Backanalysis of Deformations for Case Histories Involving Flow-Type Failures

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SYNOPSIS: The results obtained from backanalyses of two case histories of failures of small earth dams resulting from liquefaction of foundation soils are presented. The analyses have been performed in order to evaluate the deformation parameters used in a finite difference model (developed by Woodward-Clyde Associates), specifically those relevant to the liquefied soil layer. For both case histories, the final deformed shape of the earth dam is known and serves as a basis for comparing the results from the numerical study, in terms of the deformation moduli employed.

INTRODUCTION

In order to evaluate the necessary remedial measures for a coastal protection system, a series of deformation analyses have been performed for critical sections of the dykes of the Costa Oriental del Lago de Maracaibo (COLM), the critical condition occurring as a result of liquefaction of the foundation soils. The results of the deformation studies are combined with those from stability analyses to design remedial measures for the dyke system. The analyses performed correspond to the post-earthquake condition, that is to say, the situation can be analyzed statically using soil parameters that include any degradation induced by the earthquake shaking. The conditions assumed for this to be true can be summarized as:

- the design seismic event is of sufficient magnitude so as to cause initial liquefaction of the granular foundation soils,
- during the period of shaking there are no large mass movements nor failure of the dyke-foundation system,
- in terms of stability, the critical condition occurs immediately after shaking stops,
- the critical condition exists for a finite period of time after shaking and is not modified due to redistribution of stress or pore pressure,
- the principal failure mode is well-represented by the development of a continuous failure surface and not affected by the formation of sand boils or other phenomena,
- the post-seismic condition is the critical one and can be analyzed by static methods taking into account any strength or stiffness reduction that may occur as a result of ground shaking,
- as a worst case scenario for both stability and deformation analyses, the granular layers in the foundation are considered to be strain-softening (contractive) and have a strength representative of the residual condition. (For the granular layers, the residual strength, S_{uss} or S_r has been determined as being equal to 1 t/m^2 or $0.12\sigma_v'$, whichever is the greater.)

For the stability analyses, determination of the strength parameters is a reasonably straightforward process. Even for the liquefied soils, considerable amounts of published data are available, both from laboratory and field measurements, to facilitate the selection of a design residual strength, albeit conservative. However, very little information exists for assisting in the determination of deformation parameters for liquefied soils.
In order to provide information on the possible range of values for the soil moduli at or near liquefaction, it was decided to perform backanalyses of small earth dams where slope deformations had occurred as a result of loss of strength in the foundation soils due to liquefaction. This paper considers the results obtained from two analyses.

DETAILS OF CASE HISTORIES

The two cases considered correspond to the failure of small earth dams as a result of liquefaction of foundation soils. These particular earth dams have been selected as they have heights similar to those in the COLM area. The term failure is used in the broad sense to indicate the presence of large deformations, generally associated with a limit state stability condition, i.e. a post-seismic safety factor equal to or less than unity. Details of the particular conditions at each of the sites are given below.

La Marquesa Dam, Chile

The La Marquesa dam in Chile underwent a slope failure in 1985 as a result of a magnitude M_s=7.8 earthquake whose epicenter was estimated to be some 45 km from the dam site. Details of the case history are presented by De Alba et al. (1986) and will be briefly reviewed here in relation to the determination of soil parameters and the magnitude of deformations resulting from the liquefaction of the foundation soils.
In the area around La Marquesa there are numerous small earth embankments constructed for water storage purposes. The primary material used was a silty clayey sand which was placed with a relatively low degree of compaction. The estimated maximum acceleration at the La Marquesa dam is around 0.6 g in the north-south direction. The dam is approximately 10 m high

with a 220 m crest length and a storage volume of 204 000 m^3. The dam was constructed in 1943 using a local silty and clayey sand with a more plastic material being placed in the central core.

As a result of the 1985 (03/03/85) earthquake, both upstream and downstream sliding occurred, the largest movements being recorded on the upstream side. A 2 m loss of freeboard occurred in the central area of the dam. Extensive longitudinal cracking was visible in the upstream slope after the reservoir level had been drawn down. Crack widths up to 0.8 m were measured with crack depths achieving 2.0 m. Horizontal displacements of about 11 m at the toe of the upstream slope and 6.5 m at the downstream toe were measured. De Alba et al. (1986) argued that the level and pattern of deformation experienced by the dam slopes were indicative that the principal mode of failure was that associated with liquefaction of the granular soils in the dam foundation. Details of the initial and failed cross-sections are given in Fig. 1.

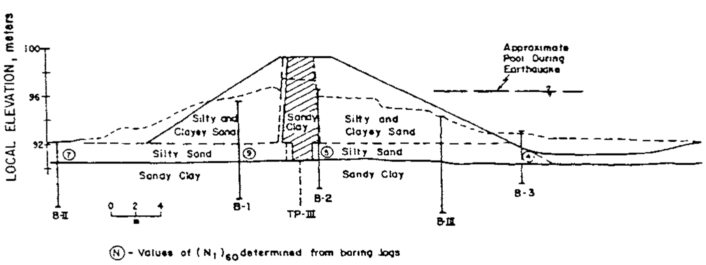

Fig. 1 Cross-section of La Marquesa (De Alba et al. 1986)

Subsequent boreholes B1 and B2 (Fig. 1) and SPT testing (energy calibrations using a Binary Instruments 102 SPT Calibrator) gave $(N_1)_{60}$ values of 4 for the upstream foundation and 9 for the downstream foundation below the dam. The respective fines contents at these two locations were 30% and 20%, giving fines-content corrected $(N_1)_{60}$ values of 6 and 11.

Based on a series of backanalyses for the failed upstream and downstream sections, De Alba et al. (1986) evaluated a range of yield accelerations and residual strengths for the liquefied material. Yield accelerations varied from 0.012 g (upstream) to 0.03 g (downstream) and the backcalculated residual strength values were in the range 76 to 340 psf (3.6 to 16.3 kPa) for the upstream area and 266 to 580 psf (12.7 to 27.8 kPa) for the downstream foundation. (It is interesting to note that these values would correspond to a S_r/σ_v' ratio varying from approximately 0.09 to 0.18).

Lake Ackerman Roadway Embankment, Michigan USA

The Lake Ackerman flow slide is unusual in that the failure was triggered by vibratory tampers during a seismic reflection survey. Full details are given by Hryciw et al. (1990).

The fill and embankment were constructed in the mid-1950's to carry a two-lane highway across the northern tip of Lake Ackerman. The clean, medium to fine, sand fill was placed by end-dumping into the water which resulted in a very loose hydraulic fill as deep as 3.7 m. Above lake level, the fill was moderately compacted.

On July 24, 1987, a six-truck crew were performing seismic reflection studies on the Lake Ackerman fill when a 91 m long section of the embankment failed as a result of liquefaction of the loose hydraulic fill described earlier. The signal generation technique was estimated to have induced a cyclic stress ratio of 0.12 in a material having corrected SPT $(N_1)_{60}$ values of between 1 and 4. Residual strength upper and lower bound estimates ranged from 8.1 to 12.4 kPa, corresponding to approximate strength ratios of 0.19 to 0.30.

No detailed information on the magnitude of the deformation of the embankment/slope is given in the paper. However, from Fig. 8 of the paper by Hryciw et al. (1990), at Station 871 + 00 the deformed embankment can be seen to have suffered a vertical displacement of about 10 ft (3.05 m) below the crest and is, on average, just above lake level. Lateral displacements are much more difficult to estimate from the information provided, but are roughly of the order of 40 to 50 ft (12 to 15 m).

The slope geometry at Station 871 + 00 is shown in Fig. 2 and the schematic failure profile in Fig. 3. This section was chosen for performing the deformation backanalysis.

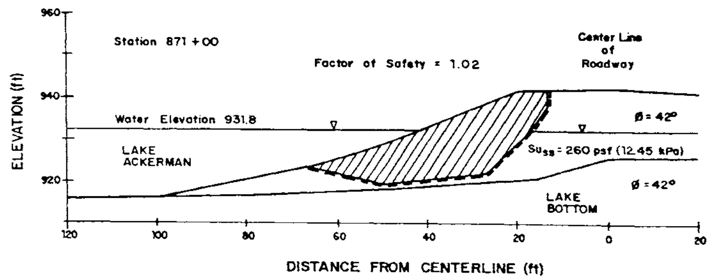

Fig. 2 Geometry for Lake Ackerman failure (Hryciw et al. 1990)

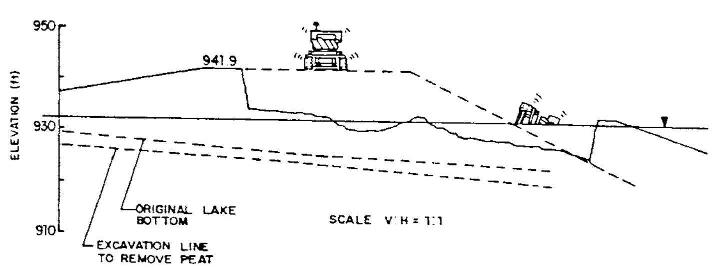

Fig. 3 Failed cross-section at Lake Ackerman (Hryciw et al. 1990)

METHOD OF ANALYSIS

Program DYNARD

The program DYNARD was used to analyze the two previously described failures in order to estimate the values of the moduli in the liquefied granular materials. The behavior of the liquefied layer(s) controls the pattern and magnitude of the deformations, the moduli of the other layers being of secondary importance provided the undrained condition is essentially maintained (i.e. no volume change). For this reason, it was possible to perform a sensitivity study as to how the modulus value of the liquefied layer affected the overall pattern and magnitude of deformations. Details of the DYNARD program and capabilities can be found in references by Woodward-Clyde Associates (1992) and Fernández and Sully (1992).

Modulus for Liquefied Soil

The modulus values for the liquefied soil layers were determined from the results of undrained triaxial compression tests performed on undisturbed samples of the foundation soils from Lagunillas and Tia Juana. The tests were carried out as part of a study into the residual strength of the granular soils in the subsidence areas where dikes have been constructed. The modulus so determined corresponds to the secant value at the strain corresponding to the initiation of steady state conditions.
The results from the laboratory tests were interpreted in terms of the hyperbolic-type soil model used in DYNARD and a possible range of K_G values were used in the backanalyses. K_G values in the range 0.76 to 48 were initially used in the analyses. For the bulk modulus, K_B values were adjusted to ensure an undrained analysis, i.e. zero volume change. For the liquefied layer, a K_B value of 60 000 was used. These parameters are similar to those employed in the analyses of the COLM dyke sections as presented by Sully et al. (1993).

RESULTS FROM BACKANALYSES

La Marquesa Dam, Chile

Based on a series of sensitivity analyses in which the shear moduli of the non-liquefiable materials were varied, it became apparent that the deformations were controlled almost entirely by the moduli values in the liquefied layers below the dam. All subsequent analyses were performed to evaluate the effect of the liquefied soil modulus on the magnitude and pattern of deformation.
In the hyperbolic-type model, both the bulk and shear moduli can be obtained from the following relationship:

$$M = K_M(\sigma')^n \qquad (1)$$

where M is the modulus of interest (shear (G) or bulk (B), in this case), K_M is the modulus factor, σ' is the effective stress and n is a laboratory determined exponent describing the dependence of M on σ'.
K_B values of 60000 and K_G values between 0.76 and 48 (for modulus values in units of t/m^2) were used as stated earlier for the backanalyses. The range of K_G values was obtained from laboratory test data and chosen to reflect possible upper and lower limit estimates for the post-seismic deformations, should the foundation soils liquefy as a result of ground shaking. The residual strength of the liquefied layer is also important in terms of controlling the mode of deformation. For these analyses, a residual strength of 1 t/m^2 was used for all the liquefied layers. This value is within the range determined by De Alba et al. (1986) based on stability calculations.
The deformations at several locations on the dam are presented in Table 1. As can be seen the variation of the modulus number from 0.76 to 48 does not have a major effect on the final displacements.
The general deformation pattern for the dam is shown in Fig. 4 where it can be seen to be in good agreement with the measured deformed section shown in Fig. 1.

Lake Ackerman Roadway Embankment, Michigan USA

Using similar parameters to those described for La Marquesa, deformation analyses were also performed for the slope failure at the Lake Ackerman embankment near Michigan.

Table 1 Results of deformation analyses for La Marquesa dam

Location on dam	$(\delta_v)_{max}$ (m)	$(\delta_h)_{max}$ (m)
D/S toe	1.0 to 1.3	2.3 to 2.7
D/S crest	1.5 to 1.7	2.4 to 2.7
U/S crest	0.6 to 1.1	3.2 to 3.7
U/S toe	0.4 to 0.7	3.2 to 3.7

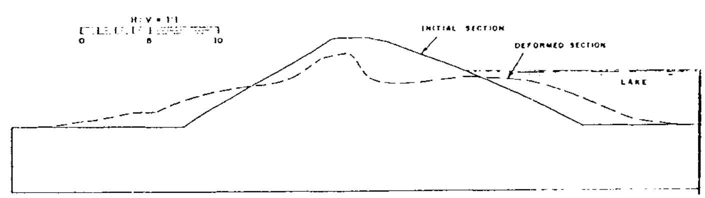

Fig. 4 Results of deformation analyses for La Marquesa dam

The deformations in the area of the slope crest were compared with the values estimated from Fig. 8 of the paper by Hryciw et al. (1990) (Fig. 3 in this report). In terms of vertical movements, a range of values between 1.6 m and 2.0 m was obtained, while the horizontal displacements ranged between 2.6 m and 3.4 m. However, as indicated in Fig. 5, the deformations do not represent the profile shown in Fig. 3. Furthermore, even varying the values of the modulus parameters outside the range given above it was not possible to obtain a good representation of the field measured response. In an attempt to consider all possible factors, it was decided to consider the static loading imposed by the presence of the trucks performing the seismic refraction survey. A static distributed surface pressure of 8 t/m^2 over a 2.5 m width (to represent a 20 t truck) was considered at the top of the slope.

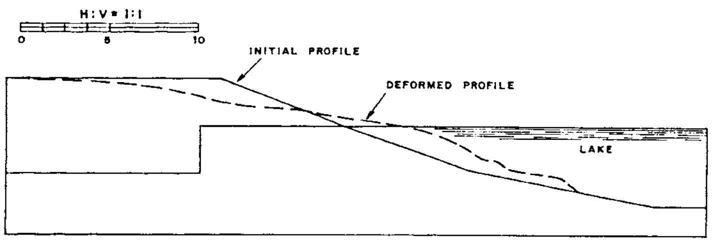

Fig. 5 Results from initial deformation analyses for Lake Ackerman embankment

The results from the deformation analyses considering the presence of the surface distributed pressure are shown in Fig. 6. As can be seen, the results are more representative of the deformed profile measured in the field after the failure due to liquefaction. For this analysis, the calculated vertical deformations in the area of the surface load were in the range 2.7 m to 3.6 m while predicted horizontal deformations were in the range 3.6 m to 4.7 m. These values are in better agreement with the field measured values.

CONCLUSIONS

The results presented from the above analyses would appear to validate the modulus parameters being used in conjunction with the DYNARD program. However, while the vertical deformations predicted by the program are in good agreement with the magnitudes measured in the field, the calculated horizontal displacements are considerably less than the measured values. This may be a result of the inability of the program to truly consider the flow-type failure response and the necessity to maintain an undrained no volume-change condition. This aside, based on the results of these two backanalyses, it appears that the program DYNARD, in association with the residual strength and liquefied modulus suggested by laboratory results from undrained triaxial tests, may provide realistic vertical deformations of earth structures on liquefied foundation soils, especially considering the pseudo-dynamic nature of the analysis and the assumptions involved. Further case studies where flow-type failures have occurred are being reviewed to provide additional verification of the procedure for varying geometries and soil conditions.

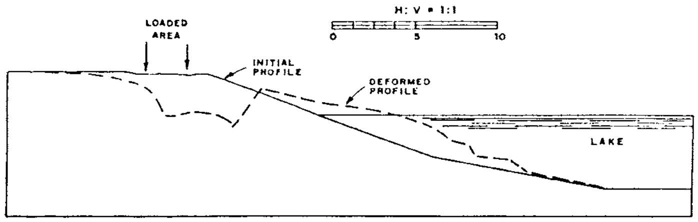

Fig. 6 Deformation pattern considering surface distributed pressure

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