SEISMIC SAFETY ASSESSMENT OF EARTHDAMS : A CASE STUDY

Bachir N. Touileb, Hydro-Québec, Montreal, Quebec, Canada
Hani Keira, SNC-Lavalin, Montreal, Quebec, Canada

ABSTRACT
The seismic safety assessment of earthfill dams is usually performed by means of a progressive methodology, starting by the consideration of screening level procedures and simple methods first. Then, based on the obtained results, more sophisticated methods such as a full dynamic analysis are realized. The main orientations of this approach are presented hereafter with the consideration of the seismic safety assessment of Manic-3 dam.

Manic-3 dam is a 108-meter high zoned earth fill embankment with a central till core. It is founded on a 126-meter deep canyon filled with granular material through which under-seepage is controlled by two deep cut-off walls. The construction of the dam, the reservoir filling and the dynamic response of the dam-foundation system were analyzed by means of the finite element method. The post-earthquake stability of the slopes of the dam and the permanent post-seismic crest settlement are evaluated.

RÉSUMÉ
L'analyse de la sécurité des barrages en terre sous l'action des séismes est habituellement réalisée au moyen d'une méthodologie progressive qui commence avec la considération d'un niveau dit de tamisage et de méthodes simples. Puis, selon la nature des résultats obtenus, des méthodes plus sophistiquées, telle que l'analyse dynamique complète, sont réalisées. Les orientations principales de cette approche sont présentées ci-après, avec la considération du cas de la sécurité sismique du barrage Manic-3.

Le barrage Manic-3 est un barrage zoné de 108 m de hauteur dont l'étanchéité est assurée au moyen d'un noyau central en moraine. Il est fondé sur un sillon alluvionnaire de 126 m d'épaisseur. La construction du barrage, le remplissage du réservoir ainsi que la réponse dynamique du système barrage-fondation ont été analysés au moyen de la méthode des éléments finis. La stabilité post-sismique des pentes du barrage et le tassement irréversible de la crête ont été évalués.

1. INTRODUCTION

Procedures involved in the seismic safety assessment of earthfill dams evolved very rapidly since the last thirty years. The lessons learned from liquefaction case histories such as Niigata earthquake (1964), Alaskan earthquakes (1964), and later on during the flow failure of Lower San-Fernando dam (1971) allowed for the recognition of the most influencing soil and earthquake parameters.

The compilation of several liquefaction case histories by B. Seed and co-workers led to the development of a practical and sufficiently accurate method based on the standard penetration test (Seed and Idriss 1971, Seed et al. 1981).

This procedure was later on refined by means of the introduction of appropriate correction factors, such as C_M (magnitude), K_α (pre-shear stress) and K_σ (confinement stress), taking into account the fact that most of the in-situ observed liquefaction were associated to free field shallow soil layers under earthquakes of magnitude 7,5.

The seismic safety assessment of a large dam is discussed based on a progressive methodology, and with reference to the assumptions at design stage.

2. MAIN FEATURES OF THE CASE STUDY

Manic-3 dam is a 108 m high and 390 m long zoned earth fill structure that was commissioned in 1975 (Figure 1). It is located 580 km North-East of Montreal (Quebec, Canada). The average upstream and downstream slopes of the dam are 3,0H:l,0V. The structure is composed of compacted sand and gravel and a central core comprising compacted till (Figure 2). The soil deposit is 126 m deep and contains a variety of pervious granular materials ranging from fine silty sand in the upper part to cobbles and boulders near the base. The watertightness of the foundation required the installation of a double cut-off barrier, consisting of two 0,6 m thick concrete walls spaced at 3,0 m centers. At the top of both walls, an arch steel and concrete gallery was provided for inspection and as an access to the instruments (Figure 3).

Taking into consideration the potential of differential deformations between the soil foundation and the concrete walls, a 3,0 m x 6,1 m bentonite cushion was incorporated at the top of the gallery which was designed to absorb any possible pinning or punching of the till core into the stiffer concrete walls; the settlement of the core being higher than the vertical deformation of the walls. Several bleeding pipes were installed all along the roof of the gallery to prevent high stress concentration.

Figure 1. Downstream view of Manic-3 dam

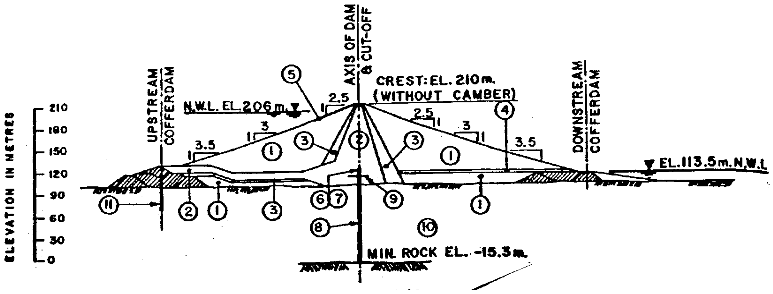

Figure 2. Cross section of the dam

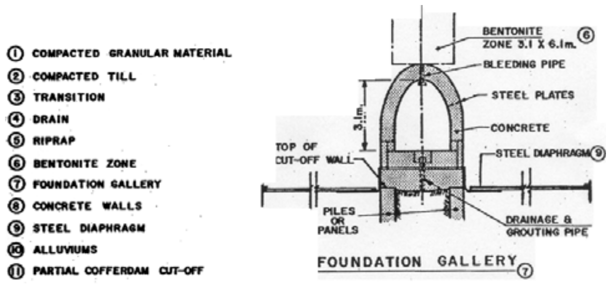

Figure 3. Inspection gallery and upper part of cut-off walls

3. SEISMIC STABILITY AT DESIGN STAGE

During the design stage in the beginning of the 70's, a seismic coefficient of 0,10 was used for the pseudo-static method. Furthermore, the effect of a strong earthquake in terms of seismically induced pore water pressures was considered and stated as follows : " The upper third (that is about 40 m) of the foundation soil deposit would loose 50% of its effective shear resistance under the impact of an earthquake. Under this condition, a factor of safety of 1,3 is required by means of a slope stability limit equilibrium analysis."

Such an assumption could be translated by means of the actual and more recent developments by stating that the earthquake would induce an excess pore water pressure ratio R_u of 0,5. And the effective shear resistance would be reduced as follows :

$$\tan(\phi_m') = (1 - R_u) \cdot \tan(\phi'_o) \qquad [1]$$

Where,

$$Ru = (\Delta U_s) / \sigma'_{vo} \qquad [2]$$

And,

ΔU_s = Induced pore water pressure,
σ'_{vo} = Initial vertical effective stress,
ϕ'_o = Initial effective friction angle,
ϕ_m' = Post-earthquake effective friction angle.

Consequently, if the initial soil friction angle before the earthquake is 35^o then, based on equation 1 above, an angle of 19^o is to be considered for the post-earthquake stability.

Under these conditions, a minimum post-earthquake factor of safety of 1,58 is calculated, which is sufficiently above the factor of safety of 1,30 that was specified at design stage.

4. SEISMIC HAZARD

The seismicity of the damsite is controlled by the LSL region and the expected earthquake at the site corresponds to the maximum magnitude of LSL which is M = 6,0.

The seismic hazard was evaluated by means of a probabilistic method that took into account the history of all recorded epicenters and the adopted seismic zoning for Eastern Canada. It leads to a design basis earthquake (DBE), corresponding to a probability of exceedance of 5×10^{-4} or 2% exceedance in 50 years, with a maximum horizontal acceleration of 0,15 g; g being the gravitational acceleration.

The seismic coefficient k to be considered for the pseudo-static method is 0,10 which is equal to the coefficient that was considered at design stage. Today, the multiple limitations of the pseudo-static method are known, and the method is being used more prudently than in the past such as discussed by Lefebvre et al. (2001).

A semi-deterministic approach, including the regional seismo-tectonic features and the nearest seismic zones known as Lower St.Lawrence (LSL) and Charlevoix (CHV), was also used for the evaluation of the maximum credible earthquake (MCE). Earlier, Leboeuf and Lefebvre (1989) considered that the MCE is produced by the maximum magnitude 5,5 earthquake ever recorded in the LSL seismic zone, at a depth of 20 km. The horizontal distance to the damsite is then 5 km only, and the calculated peak ground acceleration is about 17% g at damsite. If the maximum magnitude M_X = 6,0 of the LSL zone is used, then the peak acceleration range between 0,24 and 0,29 g. It is recalled that such a scenario is a very conservative one, since the probability of occurrence of the maximum earthquake of the LSL seismic zone on the nearest boundary to the damsite is very low.

The seismic zone CHV with a maximum magnitude M_x = 7,5 km is located at 200 km from the damsite. Based on the available seismic attenuation relationships for Eastern Canada (Hasegawa et al. 1981), CHV cannot induce a PGA of 0,30 g at the damsite. The maximum PGA due to CHV is found to be 0,17 g such as determined by Leboeuf and Lefebvre (1989).

Finally, the peak ground acceleration corresponding to a probability of annual exceedance of 10^{-4} was also evaluated. The corresponding peak ground acceleration is about 0,27 g.

It is important to notice here that the peak ground acceleration (PGA) corresponding to a probability of 1×10^{-4} is very close to the PGA corresponding to a magnitude 6 earthquake located 5 km away, along the nearest boundary of the LSL seismic zone; that is practically a magnitude 6 epicenter located under the dam. Here is then a case where the PGA corresponding to an MCE determined on the basis of a 10,000 years return period could not be conceivable except if the epicenter is located under the dam.

5. GENERAL PROCEDURE

The main steps of the analysis involved in a seismic safety assessment are :

5.1 Normal operating conditions

Determine the factor of safety of the dam under normal operating conditions.

5.2 Yield acceleration

Determine the yield acceleration K_y for the slopes of the dam. K_y is defined as that average acceleration producing a horizontal inertia force on a potential sliding mass so as to produce a factor of safety of unity and thus cause it to experience permanent displacements.

5.3 Initial stress-strain state

Determine the initial stresses and deformations by means of a numerical simulation of the construction and the filling of the reservoir.

5.4 Validation of the results

Compare the estimated stresses and deformations with measured values where applicable.

5.5 Initial shear stress

Calculate the initial shear stress ratio $\alpha = (\tau_o/\sigma'_{vo})$ in each finite soil element.

5.6 Stiffness and damping parameters

Select shear moduli and damping factors versus shear strain relationship curves for all soil materials by means of resonant column laboratory tests and refine them with Seed and Idriss (1970) curves if required.

5.7 Dynamic analysis and cyclic shear stresses

Estimate the cyclic stress ratio induced by the earthquake by means of the dynamic response analysis:

$$(CSR)_{eq} = 0{,}65\ (\tau_{max} / \sigma'_{vo}) \quad [3]$$

Where,
τ_{max} = induced maximum shear stress
σ'_{vo} = vertical effective stress

5.8 Resistance to liquefaction

Evaluate the nominal cyclic resistance ratio $(CRR)_{LO}$.

Estimate the field cyclic resistance ratio $(CRR)_L$ against liquefaction:

$$(CRR)_L = (CRR)_{LO}\ C_M\ K_\alpha\ K_\sigma \quad [4]$$

Where,
C_M : magnitude correction factor,
K_α : pre-shear stress correction factor,
K_σ : confinement stress correction factor.

5.9 Factor of safety against liquefaction

Estimate the local factor of safety against liquefaction :

$$F_{sL} = (CRR)_L / (CSR)_{eq} \quad [5]$$

5.10 Liquefaction criteria

For each finite soil element (Harder 1992) :

$F_{sL} < 1{,}2$: liquefaction is likely to occur. Assign corresponding undrained residual resistance for the post-seismic stability analysis,

$1{,}2 < F_{sL} < 1{,}4$: pore water pressure (ΔU_s) are induced by the earthquake cyclic shearing, causing a reduction of the initial effective strength,

$F_{sL} > 1{,}4$: liquefaction does not occur.

5.11 Post-earthquake stability

Evaluate the factor of safety F_s against sliding for each slope of the dam, using the post-liquefaction parameters (i.e. : residual strengths and R_u).
$F_s > 1{,}15$: the dam is stable, however it could experience some permanent deformations or settlements.
$F_s < 1{,}15$: the dam could need corrections and remedial measures. However, it is advised to consider undisturbed soil sampling first, such as by means of soil freezing techniques (Sego et al. 1993, Konrad and Pouliot 1997). The consideration of an effective stress dynamic analysis of the dam-foundation system by means of more sophisticated soil models, such as described for instance by Byrne (1991), Finn et al. (1986) and Finn (1991) could also be considered. A comprehensive description of the methodology was described with the consideration of the seismic safety assessment of a large dam by Byrne et al. (1993).

6. GEOTECHNICAL INVESTIGATIONS

The review of available geotechnical investigations and laboratory data, such as reported by Tavenas et al. (1970) for instance, and structural data based on the monitoring of the concrete of cut-off walls allowed for the evaluation of the parameters required for the preliminary analyses. A comprehensive in-situ and laboratory investigation program was established and realized in 1991 (Figure 4).

Figure 4. Geotechnical investigations at the downstream toe of the dam

The actual field investigation program that was conducted downstream of the dam consisted of four (4) standard penetration test (SPT) boreholes, six (6) large diameter - 140 mm - Becker penetration density tests (BPT), and geophysical down-hole and cross-hole tests (Table 1). Some down-hole tests were also successfully performed in the body of the main dam through existing vertical inclinometers.

Table 1. Field investigation program

Field test	Principal investigators
SPT	Monter-val (Montreal)
BPT	Western Caissons (Calgary)
BPT Energy measurements	Terratech (Montreal)
CPT	Hydro-Ontario (Toronto)
Geophysical	Géophysique Sigma (Montreal)

Three main critical granular units were identified in the alluvial foundation : soil unit # 1 (fine sand and silt), soil unit # 2 (medium sand) and soil unit # 3 (fine silty sand).

The average shear wave velocity measured by means of the cross-hole tests was found to about 400 m/s. Figure 5 shows a typical SPT index profile along the soil deposit.

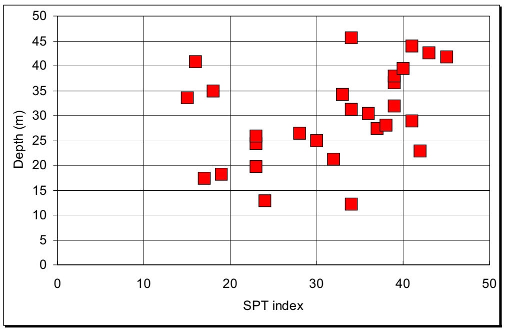

Figure 5. Typical SPT index values

7. LABORATORY TESTS

Fourteen (14) drained triaxial tests were performed in the design phase of the dam (1966-67) on loose and dense reconstituted foundation soil samples.

The new laboratory program consisted of a minimum and maximum density tests, a series of drained and undrained static triaxial tests, resonant column tests, and static and cyclic simple shear tests (Table 2).

Table 2. Laboratory tests program

Laboratory test	Principal investigators
• Triaxial tests • Simple shear cyclic tests • Resistance to liquefaction • Post-cyclic behaviour • Resonnant column	Prof. Guy Lefebvre (University of Sherbrooke)
• Steady state line	Prof. Jean-Marie Konrad (Laval University)

The number of representative cycles needed for the laboratory cyclic tests was determined following the findings of Seed et al. (1975) and Annaki and Lee (1977), and the results of Leboeuf and Lefebvre (1989) in the specific case of Eastern Canada such as presented in Table 3.

Table 3. Laboratory tests program

Magnitude	Representative cycles	
	Eastern Canada	Western USA
7,5	12 to 16	15
6,75	-	10
6	7 to 10	5 to 6
5,25	-	2 to 3

8. RESISTANCE TO LIQUEFACTION

The evaluation of the liquefaction resistance of sand samples was conducted using constant volume simple shear cyclic tests. The nominal cyclic resistance ratio against liquefaction $(CRR)_{Lo}$ was established to be in the vicinity of 0,32 based on the laboratory tests conducted on reconstituted soil foundation samples. It was determined based on 5% deformation or true liquefaction at a maximum of 10 cycles. Most of the liquefaction types observed in the laboratory are related to the limited liquefaction type; meaning that an important gain of strength is mobilized under any further shearing.

The K_α factor, which takes into account the initial existing static shear stress (τ_o) effect on the liquefaction resistance by means of the existing shear stress ratio $\alpha = (\tau_o / \sigma'_{vo})$ was determined using a series of simple shear cyclic tests at constant volume (Vaid and Chern 1983). Regarding the three sand units under consideration, the K_α factor or was established to vary as $K_\alpha = (1 + 1{,}56\ \alpha)$ for soil units #2 and #3, and as $K_\alpha = (1 - 0{,}43\ \alpha)$ for soil unit #1.

The nominal resistance to liquefaction being determined in the laboratory on Manic-3 specific samples showed that K_σ is independent of confining stress ; so as no correction is applied for the confinement. However, when the resistance against liquefaction is determined following the Seed procedure, it is recommended to apply a correction factor that is lower than unity (Pillai, 1994).

The C_M factor is set to unity when the magnitude of the considered earthquake is 7,5. For a magnitude 6,0 controlling earthquake, the factor C_M is set to 1,32. Such a correction is applied only if the nominal CRR is derived from Seed's chart.

9. STEADY STATE LINE

The steady state line was estimated using undrained triaxial tests (Castro and Poulos 1977, Poulos 1981, Konrad and Pouliot 1997). It allowed for the estimation of a residual resistance for the fine sand ranging from 36 kPa to 65 kPa. Based on this approach also, a value of 40 kPa, close to the minimum value, was found to be representative. This corresponds to an initial void ratio of 0,65 and a relative density index of 60%.

10. RESIDUAL STRENGTH

The undrained residual strength S_{ur} of the different sand units under consideration was determined by means of two different methods.

Based of an average normalized SPT index $(N_1)_{60}$ of 16, the chart established by Seed produced a residual shear strength of 40 kPa (Seed and Harder 1990).

On the other hand the steady state tests conducted to a similar S_{ur} of about 36 kPa. Accordingly, a residual strength of 40 kPa was used.

11. STRESS AND STRAIN ANALYSIS

Prior to the dynamic analysis, an evaluation of available computer programs for the calculation of stresses and deformations was performed such as reported later on in Keira et al. (1995). The pre-earthquake state in the whole continuum was then determined by means of CON2D-90 finite element program (D'ozario et al., 1991) which involves both structural concrete beam and interface elements, in addition to the built-in modified elasto-plastic Cam-Clay model for soils.

The introduction of interface elements proved mandatory due to the major contrast of stiffness between different parts of the dam-foundation system. Higher quality results were obtained when interface elements were introduced between the bentonite cushion and concrete walls and between the concrete walls and soil materials. In the latter case, the friction coefficient at the interface was estimated using the results of a full scale pile pull-out test that was undertaken in the natural soil deposit, before the construction of the dam. An angle of friction of 8^o for initial slippage was derived from the test and this value was used in the analysis.

Several specific features of Manic-3 dam were indicated by this analysis, such as the arching effect at the base of the core and the differential settlements along the concrete of the cut-off walls causing a negative friction pile behavior (Lefebvre et al. 1977).

The calculated dam crest settlement is about 0,70 m while the measured settlement was about 0,89 m. The calculated horizontal displacement of the gallery floor is about 0,17 m, while the measured value was about 0,20 m. These targeted comparisons between calculated and measured values, as well as other comparisons, for instance along the cut-off walls, were found to be acceptable confirming that the soil and structural parameters used are reflecting adequately the behavior of the dam.

12. DYNAMIC ANALYSIS

The dynamic analysis of the dam-foundation system was performed using the computer finite element program FLUSH (Lysmer et al., 1975) that is based on Seed's linear equivalent to simulate the cyclic behavior of the soil materials, allowing for shear moduli and damping factors to vary for each finite element.

The choice of that particular computer code is due mainly to the fact that the concrete walls should be considered as structural elements. Concrete is a different medium through which seismic waves travel differently than neighbor soil of the foundation.

The upper layer of the foundation, extending under a portion of the upstream cofferdam (Figure 2), was found to approach liquefaction (F_{sL} < 1,2) under the MCE conditions. On the other hand, the additional dynamic stresses, shears forces and flexural moments acting on the structural elements such as the concrete walls and the steel-concrete gallery were determined and combined with the existing stresses.

All the combinations were found satisfactory, except near the contact of the walls and the bedrock, where a shear failure is likely to occur in the walls under a strong earthquake. Considering the in-situ confinement at this depth - 126 m under the surface of the soil deposit - and the limitations involved with the actual elastic modeling of the concrete, it is believed that such a local failure could open a new crack into the concrete allowing for a concentrated seepage but does not reduce the integrity of the cut-off nor the foundation due to the depth and the length of the seepage path.

Additional parametric studies centered on the variation of the modulus of deformation of the concrete walls due essentially to creep effects were also performed.

13. DYNAMIC RESPONSE DURING THE M_S5,9 1988 SAGUENAY EARTHQUAKE

The dam is equipped with five seismographs which were triggered during the M_s = 5,9 Saguenay earthquake of November 26th, 1988 which is located at 255 km from the damsite. Despite a very low peak ground acceleration of only 0,5% g at the damsite, due to its long distance from the epicenter, the analysis of the dynamic behavior of the dam allowed for the calibration of stiffness and damping material parameters at small strains.

Moreover, an analysis of the recorded accelerograms allowed Rainer and Dascal (1991) to determine the fundamental periods of the dam (Table 4).

Table 4. Fundamental periods of vibration of the dam

Direction	Fundamental period of vibration (s)
Transversal	0,72
Longitudinal	0,63
Vertical	0,28

14. POST-SEISMIC STABILITY

A yield acceleration K_y of 0,20 was determined in the case of the upstream slope of the dam. A post-seismic factor of safety sufficiently higher than 1,15 was determined using the appropriate soil parameters. The dam is thus stable even if it would experience some minor permanent settlements at the crest.

15. POST-EARTHQUAKE SETTLEMENT

Different simple methods are available for the evaluation of the permanent post-seismic deformations : Makdisi and Seed (1978), Jansen (1988) and Newmark (1965). However, all of these methods apply only if seismically induced pore pressures remain small with no trigger of liquefaction.

The application of the Newmark method such as it is implemented in the program DISP developed by Chugh (1980) with the consideration of the acceleration time history acting on the sliding mass and the residual post-earthquake parameters led to a maximum crest settlement of 0,45 m.

Another procedure consisting on the determination of the additional post-seismic displacements that are required to balance the existing gravity loads, while residual resistance, reduced stiffness and appropriate pore water pressures are assigned to the soil elements based on the liquefaction criteria discussed in section 5.10 above.
The post-seismic permanent settlement of the crest was then about 0,30 m and the horizontal displacement towards the upstream was about 0,10 m.

These deformations are small and tolerable compared to the 4,5 m high available freeboard. According to the past North-American practice, such as the U.S Bureau of Reclamation and U.S. Corps of Engineers, post-seismic settlements up to 0,60 m were considered acceptable under a strong earthquake.

Such a post-earthquake settlement is now thought to be not an absolute issue as long as an appropriate freeboard is guaranteed after the earthquake, and that the reservoir is safely maintained despite minor damages to the structures. Babitt and Stephen (1996) stated that a permanent settlement below 1,5 m is acceptable for the dams of California Department of Water Ressources.

16. CONCLUSION

The dynamic response analysis of Manic-3 dam was performed by means of the Seed's linear equivalent method that is incorporated in the finite element program FLUSH. An extensive field and laboratory investigation of the granular alluviums of the foundation was performed to provide the necessary soil parameters.

The dam is stable under the considered earthquakes both under the conditions of the maximum design earthquake and the maximum credible earthquake.

17. ACKNOWLEDGMENTS

The authors are grateful to Dr. Liam Finn (University of British-Columbia) who reviewed the procedure established at the beginning of the actual study.

They are also grateful to Dr. Guy Lefebvre (Université de Sherbrooke, Québec) and Dr. Jean-Marie Konrad (Université Laval, Québec) for their advice and valuable comments during the preparation and the realization of the laboratory tests program.

The authors would like also to thank MM. Robert Piché, Yousef Hammamji, Kent Murphy, Phuong H. Nguyen and Gérard Verzeni (Hydro-Québec) for their valuable comments and for supporting both the project and the actual contribution.

Dr. Robert D. Jansen (Ohio, USA) and Dr. Giovanni Lombardi (Lugano, Switzerland) reviewed the conclusions of this study as members of the Hydro-Québec board of experts.

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