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A. Yziquel

Coyne et Bellier- Consulting Engineers, France

M. Lino

Coyne et Bellier- Consulting Engineers, France

G. Post

Coyne et Bellier- Consulting Engineers, France

B. Tardieu

Coyne et Bellier- Consulting Engineers, France

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Seismic Analysis as a Tool in the Design of Two Earth Dams

A. Yziquel, M. Lino, G. Post, B. Tardieu
Coyne et Bellier - Consulting Engineers, France

SYNOPSIS : Two dynamic analysis studies of embankment dams are described. One dam is 43m high on alluvium, the other is 140m high on a rock foundation in a highly seismic area. The main emphasis is on the practical nature of the analytical methods and their value as design tools. The earthquake design features of both projects are described and the closing paragraphs attempt to draw attention to the main points to be considered when running an earthquake analysis of earth dams.

INTRODUCTION

Both projects described are embankment dams built of materials which can reasonably be expected not to be subject to liquefaction, with no any major development of pore pressure or any significant loss of shear strength during an earthquake. Under these conditions, the main effect of an earthquake is the appearance of permanent displacements liable to cause serious damage and even result in failure such as overtopping through slump of the crest or by the wave produced by sloughing on the upstream face, cracking in the seepage control works leading to internal erosion, etc.

The first analysis method for this type of dense material was described by Professor H.B. Seed in his Rankine lecture (Makdisi and Seed, 1978).

- .Select a typical profile of the dam.
- .Determine the static properties of the material.
- .Calculate static stresses in the dam during an earthquake (by means of a finite element non-linear incremental analysis simulating the stages of dam construction and reservoir filling).
- .Select the design accelerogram in consultation with the geologist and seismologist.
- .Determine the characteristics of the material by means of laboratory tests, in situ tests or from published data.
- .Calculate the dynamic response of the dam. Non-linear behaviour is dealt with by the Seed linear equivalent method (program QUAD4) (Idriss - 1973).
- .Analyse dynamic deformations and check particularities in the dam (upstream facing, diaphragm wall, thin membranes, etc.).
- .Select potential slip surfaces and mechanical failure properties (cohesion and internal angle of friction) and use static equivalent method to calculate the critical acceleration giving a safety factor of unity.
- .Calculate the displacements caused by the earthquake on these potential slip surfaces by the Newmark method .

The second approach was described by K.L. Lee (1974) by :

- .Evaluate the distance from the intrinsic curve for stresses at each point, obtained by superimposing the static stresses on the maximum dynamic shear. This is an indication of the most highly stressed zones in the dam.

EARHQUAKE RESISTANCE OF A DAM ON A DEEP ALLUVIUM FOUNDATION

Geological context

Verney dam on the Eau d'Olle river is the lower dam in the Grand'Maison pumped storage scheme built by Electricité de France some 30km from Grenoble in the french Alps. The river flows through a Liassic depression surrounded by the Belledonne and Grandes Rousses crystalline structures. Where it emerges into the Allemont plain, it is very steep before entering the relatively flat valley about 300m wide containing the Verney dam. The valley flanks are Liassic schists and there is a thick filling of alluvium and moraine material on the valley bottom.

Description of project

Verney dam (fig. 1 and 2) is 43m high above the foundation at its highest point. It is built of sand and gravel material, with a fill volume of 1.55 million cubic metres. The impervious upstream facing is tightened to a plastic concrete diaphragm wall cut-off in the foundation. The dam is designed to be absolutely safe in the event of an earthquake. There is no risk of liquefaction in the very compact alluvium and morainic foundation. The bituminous concrete facing and the downstream drainage blanket prevent the fill from becoming saturated. The flexibility of the partial cut-off, down to the less pervious moraine material, enables it to accommodate settlement as the dam is built, seepage through the foundation and any dynamic loading to which it may be subjected. The connection between the upstream facing and the cut-off has been kept as simple and flexible as possible for the same reasons. Even if leakage should occur through the facing for any reason (construction defect, damage by an air craft crash, sabotage...), the upstream shell and the chimney drain would maintain the stability of the dam and control seepage. Freeboard is 3 metres above Normal Water Level plus a watertight parapet 1.23m high.

Fig 1. LONGITUDINAL SECTION OF VERNEY DAM

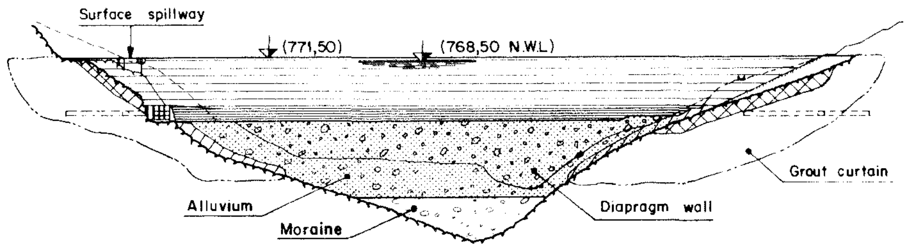


Fig 2. TYPICAL SECTION OF VERNEY DAM

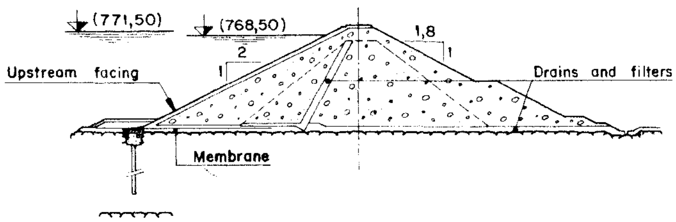


Fig 3. DISCRETISED MODEL

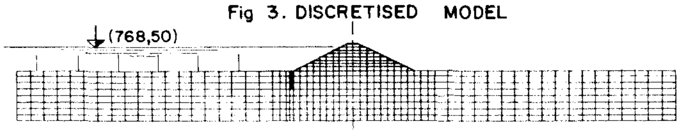


Fig 4. R RATIO CONTOUR FOR THE 3 SECTIONS

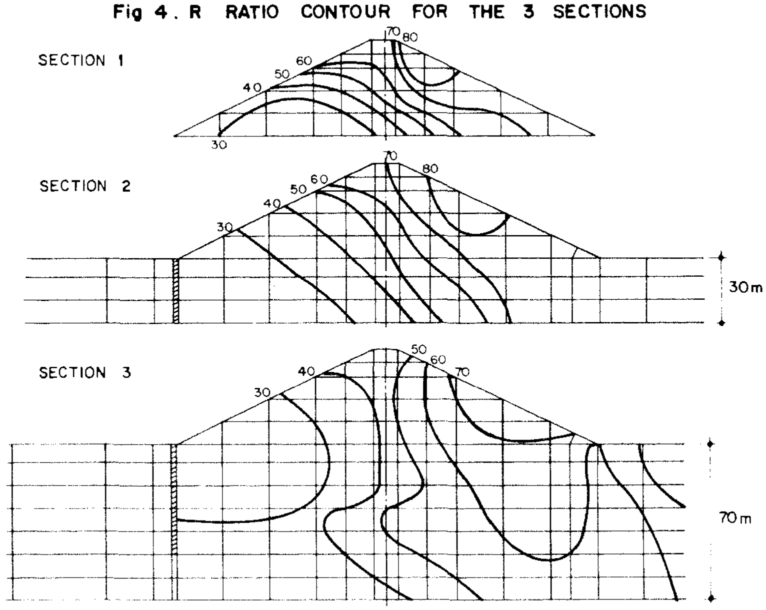


Fig 5. Maximum horizontal acceleration

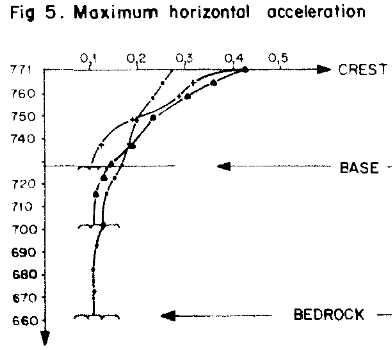


Fig 6. Maximum horizontal shear stress

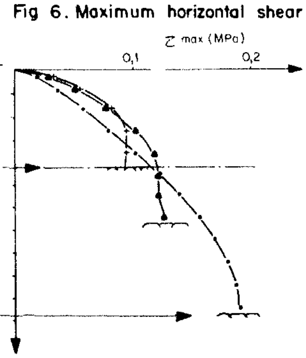


Fig 7. SECTION 1

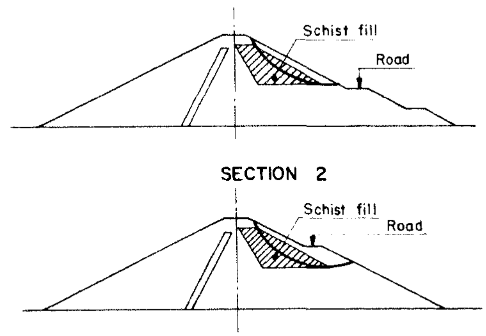


Fig 9. TYPICAL RESULTS
From Newmark method

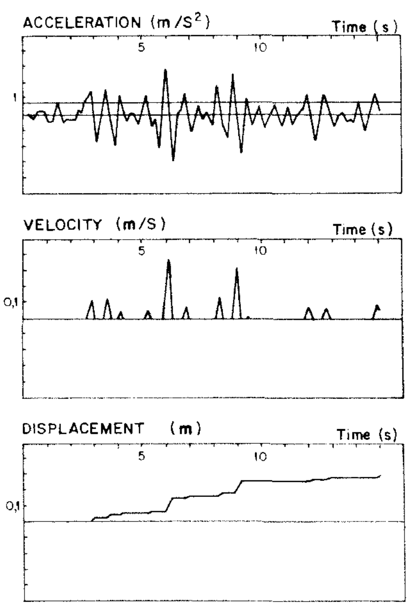
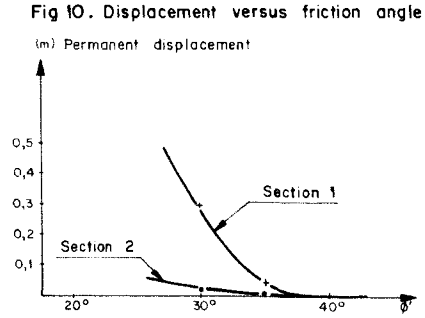


Fig 10. Displacement versus friction angle



Seismicity

A detailed study of the seismicity of the site indicated that an appropriate choice of reference earthquake would be the 22 July 1881 event which occurred at the site itself. The maximum probable earthquake was taken as one with an MSK intensity of VI or VII, producing a maximum rock acceleration of 0.1g. The duration was estimated at 15 seconds. The accelerogram used for the analyses is a synthetic accelerogram with a spectrum similar to the NRC spectrum.
The designers paid particular attention to the magnification effect of the alluvial foundation and its consequences on the earthquake resistance of the dam.

Description of materials

The Quaternary filling on which the dam sits consists of two quite distinct materials, sand and gravel alluvium overlying more compact moraine material. The body of the dam is built mainly of alluvium from the reservoir area, consisting of sand and gravel of crystalline or Liassic origin, generally containing little silt or clay.
The main geotechnical characteristics of these material are described in the Table I.

Table I

	e void ratio	c' cohesion	Ø' friction angle
Dam	0,27	0	43
Foundation 0m - 30m	0,30	0	37
Foundation 30m - 70m	0,27	0	-

Static analysis

A detailed static analysis was made in order to determine the static stresses in the dam and foundation. It was performed in three stages.

- Determination of the state of stress in the dam and foundation due to construction of the dam, by the finite element method (figure 3).
- Determination of the flow net in the foundation after reservoir filling As the cut-off is only partial, there are seepage forces in the foundation, under the dam especially.
- Application of these seepage forces and pressure forces on the upstream facing and watertight membrane to obtain the total state of stress.

Dynamic analysis

From the results of the static analysis, it was possible to calculate the mean effective confining pressure and derive, for each element, a maximum shear modulus by means of the Hardin formula :

$$G_{max} = 102.15 \frac{(2.973 - e)^2}{1 + e} (\sigma'_m)^{1/2} \quad \text{(MPa)}$$

Three dam sections were examine to find the most unfavourable foundation thickness (fig.4).

- **Bank section on rock foundation**
- **Section on alluvium foundation 30m deep**
- **Section on alluvium and moraine foundation 70m deep.**

The maximum accelerations along the dam axis are shown on figure 5. The maximum magnification occurs on the 30m foundation, with a value of 4.5. This produces 0.45g at the dam crest.

The maximum horizontal shear stresses on the dam axis are shown in figure 6. The most severe shears in the dam occur when the alluvial foundation is 30m deep. The final state of stress consists of superimposing the static and dynamic stresses. A simple mean of assessing the danger of irreversible deformation is to examine whether the point representing the total state of stress on the Mohr chart crosses the intrinsic line.

If (σ_c, τ_c) represents the static state of stress on a horizontal surface and τ_d the maximum alternativy

shear at this point during the earthquake, there are no irreversible deformations if :
$\tau_c + \tau_d < \tau max$. The ($\sigma_c$, τmax) curve represents

the intrinsic curve, which is only drawn here from static tests. The value of :

$R = \frac{\tau_c + \tau_d}{\tau max}$ is an indicator of the risk of

irreversible deformations. The value of the ratio R multiplied by 100 is shown on figure 5. It is never numerically higher than 100 either for the dam or the foundation, and it can be concluded that there should be no irreversible displacements in the dam during an earthquake of intensity VI or VII on the MSK scale.

Shear deformations caused by the earthquake near the upstream facing, more especially at its interface with the foundation or cut-off wall are never more than

5.10^{-4} and so neither the facing nor the wall should suffer any damage from an earthquake.
During the construction of the dam in the summer of 1980, it was thought useful to analyse the possibility of incorporating schist spoil from the powerstation tunnel excavations into the dam. Since the alluvium had good mechanical properties, it was thought it could be more beneficially used in other parts of the works whereas the schist would have been wasted. The position of this material, whose mechanical characteristics are not so good as the alluvium, was dictated by the need to prevent subsequent settlement, especially in the upstream zone affecting the facing. The material was therefore placed at a place where they have least effect on the stability and settlement of the dam under normal conditions. But this position is more critical during an earthquake. The earlier earthquake analyses had shown that the most critical case was were the foundation was 30m of alluvium; the stability of two sections was examined for this foundation condition, one on the right bank and the other on the left bank, the road along the downstream slope occupying a different position. A static and dynamic stability analysis was run on both sections with different mechanical characteristics for the schist fill :

Ø' = 40°
Ø' = 35°
Ø' = 30°

With the Newmark method, it is possible to determine the permanent displacement of this fill along a potential failure line after an earthquake. These permanent displacements which depend on the internal friction angle of the material are shown on figure 10. It can be seen that, for the right bank section (section 2), **final displacements are acceptably small and there is no danger on this section. There is a much greater risk on the left bank (section 1) although displacements are not more than 30cm.**

The friction characteristics of this schist material were determined from shear tests with a 1.20 x 1.20m box capable of testing scalped material reproducing 85 % of the grain sizes used in the dam. From these tests it is possible to estimate a value of the friction angle of about 30°. In the right bank section therefore, it is necessary to reduce or redesign the zone in order to use a greater proportion of good engineering material, as in section 2.

STABILITY OF A LARGE DAM IN AN AREA OF HIGH SEISMIC ACTIVITY

Description of project

The Aït Chouarit dam on the Lakhdar river in the Atlas mountain range is to impound a reservoir of 270 Mm3 capacity for over-year regulation to guarantee the supply of 350 Mm3 of water yearly, chiefly for agricultural purposes. A diversion dam 26km downstream from Aït Chouarit - the Sidi Driss dam in the process of construction - will divert water into a canal to the Haouz Plain irrigation area in the Marrakech region.

Aït Chouarit is an embankment dam with a maximum height of 140m and a crest length of approximately 360m, situated at a bend in the valley, a position dictated by topographical and geological considerations (figure 11). The valley at this point is V-shaped and about 20 metres wide at the bottom with the flanks sloping up at an average of 1.5 horizontal for 1 vertical.

The site lies in the heart of the Guettoua syncline consisting of red clayey sandstone with facies varying from coarse grained sandstone to clayey siltstone or claystone. It is moderately fissured but there is no significant fracture or fault near the site.

The typical dam profile shown in figure 2 points out how the materials available nearby have been used. The downstream shell, built entirely of random clayey sandstone fill material, has a slope of 2.75H/1V from the river bed up to the berm at elevation 880.00, after which it steepens to 2.5H/1V to the crest. The upstream shell with a slope of 2.75H/1V consists of a clayey sandstone body sloping 1.6H/1V on which is laid a hard sandstone rockfill. An impervious clay core is located between the shells with a filter, drain and another filter behind it to collect seepage, and continued as a drainage blanket at the bottom of the valley.

The alluvial filling material is removed to sit the dam directly on the sandstone bed rock. The total fill volume is approximately 8.5 Mm3.

Seismicity

The dam site lies in the Atlas range between two major and potentially active faults.

The SW-NE South Atlas Fault at 50 km from the site is a large vertical or reverse fault with a throw of more than 1000m, separating the folded range from the pre-Saharian depressions. Many auxiliary faults are associated with it.

The North Atlas Fault at 40 km from the dam runs roughly parallel to the South Fault; it is subvertical with throws of several hundreds of metres.

The degree of seismic activity associated with the North Atlas Fault is the more important of the two and can be expected to cause the most severe earthquake likely to be experienced at the site. Its epicentre would lie on a line passing within 40km of the site.

The analysis of geological and seismological data and of historically recorded earthquakes made by Professor J.P. Rothe, a specialist in Moroccan seismicity, shows that it is unreasonable to expect a maximum probable

Fig 11 . General view of Aït Chouarit dam

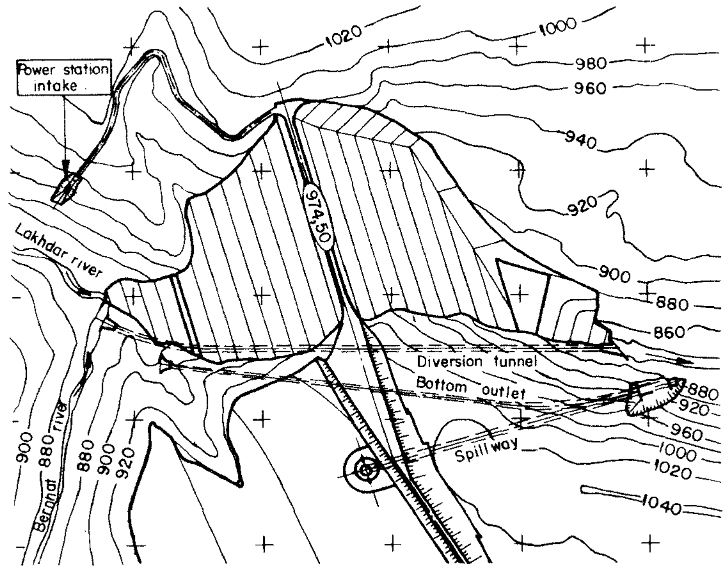


Fig 12 . Typical section of Aït Chouarit dam

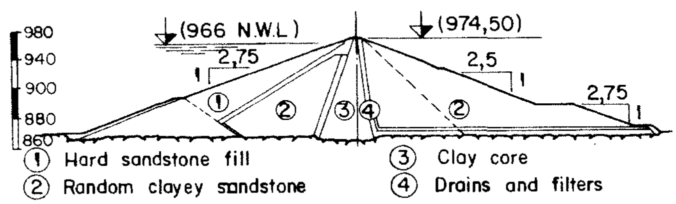


Fig 13 . Mean stress $\sigma'm$ contour at initial time (MPa)

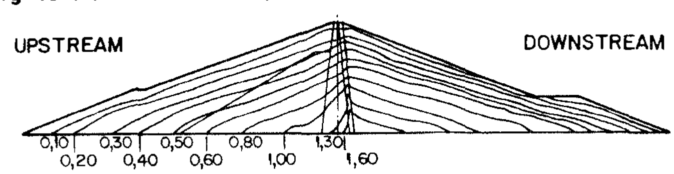


Fig 14 . Maximal acceleration contour during earthquake

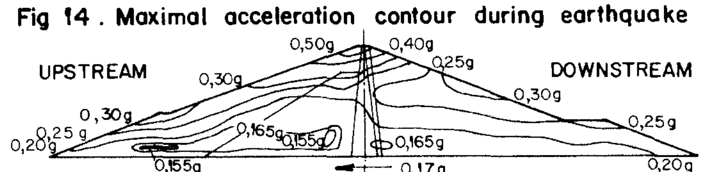
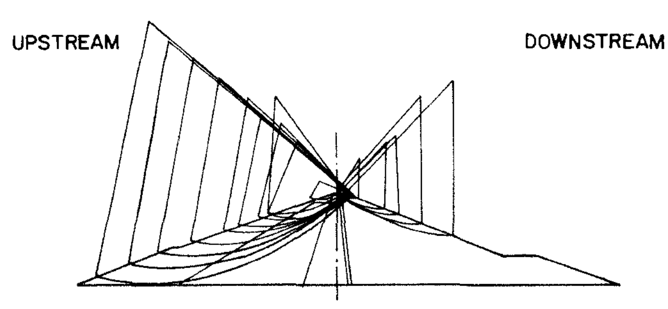


Fig 15 . Slip surfaces analysed by Newmark method



intensity near the site of VII on the MSK scale. The design intensity has therefore been set one point higher than Professor Rothe's recommendation, as an additional safety measure, with a Richter magnitude of 6.
From intensity-acceleration correlations, an earthquake of this magnitude with its epicentre 40km from the site would produce a maximum acceleration at the dam of 0.17g. The significant duration of the earthquake estimated from its magnitude and epicentre distance is taken as 15 seconds.

Design accelerogram

Several historical accelerograms (San Fernando, El Centro East-West and El Centro North-South) and a synthetic accelerogram (USNRC) whose spectrum is the envelope of a large number of recorded earthquakes were adjusted for 0.17g and tested by means of the rapid analysis procedure put forward by Seed and Makdisi in order to determine the most severe one for the dam (Makdisi and Seed - 1979). The El Centro North-South and the synthetic USNRC accelerogram give the highest maximum accelerations at the crest (0.64g approx.). Results for these two accelerograms are also quite consistent as regards the predominant period of the fill (1.09 and 1.05 respectively). The final choice was therefore for the El Centro North-South accelerogram which, in the relevant frequency range, has a spectrum very similar to the USNRC spectrum, very rich in low frequencies of the order of one Hertz and which severely stresses the dam fill.

Static analysis

The stress state in the dam with full reservoir is calculated on the typical highest profile by simulating the construction stages and reservoir filling. Triaxial tests on the clayey sandstones and the core clay provided the parameters for the Duncan hyperbolic relationship. Because of the very low permeability of the clayey sandstone, its shear strength during construction is taken at an intermediate value between its undrained and residual long term shear strength.
Figure 13 shows the lines of equal mean effective stress. It can be seen that there is a buoyancy effect reducing the load on the upstream fill, a reduction of the load on the core and a load transfer to the filter and downstream shell.

Materials dynamic properties (Table III)

The clayey sandstone forming the bulk of the dam is a well graded random material with a very low void ratio (0.22 approx.). Hardin's formula is not applicable to such a dense material, the maximum shear modulus Gmax calculated with this formula being Gmax = 650 $(\sigma'm)^{1/2}$ in which $\sigma'm$ is the mean effective stress. Gmax has been evaluated from the shear wave velocities measured by cross-hole method on a test embankment, the result being Gmax = 1700 $(\sigma'm)^{1/2}$ or 2.6 times higher than predicted by the Hardin formula.
The maximum shear modulus for the clay core was evaluated by correlation with the undrained shear strength.

Table III : Material Dynamic properties

	e void ratio	$Kmax=\frac{Gmax}{(\sigma'm)^{1/2}}$ (MPa)	c_u undrained shear strength (MPa)	ν Poisson ratio
Hard Sandstone	0.30	560	-	0.30
Clayey Sandstone fill	0.22	1 700	-	0.30/0.45
Clay core	-	-	0.10	0.45
Filters and drains	0.35	520	-	0.30

Dynamic response of dam

Figure 14 shows the maximum accelerations obtained in the fill. The highest peak acceleration is found at the crest; it is 0.54g, representing a magnification factor of 3.2 over the original ground motion. The highest accelerations occur on the upstream side, between elevations 950.00 and 966.00, corresponding respectively to the top of the random clayey sandstone in the upstream shell and to the Normal Water Level.

Permanent deformation

The permanent deformations of the fill are evaluated from the dynamic response, by the Newmark method. The mechanical characteristics used in the stability analysis are the residual characteristics, since sliding occurs in several stages (Table IV).

Table IV: Residual shear strength for pseudo-static analysis

	Friction angle Ø'	Shear Strength c'(MPa)
Hand sandstone rockfill	40°	0
Upstream clayey sandstone shell	28°/20°	0
Downstream clayey sandstone shell	30°	0
Clay Core	15°	0

Fifteen slip surfaces were studied (fig.15). The largest deformations were found on the upstream slip surfaces at the top of the fill. This is quite normal in so far as :

- the peak accelerations in the upstream shell are higher than in the downstream shell at the top of the dam,
- the critical seismic coefficients of the upstream surfaces are lower than on the downstream side because of the unfavourable effect of the reservoir in the static equivalent analysis, despite the flatter slope on the upstream side,
- the mean accelerations at the centre of gravity of the higher potential slide masses are relatively small because the accelerations at the bottom and top of these masses are not in phase. The maximum displacement was obtained on circle 1 (0.82m approx.); there were only negligible deformations on the deep circles.

Effects on design

The analysis of the dynamic behaviour of the fill under earthquake conditions shows that one can expect permanent slides of less than 1m amplitude, occurring mainly at the top of the dam.
Such deformations are in no way dangerous for the stability of the dam. The chief risk is cracking in the core, accompanied by serious leakage through the dam. This risk can be averted by providing an ample chimney drain behind the core with a special upstream self-healing fine filter at the top of the dam in front of the core.
In addition, the crest width is 12m and a freeboard of 8.5m is provided above Normal Water Level in order to prevent overtopping of the dam even in the event of a slide.

CONCLUSION

The analysis methods used for these two examples are well within the capabilities of engineers both during design and construction, provided the design is modified to suit the site. They are relatively economical and simple to use and interpret.

The lack of sufficiently quantified historical earthquake events, which is a quite frequent situation, has been overcome here by a sensitivity analysis to select the less favorable spectrum.

The amount of input data required is limited and is correlated with the void ratio and mean stress for the more conventional materials; for non conventional materials, special in situ and laboratory tests have to be performed.

The value of the mean static stress has an important impact on the values of the parameters and thereby, on the results. Determining this stress assumes a complete static analysis is made :

.construction in layers
.adherence to stages of construction
.proper behaviour patterns for the materials
.analysis of seepage forces, including in the foundation if necessary.

The three-dimensional analysis may be necessary for a dam in a narrow or steep-banked valley as Aït Chouarit dam, or when the dam site lies on a bend in the valley. Three-dimensional non-linear dynamic analysis will not be accessible to practising designers for many years yet but it is quite feasible to combine a three-dimensional non-linear static analysis with a simplified two-dimensional dynamic model.

The results described confirm that a good static design is generally a good dynamic design in the absence of any loose materials and for earthquakes of moderate intensity. Dynamic analysis does however enable this to be checked in detail and modify the design as necessary with reference to the following questions :

.If there are permanent displacements, where do they occur and can they affect the essential parts ot the dam like seepage control components (core, upstream facings, membranes) or the filters and drains or outlet works? Can they lead to the dam being overtopped?
.Are dynamic deformations acceptable in the different materials, notably special materials like the bituminous concrete in the facing, the plastic concrete in the cut-off or the membranes? But then an still laboratory tests has to be developped for determining the dynamic properties of such materials such as bituminous concrete or cement plastic concrete.
.Is there any danger of the intrinsic curves for the materials being reached during an earthquake?

The last question, which was the subject of the first example presented, must be answered before using the Newmark method, and may make it unnecessary. It does not in fact lead to absolutely identical conclusions, as shown by the analyses in the first paragraph. In both cases, irreversible displacements occur for friction angles of around 35°. But with the Newmark method, the dynamic stability diminishes from top to bottom in connection with the acceleration curve (fig. 5 and 6) whereas the R ratio contour (fig. 4) shows that the most highly stressed material varies from the top in the parts on a rock foundation to the middle and bottom where the alluvium thickness is maximum. The R ratio contour analyse, for a given section, shows the extreme state reached during the earthquake whereas the Newmark method averages accelerations varying from bottom to top and which are more or less out of phase. In fact, there is every advantage in studying the R ratio contour before calculating the permanent displacements by the Newmark method.

AKNOWLEDGEMENTS

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