Selection of Most Appropriate Procedures for Seismic Evaluation of Levees Based on Case Histories

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ABSTRACT

The current methodology for embankment dams evaluation is not appropriate for levees of low heights, which have little effect on the stress state of the foundation soil and, therefore, on its response to the seismic action. In many cases the liquefaction potential of the alluvial deposits is not affected by the levee presence, although is the main factor in levee degradation. Lateral spreading of liquefiable foundation is the main cause of small levee cracking and settlement induced by earthquakes. On the other part, the procedures recommended for evaluation of lateral spreading are not directly applicable to the analysis of levees. In this paper both categories of procedures (for dams and for free field affected by lateral spreading) are applied comparatively for evaluation of a case history. This paper summarizes results of different procedures on a case history, where a California levee was severely damaged during Loma Prieta earthquake. Recommendations are made for the analysis of various categories of levees.

INTRODUCTION

Levees are generally not designed for the seismic action, based on the relatively low probability of simultaneous occurrence of a flood event and a strong earthquake. However, failure of levees, especially when they are frequently hydraulically loaded, may have catastrophic consequences. Levees with permanent water retention should be analyzed similarly with dams and their seismic behavior evaluation is mandatory.

The authors are currently involved in the development of two documents referring to seismic evaluation of levees, which have the same general objective but different purpose, scope, and applicability: Guidance Document for performing a screening level seismic vulnerability analysis for urban levees under the jurisdiction of California Department of Water Resources (DWR) and US Army Corps of Engineers (USACE) draft Engineer Technical Letter (ETL) on Guidelines for Seismic Evaluation of Levees, which has broader potential applicability.

The main purpose of both documents is primarily the evaluation of levee vulnerability under existing conditions. Depending on the potential consequences of failure, the recommended procedures can be used for design of mitigation measures. The major mechanism of levee degradation under seismic action is considered the liquefaction of the alluvial levee foundation; therefore, most of the recommendations refer to evaluation of the foundation soil liquefiability and its possible effect on the levee integrity.

SEISMIC LOADING ASSESSMENT

There are two main parameters in common use for defining the earthquake loading for liquefaction assessment purpose:

- Peak ground acceleration (PGA), which is the largest value of the acceleration at the free field ground surface;
- Earthquake magnitude (M), which is a measure of the earthquake size/energy; the preferred definition is the moment magnitude.

The evaluation of levees does not usually require a site-specific seismicity assessment, but is generally based on existing evaluations (e.g. ground motion maps developed by

United States Geological Survey, USGS). Currently the 2008 Interactive Deaggregation site developed by USGS is an attractive solution, accessible via: https://geohazards.usgs.gov/deaggint/2008/. Although this is a preliminary version in beta test stage, for California the most recent Next Generation Attenuation (NGA) developments have been incorporated. The level of modernization of the interactive site is different for various zones of the US, but it is expected that USGS updates this software; the latest USGS ground motion calculation tool should always be used to compute ground motion intensity (defined through PGA) and deaggregation (useful for M evaluation).

The interactive USGS web site requires three main input data:

- Location, through either postal address or longitude/ latitude;
- The ground motion return period, defined through the exceedance probability;
- Site condition, through the average shear-wave velocity in the top 30 m, V_{s30}.

Generally the selected return period should be about the same level as the flood return period. DWR currently requires a 200-year return period for seismic evaluations, which is consistent with the targeted 200-year flood protection level.

The ground motion amplification is a function of site conditions, which is generally evaluated through V_{s30}. In alluvial liquefiable cohesionless deposits, where shear wave velocity measurements are not available, the best way of site condition assessment is through the average Standard Penetration Test (SPT) blow counts (N) for the top 30 m. As N is a proxy for V_{s30}, the average value should be obtained through the harmonic mean, which gives much more weight to low values, encountered generally near the surface, than to deep high values.

FIRST SCREENING IN SEISMIC VULNERABILITY EVALUATION

The USACE draft ETL states that there is no need for seismic evaluation of agricultural or wetland levees, if there is no landside human habitation or infrastructure that could be damaged by flooding.

Additionally, there is no need for seismic evaluation if a PGA < 0.1g at the levee's location. This value is derived from observations of levee damage as the result of past earthquakes.

COINCIDENT WATER LEVEL

As only saturated materials should be assumed potentially liquefiable, a typical water surface elevation should be considered during liquefaction triggering analysis and seismic slope stability analysis.

The highest of the following three levels should be used to determine the coincident water level for combining with a 200-year return period or a less frequent seismic event:

- The median annual water level; that is, the river level or groundwater level, whichever is higher.
- The typical seasonal water level. For levees where the impact of failure would be low, the typical seasonal water level should be the average water level during the wettest month of the year, and is preferably a 10-year average (e.g. February for California's Central Valley levees). For levees where the impact of failure might be severe, 84^{th} percentile of seasonal water level should be considered as the typical seasonal water level.
- The mean high tide elevation, for levees affected by tides. In these cases, consideration should be given to the predicted sea level rise expected in the decades ahead.

LIQUEFACTION TRIGGERING ASSESSMENT

It is postulated that levees would be significantly damaged by a strong earthquake only if the foundation soil is liquefiable. To simplify the problem in the levee analysis case, evaluations should generally focus on potentially liquefiable coarse-grained soils and fine-grained soils with low plasticity (sand-like). Fine-grained clay-like soils, defined as soils with the plasticity index, PI ≥ 10 are assumed non-liquefiable. Borderline materials, like CL-ML, CL, and ML soils with PI < 10 are analyzed using criteria for sand.

Although currently there are several widely accepted procedures for liquefaction triggering assessment, that consider recently observations from case histories, the USACE draft ETL recommends the methodology based on the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils (Youd et al., 2001) that is widely accepted as state-of-the-practice and represents a 5-year joint effort among a group of specialists from the United States, Canada, and Japan. The paper's 21 authors include 3 representatives from USACE. The DWR Guidance Document recommends more recently published procedures (Seed et al., 2003, Moss et al., 2004).

As the main result of liquefaction triggering assessment the factor of safety against liquefaction, FS_{liq}, is obtained. The resulted liquefiability Index [$(N_1)_{60\text{-}cs}$ for SPT and q_{c1N} for CPT, Cone Penetration Test] values are used for both calculation of FS_{liq} and to develop residual undrained shear strength (S_r) values for the seismic deformation analyses. Two procedures for evaluation of S_r are recommended by the USACE draft ETL: Seed and Harder, 1990 and Olson and Stark, 2002.

As a second screening in seismic vulnerability evaluation of levees, further consideration is not needed if the factor of safety against liquefaction, FS_{liq}, is greater than 1.0 within all investigated depths.

LEVEE SEISMIC BEHAVIOR EVALUATION

It is considered that two basic modes of distress may be induced in levees, depending on the in situ stress condition after seismic liquefaction occurrence:

- When the static driving shear is greater than the post-earthquake residual strength along a critical slip surface, **flow slide or post-earthquake instability** is probable;
- When the static driving shear is less than residual strength, but static shear stresses plus the inertial shear stress during shaking periodically exceed the available shear strength, **lateral spreading or earthquake induced deformation** is possible.

Both natural phenomena are expected to induce major levee distress, through loss of freeboard due to settlement, and through longitudinal and transverse cracking that may lead to internal erosion.

The distinction between the two cases above is made for selection of different analysis methodologies. Although lateral spreading or earthquake induced deformation generates less displacement than a flow slide, a larger deformation may sometimes occur. Additionally, both mechanisms may occur during the same event. It is difficult to predict which case is most probable under specific conditions because of the many parameters involved, including: levee height, shaking intensity, and the foundation soil's liquefiability and post-liquefaction strength. Therefore, it is recommended to begin analysis assuming the occurrence of a flow slide and, if results indicate that no flow slide is probable, to next analyze the levee assuming possible lateral spreading.

FLOW SLIDE ANALYSIS

The flow slide failure occurs when the post-liquefaction strength of a soil is not sufficient to maintain stability under static loading alone (i.e., after earthquake shaking is over). In this case, static instability can result in deformation, additional to those occurred during shaking, leading to a greatly deformed post-earthquake geometry. The factor of safety (FS) against flow slide can be obtained by limit equilibrium methods for post-earthquake strengths, and under static conditions.

If the limit equilibrium analysis calculates a FS against flow slide less than 1.0, significant damage is likely to occur and that the levee is likely to be compromised. No further analyses are required, as complete loss of the levee should be expected. However, if it is necessary to evaluate the post-sliding stable geometry of the levee, either successive post-earthquake limit equilibrium analyses (until an FS in excess of one is reached) or nonlinear analyses using finite element or finite difference programs should be performed.

If the factor of safety against flow slide is greater than 1.0, it is not likely that the levee will be affected by flow failure. However, it may still be vulnerable to damage by lateral spreading and stability under the lateral spreading condition, which should be investigated.

LATERAL SPREADING ANALYSIS

In this case, large lateral displacements can be expected in the levee, which can induce both cracking and settlement. In addition to the potential liquefaction extent of foundation, a major factor affecting the displacement is the distance from the levee to a free surface, open channel slope, or river bank.

All of the evaluation methods fall into one of two categories regarding assumptions. The first category of evaluation assumes a levee's presence has little effect on overall stability, if any, and that levee damage is induced primarily by foundation soil failure. The second category of evaluations considers displacements primarily generated by the embankment loading. It is considered appropriate of using the second category of evaluation when a levee is more than approximately 4.6 m (15 feet) in height and/or if the levee is close to the river bank.

Use these types of evaluation methods when evaluating a levee that is less than approximately 4.6m (15 feet) tall, and when this levee is located some distance from a river bank (e.g. more than 10 m or 30 feet).

<u>Methods for Evaluation of Lateral Spreading</u>

The methods for evaluating lateral spreading potential of near level ground largely ignore the presence of the levee, but assume levee integrity will be affected if an earthquake induces large displacements in foundation soil. There are several widely accepted methods; some methods are listed below:

- Shear Strain Potential Procedure by Zhang et al. (2004). The shear strain potential procedure does not take into account local site seismicity but does evaluate capacity of soil to deform; it, therefore, represents an upper limit of the potential displacement, indifferent on the intensity of earthquake shaking.
- Multi-Linear Regression (MLR) empirical model by Youd et al. (2002). This model considered a large database of lateral spreading case histories from Japan and the western United States. The recommended equations differ depending on the site's general slope conditions: gently sloping ground and relatively level ground with a free face toward which lateral displacements may occur.
- Empirical Predictions of Liquefaction-Induced Lateral Spreading (EPOLLS) computer program (Rauch and Martin, 2000). EPOLLS predicts lateral spread occurrence, and the average and standard deviation of the displacements across it. These predictions are based on

regression of large numbers of field case histories, as with the procedure of Youd et al. (2002).

- Regional Modeling of Liquefaction-Induced Ground Deformation by Bardet et al. (2002). This regional modeling is also similar to the MLR empirical model by Youd et al. (2002) from a geotechnical and topographical site characterization point of view; in addition to the MLR model, a site's seismicity is defined through earthquake moment magnitude and epicentral distance.
- Performance-Based Evaluation of Lateral Spreading Displacement by Baska (2002) and Kramer et al. (2007). This performance based evaluation computes the median lateral spreading displacement (and probability of distribution) as a function of thickness of saturated cohesionless soil, earthquake magnitude, hypocentral distance, and geometry of the site.
- Semi-Empirical Model by Faris et al. (2006). This semi-empirical approach combines a mechanistic understanding with data from laboratory testing and data from full-scale earthquake field case histories; it evaluates the displacement potential index (DPI) based on SPT results.

<u>Methods for Evaluation of Displacements When Loading by Embankment Is Significant</u>

These methods are generally Newmark-type approaches, which are based on the concept that shear stresses induced during an earthquake, together with existing static shear stresses, may momentarily exceed the available shear strength along the base of a slide mass during cyclic shaking. The available strength can be expressed as a yield acceleration k_y, which is that acceleration that causes yielding on the slide plane when applied uniformly to a slide mass. The applied loading is expressed as the average acceleration of the slide mass, assuming there is no yielding on the slide plane (i.e., a de-coupled analysis). Another basic assumption of Newmark-type methods is the sliding of a rigid block over a well-defined slip surface. This approach was first presented by Newmark in 1965. Some of the procedures in this category are:

- The USGS computer program by Jibson and Jibson (2003). This program, currently available online, makes it easier to perform Newmark-type analyses using earthquake records that can specifically be used for a given project. The computer program includes a database of 2,160 earthquake records from 29 different earthquake events.
- The procedure Developed by Bray and Rathje (1998). This procedure was primarily developed to evaluate earthquake-induced displacements of solid-waste landfills at high levels of earthquake shaking, but can be used for levee response evaluation under similar conditions. Charts with normalized parameters are available, which can be used to develop preliminary estimates of expected seismic loading and displacements.
- The procedure recommended by Olson and Johnson (2008) is based on the back-analysis of 39 documented earthquake case histories, where SPT and/or CPT results were available. It used the Newmark-type sliding block analysis and the software by Jibson and Jibson (2003) to develop a relationship between yield acceleration and the computed displacement; comparing computed displacements with the actual ones, the authors determined mobilized strength ratios that can be used in Newmark-type modeling of lateral spreads. It was found that back-calculated Newmark-type analysis-based strength ratios coincide with liquefied strength ratios that are back-calculated from liquefaction flow failures (Olson and Stark, 2002).
- The simplified approach by DWR/URS Shewbridge et al., 2009). This methodology, based on a Newmark-type deformation evaluation, was prepared for the seismic vulnerability assessment of urban levees under study for DWR's Urban Levee Geotechnical Evaluations Program. Three typical levee and foundation models, representing conditions in the Sacramento and San Joaquin Rivers Central Valley, were considered in the simplified approach development.
- The USACE Seismic Crest Deformation Toolbox by O'Leary and Schaefer (2009). Seismic crest deformation evaluation is part of the Best Practices Guidance Document, a comprehensive set of toolboxes developed for risk assessment of USACE dams and levees. Although the document was primarily developed for to analyze dams, it applies to levees also (i.e., the height of the embankment input valid range is 10 to 300 feet). Crest settlement is estimated based on a parametric study of 20,000 cases that were analyzed using the computer program FLAC.

<u>Advanced Methods for Seismic Displacement Evaluation</u>

More sophisticated nonlinear analysis methods typically require detailed characterization of the levee and site conditions, and can be difficult to apply under the specific conditions of wide variability in both site conditions and seismicity. However, they may be justified in some high hazard conditions and, when performed properly, can provide better assessment of seismic deformations (both horizontal displacements and settlement) of a levee under complex conditions; but they require experience and judgment, and they can be subject to problems such as over-damping or failure to capture the key elements of seismic embankment response, etc. which can lead to unconservative results.

It is not recommended that these types of higher-order analysis tools be used without first performing more simplified analyses in order to obtain approximate estimates of expected performance as a basis for comparison. The USACE draft ETL requires that higher-order analyses be subject to expert review. When this is done, the results of higher-order analyses can be taken as over-riding the results of more simplified approaches.

In this study, fully nonlinear analyses using the FLAC (Fast Lagrangian Analysis of Continua) computer program (Itasca, 2010) are used for comparison with the empirical and limit equilibrium methods. For modeling liquefaction, the user-defined constitutive model UBCSAND (Byrne et al., 2003) was considered. UBCSAND is a modified Mohr-Coulomb model that directly assesses plastic shear and volumetric strains during every loading step. Each increment of plastic volumetric strain is directly related to the current stress ratio, the increment of plastic shear strain, and the cyclic stress history. For saturated soil elements, the tendency for contraction of the soil skeleton increases the pore pressures while the tendency for dilation decreases the pore pressure. The model incorporates a hyperbolic relationship between stress ratio and plastic shear strain. Unloading is linear elastic, so hysteretic stress-strain loops are produced during cyclic loading.

Evaluation of Seismically Induced Settlement

When liquefiable soils are present, earthquake-induced settlement of levees can generally result via four different mechanisms:

- Flow or bearing failure
- Lateral spreading
- Ground loss due to sand boil ejection
- Dissipation of excess pore water pressure (i.e., post-liquefaction reconsolidation settlement)

Settlement associated with the first two mechanisms is called deviatoric settlement. Settlement associated with the last two mechanisms is called volumetric settlement. Total settlement is often the result of a combination of the deviatoric and volumetric components.

In the case of flow slides the evaluation of settlement is of little interest, as the levee should be considered compromised, not capable of water retention.

Of some interest is the evaluation of settlement in conjunction with lateral spreading, although the levee may become compromised due to horizontal displacements and the associated cracking. With the exception of the advanced methods, the empirically developed models do not make a distinction between horizontal and vertical displacements. Generally (and probably conservatively), it is considered that the vertical displacement varies in proportion to the total (or horizontal) displacement with a ratio of vertical to horizontal displacement of 0.7.

Volumetric settlement should be added to the deviatoric settlement when deep, loose deposits are evaluated (e.g., deposits thicker than 6 m or 20 feet of cohesionless soil with $(N_1)_{60\text{-}cs}$ less than 15). Two well-known and widely practiced procedures are:

- Procedure recommended by Yoshimine et al. (2006). It is assumed the settlement is equal to the volumetric strain, i.e. the reconsolidation is a one-dimensional phenomenon, without lateral spreading movements.
- Procedure recommended by Tokimatsu and Seed (1984). The simplified method for estimation of post-liquefaction settlement of saturated sand is based on the finding that the primary factors controlling induced settlement are the cyclic stress ratio and the maximum shear strain, together with the density of the sand deposit (represented by the SPT N-value) and the magnitude of the earthquake.

CASE HISTORY EVALUATION

The only well known case of seismically induced degradation of levees in the United States is that of Pajaro River levees near Watsonville, California, during the 1989 Loma Prieta earthquake. This case history was evaluated in several studies, of which some of the most comprehensive were presented by Charlie et al. (1998), Tinsley III et al. (1998) and Miller and Roycroft (2004). In this study, the evaluation of the levee follows the recommendations of the USACE draft ETL in view of validation of the suggested methodology.

Loma Prieta Earthquake

The shaking of the October 17, 1989 magnitude 6.9 (M_W – moment magnitude; 7.1 M_S; 7.0 M_L) earthquake was recorded at 93 stations, for a total of 125 records, according to Shakal et al. (1989). The stations close to the zone of interest (Pajaro river levee, near Watsonville) are shown on Fig. 1.

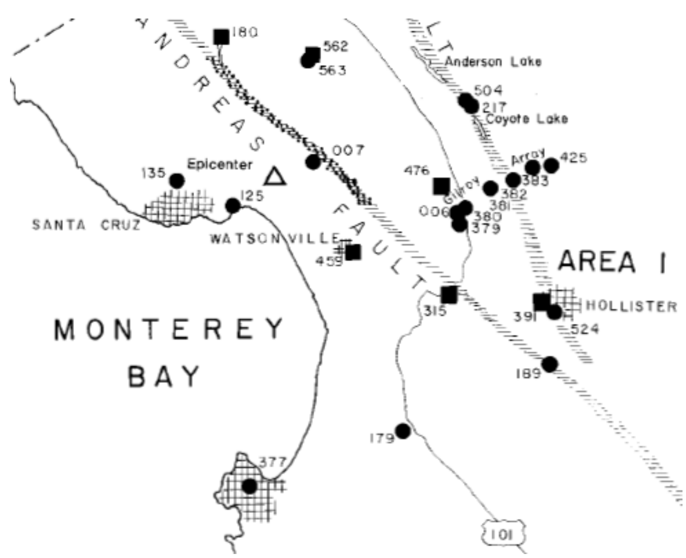


Fig. 1. Strong Motion records available in the vicinity of the study zone (Shakal et al., 1989).

The closest station from the zone of interest was in Watsonville (No. 459 on the map in Fig. 1) but was located in a 4-story building; the instrument on the ground floor recorded a horizontal peak acceleration of 0.39g and a vertical component peak of 0.66g. The next closest station was Corralitos (No. 007) near the San Andreas Fault and 5 km from the epicenter; the peak accelerations were 0.64g horizontal and 0.46g vertical. At approximately the same distance from the zone of interest was station Capitola (No. 125 on map; peak accelerations 0.54g horizontal and 0.60 vertical); this was the record assumed in this study to represent the time history at the Pajaro River levee. Station Salinas to south (No. 179) measured the peak accelerations of 0.12g and 0.11g horizontal and vertical, respectively.

Damage to Pajaro River Levee

Extensive liquefaction occurred in free field in the vicinity of Watsonville and Pajaro River levee, as presented in Fig. 2.

Fig.2. Sand volcanoes along fissures in agricultural field near the levee (source: US Geological Survey, photo by J.C. Tinsley)

Extensive damage to the levee was induced by seismic liquefaction of the alluvial deposit underneath along the entire 10 km (6-mile) levee reach between City of Watsonville and the Pajaro River mouth at Monterey Bay (Fig. 3,a). The photos (Figs. 3,b and 3,c) show cracks at the most damaged section, where the longitudinal cracks were up to 0.5 m (18 inches) wide and 2.4 m (8 feet) in depth.

With the exception of the most damaged 300 m (1000 feet) levee reach, almost continuous longitudinal cracks occurred both on the levee crest and in the field nearby, with little or no associated settlement. Of major concern were considered transverse cracks intermittently located along the inspected reach (Fig. 4).

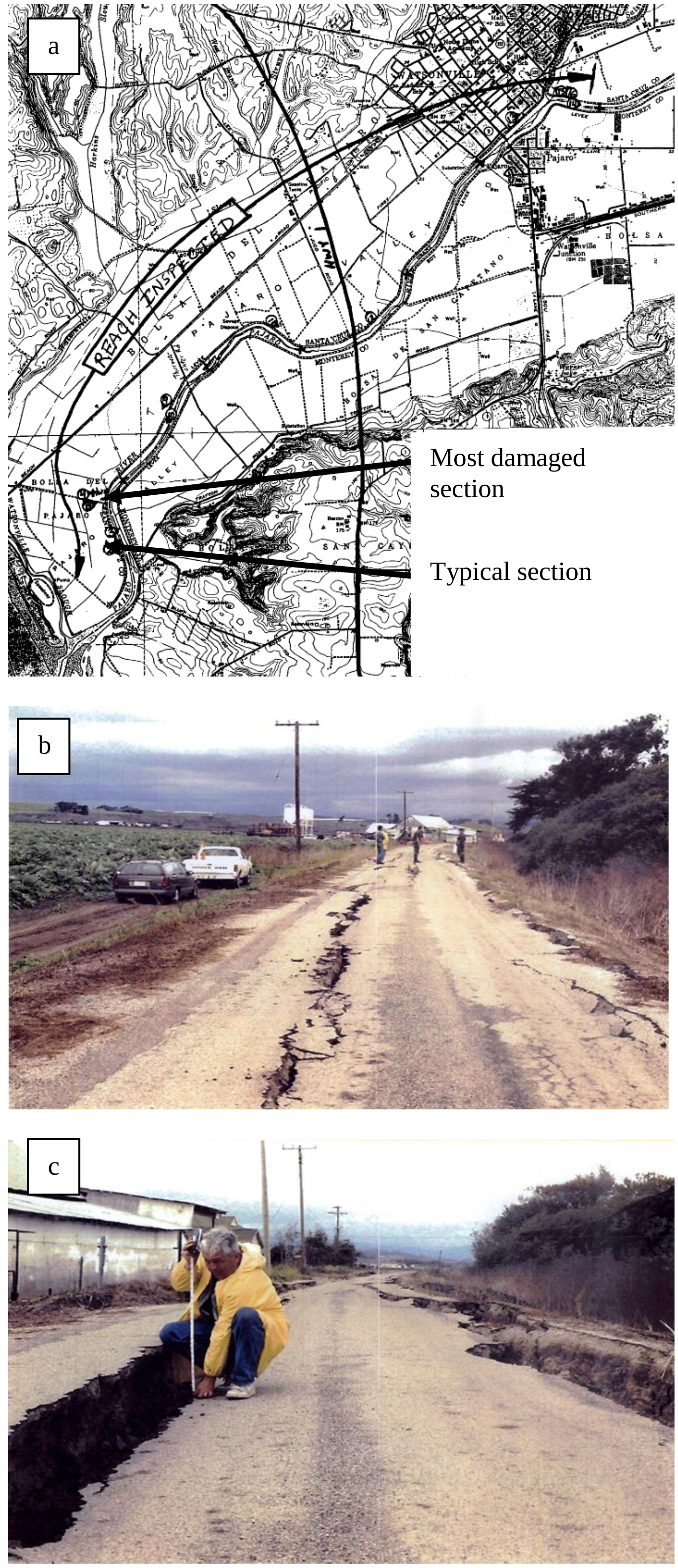


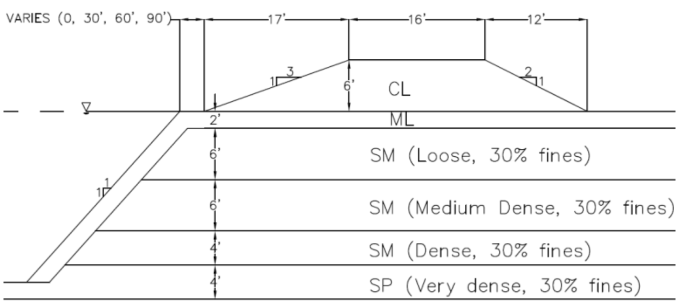
Fig. 3. Excerpts from the US Army Corps of Engineers Memo for Record dated 23 October 1989 (author: David Ricketts): a. Map of inspected damaged levee; b. Typical cracks and slumping of levee; c. Most damaged location.

Fig. 4. Transverse crack through entire levee section (photo David Ricketts).

It is believed that the transverse cracks occurred at sharp changes in foundation conditions, from liquefiable to non-liquefiable, as levee crossed old river meanders.

Evaluated Cross Sections

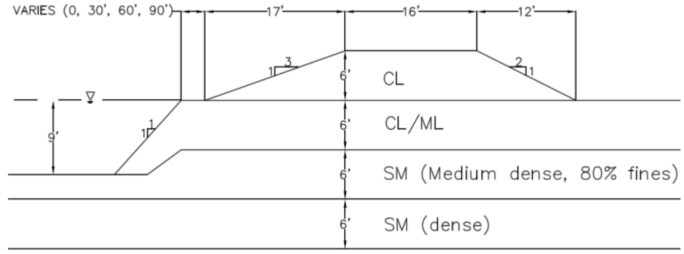
Two cross sections were considered in evaluation: the most damaged section and a typical section, as located in Fig. 3,a. The main parameters of these sections are presented in Figs. 5 and 6.



Layer	γ (pcf)	c' (psf)	ϕ (Degree)	N	Fines (%)
CL Embankment	120	300	30	-	-
ML	120	200	30	-	-
SM-Loose	120	0	30	0	30
SM-Medium Dense	120	0	33	10	30
SM-Dense	125	0	36	22	30
SP-Very Dense	125	0	38	>40	5

Fig. 5. Most damaged section.

The distance from levee to the top of the river bluff varies; as a sensitivity analysis various distances were considered, between zero and 27 m (90 feet); in what follows this distance will be named "berm".



Layer	γ (pcf)	c' (psf)	ϕ (Degree)	N	Fines (%)
CL Embankment	120	300	30	-	-
CL/ML	120	300	30	-	-
SM-Medium Dense	120	0	33	11	30
SM-Dense	125	0	36	>40	30

Fig. 6. Typical section (except for the most damaged zone).

Liquefaction Assessment

Two earthquake parameters are needed for liquefaction assessment per Youd et al. (2001); magnitude (6.9 was considered) and peak horizontal ground acceleration, PGA. Miller and Roycroft (2004) evaluated PGA = 0.33g; we found this value reasonable and in good agreement with the records at stations Watsonville (0.39g, No. 459 on Fig. 1), Corralitos (0.64g, No. 007), Capitola (0.54g, No. 125), and Salinas (0.12g, No. 179), as mentioned before. Based on USCS maps, this value is between a local 100-year event (PGA = 0.28g) and 200-year event (PGA = 0.35g), which is also credible.

Liquefaction susceptibility evaluation was based on SPT results. The factor of safety against liquefaction (FS_{liq}) was calculated for free field; the results are listed in Table 1.

Table 1. Factors of Safety against Liquefaction

Cross Section	Layer Depth (feet)	SPT, blows/foot		FS_{liq}
		N	$N_{1,60-cs}$	
Most damaged	2 - 8	0	4.7	0.21
	8 - 14	10	19.5	0.63
Typical	6 - 12	11	19.0	0.61

Note: N is the raw SPT blowcount and $N_{1,60-cs}$ the normalized parameter, corrected for fines (clean sand equivalent).

From Table 1 it is evident that with both sections there are liquefiable layers, so deformations due to liquefaction are probable.

Post-Earthquake Limit Equilibrium Evaluation (flow slide check)

For this evaluation, potentially liquefiable soils were assigned the residual shear strength that was estimated through two different procedures: Seed and Harder, 1990 (noted S&H in what follows) and Olson and Stark, 2002 (O&S). The analyses were performed with the computer program UTEXAS4 (Wright, 2008). An example of output is presented in Fig. 7 and the results are summarized in Table 2 and Fig. 8.

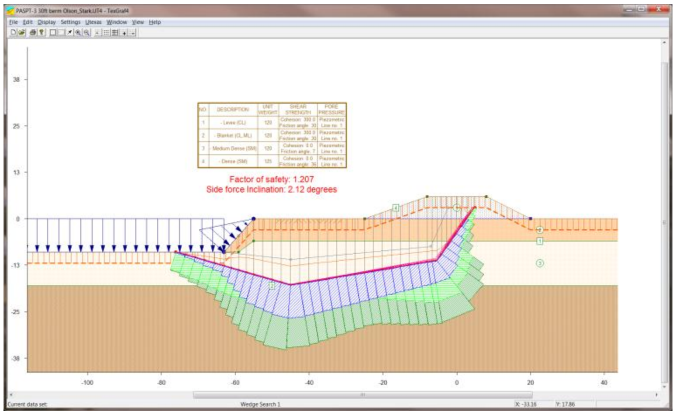


Fig. 7. Example of post-earthquake (static) stability analysis: most damaged section, O&S.

Table 2. Summary of Stability Analyses

Cross Section	Distance Levee to River Bank	Factor of Safety (FS)		
		Pre-Earthquake	Post-Earthquake	
			S&H	O&S
Most damaged	0	1.85	0.76	0.60
	30 feet	2.87	0.88	0.59
	60 feet	3.50	0.98	0.78
	90 feet	3.50	0.98	0.78
Typical	0	2.36	1.49	0.58
	30 feet	3.69	2.37	1.21
	60 feet	4.60	3.08	1.96
	90 feet	4.60	3.08	1.96

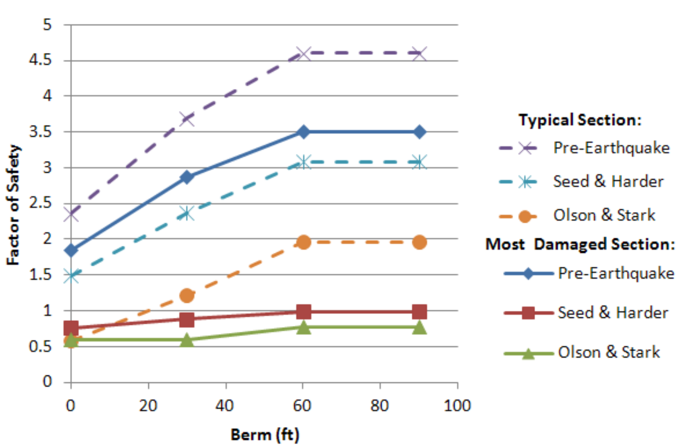


Fig. 8. Increase of stability factor of safety with berm width and its drop from pre-earthquake to post-earthquake condition (S&H and the more conservative in this case O&S options).

For the most damaged section the post earthquake limit equilibrium evaluation indicates flow failure, potentially leading to large displacements, indifferent on the distance from levee to river, i.e. berm width. The actual most damaged section (see Fig. 3, a and c) had a berm of about 9 m (30 feet) and experienced significant damage, but not flow failure; according to the USACE inspection report the longitudinal cracks were up to 0.5 m (18 inches) wide and 2.4 m (8 feet) deep, with vertical displacement at crack of 0.3 m (1 foot).

The typical section at the analyzed location (see Fig. 3, a and b) was close to the river (no berm). The actual damage was relatively minor, so a post-earthquake FS greater than one (as obtained with S&H definition of the residual strength) is considered correctly describing the field condition.

Lateral Spreading Evaluation

For the typical section with no berm it is justified to continue the evaluation of the potential seismic displacement assuming a flow failure is not expected. Three methods have been used in this respect.

Shear Strain Potential Procedure by Zhang et al. (2004). This procedure gives an estimate of the maximum potential of soil to spread laterally under strong seismic action ($6.4 < M_w < 9.2$; $0.19g < a_{max} < 0.6g$). In the evaluated case the Lateral Displacement Index, LDI = 20 cm (0.66 feet). The authors also define an adjustment of LDI based on empirical calibration against case histories, for either gently sloping or level ground with a free surface. Disregarding the levee and assuming level ground with free face (H = 9 feet), the potential maximum lateral displacement within the levee footprint (L = 45 feet) is $LD = 6 \cdot (L/H)^{-0.8} \cdot LDI = 1.1$ feet (0.34 m).

Multi-Linear Regression (MLR) empirical model by Youd et al. (2002). The predicted horizontal ground displacement (D_H) is a function of earthquake magnitude ($M_w = 6.9$ in our case), epicentral distance (R = 19 km), $(H/L) \cdot 100$ $[(9/45) \cdot 100 = 20\%]$, thickness of saturated layers with $(N_1)_{60} \leq 15$ ($T_{15} = 6$ m), average fines content in T_{15} ($F_{15} = 30\%$), and average particle diameter in T_{15} ($D50_{15} = 0.2$ mm). It results in our case $D_H = 0.44$ m (1.3 feet).

Semi-Empirical Model by Faris et al. (2006). This model is similar to the model by Zhang et al. (2004) but uses different procedures for evaluation of the maximum Displacement Potential Index (DPI) and for calibration based on case histories in view of evaluation of the maximum horizontal displacement (H_{max}). DPI is calculated based on the cyclic stress ratio, CSR and $N_{1,60\text{-}cs}$; in our case $DPI_{max} = 18$ cm (0.59 feet), similar to Zhang's LDI. $H_{max} = \exp(1.0443 \ln(DPI_{max}) + 0.0046 \ln(\alpha) + 0.0029 M_w) = 0.17$ m (0.52 feet) in our case, about half Zhang's LD. The parameter α, representing the static load, was considered equal to H/L; however it was found that the result is not sensitive to α at all.

Advanced (FLAC/UBCSAND) Evaluation

FLAC computer program (Itasca, 2011) was used in conjunction with the UBCSAND liquefaction model (Byrne et al, 2003) as modified by Dr. Michael Beaty for better modeling liquefiable layers located at shallow depth under embankments (Ruthford et al. 2008).

Pre-Earthquake Static Equilibrium. Several variants were considered, with various berm dimensions; Figure 9 presents the meshes for the two basic considered cross sections (variants without berm; stratification below the liquefiable layers was simplified). It is mentioned that the dynamic loading requires time history being applied within the bedrock; as the granitic rock at the levee location is at a depth in excess of 760 m (2500 feet), the mesh was limited to the Purisima Formation sandstone existing below the depth of about 36 m (120 feet).

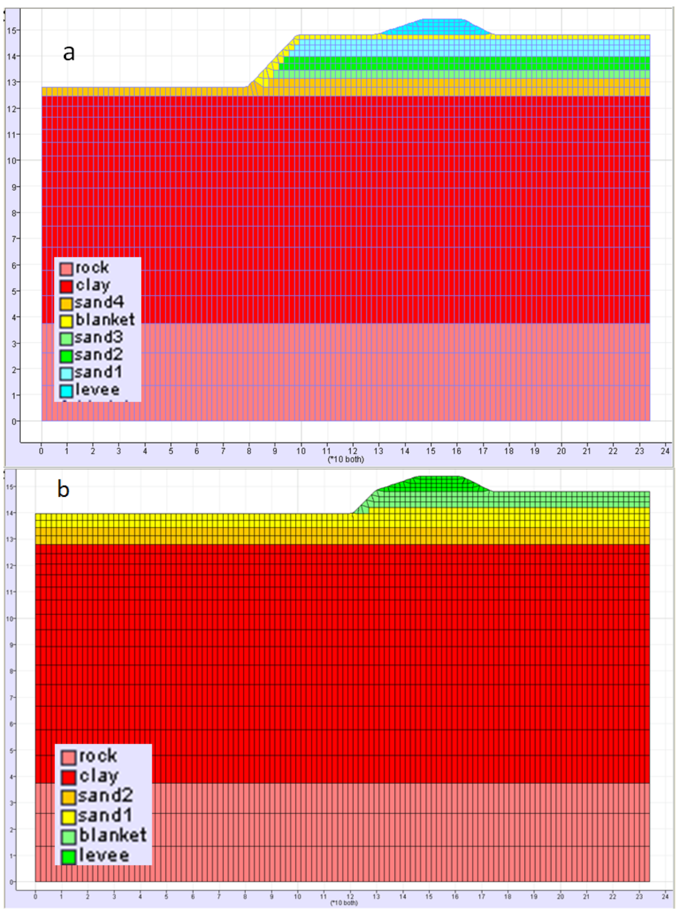


Fig. 9. Finite difference mesh for: a – Most damaged section, with 30-foot berm; b – Typical section, without berm.

Mohr-Coulomb model was assigned to all soil materials, except for three columns of elements at both sides of the mesh, where elastic model was used; the elastic model was used for modeling the rock (sandstone, the bottom three rows of elements) also. Steady state seepage equilibrium was obtained assuming the water in river at the ground surface elevation. Once the initial stress state had been achieved, the model was converted to address dynamic conditions: (a) Adjusting properties of Mohr-Coulomb and elastic zones to address the anticipated dynamic response of the elements; (b) Assigning the UBCSAND model to zones considered susceptible to liquefy (based on possible saturation and $N_{1,60\text{-}cs} < 30$, see Table 1), as shown in Fig. 10; (c) Assigning appropriate levels of viscous (Rayleigh) damping to various zones; (d) Converting the boundary conditions of the model so that free-field boundaries were used on the left and right boundaries and a compliant (non-reflecting) base was used at the bottom of the model.

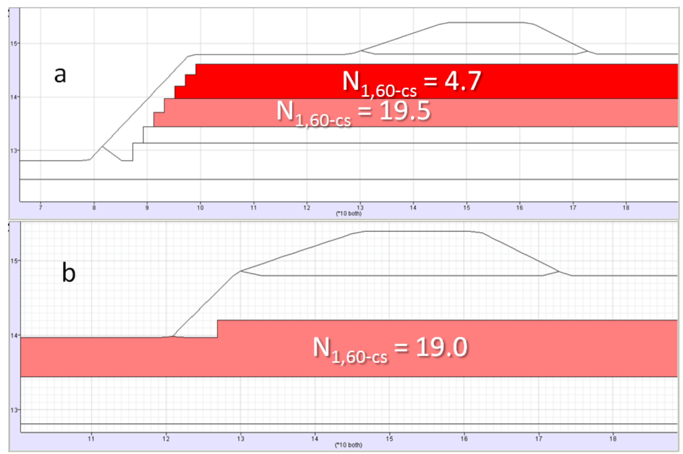


Fig. 10. Zones assigned with UBCSAND and their corresponding liquefiability parameter: a – Most damaged section, with 30-foot berm; b – Typical section, without berm.

Earthquake Simulation. The Capitola Station record, 000 horizontal component and the vertical component, was used; after filtering above frequencies of 15 Hz and baseline correction, the peak acceleration was 0.52g, higher than the target of PGA = 0.33g. The original 000 component accelerogram, with a total duration of 40 seconds, is shown in Fig. 11.

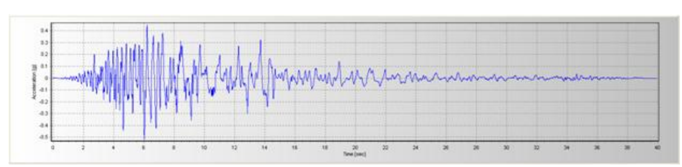


Fig. 11. Acceleration time history of the 1989 Loma Prieta earthquake, Capitola Station, 000 component.

The compliant boundary required the input acceleration history to be converted into an equivalent shear stress history before being applied to the base of the mesh (within rock). As amplification is expected within the soil layers, the original accelerograms had to be scaled before conversion; based on estimations by Miller and Roycroft (2004) for obtaining 0.33g at the ground surface, the peak bedrock acceleration should be 0.25g. Therefore, the original time histories of both horizontal and vertical components were scaled with 0.25g/0.52g = 0.48.

<u>Post-Earthquake Analysis</u>. After running for an additional five seconds, to permit decay of motions after the end of the earthquake, the liquefied zones were converted to a Mohr-Coulomb model with residual strengths. The residual strength was based on Olson and Stark, 2002 (O&S).

<u>Example of Results</u>.

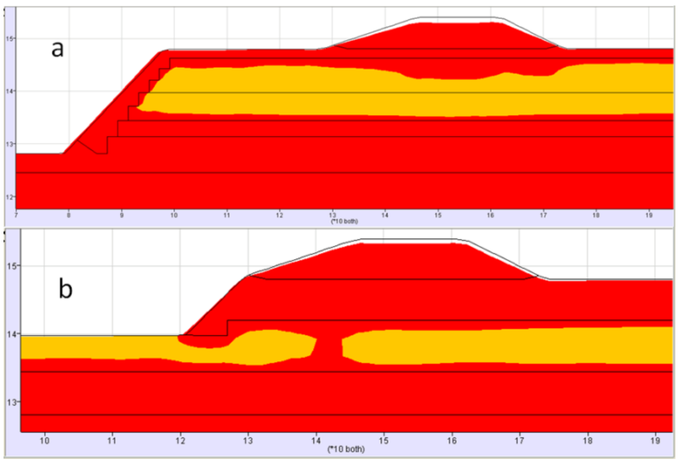


Fig. 12. Extent of liquefied zone (orange). a – Most damaged section, with 30-foot berm; b – Typical section, without berm.

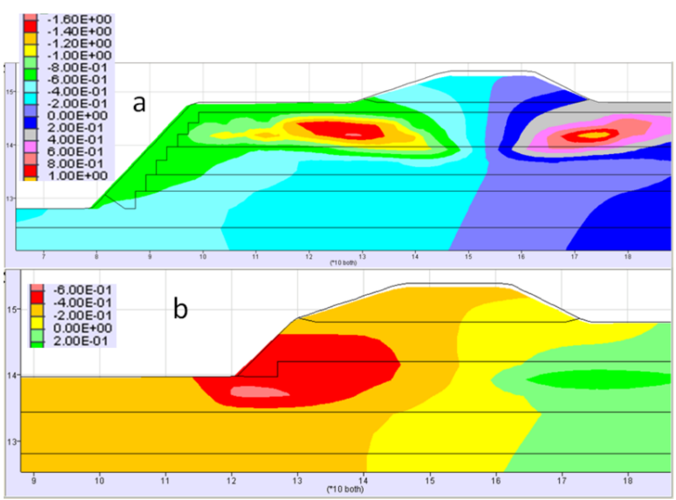


Fig.13. Horizontal displacement contours (0.2-foot intervals). a – Most damaged section, with 30-foot berm; b – Typical section, without berm.

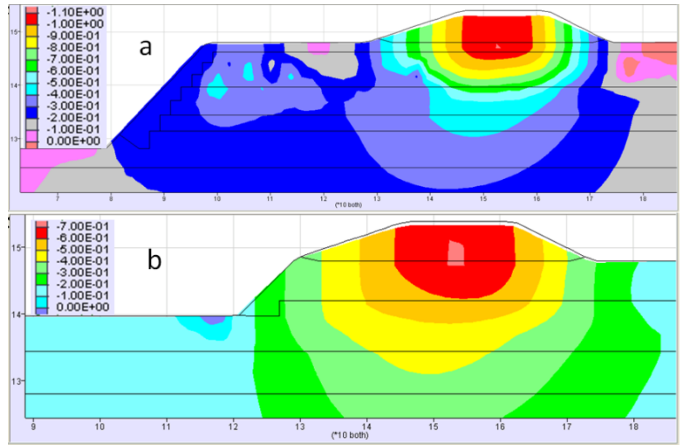


Fig.14. Vertical displacement contours (0.1-foot intervals). a – Most damaged section, with 30-foot berm; b – Typical section, without berm.

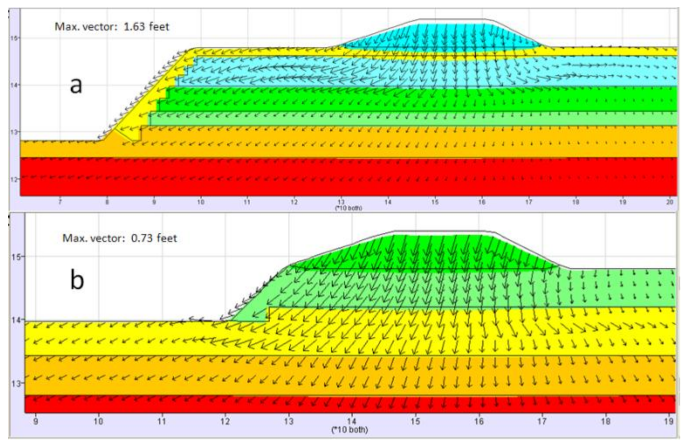


Fig. 15. Displacement vectors. a – Most damaged section, with 30-foot berm; b – Typical section, without berm.

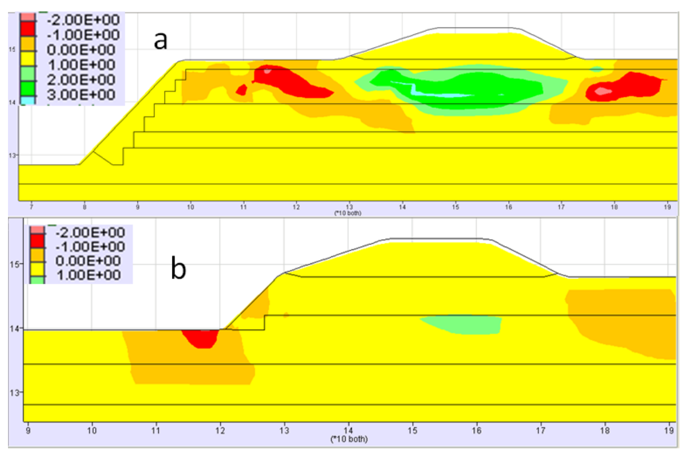


Fig. 16. Percent elongation (1% contour intervals). a – Most damaged section, with 30-foot berm; b – Typical section, without berm.

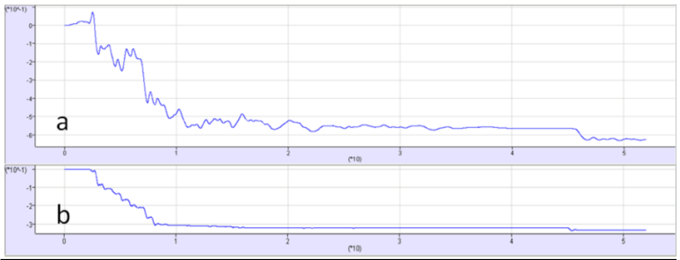


Fig. 17. Time history of horizontal displacement of the levee waterside toe; abscissa: dynamic time; end of shaking at 40seconds; start of post-earthquake stage at 45 seconds; vertical coordinate: horizontal displacement in feet. a – Most damaged section, with 30-foot berm; b – Typical section, without berm.

The results of the advanced evaluation clearly indicate that flow slide is not probable. With both the most damaged and the typical section the deformation practically stopped at the end of shaking, with very little displacements in the post-earthquake stage of computer run and quick stabilization under gravitational forces and residual strength in liquefied regions (Fig. 17).

It is noted that the most damaged section in the variant without berm predicted less displacement than the variant with 30-foot berm that better describes the condition in the field. Table 3 summarizes the results, some of them presented also graphically in Figs. 12 through 17.

Table 2. Summary of Advanced Evaluation.

Parameter	Most Damaged Section		Typical Section, No Berm
	30-foot Berm	No Berm	
Horizontal displacement (ft): - of waterside toe - of landside toe	 - 0.64 + 0.22	 - 0.66 + 0.21	 - 0.36 - 0.08
Elongation toe to toe (ft): - at ground surface - within liquefiable layer	 0.9 2.8	 0.9 1.7	 0.3 1.0
Vertical displacement at crest (ft)	- 1.1	- 1.1	- 0.7

The results of the advanced evaluation are in general agreement with the field observations, in terms of both elongation compared with sum of crack widths and settlement.

CONCLUSIONS

The most appropriate procedure of predicting seismic deformation of levees uses advanced methodologies. However, in most cases the soil information available is not detailed enough for justifying sophisticated procedures, expensive and time consuming. The post-earthquake limit equilibrium evaluation is simple and provides conservative results. When flow slide is not probable the empirical procedures can be used for the evaluation of displacements.

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