENIEROS
DE CAMINOS,
CANALES Y PUERTOS

CNEGP
SPANISH NATIONAL COMMITTEE
ON LARGE DAMS (SPANCOLD)

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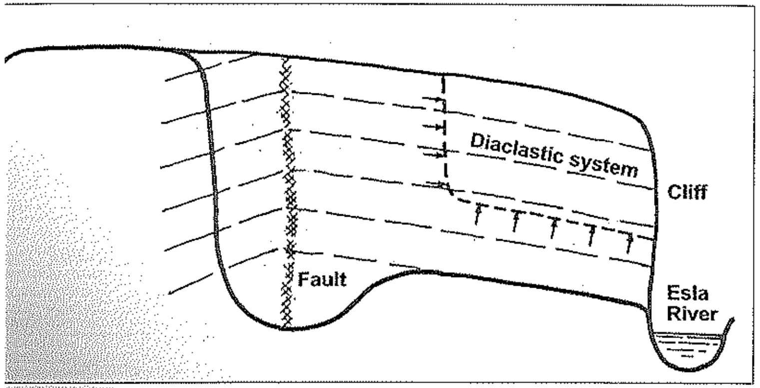

the block out and the uplift disturbing force that subtracts effect. Once the border block ...es with the closer ones, which ...t with the atmosphere and ...ls.

...pped at the middle of the ... there was a vertical fault ...stream has different ... massif has a similar ...system, which dips ...slope gradient. ... a vertical ... in this ...ping ...may continue its advance to the reservoir creating, a parallel course to the Esla River.

This sliding process, rapidly developed in a few years, may show how river courses are formed. Many times is heard that a slow continuous erosion process after centuries or millenniums may reach huge dimensions, but according to reality and the observations, is the erosion produced by floods, when there is not enough capacity, what opens or widens the river courses. So, it is not the moderate and continuous action after centuries or millenniums –the constancy–, but the floods waiting always for a bigger one – the patience – during centuries or millenniums. As stated above, if the left bank would have had a uniform structure in less than one decade there would be a new course, 90 m depth, parallel to the original one.

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Chapter 17

MONITORING AND BEHAVIOUR OF DAMS

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1. INTRODUCTION

1.1. NECESSITY AND OBJECTIVES OF MONITORING

Dams are very particular infrastructures both because of their dimensions, service life and territorial impact and for being a potential risk for the downstream area of the dam. For this reason, an increasing public interest in most parts of the world aroused in order to identify and control possible risks created by dams and to delimit them qualitatively and, as far as possible, quantitatively.

With the objective of responding to those concerns, the Spanish Water Authorities have created and adapted a legislative and normative framework during the last forty years, fruit of a long tradition and experience in dam construction and maintenance, with the aim of giving a global focus on concepts and criteria for dam safety.

After the "**Instructions for the Design, Construction and Operation of Large Dams**" published in 1967 followed the "**Technical Regulation for Dam and Reservoir Safety**" in 1996, which provided the elaboration of seven technical guidelines by the Spanish National Committee on Large Dams (SPANCOLD). One of them, Technical Guideline number 7, refers to "MONITORING OF DAMS AND THEIR FOUNDATIONS."

This Guideline comprises the entirety of activities, methods and equipment in order to understand the behaviour of a dam, including observations and visual inspections of certain aspects that are not measurable: humidity or cracks in the downstream face of the dam, colour of seepage water and existence of solid material, conditions of the downstream river valley (vegetation, erosion), state of the conservation of the monitoring equipment, accesses, communication media, etc.

The objective of the guideline is to meet the following aspects that give evidence of the necessity of monitoring dams and their foundations:

- **Anticipate** and therefore avoid that a future situation could lead to failure, accidents or other negative incidents.
- **Control** certain parameters that have an important influence on dam behaviour and safety.
- **Reduce** uncertainties of simplifying hypotheses in order to increase the confidence in future behaviour.

• **Economize**, or at least rationalize, the construction and/or operation with real data measured "in situ".
• **Understand**, with true information, if the behaviour of the dam under construction or operation is satisfying or not.

DAM BEHAVIOUR AND SAFETY

n safety depends not only on a good design and nstruction but also on all types of circumstances and dents produced in the different phases and periods, ecially during construction, first filling and the first years operation, which have to be considered.

Moreover, good and adequate monitoring allows, ides accomplishing its main purpose of knowing the ety status of the dam to avoid accidents, to obtain rmation that enriches the understanding of the real aviour of the construction and in the end to improve erimental techniques and calculation methods in order obtain a better and safer concept for future dams.

STAGES TO BE CONSIDERED

stages or phases that always have to be considered in monitoring process are:

Preliminary studies and design phase, construction, first oundment, operation and decommissioning or andoning.

UNCERTAINTIES DUE TO THE BEHAVIOUR

ords obtained from dam instrumentation are crucial in er to interpret the structure behaviour and to be able to ess its safety. However, due to the uncertainties involved he process of installing such instruments, the way dings are collected, the nature of the instruments and ir conservation state, measurements may be under ect. In fact, measurements may be misunderstood, may lead to wrong decisions and even may cause among dam engineers some degree of sceptlcism about their importance.

The technological progress during the last years has made possible to use more and more sophisticated systems with automatic data acquisition and remote transmission of data to the control centre and therefore has simplified the manual labour of recording data. Nevertheless the probability of errors may increase if no adequate service for the conservation and maintenance of the system is provided.

In this context, the study of data series by means of statistical models and the implementation of conceptual models are important to get a better understanding of the dam behaviour. However, numerical models may also add some additional uncertainties if they are not carefully and rigorously used: construction data should be examined, constitutive models properly chosen, etc. Also, a statistical analysis may be highly misleading if it is not properly used. Finally, the behaviour itself of a particular dam is a great source of uncertainties.

Therefore it can be stated that the real behaviour of a dam and its foundation is conditioned by a great number of factors and parameters of major or minor uncertainty. The quality of the safety evaluation will basically depend on an adequate monitoring system, the goodness and quality of the data provided by it and –first of all– the experience of the dam safety engineer.

2. THE SITUATION IN SPAIN

2.1. SPANISH LEGISLATION:
ANNUAL REPORTS AND PERIODIC INSPECTIONS

The increasing number of dams and the corresponding growth of affected people and economic interest, together with some accidents that have occurred and the

human and material damage associated with a dam failure, have evidenced the necessity of paying attention to dam safety and considering it as a priority factor taking into account during all stages of the technical lifetime of a dam.

Therefore in Spain exists a long tradition of dam safety control, which is reflected in the fore mentioned **"Instructions for the Design, Construction and Operation of Large Dams"** published in March 1967. The instructions oblige that every dam under operation must provide a technical operation service that is besides the ordinary operation tasks responsible for the inspections, surveillance, and safety of the dam and particularly for maintaining the control devices, outlets, accesses, and communications.

As a result of some catastrophic floods which affected the north and east of Spain in the beginning of the eighties and even lead to the failure of the Tous dam, the former General Direction of Hydraulic Works set off a program for dam safety in 1983 which had to be applied on dams operated by the state.

This program consisted of a series of phases and subprograms in order to prepare rules of operation and inspection reports which are compendiums of the main information about the design, construction and operation including monitoring data.

Afterwards the General Guideline on Civil Protection Planning for flood risks, published in 1995, established the obligation of classifying all Spanish dams depending on its potential risk in the categories A, B or C.

Category A corresponds to dams whose failure or malfunction could seriously affect urban areas or essential services and infrastructures, or produce important material or environmental damage. Category B corresponds to dams whose failure or malfunction could cause important material or environmental damage or affect a reduced number of residential buildings. Finally, category C corresponds to dams whose failure or malfunction could produce material damage of moderate importance and just incidentally the loss of human life.

In the same way, it is formulated that all dams which are classified in category A or B must provide a corresponding Emergency Plan. The elaboration of the mentioned document is the responsibility of the owner.

One of the most important aspects to be developed in the Emergency Plan of a dam is its safety analysis including the definition and admissible range of indicators in order to permit a reliable evaluation with sufficient time in advance of the diverse and potential emergency situations. Monitoring systems and the analysis of their data are of an outstanding importance for the analysis of the dam behaviour and the selection of the most important indicators.

The **"Technical Regulation for Dam and Reservoir Safety"** approved in 1996 by the former Ministry of Public Works, Transport and Environment, established a series of additional obligations for the dams owned by the state. The most relevant obligations referring to monitoring and inspection are:

• To elaborate and apply a coordinated plan for monitoring and periodic inspections of the dam and the reservoir, orientated on the verification of its safety and functional state. The plan must define both the scope and frequency of the inspections and the composition of the installed equipment for recording monitoring data, indicating the record frequency of each sensor, specifications referred to information collection and processing and its interpretation method.
• To prepare an annual report with the inspection and monitoring results, an analysis of observed deficiencies and proposals for adequate remedial measures. This report must be redacted by the responsible dam operation engineer.

• To carry out a detailed inspection of the dam and its installations, including accesses and communications after an extraordinary incident like seismic activity, abrupt changes of the reservoir level, important discharges, landslides in the reservoir or others.

In addition to these inspections, the owner is obliged to ɔarry out periodical revisions and a general analysis of the ɪfety of the dam and the reservoir. This revision has to be ıade by technical specialists that are not included in the peration team. The intervals for general revisions are 5 ɜar for dams belonging to category A and 10 years for ategory B and C.

2. STANDARD OF KNOWLEDGE ABOUT ꟷE BEHAVIOUR OF SPANISH DAMS

.lthough, as shown above, the **"Technical Regulation for ıam and Reservoir Safety"** is only applicable to state-ʼwned dams and new constructions, the owners of none tate-owned dams (two-thirds of all) are obliged by the asic Directive of Civil Protection to both make a proposal ıf the classification due to potential risk of a failure and to ·laborate an Emergency Plan for the dams belonging to :ategory A and B. The latter has to include a safety analysis ıf the dam defining in quantitative and qualitative terms ʼalues or limit states from which phenomenons and ınomalies could be dangerous. For the definition of these mits the history of the dam behaviour including the nonitoring data, natural phenomenons (floods and seismic ɔctivity) and parameters and regulations that are ıdicated in the operation rules, have to be taken into ɔccount.

Until now Emergency Plans have been elaborated ɔractically for all state-owned dams with category A or B ɔnd approximately for 50 % of the none state-owned dams ʋith category A or B.

At the same time and in the case of the state-owned dams, detailed inspections were carried out in order to evaluate the situation of the dam safety, indicating deficiencies or detected insufficiencies and, if necessary, suggest actions to maintain the dam safety. Depending on the category of the dam, these detailed inspections have to be performed in intervals of 5 or 10 years. At present, ten years after establishing the **"Technical Regulation for Dam and Reservoir Safety"** practically all state-owned dams provide their first security check.

As mentioned above, the first dam safety regulations in Spain were established in the year 1967, the second, the program for Dam Safety, in 1983, the third was published in the year 1995 and the fourth in 1996. Recently a draft document for a future law about Dam Safety has been formulated and is to be approved by the Spanish Parliament. In consequence, it can be affirmed that the situation in Spain due to the analysis, methodology, equipment and formation of qualified technicians in dam safety is on a similar level as in other developed countries.

3. MONITORING EQUIPMENT AND SYSTEMS

3.1. MONITORING EQUIPMENT AND SYSTEMS MOST COMMONLY USED IN SPAIN

For monitoring dams and their foundations various aspects are taken into consideration:

• Type of dam and geometrical characteristics.
• Geological and geotechnical properties of the foundation.
• Whether it is an old dam or a new one to be built.

The aforementioned Technical Guideline N° 7, "MONITORING OF DAMS AND THEIR FOUNDATIONS",

TABLE 1: SYSTEMS AND SENSORS USED FOR MONITORING DAMS AND THEIR FOUNDATIONS.

SYSTEM OR TYPE OF MONITORING	PROVIDED PARAMETER OR MAGNITUDE	SENSOR OR MEASUREMENT INSTRUMENT	PRECISION, SENSIBILITY AND MINIMUM RANGE P	S	R
HYDRAULIC	Discharge	Gauging stations and rating curves	SN	SN	SN
	Seepage	Gauging and/or flowmeter	± 1/100 R	1/100 R	SN
	Pore pressures or interstitial pressures	Piezometer (vibrating wire)	± 1/800 R	1/800 R	SN
	Piezometric levels	Standpipe	± 5 mm. c.a.	2.5 mm. c.a.	SN
DEFORMATION	Absolute displacements				
	- Horizontal	Inverted pendulum Geodesic triangulation	± 0.1 mm. ± 0.3 mm.	0.01 mm. 0.05 mm.	SN SN
	- Vertical	Geodesic levelling	± 0.2 mm.	0.05 mm.	SN
	Rotations	Inclinometer or clinometer	± 10-5 Ra	10-5 Ra	SN
	In joints or cracks	Differential extensometers (wire or rod)	± 0.1 mm ± 0.02 mm.	0.05 mm. 0.01 mm.	SN SN
	In geological faults	Long distance extensometers	± 0.1 mm.	0.01 mm.	SN
	Foundation	Differential extensometers (wire or rod)	± 0.01 mm.	0.01 mm.	SN
STRESS-STRAIN	Stress transmitted to the foundation	Total pressure cells (direct method)	± 1 % R	± 1 % R	0 - 60 kp/cm²
		Extensometers for measuring unitary deformations in areas close to the foundation (indirect method)	± 1 _ / m.	± 1 _ / m.	± 1.500 _ / m.
SEISMIC (in medium and highseismic activity areas)	Accelerations in foundations	Accelerograph	± 0.01 g	± 0.005 g	0 - 1 g
	Earthquakes induced by the reservoir	Seismograph	10 - 4 mm / s	10 - 4 mm / s	0.02 mm / s
	Acoustic emission	Acoustic wave detectors	-	46 pc / g	15,200 kHz
HYDROLOGIC	Precipitation (Rain – snow)	Rain gauge	0.3 l / m²	0.1 l / m²	SN
		Snow gauge	0.3 l / m²	0.1 l / m²	SN
		Meteorological radar	According to the type of rain		
	Water level in reservoir	Limnigraph	< 1 cm	< 1 cm	SN
METEOROLOGIC	Air temperature, wind speed and direction				
	Evaporation, humidity, solar radiation	Meteorological station	According to National Meteorological Institute norms		

P = accuracy; S = Sensibility; R = Range; Ra = Radian; SN = according to necessities; INM = National Meteorological Institute commends the variables to be checked and indicates e type of sensors to be used and its necessary precision able 1). The guide also includes other tables that classify ecific instrumentation according to the dam type: oncrete dams, distinguishing between gravity dams, arch ams and gravity arch dams, and embankment dams. This uideline also includes recommendations about how to nplement measurement and observations programs, and iggests recording frequencies depending on the type of am and its age.

2. MONITORING OF OLD DAMS

e main feature of the Spanish climatologic conditions, specially in the Mediterranean area and in the south, is e scarcity of annual rains (from 200 to 400 mm), which addition are very irregularly distributed in space and me. Periods without rain are especially long during the pringtime and in the summer. As a result, there is a long adition of hydraulic engineering structures, especially ams and channels, and more than half of the dams in pain were built more than 35 years ago. Many of these ld dams do not have monitoring devices or existent evices are insufficient, inadequate or useless. As onsequence of the actual dam safety regulations new ontrol systems are being implemented in these dams, llowing the analysis of their behaviour and an estimation f existing safety coefficients.

. DATABASE MANAGEMENT AND ANALYSIS TOOLS. TATISTICAL AND DETERMINISTIC MODELS

n Spain, automatic acquisition of monitoring data tarted about 20 years ago. The number of dams with utomatic systems has grown every year ever since. At e same time, computer tools for collecting, verifying, rocessing and analysing data have developed to a great extent. Admitted variation ranges for control variables are usually established in the safety analysis carried out in the framework of emergency plans.

Most commonly used analysis tools are deterministic and statistical models. Deterministic models usually consider finite elements in accordance to stress-strain laws that concern the foundation materials and the dam main body. These models are used for verifying that the behaviour that has been observed corresponds to the project hypothesis, and they are usually complex to develop, since they require a very accurate definition of the geometry and characteristic parameters of the materials that constitute the structure as well as the abutments and foundation.

Statistical models, which correlate the structure behaviour variables and external factors, as for example the water level of the reservoir and the environmental temperature, have been increasingly used during the last years. Their drawing up is not as complex as for the deterministic models, and they make forecasting of future behaviour easier for different load situations. Validated and long-term -preferably over ten years- monitoring data series are needed in order to achieve reliable results. Referring to forecasting future behaviour, it has to be considered that any change in dam operation or any particular incident can modify correlations. In that case, a new analysis would be necessary. For example, the graph in figure 1 shows an abrupt alteration in radial displacements recorded in an arch dam block as a result of the works in a hydropower station close to the dam.

Behaviour models for Spanish dams tend to be hybrid models that combine the advantages of deterministic models, generally more complex, and statistical models, which can reproduce thermal effects on the structure. Most important dams due to their size, potential risks, etc., have master systems that automatically collect and transmit data to a control centre where sudden variations and anomalies can be detected in real time by using behaviour models.

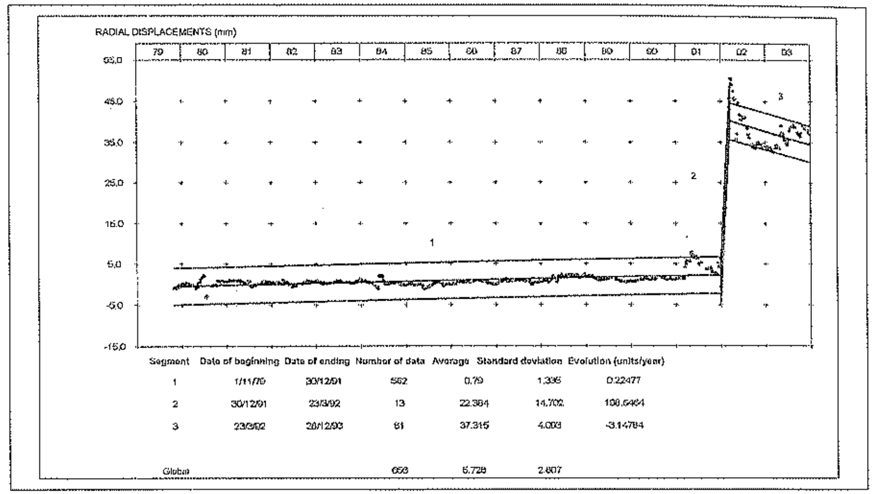

Fig. 1. Radial Displacements Recorded in La Baells Dam

5. CASE STUDIES

In this chapter, some examples of the information obtained throughout monitoring systems are shown for four different dam types: gravity arch dam, arch dam, embankment dam and RCC dam.

5.1. LA ACEÑA DAM (GRAVITY ARCH DAM)

La Aceña Dam (see figure 2) is an arch gravity structure, and belongs to the Madrid water supply system operated by the company Canal de Isabel II. The study of its behaviour has been divided according to chronological order and the source of the readings.

a) First impounding (February 1999 to May 2001)
The instrumentation and monitoring system of the dam was aimed to control displacement, joint performance stresses and seepage. The analysis of data recorded during first impounding resulted in the following outcomes:

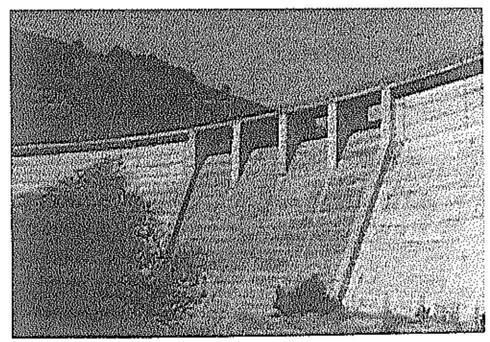

Fig. 2. La Aceña Dam.

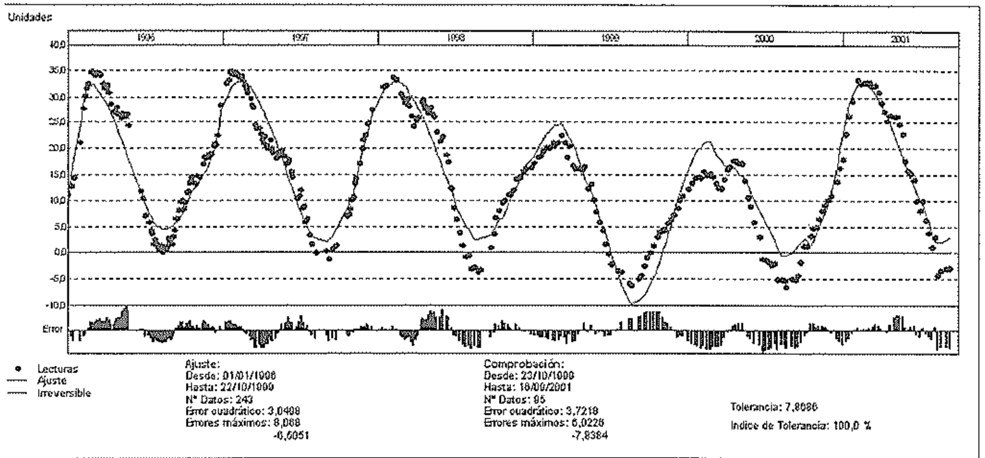

Fig. 3. Results obtained from the Statistical Model.

• Maximum displacement toward the abutments (tangential) was 3.75 mm, reached when the reservoir was at maximum normal operating level.

• Radial displacements (upstream to downstream) were in a range between 1.23 cm and 1.8 cm, with slight differences between blocks. Maximum joint movement recorded was 1.88 mm.

• Maximum seepage flow (40 l/min) also occurred for the maximum normal operating level. The maximum value for a single block was 1.7 l/min.

b) Movements after first impounding

Movements recorded after first impounding by means of four plumb lines revealed an apparently significant increase of downstream displacements (around 4 cm).

c) Summary

Despite the difficulties involved in making equivalent comparisons between first impounding and other load cycles, the behaviour trend of the dam does not seem to have changed during the years. Finally, from statistical and deterministic implemented models, some extra considerations can be pointed out:

• The statistic analysis of instrumentation records (plumbline records versus water level and temperatures) has shown that these two external variables are strongly correlated with the movements of the dam (see figure 3).

• The deterministic (conceptual) model implemented still shows how, due to an increase of water level, downstream movements should not be larger than a few millimetres and, for a 15 Celsius Degrees temperature gradient, should not be larger than two centimetres (see figure 4).

5.2. JOSÉ TORÁN DAM (ARCH DAM)

José Torán Dam (see figure 5) was built between 1985 and 1991 in the Guadalbarcar river located in Lora del Río (Seville, Spain). It was first impounded during the summer of 1991.

José Torán is an arch dam with three centres and double curvature and was built of conventional concrete with a dosage of 250 Kg/m^3. It is height is 77 meters above its foundation.

The foundation consists of a heterogeneous mixture of Palaeozoic rocks with quartzite and slate in variable proportions.

The dam has a very complete monitoring system that has been recently automated.

The data obtained by the monitoring system were analysed and point out the following conclusions:

• No important behaviour anomalies were detected during the construction phases, the first complete impounding and the subsequent ones.

• After the injection of the dam shrinkage joints and its first impounding, the structure behaviour has been monolithic and the joint opening movements have been very limited. The effects of the variation of hydrostatic loads have been less important than the effects of seasonal temperature variations.

• The structure has suffered an acceptable deformation asymmetry that can be observed in a larger horizontal displacement towards the right abutment.

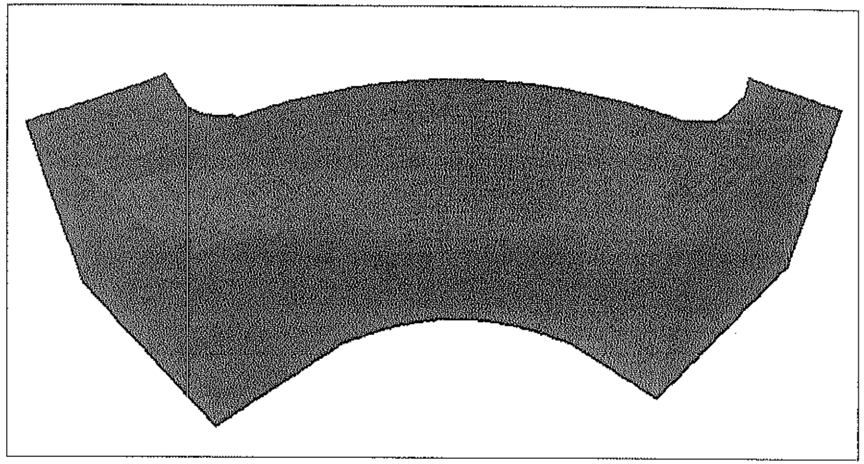

Fig. 4. Mesh of the three-dimensional deterministic Finite Element Model.

Fig. 5. José Torán Dam.

Fig. 6. Tous Dam.

• The dam foundation behaviour, which was determined by the measurements made with rod extensometers, has also been satisfactory.
• Pore pressures recorded in the right bank of the dam are slightly higher than those in the left bank, although in all blocks they are clearly lower than the values provided by the law commonly applied to dams with continuous drainage curtain.
• Seepage flow measured inside the dam is very limited, and most of it is due to the drainage. With high water level, these do not exceed 152 l/min. Seepage water measured on the right bank is approximately 2.3 times bigger than on the left one.

.3. TOUS DAM (EMBANKMENT DAM)

Iew Tous dam (see figure 6) on the Jucar river in Spain is ı 140 meter high rockfill dam with a clay core. It was onstructed between 1991 and 1996 at the place of the ıld dam that failed in 1982.

Instrumentation data (vertical and horizontal ıovements, pore pressures, total pressures, etc.) have ıeen collected since the beginning of the construction until now. The water level (real static load) has been recorded on a daily basis. Although no real significant earthquake has been recorded at the site, design earthquakes up to ten thousand years of return period have been calculated.

Static stress-strain models (see figure 7) and water effects have been modelled and calibrated with instrumentation data. In fact, initial conditions to run the dynamic model have been settled from such works that have included the simulation of construction, creep and wetting effects on strains, stresses and pore pressure distribution.

From all undertaken tasks during the last years, the behaviour of the dam has been characterized as remarked bellow:

• Core seepage behaviour differs from implemented conceptual model, and it seems that the steady-state regime has not been reached yet.
• Piezometer records show a good behaviour (as expected) of the saturated core.
• Foundation seepage records might be related to the existence of some layers significantly more permeable than others.
• The statistical model has shown how seepage variability is not completely explained from water level and rain records.
• On the other hand, movements are in general so small that the dam can be characterized as much stiffer than expected.
• Lastly, the dam has reached stability with regard to creep movements according to Dascal Criteria.

5.4. RIALB DAM (ROLLER COMPACTED CONCRETE DAM)

Rialb Dam (see figure 8) was built between 1992 and 1999, and its first filling took place in April 1999. This impounding has not finished yet, although the dam is being normally operated. It was built on the Segre river, in Lleida (Spain).

The dam is a straight-lined gravity structure, and is made of roller compacted concrete (RCC). It has a combined gated and ungated spillway. Its elevation over foundation is 101 meters, the crest length is 595 meters and the total RCC volume 1,050,000 m^3.

The rocks on the dam site date from the Stampien era, which is part of the Upper Oligocene. These materials consist of alternate sandstone, marl and calcareous lime strata.

The dam was built at the end of a meander, where the river section has the shape of a dissymmetric trough having a sharper right bank than the left one.

The dam has a very complete monitoring system, with automated measurement devices that transmit monitoring data in real time to the control centre of the technicians in charge of the dam operation work located in Zaragoza, Spain.

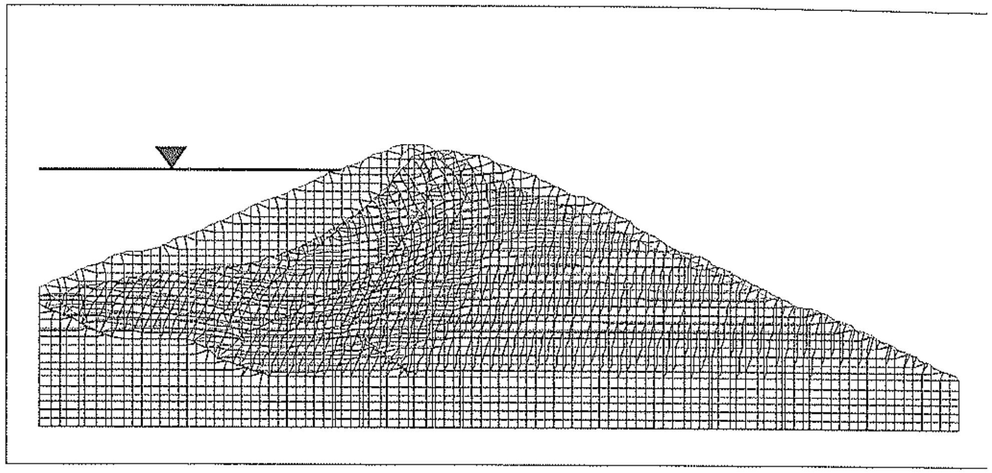

Fig. 7. Results obtained from the Dynamic Finite Element Model due to Seismic Loads (Design Earthquake).

From the monitoring data, as well as from othe observations being made, it can be drawn the conclusion that the structure and its foundation behaviour has been good in general to date, since nc important anomalies showing excessive settlement oı

Fig. 8. Rialb Dam.

…portant filtrations have been detected. However, it …st be outlined that:

- During the dam construction processes and its subsequent partial impounding it could be seen that the real deformability of the foundation was twice as much as expected in the original design.
- In general, the pore pressures recorded in the dam are quite low due to the efficiency of the injections and drainage in the foundation. Nevertheless, the hydraulic sealing of the high blocks on the left bank has not been completely satisfactory, since pore pressures in these blocks are high. Nevertheless, they do not represent a risk.
- On 21st September 2004, a 4.5 degrees on the Richter scale earthquake with its epicentre just 110 kilometres away from the dam was detected. Immediately after this incident, a general series of readings were carried out with the monitoring system, as well as an inspection of the structure and its entirety. In the end, no mentionable earthquake effects were observed.
- With the partial impounding, absolute displacements of minor importance in the structure and a slight rotation in the downstream direction were measured. The biggest settlement that was measured in the dam since it was finished has been 3 mm.

…URRENT RESEARCH AND PRESENT SITUATION

… DAM BEHAVIOUR AND RISK ANALYSIS

…er ten years of new dam safety regulations in Spain …t has promoted lots of instrumentation projects and dam behaviour analysis, an important number of lessons have been learnt. Nevertheless, a more recent approach undertaken in the context of risk analysis and risk assessment, might allow the dam engineers to focus on what really matters about instrumentation and measurements and decide about the need of new instruments at their site or remedial works in case of any problems detected.

Typically, risk analysis implies a series of tasks such as: information gathering and review, load events identification, failure modes identification, system response characterization, and consequence estimation. In such context, instrumentation, monitoring and subsequent behaviour study by means of such records and numerical models, become extremely relevant to perform risk analysis, a very valuable tool to complement classical dam safety approaches.

6.2. APPLICATION OF NEW TECHNOLOGIES

The most important innovations that were achieved during the last years concern collection and transmission of both local and remote monitoring data. Nowadays, standard equipments consist of centralised automatic systems with electronic units for data acquisition used also in other industry branches, mainly in the field of industrial process control. Moreover, fibre optics are used in order to avoid problems in the case of over voltage due to electrical storms. In that way, more reliable and solid systems could be achieved. The trend goes to an increased application of standard products and solutions from the market in order to simplify system compatibility and reduce costs.

With regard to instrumentation, we can underline the increasing use of optical fibre as a sensor element for temperature, deformation and seepage measurement.

6.3. INTEGRATION OF MONITORING DATA TO THE AUTOMATIC HYDROLOGIC INFORMATION SYSTEMS (SAIHS)

At the beginning of the 80's an ambitious program started in Spain with the aim of implementing hydrometeorological control nets in order to get real time information about precipitations, water levels in rivers, channels and reservoirs, and operation conditions of dams (volume of in and outflow, valves and gates positions, etc.). Nowadays, in most Spanish hydrographical basins exists such a system which is called Automatic Hydrologic Information System (SAIH). In general, it can be profited by the SAIHs communication systems in order to transmit dam safety information (for example information used for emergency plan protocols) and to centralise data coming from monitoring systems. A good example is the Ebro river, the largest hydrological basin in Spain: its control centre in Zaragoza automatically receives monitoring data from 40 dams through the SAIH communication net.

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RISK ANALYSIS AND RISK EVALUATION FOR THE MORA DE RUBIELOS COFFERDAM (TERUEL, SPAIN) (*)

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(*) *Évaluation et analyse du risque appliquées au batardeau de Mora de Rubielos (Teruel, Espagne).*

1. INTRODUCTION

The need for implementing a risk analysis technique as a decision support tool for dam safety management emerged in the early nineties in some of the most developed countries in the world. In a context of dam maintenance requirements, improving operating procedures and increasing regulation, estimating different types of risk (structural, operational, etc.) becomes a crucial need. Even more, the identification of tolerable risk levels (both related to the dam-reservoir system and water supply) should be an available tool for decision makers (Ref. [8]).

Nowadays in Spain, traditional dam safety assessment based on the pseudo-probabilistic load hypothesis and partial safety factors cannot respond to the mentioned questions. In fact, despite the fact that design floods and earthquakes incorporate probability related to their selected annual exceedence probability (AEP or 1/return period), the probability of occurrence of such a loading condition is not accounted for and maybe much rarer than the AEP of the design event itself if the prior reservoir level is assumed to be the maximum normal operation level. In addition, partial safety factors lead to acceptance/non acceptance criteria but cannot provide a sense of how close failure is.

One of the aims during these first years has been ensuring the consistency of the analysis, so that methodologies should allow to prioritize corrective actions for any group of dams that have been analyzed in a consistent manner. The majority of developed methodologies and lessons learned have been shared and published in different meetings, courses, workshops and symposiums.

2. SCOPE OF THE WORKS

The authors have reviewed existing documents and works all over the world concerning risk assessment applied to dam safety, in particular studies developed by Dr. David Bowles (Refs. [2] and [3]) and methodologies implemented by the Bureau of Reclamation (USBR, USA) and recommended by the Australian National Committee on Large Dams (ANCOLD, Australia). The work herein presented is related to the Mora de Rubielos cofferdam application of such techniques with some contributions on calculation procedures.

The Mora de Rubielos cofferdam (Teruel, Spain) is an earth fill structure 9 meters high, with an upstream slope 2:1 (horizontal to vertical) and a little steeper downstream slope (1.8:1). The dam lays on marl limestone foundation and its body material has been classified as GC.

The information used to perform the risk analysis and evaluation has included the Constructional Project, Foundation Treatment specific projects and monthly reports on construction of the Mora de Rubielos dam and cofferdam.

3. RISK ASSESSMENT

3.1. SCENARIOS, FAILURE MODES AND GENERAL EVENT TREE

As the structure is just a provisional dam to derive the river flow during a few weeks and whose storage capacity is very small, the methodology has been simplified (Fig. 1) in order to efficiently apply a useful tool (Ref. [9]).

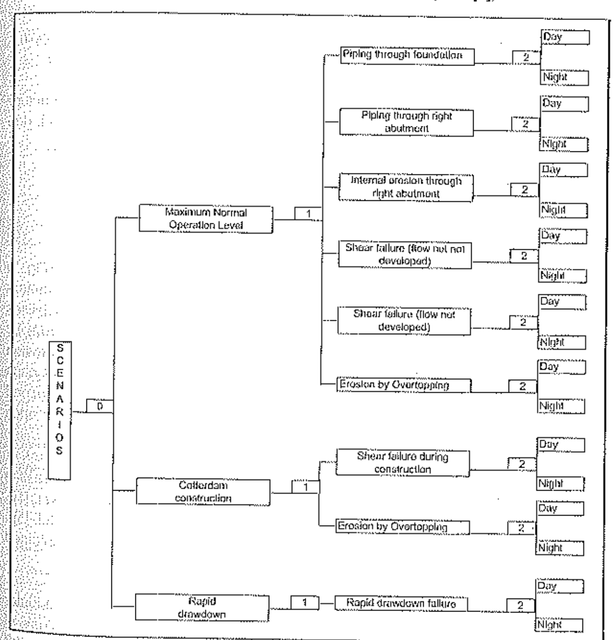

Fig. 1
General event tree
Arbre d'événement général

As shown in Fig. 1, there is no seismic scenario considered as the dam is located at an area of "low seismicity" (horizontal acceleration for 500 years lower than 0.04 g). In addition, hydrologic and static events have been unified and three scenarios distinguished: Maximum Normal Operating Level (for 4 and a half weeks that the cofferdam will be useful), Construction of the cofferdam (4 weeks) and Rapid Drawdown (only possible by pumping).

Failure modes identified for the first scenario (Maximum Normal Operating Level) are foundation piping, piping starting at the contact between the fill and the outlet concrete structure, internal erosion through the contact between the fill and the outlet structure, shear failure with a permanent flow net developed, shear failure right after construction before a flow net is developed and overtopping that erodes the dam.

Failure modes identified for the second scenario (Construction) are shear failure associated to a pore pressure increase during compaction and overtopping. Third scenario (Rapid Drawdown) includes the rapid drawdown mode of failure.

Finally, the general Event Tree used as risk model to analyze the structure includes only two different categories of population exposure (day and night), due to the short period of time that the structure will be operated.

3.2. PROBABILITY OF FAILURE

For all shear failure related modes, a numerical model built in FLAC (Fig. 2) has been used to reproduce the dam behaviour (Ref. [6]). Material parameters have been taken from the reviewed information and the conditional probabilities have been estimated making use of the second order lineal theory (FOSM) by means of Taylor Series. Failure modes related to piping and internal erosion have been analyzed by event trees, where probabilities are estimated at each node by expert judgment (Ref. [7]). For overtopping failure mode, a second simplification has been made with the assumption of failure occurring once overtopping starts.

Table 1 summarizes the total probabilities of failure (while the structure is in use) for each scenario, as recommended by ANCOLD guidelines (Ref. [1]). Table 2 summarizes the probabilities as recommended by the USBR (Ref. [5]). Herein, instead of considering failure modes as no mutually exclusive, obtained probabilities during cofferdam's service time have been simply added.

As it is shown in the tables, overtopping is clearly the maximum contributor to the total probability of failure. Also, the first and second methodology of organizing the probability of failure results are for this case giving a very similar magnitude of the total probability, due to the great importance of the hydrological event in this case.

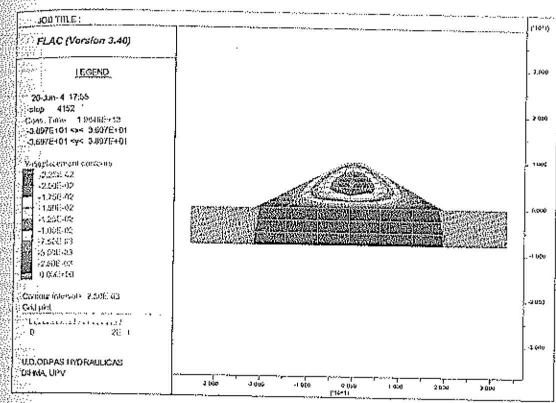

Fig. 2
Numerical model in FLAC (part of a reliability model)
Modèle numérique en FLAC (partie d'un modèle de fiabilité)

Table 1
Probabilities of failure (ANCOLD criteria)

SCENARIOS	% TIME IN EACH SCENARIO	FAILURE MODE	CONDITIONAL PROBABILITY OF FAILURE			CONDITIONAL PROBABILITY OF FAILURE FOR	OVERALL SERVICE TIME PROBABILITY OF FAILURE
			Lower Limit (L)	Mean	Upper Limit (U)		
Maximum Normal Operation Level	5.29E-01	Piping through foundation	1.00E-11	2.49E-08	5.00E-04	4.32E-03 (U) 4.32E-03 (L)	2.29E-03 (U) 2.28E-03 (L)
		Piping through right abutment	1.00E-15	2.00E-10	5.00E-07		
		Internal erosion through right abutment	1.00E-16	2.00E-11	1.00E-07		
		Shear failure with flow net not developed	1.00E-12	2.02E-10	1.00E-06		
		Shear failure with flow net developed	1.00E-15	5.58E-13	1.00E-08		
		Erosion by overtopping	5.00E-04	4.32E-03	2.00E-02		
Cofferdam Construction	4.71E-01	Shear failure during construction	1.00E-10	6.46E-08	1.00E-05	3.84E-03 (U) 3.84E-03 (L)	1.81E-03 (U) 1.81E-03 (L)
		Erosion by overtopping	5.00E-04	3.84E-03	2.00E-02		
Rapid Drawdown	2.49E-06	Rapid drawdown failure	1.00E-05	1.33E-03	1.00E-02	1.33E-03	3.33E-09
						OVERALL SERVICE TIME PROBABILITY OF FAILURE	4.09E-03 (U) 4.09E-03 (L)

Table 2
Probabilities of failure (USBR methodology)

SCENARIOS	% TIME IN EACH SCENARIO	FAILURE MODE	COND. PROBABILITY OF FAILURE			PROB. OF FAILURE FOR SCENARIO		
			Lower Limit (L)	Mean	Upper Limit (U)	Lower Limit (L)	Mean	Upper Limit (U)
Maximum Normal Operation Level	5.29E-01	Piping through foundation	1.00E-11	2.49E-06	5.00E-04	5.29E-12	1.32E-06	2.65E-04
		Piping through right abutment	1.00E-15	2.00E-10	5.00E-07	5.29E-16	1.06E-10	2.65E-07
		Internal erosion through right abutment	1.00E-16	2.00E-11	1.00E-07	5.29E-17	1.06E-11	5.29E-08
		Shear failure with flow net not developed	1.00E-12	2.02E-10	1.00E-06	5.29E-13	1.07E-10	5.29E-07
		Shear failure with flow net developed	1.00E-15	5.58E-13	1.00E-09	5.29E-16	2.96E-13	5.29E-10
		Erosion by overtopping	5.00E-04	4.32E-03	2.00E-02	2.65E-04	2.28E-03	1.06E-02
Cofferdam Construction	4.71E-01	Shear failure during construction	1.00E-10	6.46E-08	1.00E-05	4.71E-11	3.04E-08	4.71E-06
		Erosion by overtopping	5.00E-04	3.84E-03	2.00E-02	2.35E-04	1.81E-03	9.41E-03
Rapid Drawdown	2.49E-06	Rapid drawdown failure	1.00E-05	1.33E-03	1.00E-02	2.49E-11	3.33E-09	2.49E-08
OVERALL SERVICE TIME PROBABILITY OF FAILURE						5.00E-04	4.09E-03	2.03E-02

3.3. Consequences

Potential hazard is very influenced by the fact that the dam is located very close to an urban centre (Mora the Rubielos). On the other hand, the severity of the flood calculated for all cases is such that a peak flow of 300 m^3/s can be expected between the dam and Mora the Rubielos town. In addition, the river geomorphology implies high velocities and water depths where houses closer to the river are located. Table 3 shows results on life loss estimation for each scenario and failure mode, making use of USBR recommendations (Ref. [4]).

Table 3
Life loss estimation

SCENARIOS	FAILURE MODE	LIKELY LOSS OF LIFE - NIGHT (14 h)			LIKELY LOSS OF LIFE - DAY (10 h)		
		Lower Limit (L)	Mean	Upper Limit (U)	Lower Limit (L)	Mean	Upper Limit (U)
Maximum Normal Operation Level	Piping through foundation	0.0	0.0	0.0	0.0	0.0	0.0
	Piping through right abutment	0.0	0.0	0.0	0.0	0.0	0.0
	Internal erosion through right abutment	0.0	0.0	0.0	0.0	0.0	0.0
	Shear failure with flow net not developed	1.2	6.0	14.0	0.4	1.8	4.2
	Shear failure with flow net developed	1.2	6.0	14.0	0.4	1.8	4.2
	Erosion by overtopping	0.0	0.0	0.0	0.0	0.0	0.0
Cofferdam Construction	Shear failure during construction	1.2	0.0	14.0	0.4	1.8	4.2
	Erosion by overtopping	0.0	0.0	0.0	0.0	0.0	0.0
Rapid Drawdown	Rapid drawdown failure	1.2	0.0	14.0	0.4	1.8	4.2

3.4. Risk Estimation

Risk has been estimated for each scenario and also related to each failure mode separately, according to the same methodologies used for probability of failure results. It is important to remark how incremental individual risk results are again similar for both methodologies (Ref. [1] and [5]) due to the importance of the overtopping failure mode (Tables 4 and 5).

Table 4
Risk estimation (ANCOLD criteria)

SCENARIOS	% TIME IN EACH SCENARIO	FAILURE MODE	COND. PROB. OF FAILURE	LIKELY LOSS OF LIFE (mean)	INCREMENTAL RISK	INCREMENTAL RISK SCENARIO
Maximum Normal Operation Level	5.29E-01	Piping through foundation	2.49E-06	0.0	0.00E+00	4.55E-10 (lives/cofferdam's service time)
		Piping through right abutment	2.00E-10	0.0	0.00E+00	
		Internal erosion through right abutment	2.00E-11	0.0	0.00E+00	
		Shear failure - flow net not developed	2.02E-10	4.3	4.54E-10	
		Shear failure - flow net developed	5.58E-13	4.3	1.26E-12	
		Erosion by overtopping	4.32E-03	0.0	0.00E+00	
Cofferdam Construction	4.71E-01	Shear failure during construction	6.46E-08	4.3	1.29E-07	1.29E-07 (lives/cofferdam's service time)
		Erosion by overtopping	3.84E-03	0.0	0.00E+00	
Rapid Drawdown	2.49E-06	Rapid drawdown failure	1.33E-03	4.3	1.41E-08	1.41E-08 (lives/cofferdam's service time)
TOTAL INCREMENTAL RISK						1.44E-07 (lives/cofferdam's service time)

Table 5
Risk estimation (USBR methodology)

	Failure modes	Conditional Probability of Failure			Potencial Loss of Life Estimate			Total Incremental Risk Estimate		
		Lower Limit	Mean	Upper Limit	Lower Limit	Mean	Upper Limit	Lower Limit	Mean	Upper Limit
1	Piping through foundation	5.29E-12	1.32E-06	2.65E-04	0.0	0.0	0.0	0.00E+00	0.00E+00	0.00E+00
2	Piping through right abutment	5.29E-16	1.06E-10	2.65E-07	0.0	0.0	0.0	0.00E+00	0.00E+00	0.00E+00
3	Internal erosion through right abutment	5.29E-17	1.06E-11	5.29E-08	0.0	0.0	0.0	0.00E+00	0.00E+00	0.00E+00
4	Shear failure - flow net not developed	5.29E-13	1.07E-10	5.29E-07	0.9	4.3	9.9	4.50E-13	4.54E-10	5.25E-06
5	Shear failure - flow net developed	5.29E-16	2.96E-13	5.29E-10	0.9	4.3	9.9	4.50E-16	1.26E-12	5.25E-09
6	Erosion by overtopping	2.65E-04	2.28E-03	1.06E-02	0.0	0.0	0.0	0.00E+00	0.00E+00	0.00E+00
7	Shear failure during construction	4.71E-11	3.04E-08	4.71E-06	0.9	4.3	9.9	4.00E-11	1.29E-07	4.07E-05
8	Erosion by overtopping	2.35E-04	1.81E-03	9.41E-03	0.0	0.0	0.0	0.00E+00	0.00E+00	0.00E+00
9	Rapid drawdown failure	2.49E-11	3.33E-09	2.49E-08	0.9	4.3	9.9	2.12E-11	1.41E-08	2.47E-07
	Total	5.00E-04	4.09E-03	2.03E-02				6.16E-11	1.44E-07	5.22E-05

3.5. Risk Evaluation

Social criteria developed by USBR and ANCOLD have been adapted to perform the risk evaluation (Fig. 3 and 4) despite the fact that such criteria have been developed for permanent structures and not for cofferdams. As shown through the charts, Mora de Rubielos cofferdam clearly satisfies both types of criteria for risk evaluation, taking into an account that a maximum combined probability of failure pf 1 in 100 (during service time) has been adopted here.

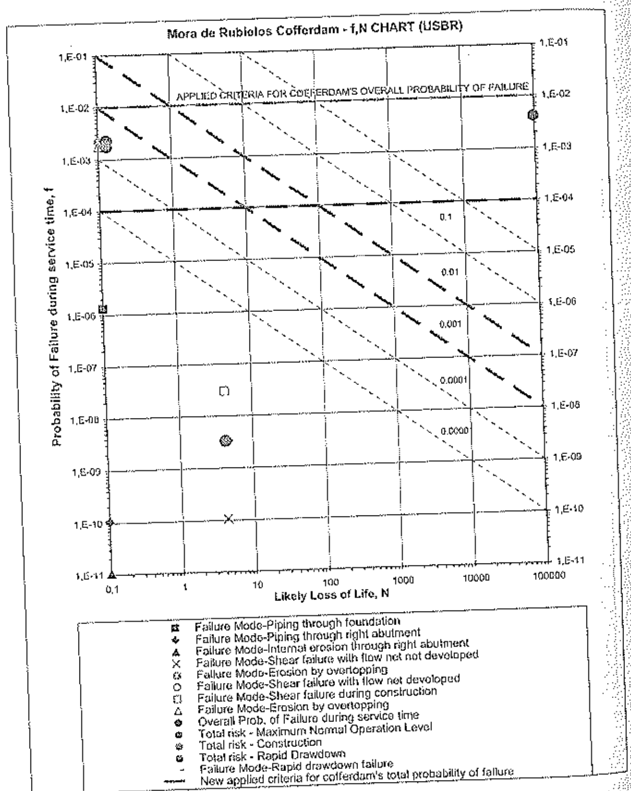

Fig. 3

Risk evaluation summary results (USBR methodology)

Résumé des résultats de l'évaluation des risques (méthodologie USBR)

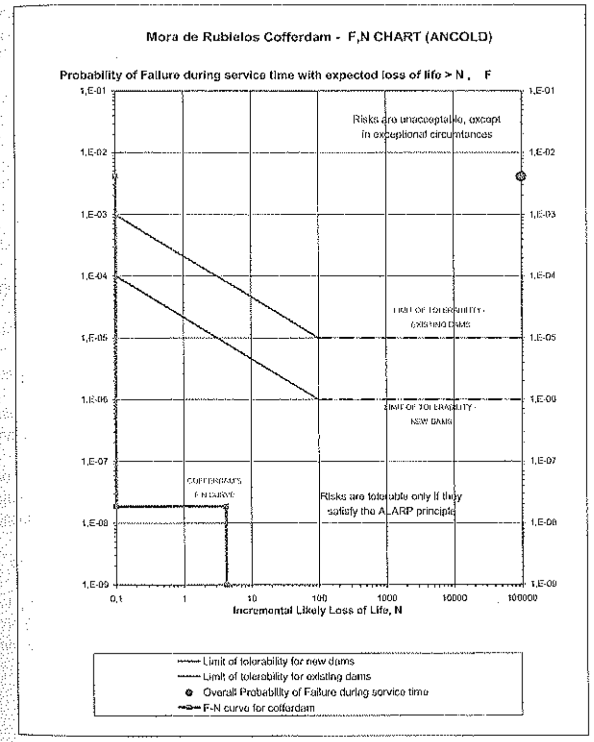

Fig. 4

Risk evaluation summary results (ANCOLD criteria)

Résumés des résultats de l'évaluation des risques (critères ANCOLD)

ACKNOWLEDGEMENTS

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SUMMARY

After reviewing existing documents and works from all over the world concerning risk assessment applied to dam safety, in particular the studies developed by Dr. David Bowles and methodologies implemented by the Bureau of Reclamation (USA) and recommended by ANCOLD (Australia), the authors present a Spanish case study on risk analysis and risk evaluation applied to an earthfill cofferdam, including some contributions on calculation which, hopefully, will contribute to future studies being developed in Spain.

RÉSUMÉ

Suite à une recherche mondiale des documents et des travaux concernant l'analyse des risques appliquée à la sécurité des barrages, notamment les études développées par le Dr. David Bowles et les méthodologies mises en pratique par le Bureau of Reclamation (USA) et ANCOLD (Australie), les auteurs présentent dans ce rapport un exemple espagnol d'évaluation et d'analyse du risque appliquées à un batardeau en terre, avec des contributions sur les calculs qui, on l'espère, feront progresser des études futures en Espagne.

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REVISED CRITERIA FOR EVALUATING GRANULAR FILTERS IN EARTH AND ROCKFILL DAMS (*)

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1. INTRODUCTION

Internal erosion is one of the most important causes of failure of embankment dams. The placement of downstream filters is considered the best way to prevent continuing erosion in dams. The trust in these filters is such that Sherard & Dunnigan (1985) affirm that: "*by providing conservative downstream filters, we can quit worrying about possible concentrated leaks through the core*".

(*) *Critères révisés pour évaluer les filtres granulaires dans des barrages en terre et en enrochement.*

Filters are mainly designed using simply applicable empirical criteria. Such criteria have been proposed by many different researchers on the basis of correlations between different base soil and filter variables that produce a satisfactory behaviour if they are tested in laboratory under extreme conditions.

No dam designed in accordance with modern filtering requirements has suffered a severe failure incident (Fry *et al.*, 1997) and it is known that in several dams which suffered the initiation of internal erosion, filters were able to stop the process even though they did not satisfy current design criteria (Foster & Fell, 2001), however, it is also true that, at least in laboratory tests, some filters that were designed using these rules, have failed. This observation reveals a possible lack of knowledge about the accuracy and margin of safety included in the available design rules, caused by the enormous complexity of the internal erosion phenomenon and the high degree of uncertainty of the involved variables.

For this reason, the predictive quality of well known filter design criteria is exemplary analysed in the presented paper. A very large data set of laboratory filter tests undertaken by several authors in the past serves as the basis of this work. This data set is supplemented by the results of further laboratory testing performed at the University of Granada, Spain, which was aimed for best defining the variables that have a major influence on the base-filter behaviour against internal erosion.

2. SHORT REVIEW OF CURRENT CRITERIA

A very good review of the state-of-the-art of filters in dam engineering can be found in ICOLD Bul. 99 (1994) and Indraratna & Locke (1999). But one of the most interesting and recent studies about filters is the one conducted at the University of New South Wales, Australia, by Foster & Fell (1999). They analyse a large number of data from different sources (see section 3) and those obtained from an extended program of laboratory tests using NEF (No Erosion Filter) and CEF (Continuing-Erosion Filter) tests and propose the filter design criteria shown in Table 1[1]. Compared to the well known work of Sherard & Dunnigan (1989), Foster & Fell (2001) propose some new features:

- It is better to modify the subdivision for soil group 2 and soil group 4
- Some filters that were expected to fail according to Sherard's criteria have succeeded and some filters that were expected to succeed have failed

[1] *The notation used in this paper is as follows: D15F = particle size of the filter material for which 15% by weight is finer; D85B = particle size of the base material for which 85% by weight is finer; P0.075B = percent finer than a particle size of 0.075 mm (fines content of the base soil); kF = filter permeability.*

- The traditional ratio D15F/D85B is dominant but there seems to be an important influence of the clay content in the base soil on the test outcome
- There is a wide variation of D15F for No erosion Boundary, (see Table 1, column 4), so the proposed criteria, (see column 5), should not be taken as exact equations
- The retention criterion for soil Group 1 is valid only if the base soil is not dispersive
- New "Continuing-erosion" criteria, less strict than "No-erosion" criteria, are proposed but only as a help for assessing filters in existing dams, not for relaxing critical filter design, (see Foster & Fell, 2001).

Table 1
Summary of results of statistical analysis of the no erosion boundary of filter tests (Foster & Fell, 2001)

Base soil group	Fines content *(1)*	Design criteria of Sherard & Dunnigan (1989)	Range of D15F for no erosion boundary	Proposed criteria for no erosion boundary
1	≥ 85%	D15F ≤ 9 D85B	6.4 - 13.5 D85B	D15F ≤ 9 D85B
2	35 - 85%	D15F ≤ 0.7mm	0.7 - 1.7 mm	D15F ≤ 0.7 mm
3	< 15%	D15F ≤ 4 D85B	6.8 - 10 D85B	D15F ≤ 7 D85B
4	15 - 35%	D15F ≤ (40-P0.075B) x (4 D85B-0.7)/25 + 0.7	1.6 - 2.5 D15F of Sherard and Dunnigan design criteria	D15F ≤ 1.6 D15Fd, where D15Fd = (35- P0.075B)(4 D85B-0.7)/20+0.7

Notes: (1) The subdivision for soil group 2 and 4 was modified from 40% passing 75μm, as recommended by Sherard & Dunnigan (1989), to 35% based on the analysis of the filter test data.

As can be seen, there is a general agreement in the major aspects of filter design but there are some non solved questions such as:

- Almost everybody agrees with the idea that D15F/9 represents the effective opening size of the filter, but compaction of the filter is also very important and in fact permeability of the filter should better represent filter capability of retaining base particles (Vaughan & Soares, 1982)
- Even for base soil Group 1, percentage of fines and clay content should have a strong influence in the ratio D15F/D85B, possibly resulting in a variable ratio
- D85B is very representative but sometimes for some soils it is difficult to calculate, especially when sieve analysis and sediment analysis produce different results, other base soil variables could be more useful
- Empirical criteria depend on a (usually very limited) data set for performing regression analyses. Instead of proposing a specific design criterion as an

exact equation, variability of results should be also shown giving more information for expert judgement.

Trying to answer those questions and to improve filter design criteria, this paper pretends to compare the results of the filter research program conducted at the University of Granada (Delgado, 2000) with the ones analysed by Foster & Fell (1999).

3. UGR LABORATORY RESEARCH PROGRAM

3.1. TEST METHOD

Usually the study of stability of base-filter systems involves direct experimentation through laboratory tests. The most well-known of these tests is the No Erosion Filter Test (NEF test) proposed by Sherard & Dunnigan (1989).

In the current investigation the NEF test has been employed. The method is described in detail in Delgado and Locke (2000). Those tests performed under the same conditions as described by Sherard & Dunnigan (1989) are denoted as "STANDARD". In addition, to best determine the effect of several influencing variables, some tests have several variations: filter compaction, base soil moisture water content, water pressure and test duration (see Delgado, 2000).

3.2. SAMPLES TESTED

The filter material used in the tests was obtained from "El Portillo" dam (in the South of Spain). After washing on the 0.075 mm sieve it was split into distinct size fractions and re-blended to obtain certain linear particle size distributions, defined by the D15F and D100F sizes. Base materials tested were clays of low and high plasticity, generally of very fine grading (see Table 2). These materials represent a good spread over the two classifications of soil Groups 1 and 2 proposed by Sherard & Dunnigan (1989). In accordance with the generally applied procedure, all grain size distributions for base soils used in this paper refer to the fraction passing the 4.75 mm sieve.

Table 2
Characteristics of the base soils tested.

Soil No.	Name	Dam of origin	Index properties		Gradation		Pinhole dispersion
			LL	PI	% fines <75μm	D85B (mm)	
1	BBA-C1	Barbate	56	35	90.68	0.047	ND-1
2	BCA-C2	Canales	60	34	86.75	0.050	ND-1
3	BFA-C2	Francisco Abellán	32	14	67.75	0.606	ND-1
4	BJV-C1	J. del Valle, (project)	23	6	66.15	0.295	ND-1
5	BJV-C2	J. del Valle, (project)	31	12	90.76	0.052	ND-1
6	BJV-C5	J. del Valle, (project)	38	18	90.99	0.042	ND-1
7	BJV-C6	J. del Valle, (project)	30	12	88.88	0.061	ND-1
8	BJV-C7	J. del Valle, (project)	30	13	63.22	1.162	ND-1
9	BP-C3	Portillo	34	14	90.12	0.054	ND-1
10	BSC-C4	San Clemente	31	15	82.50	0.108	ND-1
11	BZA-C1	Zahara	66	33	93.94	0,004	ND-3

3.3. RESULTS

180 Standard NEF tests were done following the same specifications proposed by Sherard and Dunnigan (1985), but also 113 tests modifying filter compaction energy, 21 tests mixing additives in the base soil, 27 tests reducing water pressure, 46 tests modifying base soil water content and 23 more tests with several modifications.

In Section 4, quantitative analyses are performed, but here some other qualitative conclusions are presented:

- The lower the hydraulic gradient is, the coarser the boundary filter can be, so it is not sufficient to suppose that at high velocity, erosion always occurs and therefore the hydraulic gradient stops having an effect.
- The coarsest boundary filter is obtained for the base soil optimum moisture content, decreasing for both more and less moisture content.
- Base soil plasticity seems to have no relevant influence on the results.
- The addition of aluminium sulphate to the base soil tends to flocculate the clay and reduce dispersion, this permits the use of coarser filters. Regardless of possible practical applications, this observation confirms that the dispersivity of clays effects the functioning of the base soil – filter system.

4. GLOBAL DATA ANALYSIS

4.1. INTRODUCTION

Foster & Fell (1999) collected filter erosion test results from numerous published and unpublished sources. The resulting compiled data, together with the no erosion and continuing erosion filter tests performed by Foster & Fell (1999), is reduced to the data set used in the current analysis, only containing base soil Groups 1 & 2. Table 3 provides a listing of the adapted data used in the present study, giving basic information on test methods, source authors and the specific number of available data points for each category (see also Foster & Fell, 1999). The UGR data set (Delgado, 2000) with additional 272 data points is added to the table for the purpose of completeness.

Table 3
Data sources and filter test methods used in the analysis (only soil Groups 1 & 2)

Test type	Source[2]	#
Conventional base/filter tests	Sherard *et al.*	20
	USBR	4
Base suspension tests	Sherard *et al.*	68
	US Corps of Engineers	1
	Kenney *et al.*	5
Preformed slot tests	Sherard *et al.*	72
	Water Conservation and Irrigation Commission	26
No erosion filter tests	Sherard *et al.*	105
	Foster & Fell	47
	Delgado (UGR NEF tests)	340
Total size of data set for filter stability analysis		688

4.2. ALL SOURCES DATA AND UGR STANDARD NEF TESTS

Fig. 1 shows the results of the 348 erosion tests provided by external sources and 159 "standard" no erosion filter tests from Delgado (2000) presented in the context of the well known and widely accepted soil Group 1 & 2 filter design criteria by Sherard & Dunnigan (1989) which are defined as shown in Table 1. As can be seen, the basic separation between "*no failure*" (fulfilling the criteria) and "*failure*" region is qualitatively verified by the given data. Nevertheless a considerable mixing of test outcomes in either region can be clearly identified. This observation fits to the findings by Foster & Fell (1999) that

[2] *For the lack of space, the list of references contained in table 3 could be seen in Foster & Fell (2001).*

Sherard's criterion for soil Group 1 corresponds to a probability of failure of about 0.5 to 0.55 and is therefore not always conservative. Further it comes apparent that the fixed design criterion of D15F > 0.7 mm for soil Group 2 is not clearly separating the *no failure* and *failure* regions. Especially for coarser base soils this criterion becomes increasingly conservative, while for base soils close to the soil Group 1 no safety margin is remaining and, partially, an unsuitability of this criterion can be observed in Fig. 1 respectively. In this context, already Sherard & Dunnigan (1989) judge the design criterion of 0.7 mm for the D15 of the filter as not being excessively conservative. In addition to this, Foster & Fell (2001) observe a possible decrease of the Sherard's erosion boundaries in case of dispersive base soils. In fact, proposing a constant filter value, i.e. saying that the filter is independent of the base soil, is physically not plausible. Nevertheless, improvements concerning this criterion have only succeeded insofar as ranges of values are provided, e.g. as done by Foster & Fell (2001) by more thoroughly analysing different degrees of erosion in NEF and CEF tests (see Table 1).

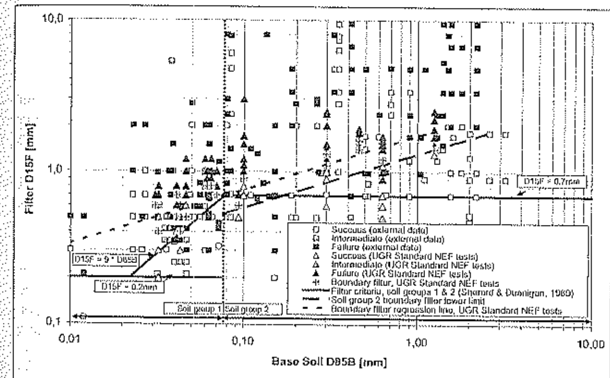

Fig. 1
D15F/D85B for all external data and UGR standard NEF tests
D15F/D85B pour l'ensemble des données externes et essais NEF ("érosion nulle filtre") normalisés de l'UGR

When following a common approach of research in the field of filter design, the purpose of the authors of the presented paper would be to establish new design criteria (at least for soil Group 2) on the basis of the available very large data set. This attempt would possibly lead to a soil Group 2 criterion basing on a potential regression for a set of 26 identified boundary filters (i.e. base-filter combinations characterising the transition from *no failure* to *failure*) in Delgado (2000). In fact, this regression can be performed very accurately (R = 0.85).

Adapting this criterion in order to conservatively enclose all boundary filters and thus to represent an erosion lower bound would lead to a new design criterion as presented in Fig. 1 (dashed line). As can be seen clearly, though the overall description of the D15F-D85B relationship is considerable good, several data points of the available test data set (348 external + 133 UGR data points) fall into the theoretical *no failure* region while showing a failure in the performed test. Similar to Sherard's criterion for soil Group 2, a generally clear separation of the *no failure* and *failure* regions is not reached.

The relationships between D15F and D70B or D15F and P0.075B have also been studied and they are slightly better for representing *no failure* and *failure* regions. Anyway, for all these cases it seems to be clear that because of the big dispersion of the results, it is not possible to give an exact design rule, only a general tendency. Nevertheless the problem can be even worse if other test conditions are modified.

4.3. ALL SOURCES DATA AND ALL UGR NEF TESTS

181 UGR tests are associated to variations of influencing soil parameters (mainly filter compaction, but also base soil moisture content, water pressure, etc.). When adding this sub-data set to the previously investigated data, additional data points fail to meet Sherard's criteria as well as the possible soil Group 2 criterion derived before. This result demonstrates the existence of noticeable effects of certain test parameters on the test outcome and thus presumably on the stability of base-filter systems in the field.

In the actual context, the filter compaction plays a very important role for the test outcome. Fig. 2 exemplary shows the functional dependency between the filter permeability kF and the D15F with respect to different filter compactions in the UGR NEF tests (standard NEF test implies duration of compaction of T = 60 s).

A consequent step in dam filter design would be the formulation of a new criterion on the basis of a parameter which is physically dependent on the compaction efforts. Vaughan & Soares (1982) and Indraratna (1996) emphasize the dependency between the compaction of filter soils, their resulting pore sizes and thus the filter permeability.

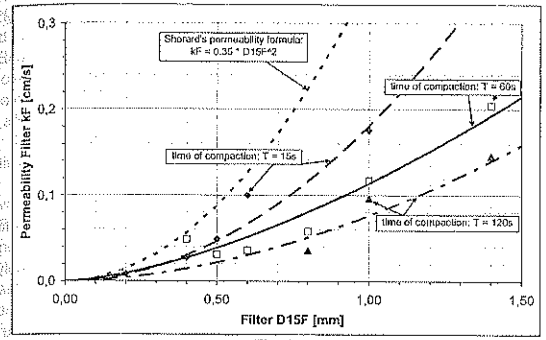

Fig. 2
Filter permeability influenced by time of compaction
La perméabilité du filtre en fonction de la durée de compaction

Fig. 3 presents the relation between filter permeability kF and percentage of base soil passing the 0.075 mm sieve (P0.075B). Filter permeability has been calculated applying the relation $kF = 0.35 \cdot D15F^2$ (Sherard & Dunnigan, 1989) to the 348 data points from external sources and using specific regression formulae (function of soil compaction) for the UGR data set, (see Fig. 2).

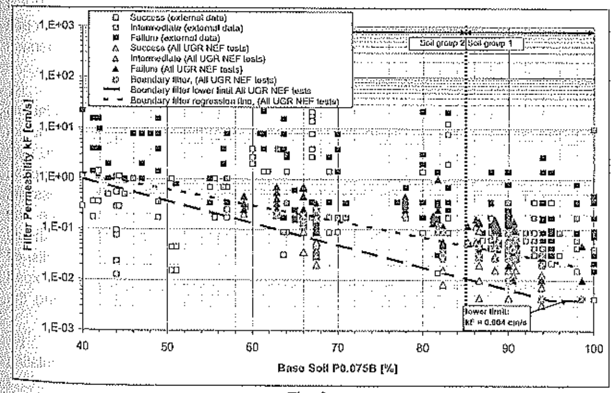

Fig. 3
kF/P0.075B for all available data points
kF/P0.075B pour chaque donnée disponible

The variable P0.075B, has been chosen because, for soil groups 1 & 2, the behaviour of the base/filter tests are clearly dominated by the fines of the base soil. It has been obtained that P0.075B is more representative than D85B. In order to obtain a general tendency for the presented parameter combination, a linear regression is performed on the basis of 59 boundary filters identified in Delgado (2000) and then fitted to enclose the minimum boundary filters. As can be seen in Fig. 3, the obtained tendency is valid only for soil Group 2. The minimum boundary filter for combined soil Groups 1 & 2 shows a permeability of approximately 0.004 cm/s.

With Fig. 3, it becomes apparent that even considering the notable effects of filter compaction does not result in a unique definition of a *no failure* region but tests undertaken at the UGR show the superiority of permeability-based analyses over the classical grain size dependent approach. Nevertheless, due to remaining uncertainties concerning the nature of the base-filter interacting effects, a perfect design criterion cannot be expected, as shown on the basis of a large data set. Therefore, a fully empirical approach is, although being easy-to-perform, not advisable if the shortcomings and uncertainties are not disclosed to the applying engineer.

5. CONCLUSIONS

It is widely established to use simply applicable empirical criteria for the design or safety assessment of granular filters for embankment dams. But if someone only looks for the criterion formulae he can get the false impression that these equations are very accurate and totally reliable while in fact the dispersion of underlying test results is very large.

This paper utilises a total of 688 filter test results and presents correlations between different filter and base soil parameters. It seems to be clear that it is not advisable to offer only strict design equations. As can also be seen in the long lasting proposals and disputes in the field of erosion stability throughout the past, an overall consensus is far from being reached and probably impossible. According to this, as yet unknown parameters and effects influence the performance of base/filter systems, causing unforeseen test outcomes, it should be better to offer to the public the total amount of results using established equations just as general tendencies rather than unambiguous criteria. Expert judgment will be always necessary when values are not exactly fulfilling the traditional empirical criteria.

Further research should pay attention on the possibility of calculating "probabilities of failure" using a data base as larger as possible to best manage the large amount of uncertainties that we have to control. This would lead to a

more probability based approach towards the design and safety assessment of embankment dam filters.

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SUMMARY

Rockfill is used to construct shoulders for large earth dams. The behaviour of earth structures with clay cores is investigated in the context of coupled hydro-mechanical modeling of unsaturated soils. Deformation of a dam takes takes place both during construction and during operation. In the latter case, the main cause is changes in moisture content due to rainfall. Laboratory experiments have led to determining parameters for recently developed constitutive models used to perform simulations. The predictive capabilities of the models have significantly improved in recent years which is shown by the good agreement between calculated and measured displacements recorded at Beliche dam during construction and operation.

RÉSUMÉ

L'enrochement est utilisé pour la construction de grands barrages en terre. Le comportement des structures en terre avec noyau d'argile est étudié dans le contexte de la modélisation couplée des sols non saturés. La déformation des barrages en terre a lieu pendant la construction et la phase opérationnelle. Au cours de celle-ci, la principale cause concerne les variations d'humidité induites par les pluies. Les paramètres des lois de comportement se déterminent à partir d'essais en laboratoire. Les calculs englobent les équations mécaniques et les équations d'écoulement d'eau en régime non saturé. Les performances de prédiction des modèles ont fait preuve d'améliorations importantes au cours des dernières années ainsi que l'on peut le voir par le bon accord qui existe entre les mouvements calculés et mesurés au barrage de Beliche pendant la construction et l'exploitation.

COMMISSION INTERNATIONALE
DES GRANDS BARRAGES

VINGT DEUXIÈME CONGRÈS
DES GRANDS BARRAGES
Barcelone, juin 2006

NUMERICAL SIMULATION OF CONSTRUCTIONAL BEHAVIOUR OF MORA DE RUBIELOS DAM (TERUEL, SPAIN) (*)

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(*) *Analyse numérique pour évaluer le comportement constructif du barrage de Mora de Rubielos (Teruel, Espagne).*

1. INTRODUCTION AND SCOPE OF THE WORKS

A cuasi-complete analysis tool has been built by the second author during the last six years in FLAC (Itasca, 1994), a code that has the capability of upgrading stresses from any strain change making use of its "built-in" models or any other written with a programming language (FISH). Main features of the written routines for static analysis of fill dams have been published in several papers (Ref. [2], [3] and [5] among others).

In this paper the authors show how the model has been initially used to estimate the constructional behaviour of a zoned embankment dam with an asphaltic concrete core. (Fig. 1: Mora de Rubielos, Teruel, Spain).

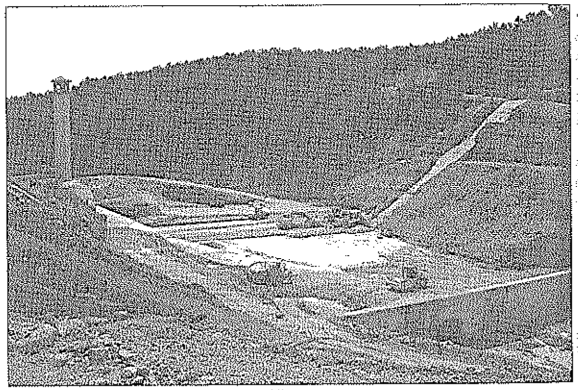

Fig. 1
Overview of some of the dam construction tasks from the downstream side.
Vue générale depuis l'aval de certaines activités des travaux de construction du barrage

2. DAM FEATURES

2.1. DAM BODY AND FOUNDATION

Mora de Rubielos dam is located in Las Tosquillas creek, province of Teruel (Spain), about 4 km upstream the same named town. It is now being

constructed by Júcar River Water Authority (Confederación Hidrográfica del Júcar), from Spanish Environment Ministry, in order to meet strategic purposes related to water supply, irrigation and rural tourism.

Mora de Rubielos is a 35 m high embankment dam with bituminous core built, the length of the crest is 215 m and has a reservoir volume of barely 1 hm^3, so it is of vital interest to optimise its use. The basis for choosing this type of dam were mainly environmental factors and the fact that asphaltic concrete cores are virtually impervious, resistant to internal erosion and ageing, workable and capable of almost continuous placing. The dam spillway combines a lateral fixed lip spillway, common in narrow gorges with limited space adjacent to the dam, with a frontal gated one and provides for optimum flood management.

At dam site cretaceous materials can be found; limestone sound rock at the foundation (with a slight dip towards the upstream side) and alternative layers of sandstone and siltstone at the buttresses and the reservoir.

Fig. 2 shows the typical cross-section with limestone rockfill at the outer dam shoulders (R), natural gravels at shells (G1 and G2), transitions (T) of 1.50 m width and the asphaltic concrete core (AC) of 0.50 m width. Due to the nature of gravels extracted at the borrow pit, it has been necessary to include an upstream filter (F) between the gravels and the quarried rock, some of the natural gravels are being screened (G1) and horizontal drains have been placed in order to prevent from pore pressure build up at the upstream shoulder.

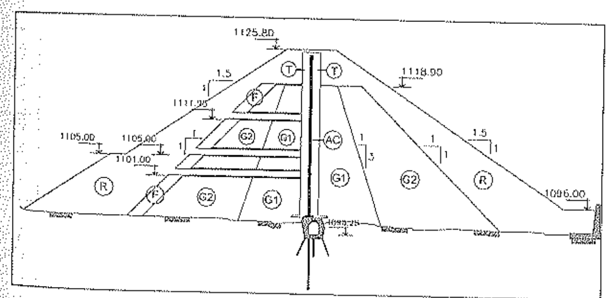

Fig. 2
Mora de Rubielos dam cross section
Coupe du barrage de Mora de Rubielos

2.2. MATERIALS AND CONSTRUCTION PROCEDURES

Table 1 summarizes fill material and asphaltic concrete core properties and specifications. Apart from that, several standard laboratory tests have been carried out to determine all normal soil mechanics values. Large scale shear and triaxial tests have been performed to determine parameters for rockfill and gravels (by CEDEX-Spain). For the core, laboratory specimens have been subjected to conditions approximating those that will exist in the field and triaxial tests have varied bitumen content, type of aggregate and lateral confining stress (by Norwegian Geotechnical Institute). In addition, placing tests in trial fills have been used to optimize and adjust construction procedures for the natural gravels, rockfill and both core and transitions.

Table 1
Material properties and requirements

Zone		Material	Compacted layer thickness (m)	Vibratory roller min. weight (tons)	Nº of passes
R	Rockfill	Maximum rock size = 60 cm Passing sieve 1" < 30 % Passing sieve no. 200 < 10 % Uniaxial compression resistance > 50 MPa	0.60	16 t + water sluicing	4
G1	Fill 1	Passing sieve no. 200 < 12 % Maximum aggregate size = 30 cm	0.20 + 0.20	12 t + water sluicing 100% mod.proctor wopt-1 – wopt+2	4
G2	Fill 2	Passing sieve no. 200 < 20 % Maximum aggregate size = 30 cm	0.20 + 0.20	12 t + water sluicing 100% mod.proctor wopt-1 – wopt+2	4
F	Filter	Meets current filter criteria between rockfill and natural gravels	0.40	12 t + water sluicing 100% mod.proctor wopt-1 – wopt+2	4
AC	Core	Aggregate with filler satisfies Fuller's curve Bitumen type (B60/70) Compacted at 160º-180º Bitumen content by total weight = 7.3%	0.20	0.5 t	Until voids< 3%
T	Transition	d100 ≤ 60mm d50 > 10mm d15 < 10mm | d10 ≤ d100core d100 ≥ 0.25d100shell k > 10^{-4} cm/s	0.20	2.5 t+water sluicing 70-80% Dr wopt-2 – wopt+2	5

Construction of dam body started off in mid-April and should be finished by the end of August (both in 2005). Due to the location of the core and transitions, which are placed and compacted simultaneously to give the core immediate lateral support, all zones of the embankment must be built up more or less at the same time and only a slight advance of the core and transitions is admissible.

3. NUMERICAL MODEL

3.1. CONSTITUTIVE MODELS

The hyperbolic (non linear elastic) model was proposed by Duncan and Chang (Ref. [1]) and some modifications to the model were made some years later (Ref [2]). This latter formulation of hyperbolic stress-strain model implied:

a) Elasticity secant and tangent modulus under primary loading and unloading-reloading situations are maintained as they were originally formulated:

$$E_s = (1-R_f\cdot(1-\sin\phi)\cdot(\sigma_1-\sigma_3)/(2\cdot C\cdot\cos\phi+2\cdot\sigma_3\cdot\sin\phi))\cdot k\cdot P_a(\sigma_3/P_a)^{nd}$$
$$E_t = (1-R_f\cdot(1-\sin\phi)\cdot(\sigma_1-\sigma_3)/(2\cdot C\cdot\cos\phi+2\cdot\sigma_3\cdot\sin\phi))^2\cdot k\cdot P_a(\sigma_3/P_a)^{nd}$$
$$E_{ur} = k_{ur}\cdot P_a(\sigma_3/P_a)^n$$

b) Dependency of bulk modulus on confining stress is incorporated:

$$B = k_b\cdot Pa\cdot(\sigma_3/P_a)^{md}$$

c) Values of friction parameter are better approached by a logarithmic function:

$$\phi = \phi_0 - \Delta\phi\cdot\log_{10}(\sigma_3/Pa)$$

d) Shear Modulus and Poisson's ratio are consistent with the Modulus of Elasticity and the Bulk Modulus (evolution of Poisson's ratio values is not reproduced by any explicit equation).

$$v = 0.5\text{-}E_t/(6\cdot B)$$

e) The stress level that separates the primary loading and the unloading-reloading behavior is clearly defined:

$$SL = ((1-\sin\phi)\cdot(\sigma_1-\sigma_3)/(2\cdot C\cdot\cos\phi+2\cdot\sigma_3\cdot\sin\phi))\cdot(\sigma_3/P_a)^{0.25}$$

where B = bulk modulus; C = cohesion; E_s secant Young's modulus; E_t = tangent Young's modulus; E_{ur} = unloading-reloading modulus; K = modulus number; k_b = bulk modulus number; k_{ur} = unloading-reloading modulus number; P_a = atmospheric pressure; m_d = bulk modulus exponent; n_d = exponent for stress-dependent modulus; R_f = failure ratio; SL = stress level; ϕ = angle of internal friction; ϕ_0 = angle of internal friction at σ_3 = 1 atmosphere; σ_1 = major principal stress; $(\sigma_1-\sigma_3)_{ult}$ = asymptotic value of stress difference; σ_3 = minor principal stress; ν = Poisson's ration and $\Delta\phi$ = reduction factor for ϕ (reduction in friction angle for a 10-fold increase in σ_3).

Preliminary parameters fitted for dam materials are summarized in Table 2.

Table 2
Estimated parameters for dam materials ($P_a = 10^5$ Pa. Units in Pascals)

Material	Kb	k	m_d	n_d	Rf	C	$\Delta\phi$	ϕ_0
Rockfill	300	600	0.2	0.45	0.79	0	15	43.5
Gravel G1	250	500	0.2	0.76	0.6	0	9	40
Gravel G2	200	400	0.2	0.8	0.7	3500	7	39
Filter	225	450	0.2	0.4	0.6	0	10	42

Note: foundation and core have been preliminary considered elastic materials with Bulk and Shear modulus of 867 MPa and 400 MPa for the foundation and 100 MPa and 60 MPa for the core.

3.2. SOFTWARE CAPABILITIES

FLAC 2D (Ref. [4]) is a bi-dimensional finite difference code (explicit scheme) that permits to simulate the behavior of soils, rocks, etc. The program is based on the Lagrangian calculation scheme and its basic formulation assumes a generalized bi-dimensional plane strain state. Each element behaves according to a prescribed stress-strain law, as a response to applied forces and boundary restraints.

FLAC is equipped with an internal programming language (FISH) that permits to define calculation schemes (i.e. complex constructive sequences) and the nature of calculations (i.e. user defined constitutive relationship). This feature makes FLAC very useful for research purposes. In sum, the main potentiality of FLAC is the software capability of upgrading stress states making use of any of its "built-in" models or user defined models.

3.3. GEOMETRICAL MODEL

The reproduction of an almost real constructive sequence has been done by equilibrating approximately one-meter height layers of calculation zones

(Fig. 3 and 4). Also, due to the necessity of realistically reproducing the interaction between core and fill, an interface was defined for the surface contact, which was characterized by the Mohr Coulomb sliding and tensile opening criteria.

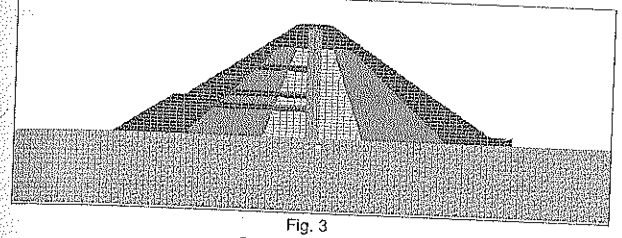

Fig. 3
Geometrical model
Modèle géométrique

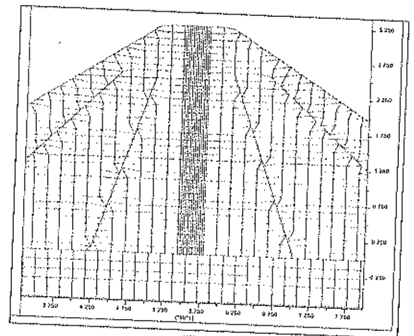

Fig. 4
Detail of the grid
Détail du maillage

3.4. STRESS-STRAIN MODEL

On the other hand, as detailed before, the stress-strain relationship used to upgrade the governing parameters correspond to latest version of the hyperbolic model (Ref. [1]). Particularly, the main difficulty related to the fact of reproducing the real constructive sequence is the necessity of defining threshold values for stresses to set a minimum value of tangent parameters. Another need identified (Ref. [2]) is to check the deviator stress to avoid values apparently consistent but higher than the ultimate deviatoric stress defined as an asymptotic value of the stress difference.

Fig. 5a and 5b show the obtained estimation for settlement and horizontal movements in the analyzed cross-section. Fig. 6 shows shear stress distribution and Fig. 7 a preliminary displacement model for first impounding.

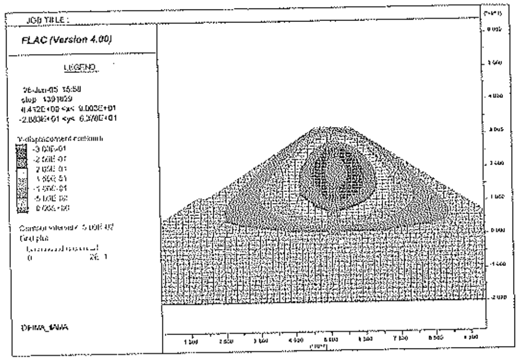

Fig. 5a
Estimated settlement in meters (during construction)
Tassements théoriques pendant les travaux (en mètres)

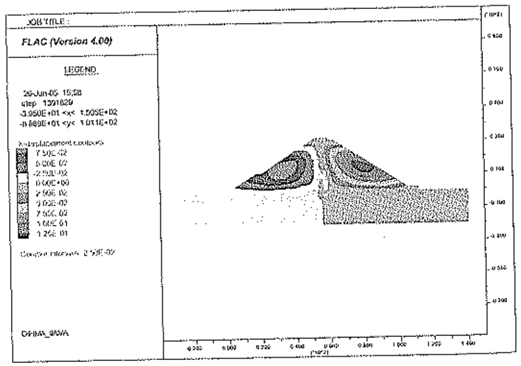

Fig. 5b
Estimated horizontal movements in meters (right after construction)
Déplacements horizontaux théoriques en fin de chantier (en mètres)

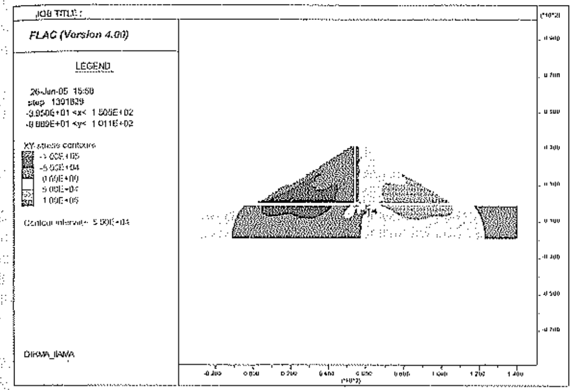

Fig. 6
Shear stress distribution in Pascals (right after construction)
Répartition des cisaillements en fin de chantier (en Pascals)

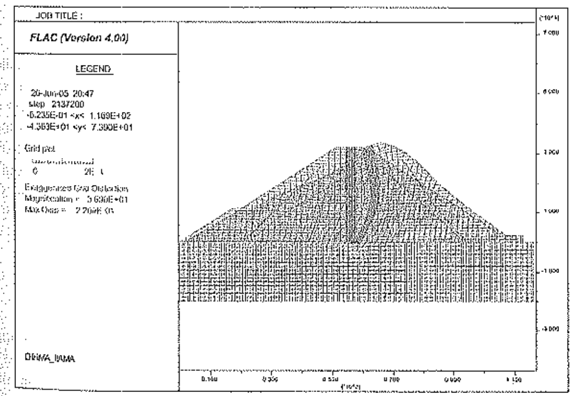

Fig. 7
Preliminar movements in meters estimated for first impounding
Déplacements préliminaires théoriques lors de la mise en eau (en mètres)

As shown in the pictures, despite the fact the model should be calibrated with instrumentation records, expected movements during construction are about 30 cm. for settlement and 12 cm for horizontal displacement. Also, first impounding should not apparently lead to movements larger than 25 cm.

4. FUTURE WORKS.

Once the base model has been built, calibration of model parameters using instrumentation records will be made when instrumentation records are available, i.e. by finding the minimum average square accumulated error between measured and calculated settlement. Also, a particular model of behaviour for the asphaltic concrete core should be incorporated so that time dependent properties might be accounted.

Finally, rigorous estimation of wetting effects and creep on the dam will also be carried out while construction takes place in order to provide a tool to help in decision-making related to changes in material types, the location of such materials or compaction procedures.

ACKNOWLEDGEMENTS

The work has been developed as part of a research project entitled "Análisis de la influencia de la disminución de riesgos de rotura de presas sobre el incremento de riesgos de la insatisfacción de las demandas en sistemas de recursos hídricos" sponsored by the Spanish Ministry of Science and Technology (30%) and FEDER funds of the European Union (70%). Also we would like to thank Jose Estaire (CEDEX, SPAIN) and KAARE HÖEG (Norwegian Geotechnical Institute).

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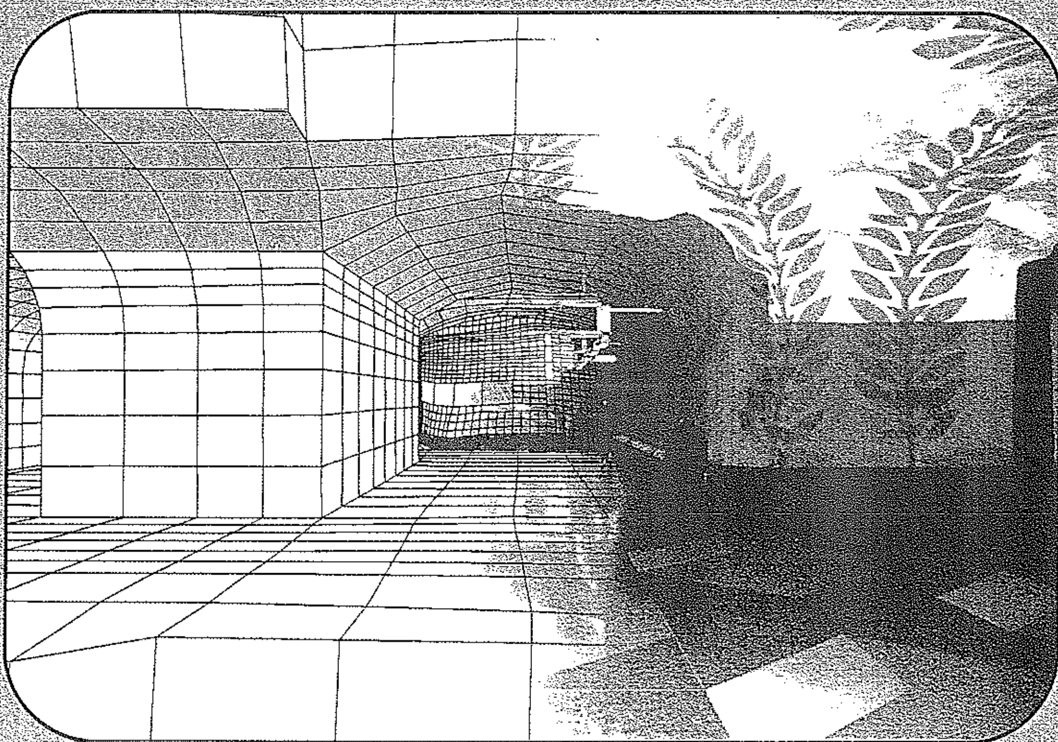
SUMMARY

A quasi-complete fill dam numerical analysis tool, whose features for static and dynamic analysis of such structures have been presented in previous ICOLD Meetings (Montreal 2003 and Teheran 2005), has been built by the second author during the last six years in FLAC (Itasca, 1994). In this paper the authors show how the model has been used to estimate the constructional behaviour of an embankment dam with an asphaltic concrete core. (Mora de Rubielos, Teruel, Spain).

RÉSUMÉ

Un outil quasi-complet d'analyse numérique des barrages en remblai, dont les caractéristiques pour l'analyse dynamique et statique de ces structures ont été déjà présentées dans des réunions précédentes de la CIGB (Montréal, 2003 et Téhéran, 2005), a été développé par le second auteur de ce rapport depuis six ans dans FLAC (Itasca, 1994). Dans ce rapport les auteurs présentent comment le modèle a été utilisé pour évaluer le comportement constructif d'un barrage en remblai avec noyau central bitumineux.

FLAC and Numerical Modeling in Geomechanics — 2006



Pedro Varona
Roger Hart
Editors

FLAC and *FLAC*^{3D} are explicit finite-difference computer codes for geomechanics applications. *FLAC* has been distributed by Itasca Consulting Group, Inc. since its first commercial release in 1986; it was joined by *FLAC*^{3D} in 1994. Both programs now enjoy extensive application worldwide.

This volume contains a collection of 62 papers selected for presentation at the fourth *FLAC* Symposium, held May 29-31, 2006 in Madrid, Spain. The contributions cover a wide range of topics in the areas of: slopes and embankments; underground structures; coupled processes and fluid flow; dynamic analysis; soil-structure interaction; tectonics; numerical techniques; and constitutive models/material behavior.

The proceedings illustrate the great variety of *FLAC* applications in geomechanics. The volume provides descriptions of both engineering applications and theoretical developments and may be used as a guide to help *FLAC* users in their geotechnical analyses.

ISBN 0-9767577-0-2

4 COMMENTS AND RECOMMENDATIONS

Two examples are presented that illustrate that a *FLAC* model with the new hysteretic damping logic can provide a reasonable match to the model using Rayleigh damping, provided that both damping types are adjusted to best-fit shear modulus reduction and damping ratio curves that represent the dynamic cyclic energy dissipation of the materials.

These examples illustrate the benefits of using the hysteretic damping logic for seismic analysis. A primary benefit is that this logic directly accounts for the strain-dependent modulus and damping change during the cyclic motion. In addition, hysteretic damping can provide a substantial increase in calculation speed compared to Rayleigh damping.

The following recommendations should be kept in mind when applying hysteretic damping:

1 The hysteretic-damping logic can be directly applied provided that physical data are available that relate the reduction in shear modulus of the materials to the cyclic shear strain. The hysteretic damping function and parameters that are fit to these data should also produce a good fit to physical data that relate the change in damping ratio to cyclic shear strain over the expected range of cyclic strains. It is important that the fit be satisfactory before applying hysteretic damping to field–scale analysis.

2 It is also important, before applying hysteretic damping, to check the initial shear-stress state of the model, i.e., the equilibrium state before dynamic loading is applied. The hysteretic damping formulation is assumed to initiate hysteresis from an initial value of zero shear strain. If the initial shear stresses are high, then the shear stress and shear strain state for hysteretic damping may not be compatible. In the above examples, initial (static) shear stresses are small. However, if shear stresses are significant, then, in order to ensure that shear stresses and strains are consistent during the dynamic phase, hysteretic damping should be invoked before the model is brought to the initial equilibrium state. This can be done in *FLAC* by applying local damping and hysteretic damping concurrently in place of the default static (local) damping logic, when the model is brought to initial static equilibrium (see Cundall 2006).

3 The simple examples described here assume that the initial shear modulus (G_{max}) is uniform throughout each material unit. It may be more appropriate to vary G_{max} for soils as a function of the in-situ effective stress (e.g. see Kramer 1996). For example, it may be considered that the maximum shear modulus varies as a function of effective stress as defined by the Seed et al. (1986) expression:

$$G_{max} = 21.7 \times P_a \times K_{2,max}\left[\frac{\sigma'_m}{P_a}\right]^{0.5}$$

where P_a is the atmospheric pressure, $K_{2,max}$ is a constant determined from the relative density, and σ_m is the effective mean stress. This initial modulus variation can be implemented during the static loading stage. The effect of including this initial variation in G_{max} in the earthfill dam seismic simulation is shown in Figures 12 & 14.

4 The hysteretic damping logic can also be included with models incorporating other cyclic material response, such as liquefaction. For example, the liquefaction models proposed by Byrne (1991) or Dawson et al. (2001) are based on the Mohr-Coulomb model, and can also include the hysteretic damping logic to represent the cyclic energy dissipation. Note that more complex liquefaction models, such as the UBCSAND model (Beaty & Byrne 1998), include a hysteretic damping component as part of the model; the hysteretic damping logic is not needed for these types of constitutive models.

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FLAC numerical models applied to safety assessment of dams

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Universidad Politecnica de Valencia, Valencia, Spain

ABSTRACT: *FLAC* numerical models have shown to be very useful to assess safety of dams. In fact, behavior during construction, creep analysis, wetting effects on movements and stresses, pore pressure distribution, and the simulation of the non linear dynamic response have been approached in different and successful manners. Herein, a number of study cases for different problems on fill and concrete dams are shown with emphasis on the calculation needs, difficulties found and the well-established solutions published by the authors.

1 INTRODUCTION

Over the past eight years the dam safety legislation in Spain has promoted many instrumentation projects and analyses of dam behaviors. Through these projects many lessons have been learnt.

Concerning the safety of large dams and reservoirs three legal codes are currently applicable in Spain:

1 The "Instruccion para el Proyecto, Construccion y Explotacion de Grandes Presas" (Instruction for the Project, Construction and Operation of Large Dams) of 1967 (applicable to about 66% of Large Dams)
2 The "Directriz de Planificación de Protección Civil ante el Riesgo de Inundaciones" (Civil protection planning guidelines against flooding risk) of 1995 (applicable to 100% of Large Dams)
3 "Reglamento Tecnico sobre Seguridad de Presas y Embalses" (Technical Rules on Dams and Reservoirs Safety) of 1996 (applicable to about 33% of large Dams)

The development of technical codes related to any dam aspect has been sponsored by the Comite Nacional Español de Grandes Presas (Spanish National Committee on Large Dams) and by Direccion General de Obras Hidraulicas y Calidad de Aguas (An Office of the Spanish Ministry on Environment and Water)

The Technical Guides are a set of recommendations on how to apply the Reglamento 1996. In fact, while Instrucción 1967 was very concrete and contained very rigid calculation criteria, the Reglamento 1996 is very general. However, it is important to remember that Reglamento 1996 is mandatory (for those dams previously determined) and Technical Guides are only recommendations on how to apply such legal document.

For those dams to which Reglamento 1996 is applicable, the following tasks are to be done: Constitution of Dam Technical (Safety) File, Potential Hazard Classification, Operating Procedures, Emergency Action Plan, Complete Safety Review, Inspection Reports and Annual Reports.

The study of the behavior of dams becomes especially relevant in order to undertake the three last mentioned tasks. Finally, as a more recent and very interesting approach, the failure mode analysis undertaken in the context of risk analysis and risk assessment, might allow the dam engineers to focus on what really maters about instrumentation and measurements and decide about the need of new instruments at their site

2 IMPLEMENTED NUMERICAL MODELS

A complete analysis tool has been built by the second author during the last six years using *FLAC* (Itasca 2000), a code that has the capability of upgrading stresses from any strain change making use of its "built-in" models or any other written with a programming language (FISH) available for users. Main features of the written routines for static analysis of dams are:

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AMS STUDY

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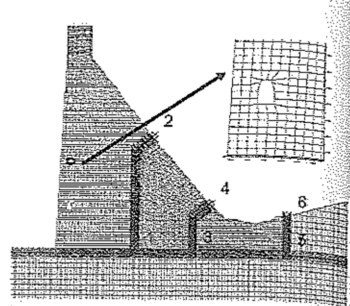

Figure 1. Geometry of a cross section.

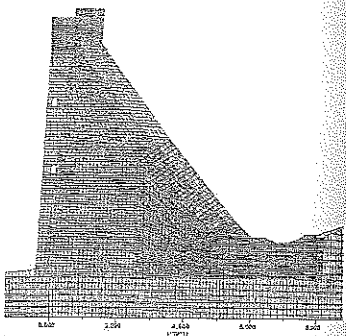
Figure 2. Movements plot after impounding.

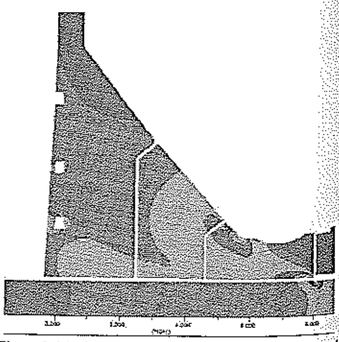
Figure 3. Maximum principal stresses plot alter impounding.

4 ROCKFILL DAMS STUDY CASES

Safety assessment of central clay core dams typically involves stability analysis (shear static and dynamic failure), stress-strain analysis with special emphasis in pore pressure development and other types of checks such as filter criteria, hydraulic fracture potential, piping susceptibility that are not generally undertaken by means of numerical models.

Routines to simulate stress-strain behavior under static and dynamic loads, including the liquefaction potential have been previously published and widely used.

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

In addition, irrecoverable displacements due to earthquakes will also be overestimated and a non-objective sense of lack of safety might be transmitted.

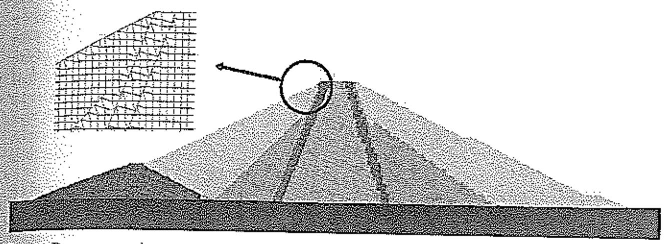
Figure 4. Dam cross section geometry and different regions

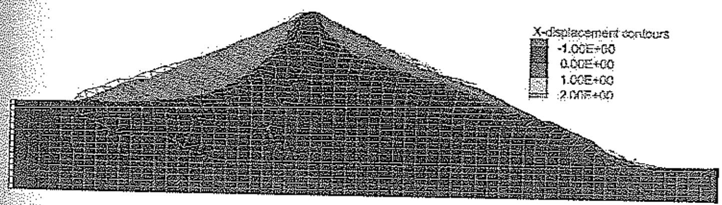

Figure 5. Irrecoverable horizontal displacement after earthquake

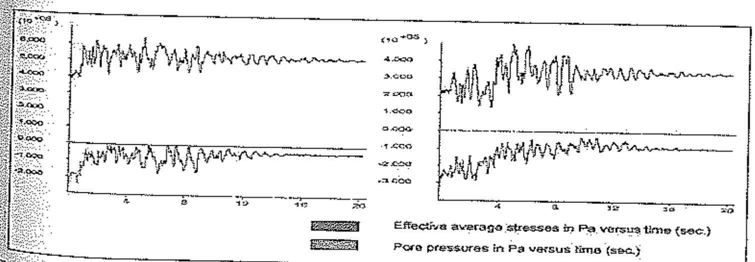

Figure 6. Pore pressures time history during earthquake.

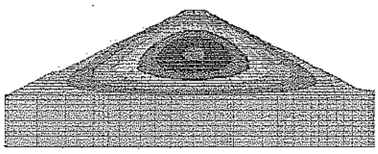

Figure 7. Settlement plot during construction for a rockfill dam with central clay core.

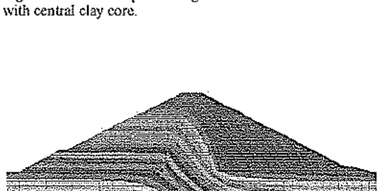

Figure 8. Steady pore pressure plot for a rockfill dam with central clay core.

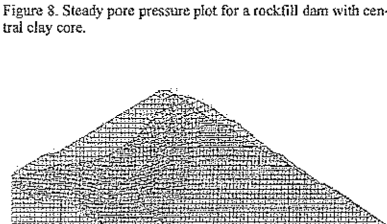

Figure 9. Movements plot for a rockfill dam with central clay core after impounding.

5 CONCLUSIONS AND RESEARCH NEEDS

Any legislation on dam safety that prescribes the monitoring and study of the behavior of dams provides good opportunities to get a better understanding of all dams but also, such studies should be carefully undertaken.

In summary, combination of a good knowledge of the instruments, an appropriate conservation and reading procedure, a realistic data management program and the implementation of numerical models are very important. Also, the behavior of a dam itself is a source of uncertainties.

Finally, in the context of risk analysis applied to dam safety, some research needs are identified and summarized bellow:
- Extend the range of failure modes that can be reproduced by numerical modeling.
- Integrate as many different failure modes as needed in the same model.
- Increase the calculation efficiency to make possible direct links to Montecarlo analysis.
- Develop simulation procedures and criteria to obtain "failure curves" from a limited number of calculations making possible computation of as many simulations as needed.

ACKNOWLEDGEMENTS

Special thanks to OFITECO SA, the firm that supported the application of all this work to real cases. Also, some of the works have been developed as part 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%) and FEDER funds of the European Union (70%).

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4th International FLAC Symposium on Numerical Modeling in Geomechanic

Evaluation of the dynamic stability of a tailings da

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ABSTRACT: *FLAC* (version 5, with the dynamic option) was used to sim and the impounded tailings under seismic loading. The characteristics o cone penetration testing as well as conventional and dynamic laboratory Saguenay, Quebec (1988, M_W = 5.9) and Northridge, California (1994, M the simulations. Hysteretic damping was incorporated into the model to p propagation of shear waves through the dam and tailings and the resulting excess porewater pressures was simulated using the Byrne model. Results ior of tailings can be incorporated into the dynamic analysis of a tailings da

1 INTRODUCTION

Failures of tailings dams due to seismically-induced liquefaction of the retained tailings have occurred worldwide, including at the Barahona and El Cobre Mines in Chile in 1928 and 1965, respectively, a mine in Zaire in 1970, at multiple locations in Mochikoshi, Japan in 1978, and most recently, at the Tapo Canyon tailings dam as a result of the 1994 Northridge, California earthquake (Ishihara et al. 1980, Harder & Stewart 1996, Davies & Lighthall 2001, Davies 2002, Wise 2004).

Realistic evaluations of the dynamic stability of tailings dams must consider the dynamic behavior of the tailings, including porewater pressure development, strength loss and deformation.

Existing analytical techniques for the evaluation of porewater pressure development and liquefaction under seismic loads are largely based on the dynamic behavior of sand, as limited research has been conducted on the dynamic behavior of tailings. However, observations and research indicate that some types of tailings, specifically those from hard rock mines, are highly susceptible to liquefaction (Ishihara et al. 1980, Garga & McKay 1984, Troncoso 1985, Wijewickreme et al. 2005b).

Tailings from hard rock mines typically consist of hydraulically placed, normally consolidated, nonplastic silts and sandy silts (Vick 1990, Aubertin et al. 1996, Qiu & Sego 2001). Conventional liquefaction evaluation techniques applied to sands and silty

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