D R A F T   R E P O R T

IN-DELTA STORAGE PROGRAM SEISMIC ANALYSIS

Prepared for
Department of Water Resources
901 P Street
Sacramento, CA 94236

June 2003

URS Corporation
500 12th Street, Suite 200
Oakland, CA 94607

26814104

TABLE OF CONTENTS

Tables

TABLE OF CONTENTS

Figures

TABLE OF CONTENTS

Attachment

1.1 PURPOSE

As part of the feasibility study, the Department of Water Resources requested that URS Corporation (URS) undertake a risk analysis and integrate the physical design with a desirable level of protection through seismic, flooding, operational, environmental and economic analyses. Other objectives were to recommend a desirable level of protection and appropriate factor of safety for the project.

1.2 SCOPE OF WORK

The specific scope presented under this Task Order was to address the vulnerability and reliability of the existing conditions and In-Delta Storage Re-engineered project (embankment and integrated facilities) under seismic loads. The work for this Task Order included the evaluation of the existing conditions and the proposed re-engineered reservoir project at Webb Tract and Bacon Island. Specifically, the following subtasks were performed:

- Collected and reviewed existing information.
- Conducted a seismic hazard analysis and evaluated expected ground motions at the reservoir island sites. (The probability seismic hazard analysis is presented in Attachment 1 of this report).
- Performed seismic stability analyses of the existing conditions and the re-engineered project.
- Estimated failure probabilities under seismic loading.

The work was conducted in accordance with all applicable standards and guidelines contained in Standard Agreement No. 4600001747 and in coordination with Department staff.

Dynamic response analyses of the embankments were performed to calculate time histories of seismic-induced inertial force acting on the critical sliding masses. We utilized the computer program QUAD4M (Hudson et al., 1994) for these analyses. QUAD4M is a two-dimensional, plan-strain, finite element code for dynamic response analysis. It uses an equivalent linear procedure (Seed and Idriss, 1970) to model the nonlinear behavior of soils. The softening of the soil stiffness is specified using the shear modulus degradation (G/G_{max}) and damping vs. shear strain curves. QUAD4M also incorporates a compliant base (energy-transmitting base), which can be used to model the elastic half-space.

Our review of the soil data indicates that there are some sections under the perimeter levees where the upper 5 feet of the underlying sand deposits may liquefy during earthquake events. In addition, part of the existing levee, on the island side, may contain loose sands, which have the potential to liquefy when they become saturated during the reservoir filling. One of the consequences of the liquefaction of the loose saturated sand is the reduction in shear resistance along the critical slip surface during earthquake shaking. In the context of this analysis, this translates into lower yield acceleration, k_y, which in turn, induces larger deformations. Dynamic analyses for both cases involving non-liquefied and liquefied sandy layers were performed. Embankment deformations for these cases were then estimated.

2.1 DATA REVIEW

The information from the following studies was reviewed:

- Dynamic Properties of Sherman Island Peat by Boulanger et al. (1997). Report No. UCD/CGM-97/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California at Davis
- Three deep boring logs and geophysical measurements at Webb Tract and Bacon Island obtained from Department of Water Resources
- Nonlinear Dynamic Properties of a Fibrous Organic Soil by Wehling et al. (2001). Paper accepted for publication in ASCE Journal of Geotechnical Engineering
- Seismic Vulnerability of the Sacramento-San Joaquin Delta Levees, December 1998, Calfed Bay-Delta Program, Seismic Vulnerability Sub-Team
- Department of Water Resources, 2002, Draft report on engineering investigations, In-Delta Storage Program, CALFED Bay-Delta Program.

2.2 ANALYSIS PARAMETERS

2.2.1 Embankment Cross Sections

Two embankment alternatives were considered. The first alternative consists of building the embankment on the island side with a slough-side bench (bench alternative). This alternative results in a relatively off-set embankment from the existing levee, and provides for a flat slough side slope of 4H:1V or flatter. The second alternative consists of building the embankment on the existing levee and placing a rock toe berm on the slough-side slopes with an average slope of 3H:1V (rock berm alternative).

For each of these alternatives, two cross-sections representing the variation in the subsurface conditions were developed for analysis. These cross-sections represent the upper and lower base elevation of the peat underlying the existing levees. Figure 1 shows the finite element model for Cross Section I, where a thinner peat deposit was encountered (peat bottom elevation at –20 feet). The finite element model for Cross Section II with a thicker peat deposit (peat bottom elevation at –40 feet) is illustrated in Figure 2. These cross sections are considered to be representative at both Webb Tract and Bacon Island sites.

2.2.2 Material Properties

Dynamic soil parameters used in our previous study (URS, 2000) were reviewed and updated using the more recent information. Specifically, the shear and compressive wave velocities obtained from the geophysical measurements at the Sherman Island (Boulanger, 1997) and at the Webb Tract and Bacon Island (Wehling, 2001) were used. The relationship that relates maximum shear modulus, over consolidation ratio (OCR) and effective pressure proposed by Wehling (2001) for peat was also utilized to account for the dependency of shear modulus (or shear wave velocity) on effective pressure.

The shear modulus degradation (G/G_{max}) and damping curves of Kokusho (1980) and Vucetic and Dobry (1991) were applied for the sandy soils (embankment fill and alluvium) and clay, respectively. For peat, the relationships of Wehling (2001) were utilized. The selected dynamic soil properties used for the response analyses are summarized in Table 1. Plots of the selected G/G_{max} and damping vs. shear strain relationships are presented in Figures 3 and 4. It should be noted that analysis results (Section 2.4) showed high seismic induced shear stresses within the peat; i.e., stresses that are higher than the undrained shear strength of the peat. To reduce the calculated stresses from the equivalent linear procedure, the G/G_{max} vs. shear strain relationship of Wehling (2001) was slightly lowered at large shear strain values.

For liquefied sand, small-strain shear wave velocities of 300 and 400 ft/sec were used for deposits outside and within the footprint of the embankment, respectively. No shear modulus degradation was applied for the liquefied soil, and the damping values were kept constant at 8% to 10% of the critical damping value.

2.2.3 Reservoir Stages and Slough Water Levels for Analyses

Two operating water elevation scenarios were selected to represent the fluctuation of water elevations in the reservoir and the slough, and are as follows (see Embankment Design Analysis Report):

- High Tide and Low Reservoir: a low reservoir and high slough water at elevation +3.5 feet. This condition was assumed to prevail 2/3 of the time.
- Low Tide and High Reservoir: a high reservoir water at elevation +4.0 feet and low slough water at elevation –1 foot. This condition was assumed to prevail 1/3 of the time.

These scenarios represent normal fluctuation in tidal water at the project site. They do not correspond to “extreme” conditions associated with flooding.

Table 1
Dynamic Soil Parameters Selected for Analysis

Description		Moist Unit Weight (pcf)	K_{2max}	Shear Wave Velocity (ft/sec)	Modulus and Damping Curves
Embankment Materials					
New fills: sand		120	80	-	Sand[1]
Peat	- free-field	70	-	See note[4]	Peat[2]
	- under embankment				Peat[2]
Foundation Materials					
Sand	(non-liquefied)	120-125	80	-	Sand[1]
	(liquefied)	120-125	-	300-400	See Note [5]
Clay		127	-	1000	Clay[3]

Note: 1. Relationships of Kokusho (1980), function of confining pressure
2: Relationships of Wehling et al (2001)
3: Relationships of Vucetic and Dobry (1991) for PI = 50
4: Shear wave velocity was estimated using the following equations (Wehling et al. (2001):

$$v_s = \sqrt{\frac{G_{max}}{\rho}} \geq 75 \ ft/\sec$$

$$\frac{G_{max}}{Pa} = 75.7 \left[\frac{\sigma'_{1c}}{Pa}\right]^{0.87} OCR^{0.65}$$

Where Pa and σ'_{1c} are the atmospheric and effective vertical pressures, respectively
5. For liquefied sand, no reduction in G is allowed and the damping is fixed at 8%-10% of critical damping.

2.3 EARTHQUAKE LOADS

A site-specific probabilistic seismic hazard analysis was performed for the current study to provide estimates of ground motions for future earthquake occurrences. A discussion of the approach, assumptions and results is represented in Attachment 1 to this report.

2.3.1 Earthquake Response Spectra

Three seismic events representing a small, a moderate, and a large earthquake in the region are considered. The three selected events correspond to ground motions having probabilities of exceedance in 50 years of about 69%, 10% and 2%. These correspond to ground motions with return periods of about 43 years, 475 years and 2,500 years, respectively. Figure 5 depicts the 5%-damped response spectra of these ground motions. These response spectra represent free-field motions for the outcropping stiff soil site condition. The peak ground accelerations (PGA's) at the site are as follows:

- 43 year return period: 0.14g
- 475 year return period: 0.33g
- 2,500 year return period: 0.52g

2.3.2 Spectrally-Matched Time Histories

To perform the dynamic response analyses, earthquake acceleration time histories are needed as input. We have used the same time histories as in the previous URS, 2000 study. These records are from the 1992 **M** 7.3 Landers earthquake, recorded at Fort Irwin station (station #24577), and the **M** 6.0 1987 Whittier Narrows earthquake, recorded at Altadena, Eaton Canyon station (station #24402). Table 2 lists these recorded motions along with their closest distances from the rupture planes and recorded peak accelerations. The site conditions at these recording stations are classified as stiff soil sites. The record from the 1992 Landers earthquake was selected to represent the larger and more distant earthquakes on the San Andreas and Hayward faults. The 1987 Whittier Narrows earthquake was selected to represent seismic events on the local seismic sources.

The response spectral values calculated from the selected acceleration time histories (natural time histories) have peaks and valleys that deviate from the smooth analysis response spectra (target response spectra). To develop acceleration time histories with overall characteristics that match the target response spectra, modifications to the natural time histories were necessary.

The two acceleration time histories were spectrally matched to the selected response spectra (i.e., response spectra for return periods of 43 years, 475 years and 2,500 years) using the method proposed by Lilhanand and Tseng (1988) and modified by Abrahamson (1993). The plots of the acceleration, velocity and displacement time histories of these spectrally matched motions are presented in Figures 6 through 11. The 5% damped response spectra for the modified motions are shown in Figures 12 through 14 along with the target spectra. It can be seen from these figures that the response spectra calculated from the modified time histories closely match the target spectra.

Table 2
Summary of Earthquake Records Used in the Dynamic Response Analysis

Earthquake	M_w	Recording Station			Comp.	Recorded PGA (g)
		Distance (km)	Station	Site Condition		
1987 Whittier Narrows	6.0	18	Altadena – Eaton Canyon Station	Soil[a]	90°	0.15
1992 Landers	7.3	64	Fort Irwin Station	Soil[a]	0°	0.11

Note : a = Deep stiff soil site

2.4 ANALYSIS RESULTS

Dynamic response analyses were performed by using compliant bases at the bottom of the finite element models to prevent total reflection of wave energy at the fixed boundaries. The shear wave velocity for the underlying elastic half space was taken equal to that of the stiff clay deposit beneath the sand layer. The spectrally-matched acceleration time histories were input to the finite element models at an elevation of about -100 feet. These input acceleration time histories were obtained by deconvolving the spectrally matched time histories to that elevation. We used the one-dimensional wave propagation computer program SHAKE (Schnabel et al, 1972) to deconvolve the ground motions at elevation –100 feet.

The results of analyses are expressed in terms of average horizontal acceleration (K_{ave}) time histories of the potential (critical) slide masses within the embankments. The critical slide masses for each embankment alternative and for the two cross sections were identified in the static slope stability analyses (Embankment Design Analysis Report), and are presented in Figures 15 through 18. The average horizontal acceleration was calculated by computing the dynamic response of the embankment and averaging various stresses within or close to the sliding surface. Examples of the calculated K_{ave} time history are presented in Figures 19 through 22 for the 475-year return period ground motion.

Seismic-induced deformations of the embankments were estimated for the three ground motion levels selected for this study. The estimated deformations and their associated ground motion levels were used to evaluate the seismic risk of the proposed embankment alternatives.

3.1 METHODOLOGY

Seismic-induced permanent deformations of the embankment slopes were estimated using the Newmark Double Integration Method (1965) and the Makdisi and Seed Simplified Procedure (1978). The Newmark Double Integration Method is based on the concept that deformations of an embankment will result from incremental sliding during the short periods when earthquake inertia forces in the critical slide mass exceed the available resisting forces. This method involves the calculation of the displacement (deformation) increment of a critical slide mass at each time step using the average horizontal acceleration (k_{ave}) and the value of yield acceleration (k_y) calculated for the slide mass. The development of the k_y is discussed in the Embankment Design Analysis Report. The displacement increment is calculated by double integrating the difference between k_{ave} and k_y values acting on the slide mass. The estimated permanent deformation of the slide mass is then taken as the sum of the displacement increments at the end of ground shaking.

The simplified procedure of Makdisi and Seed (1978) was developed based on observations of dam performance during past earthquakes and analysis results. In this method, the inertial force on the slide mass is represented by the peak average horizontal acceleration (k_{max}) induced by the design earthquake. Empirical relationships relating the ratio of k_y and k_{max} (k_y/k_{max}) and the average deformation were used to estimate embankment deformations.

3.2 RESULTS

3.2.1 Bench Alternative

The slope deformations calculated using the Newmark Double Integration Method for non-liquefied sandy soils are tabulated in Tables 3 and 4 for Cross Section I (bottom of peat at elevation –20 feet) and Cross Section II (bottom of peat at elevation –40 feet), respectively. For the non-liquefied cases, the results of the analysis suggest that up to about 1.65 feet and 0.4 feet of slope deformations on the slough and reservoir sides, respectively, can be expected during an earthquake event having a 475-year return period. Under the 43-year return period ground motions, the seismic induced slope deformations are expected to be small. The Simplified Makdisi and Seed procedure was also used to estimate slope deformations for comparison purposes. The comparisons are shown in Tables 3 and 4.

The results for the liquefied cases are tabulated in Tables 5 and 6 for Cross Sections I and II, respectively. As expected, under the 475-year return period event, much larger slope deformations were estimated. For Cross Section I, up to about 3.3 feet and 1.35 feet of deformations were calculated for the slough and reservoir slopes, respectively. Slough side slope deformations of about 9 feet and reservoir side slope deformation of about 2.25 feet were estimated for Cross Section II. Under the smaller ground motions of 43-year return period, maximum deformations of about 0.6 feet and 1.15 feet were calculated for the slough and reservoir slopes, respectively, for Cross Section I. The maximum slope deformations for Cross

Section II were calculated to be about 1.5 feet, for the slough slopes, and 1.35 feet, for the reservoir slopes.

As noted in Tables 3 through 6, convergence was not obtained for some of the cases with larger earthquakes (2500-year and some 475-year events). For these cases, the average horizontal acceleration time histories could not be computed in the QUAD4M runs. These numerical problems were caused by large deformations (shear strain in excess of 40%) calculated in the peat deposits due to large earthquake shaking. The procedure of Makdisi and Seed (1978) was not judged appropriate for these cases where substantial strength loss takes place. For embankments experiencing large seismically induced strains, the average acceleration may not continue to increase with increasing levels of seismic shaking and deformations. However, for the purpose of this study, a deformation of over 12 feet was assumed to have a 95 percent probability of embankment failure. This condition was considered to represent the expected embankment performance under severe earthquake events.

The results of the seismic deformation analyses for the bench alternative are summarized in Table 10A for the 475-year earthquake event.

3.2.2 Rock Berm Alternative

For rock berm alternative, the calculated slope deformations considering non-liquefied sandy soils are tabulated in Tables 7 and 8 for Cross Section I (bottom of peat at elevation –20 feet) and Cross Section II (bottom of peat at elevation –40 feet), respectively. For the non-liquefied case, the results of the analysis suggest that up to about 0.4-foot of slope deformation can be expected during an earthquake event having a 475-year return period. Under the 43-year return period ground motions, the seismic induced slope deformations are expected to be small. The Simplified Makdisi and Seed procedure was also used to estimate slope deformations for comparison purposes. The comparisons are shown in Table 7 and 8.

The results for the liquefied cases are tabulated in Tables 9 and 10 for Cross Sections I and II, respectively. As expected, under the 475-year return period event, larger slope deformations were estimated, For Cross Section I, up to about 1.4 feet and 0.6 foot of deformations were calculated for the reservoir and slough slopes, respectively. Maximum deformations of about 2.0 feet were estimated for the reservoir and slough slopes of Cross Section II. Under the smaller ground motions of 43-year return period, maximum reservoir slope deformation of about 1 foot was calculated.

As noted in Tables 7 through 10, convergence was not obtained for some of the cases with larger earthquakes (2500-year and some 475-year events). Similarly to the above discussion, a deformation of over 12 feet was assumed to have a 95 percent probability of embankment failure. This condition was considered to represent the expected embankment performance under severe earthquake events.

The results of the seismic deformation analyses for the rock berm alternative are summarized in Table 10A for the 475-year earthquake event.

This section of the report summarizes the estimated probability of failures for the various cross sections analyzed under the different earthquake scenarios. The modes of failure considered for this study included those caused by an earthquake event, such as seismic-induced slumping, slope failure, liquefaction-induced sliding and lateral spreading and other related secondary failures (i.e., piping through an open crack, etc.).

4.1 EMBANKMENT FRAGILITY CURVE

The embankment fragility curve developed by the Seismic Vulnerability Sub-Team (CALFED, 1998) was used for this study for both the liquefied and non-liquefied cases. This curve was then utilized to evaluate the probability of failure of an embankment cross section with given earthquake-induced deformations.

4.2 FAILURE PROBABILITY

Failure probabilities for the two project alternatives (bench and rock berm) and the two embankment cross sections (Cross Section I and II) were calculated by combining the various weights (probabilities) associated with reservoir and slough water levels, earthquake ground motion and liquefaction scenarios. Weights assigned to the reservoir and slough water level scenarios were estimated based on the time percentage of each scenario to occur annually. Weights for the earthquake ground motion scenarios were estimated by assuming a time-independent Poisson process for earthquake occurrence and a project life cycle of 50 years. In estimating the weights for the three ground motion scenarios, we assumed that the 43-year, 475-year and 2,500-year ground motions are represented by ground motions with return periods less than about 130 years, 130 years to about 1,000 years and greater than 1,000 years, respectively. The failure probabilities were calculated considering the contributions from the large/distant and moderate/near earthquakes and critical slide masses on the reservoir and slough sides. Weights for the liquefaction scenarios were selected based on judgment and evaluation of sampler blowcounts recorded in the sandy deposits.

Tables 11 through 14 summarize the contributions of the various scenarios and provide estimates for the total probability of failure for each project alternative and each cross section for a 50-year life cycle. The bench alternative with peat at elevation –20 feet has about 19 percent chance of failure (Table 11), while the cross section with peat at elevation –40 feet has about 28 percent chance of failure (Table 12). For the rock berm alternative, the cross section with peat at elevation –20 feet has about 17 percent chance of failure (Table 13), while the cross section with peat at elevation –40 feet has about 23.5 percent chance of failure (Table 14).

This report presents the results of estimated seismic performance of the two embankment design alternatives, and addresses the probability of earthquake-induced embankment failure.

Table 10A shows that the calculated seismic deformations are large for several conditions for the 475-year earthquake event. The results of the evaluation appear to suggest that the rock berm alternative would provide for a lower probability of failure than the bench alternative. The rock berm alternative is preferable to the bench alternative because it places the embankment over the existing levee and, therefore, makes use of the stronger peat under the levee as opposed to the weaker free-field peat. In addition, the rock berm alternative provides a more stable slough side slope.

Because liquefaction would lead to large deformations that would affect overall stability of the embankment, further investigation and evaluation of the existing levee materials are recommended. Depending on the extent of the potentially liquefiable sands within the existing levee, removal of the loose sands may need to be implemented.

Due to the limitations of the QUAD4M computer program for large earthquake loads, a uniform assumption has been made for estimating the expected embankment deformation. Although this assumption is considered conservative, a more rigorous non-linear analysis would probably be useful and could provide more insight into the deformation patterns associated with large strains under the large earthquake shaking. This analysis could also provide more insight into the comparative performance of the embankment alternatives under the larger earthquakes.

The calculation of the overall risk is presented in the URS Risk Analysis report. The risk analysis combines the probabilities of failure from various events (seismic, operational and flood) and their failure consequences.

Abrahamson N., 1993, Non-stationary spectral matching program, personnel communication.

Boulanger R. et al., 1997, Dynamic properties of Sherman Island peat, Report No. UCD/CGM-97/01, Center for Geotechnical Modeling, Department of Civil and Environmental Engineering, University of California at Davis.

CALFED, 1998, Seismic vulnerability of the Sacramento-San Joaquin Delta Levees, Calfed Bay-Delta Program, Seismic Vulnerability Sub-Team, December.

Department of Water Resources, 2002, Draft report on engineering investigations, In-Delta Storage Program, CALFED Bay-Delta Program.

Hudson, M., Idriss, I.M., Beikae, M., 1994, User's Manual for QUAD4M, Center for Geotechnical Modeling, Department of Civil & Environmental Engineering, University of California, Davis, California, May.

Kokusho T., 1980, Cyclic triaxial test of dynamic soil properties for wide strain range, Soils and Foundations, 20, 45-60.

Lilhanand, K. and Tseng, W.S., 1988, Development and application of realistic earthquake time histories compatible with multiple-damping design spectra, Proceeding of the 9th World Conference on Earthquake Engineering, Tokyo-Kyoto, Japan, August.

Makdisi, F.I. and Seed, H.B., 1978, Simplified procedure for estimating dam and embankment earthquake-induced deformations', Journal of the Geotechnical Engineering Division, ASCE, Vol. 104, No. GT7, July.

Newmark N.M., 1965, 'Effects of earthquakes on dams and embankments', Geotechnique, Vol. 15, No. 2, June.

Schnabel, P.B., Lysmer, J., and Seed, H.B., 1972, 'A computer program for earthquake response analysis of horizontal layered sites', Earthquake Engineering Research Center, Report No. EERC 72-12, December 1972.

Seed, H.B. and Idriss, I.M., 1970, Soil moduli in damping factors for dynamic response analysis, Report No. EERC 70-10, University of California, Berkeley, December.

URS, 2000, Geotechnical services report in support of the supplemental EIS/EIS, Report prepared for Jones & Stokes Associates, Inc. March 17.

Vucetic, M. and Dobry, R., 1991, Effect of soil plasticity on cyclic response, Journal of Geotechnical Engineering, ASCE, Vol. 117, No.1.

Wehling, T.M. et al.,2001, Nonlinear dynamic properties of a fibrous organic soil, Paper accepted for publication in ASCE Journal of Geotechnical Engineering.