Performance of Port Facilities During the Northridge Earthquake

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SYNOPSIS During the January 17, 1994, Northridge earthquake, two of the Port of Los Angeles' facilities called Berths 121-126 and Pier 300 sustained moderate damage. Lateral displacement of dikes up to five inches and liquefaction of hydraulic fills were observed. Several geotechnical analyses from simplified SPT-based method to sophisticated fully-coupled analyses are presented. Observed lateral displacements are predicted reasonably well by the fully-coupled analysis procedure and an intermediate analysis procedure which incorporates some results from a fully-coupled analysis in to a simplified Newmark-type deformation analysis. The potential for higher pore pressure generation underneath the dike compared to a level ground is also discussed.

INTRODUCTION

During the January 17, 1994, Northridge earthquake (moment magnitude, M_w = 6.7), approximately 32 miles southeast of the epicenter, two of the Port of Los Angeles' (POLA's) facilities called Berths 121-126 and Pier 300 sustained moderate damage. The general locations of these facilities within POLA are shown in Figure 1.

The Berths 121-126 site consists of two wharves and associated backland area used for container storage and related activities. The wharves and the backland were constructed in 1982. The southern portion of the backland was formed by hydraulically placing materials dredged during the construction of the project. During the Northridge earthquake, several cracks formed in the asphalt pavement covering the backland area. These cracks were primarily confined to the area where backland was formed by hydraulically placing the dredged fill. In an area approximately 300 feet behind the Berth 121 wharf, hydraulic fill material was brought to the surface through cracks in the pavement as a result of liquefaction of the hydraulic fill. The pavement in the immediate vicinity of the liquefaction area settled approximately six to eight inches. The wharf structures also experienced lateral movements of approximately two inches at Berth 121 and approximately three inches at Berth 126. Cracks, approximately two to three inches wide, formed in the asphalt just behind the wharves due to seaward movement of the wharves. The hydraulic fill immediately behind the Berth 121 dike settled approximately four to six inches which resulted in settlement of the pavement.

The Pier 300 site was undeveloped when the Northridge earthquake occurred. This site is a 190-acre landfill created by hydraulically placing materials dredged during the Harbor Deepening Project from January 1981 through April 1983. The fill was retained by rock dikes along the southern and eastern boundaries of the site (see Figure 1). Post-dredging investigations indicated that the hydraulic fill materials varied significantly, in both lateral and vertical directions, and

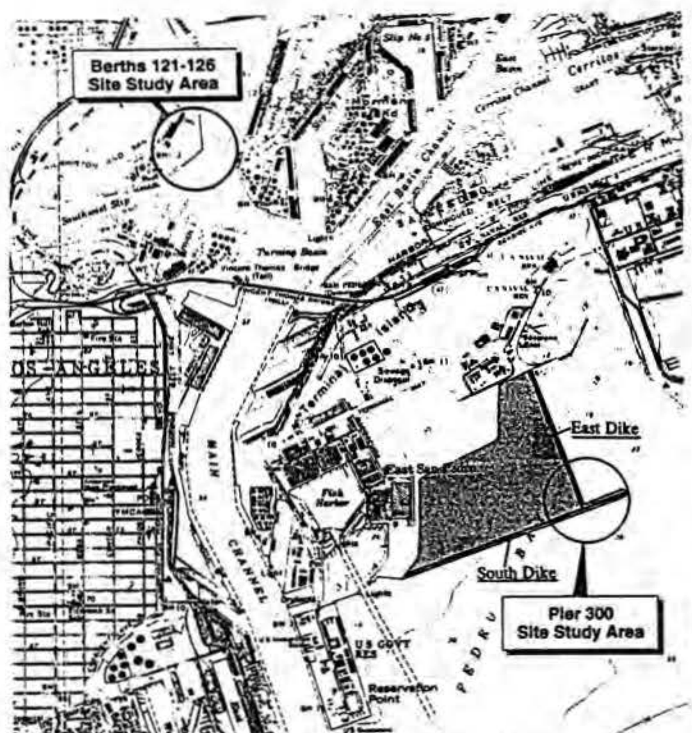

Fig. 1. Locations of Berths 121-126 and Pier 300 Sites within Port of Los Angeles

consisted predominately of soft clays and loose sands. Since the time of hydraulic fill placement, several ground improvement programs have been completed at the Pier 300 site to improve the clays. At the southeastern portion of the site an approximately 24 feet-high surcharge, used in the ground improvement program, was present at the time of the Northridge earthquake. The southern toe of the surcharge was parallel to and approximately 150 feet behind the south dike. The eastern toe of the surcharge was approximately 50 feet behind and parallel to the east dike.

The southeastern portion of the Pier 300 site sustained some damage during the earthquake. In this area, several cracks formed along the south and east dikes. The cracks along the south dike were approximately 0.5- to 1-inch wide and 200 feet long. The east dike sustained cracks two to five inches wide and approximately 1000 feet long. Lateral movements of the dikes are believed to be the cause of these cracks. Sand boils were also observed at three locations in the southeastern corner of the site immediately behind the dikes.

This paper presents the evaluation of the performance of the Berths 121-126 and Pier 300 East Dike during the Northridge earthquake using a variety of analyses. Analyses of Pier 300 South Dike and more details about the analyses, soil profiles, and properties can be found else where (Earth Tech, 1995). The analyses performed include the following: simplified Standard Penetration Test (SPT)-based liquefaction analyses; one-dimensional total stress dynamic analyses using the computer code SHAKE (Schnabel et al., 1972); one-dimensional, fully-coupled, effective stress, dynamic analyses using the computer code DYSAC2 (Muraleetharan et al., 1988, 1991); a two-dimensional fully-coupled, effective stress, deformation analysis using DYSAC2; two-dimensional static and pseudo-static slope stability analyses; and Newmark's sliding block deformation analyses.

Simplified SPT-based procedures are useful in predicting liquefaction potential of soils and settlement of soil deposits. Simplified Newmark-type methods are useful in predicting lateral displacement of slopes. While simplistic and easy to use, these methods do not provide insights into the mechanisms involved in a problem. On the other hand, two-dimensional, fully-coupled, non-linear, dynamic analyses procedures such as DYSAC2 can provide a complete picture about the failure and deformation mechanisms for a problem. However, these dynamic analysis procedures are too costly to use on small projects or to analyze each and every cross-section on a big project. Therefore, a class of intermediate analysis procedures, which are problem-specific and retain the insights gained from a limited number of fully-coupled analyses, yet, simple enough to analyze numerous design scenarios would be very useful for design projects. One example of such an intermediate analysis procedure is utilizing average excess pore pressures developed in various layers obtained from a fully-coupled analysis procedure in calculating the yield acceleration using the pseudo-static slope stability analysis. This yield acceleration can be subsequently used to calculate the lateral displacement from the Newmark's method. The above described approach was also used to evaluate the performance of Berths 121-126 and Pier 300 sites during the Northridge earthquake and the results are presented in this paper.

SOIL PROFILES AND PROPERTIES

Simplified 1-dimensional soil profiles for Berths 121-126 and Pier 300 East Dike are shown in Figures 2 and 3, respectively. Soil properties of the various layers are listed in Table 1. Soils are grouped into the generalized stratigraphic units identified during the POLA/2020 Plan Geotechnical Investigations (Fugro-McClelland, Inc., 1992). These stratigraphic units are also shown in Figures 2 and 3. All of the soil parameters used in the analyses were obtained from review of past geotechnical investigations at or in the vicinity of the sites. Extensive in situ and laboratory tests have been performed on soils at or near the Pier 300 site during previous geotechnical investigations. Therefore, better estimate of soil parameters were made for the Pier 300 East Dike profile compared to the Berths 121-126 profile. No information could be obtained on the pre-Northridge earthquake properties of the hydraulic fill at the Berths 121-126 site. Properties of this hydraulic fill were estimated using post-earthquake Cone Penetration Tests (CPTs) and using properties of other similar hydraulic fills.

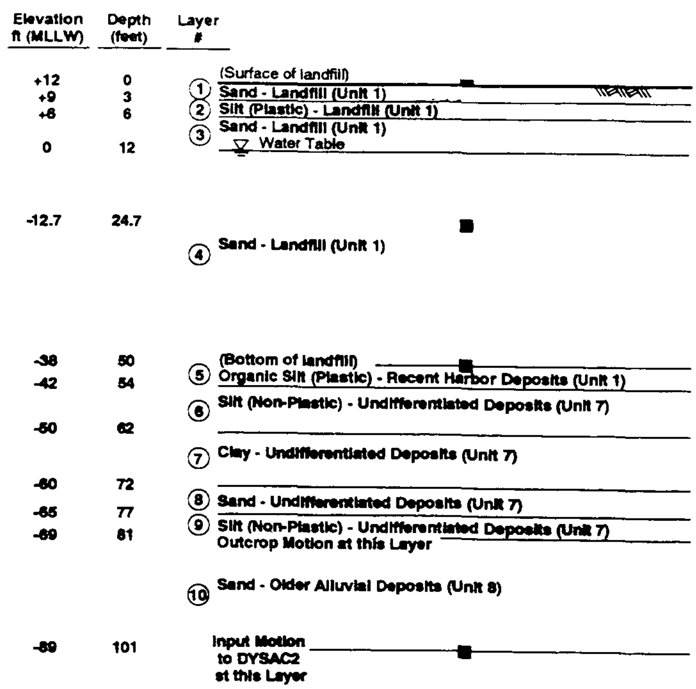

Fig. 2. Simplified One-Dimensional Soil Profile for Berths 121-126 Site (MLLW = Mean Lower Low Water Level)

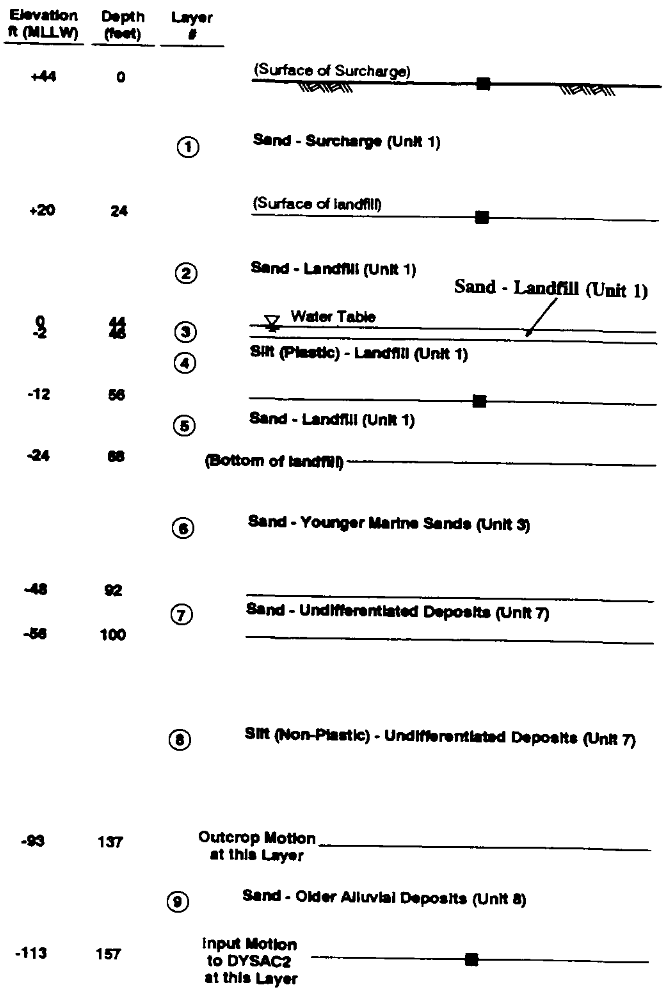

Fig. 3. Simplified One-Dimensional Soil Profile for Pier 300 East Dike Site (MLLW = Mean Lower Low Water Level)

TABLE 1. Soil Properties

Layer #	Total Unit Wt. (pcf)		N_1		N_1'	
	Berths 121-126	Pier 300 East Dike	Berths 121-126	Pier 300 East Dike	Berths 121-126	Pier 300 East Dike
1	110	105	17	13	22	17
2	100	110	15	7	--	11
3	110	118	14	5	18	9
4	118	110	9	4	13	--
5	118	118	4	4	--	9
6	122	122	26	16	36	19
7	120	125	12	18	--	25
8	127	122	24	16	32	24
9	122	130	20	36	30	40
10	127	---	41	--	46	--

Notes: N_1 = Overburden-normalized SPT value
N_1' = Overburden-normalized and fines content-adjusted SPT value

REPRESENTATIVE ACCELERATION-TIME HISTORY FOR THE SITES

Based on the review of strong motion records made available by the Division of Mines and Geology (DMG) of the California Department of Conservation (DMG, 1994), a peak firm-ground acceleration of 0.12g was selected as the representative value for the Berths 121-126 and Pier 300 sites. Since no digital acceleration-time histories were available near the sites at the time of the analyses, acceleration-time history recorded at the Long Beach City Hall grounds, approximately 33 miles from the epicenter and about 5 miles away from the sites, was chosen as the representative acceleration-time history. The digital acceleration-time history from the Long Beach City Hall grounds strong motion instrument, scaled to 0.12 g, was selected as the representative firm-ground outcrop motion for both sites. This acceleration-time history is shown in Figure 4.

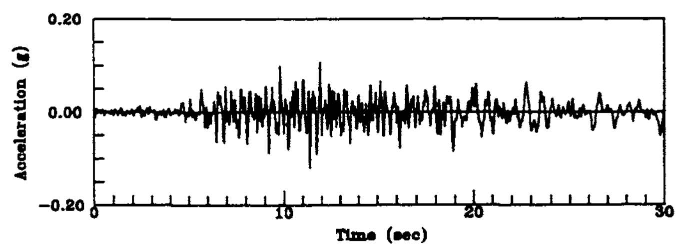

Fig. 4. Firm-Ground Outcrop Acceleration-Time History Selected for the Analyses

SHAKE ANALYSES

The computer code SHAKE is based on the 1-dimensional propagation of shear waves through horizontally-layered soils (Schnabel et al., 1972). This is a total stress analysis; i.e., the effects of pore pressure are not considered in the analysis. The frequency-domain-based SHAKE can compute response motions anywhere in the deposits for an input motion applied anywhere in the deposit. The program incorporates an elastic half space at the bottom of the deposits analyzed, and equivalent-linear soil properties that can vary with cyclic shear strain.

Older Alluvial Deposits shown in Figures 2 and 3 were considered as firm-ground at Berths 121-126 and Pier 300 sites, respectively. Acceleration-time history shown in Figure 4 is used as the input motion for SHAKE analyses at outcropping Older Alluvial Deposits. For the subsequent DYSAC2 analyses, motions calculated at the bottom of 20-feet-thick Older Alluvial Deposits were considered as input motions. These 20-feet-thick Older Alluvial Deposits were included in DYSAC2 analyses to account for any possible pore water pressure dissipations through these coarse-grained deposits.

Typical modulus-shear strain and damping-shear strain curves were used in the SHAKE analyses for clays and sands. Sand and clay curves were also used for non-plastic and plastic silts, respectively.

The results of the SHAKE analyses are presented in Figures 5 and 6. These figures illustrate the acceleration-time response at the top surface of the landfill and within the Older Alluvial Deposits. Based on these analyses peak ground acceleration values of 0.17g and 0.14g were obtained for the Berths 121-126 and Pier 300 sites, respectively. The value of 0.14g was obtained from the Pier 300 South Dike analysis which is not shown here. These peak ground acceleration values were used in the simplified SPT-based liquefaction analyses described in the next section.

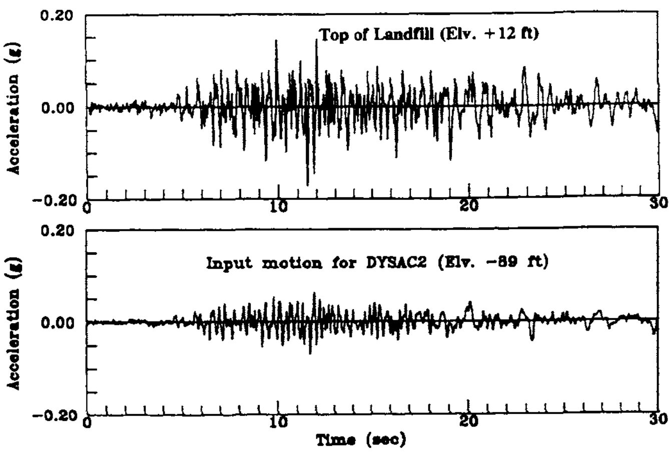

Fig. 5. Acceleration-Time Histories Predicted by SHAKE for Berths 121-126 Site

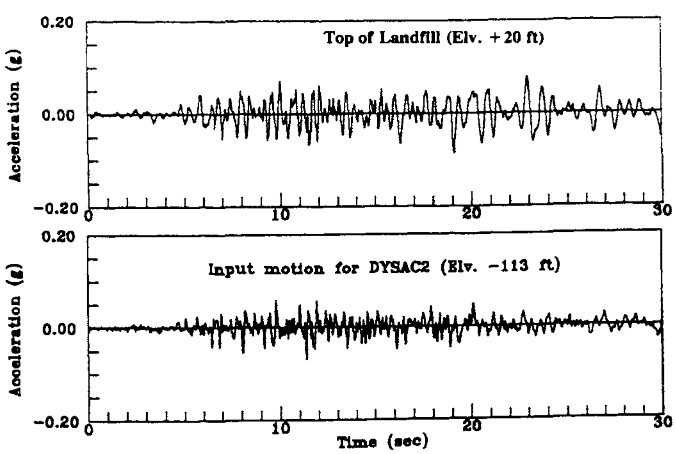

Fig. 6. Acceleration-Time Histories Predicted by SHAKE for Pier 300 East Dike Site

SIMPLIFIED SPT-BASED LIQUEFACTION ANALYSES

Simplified liquefaction analyses were performed using the methodology outlined by Seed et al. (1985). Cyclic shear stresses induced during the earthquake were calculated and converted to equivalent overburden-normalized and fines-content adjusted SPT values required to cause liquefaction, $(N_1')_{60}$, and were compared against the average $(N_1')_{60}$ values at both sites. Marginal liquefaction was predicted at Berths 121-126 site in Layer Number 4 and no liquefaction was predicted at the Pier 300 East Dike site. When $(N_1')_{60}$ values from individual borings were compared against $(N_1')_{60}$ values required to cause liquefaction, number of depths in certain borings within Pier 300 landfill indicated potential liquefaction. No pre-earthquake $(N_1')_{60}$ values were available within the landfill for Berths 121-126 site.

ONE-DIMENSIONAL DYSAC2 ANALYSES

DYSAC2 (Muraleetharan et al., 1988, 1991) is a finite element computer code for two-dimensional dynamic analysis of geotechnical engineering structures. DYSAC2 is based on the rigorous mathematical formulation of the coupled dynamic behavior of soil skeleton and pore fluid (Biot, 1962). Linear and non-linear material behavior can be modeled in DYSAC2. Non-linear material behavior is modeled using two bounding surface, elastoplastic, effective stress models. One for cohesive soils (Dafalias and Herrmann, 1986) and another for non-cohesive soils (Yogachandran, 1991). Description of these constitutive models and model parameters can be found elsewhere. Some of the key parameters for various stratigraphic units used in the DYSAC2 analyses are summarized in Table 2. The Constitutive model for cohesive soils was used to represent stress-strain behavior of clays and plastic silt and the constitutive model for non-cohesive soils was used to represent sands and non-plastic silts.

TABLE 2. Selected Input Parameters Used in the DYSAC2 Analyses

Parameters	Landfill Soils (Unit 1)		Younger Marine Sands (Unit 3)	Recent Harbor Deposits (Unit 1)	Undifferentiated Materials (Unit 7)			Older Alluvial Deposits (Unit 8)
	Sand	Silt (Pla.)		Organic Silt (Plastic)	Sand	Silt (Non Pla.)	Clay	Sand
Dry Unit Weight (pcf)	87	75	93	86[1]/ 50[2]	98	92	90	107
Permeability (cm/sec)	1.0 E-03	1.0 E-07	4.6 E-04	1.0 E-07	4.6 E-04	2.9 E-05	7.6 E-08	1.2 E-03
Poisson's Ratio	0.3	0.3	0.20	0.3	0.2	0.3	0.3	0.2
Compression Index (C_c)	0.037	0.445	0.039	0.445[1]/ 0.900[2]	0.039	0.092	0.445	0.023
Swelling Index (C_s)	0.007	0.044	0.005	0.044[1]/ 0.092[2]	0.005	0.021	0.044	0.009
Equivalent Angle of Internal Friction in Compression (Degree)	33.0	28.0	36.0	26.0	36.0	35.0	34.0	37.0
Equivalent Angle of Internal Friction in Extension (Degree)	32.2	27.9	38.1	25.5	38.1	36.5	37.3	32.2
Over Consolidation Ratio	--	1.0	--	1.0	--	--	2.5	--

Notes: 1. Underneath dike and landfill
2. Outside dike and landfill

Excess pore pressure ratio-time histories predicted by DYSAC2 in a sand and a plastic silt layers at Berths 121-126 and Pier 300 East Dike sites are shown in Figures 7 and 8. These and other excess pore pressure ratio-time histories, not shown here, indicated that maximum excess pore pressure ratios predicted by 1-dimensional DYSAC2 analyses within sand and non-plastic silt layers are less than 15 percent. However, DYSAC2 predicted approximately 65 percent pore pressure ratio within the plastic silt layers as shown in Figures 7 and 8.

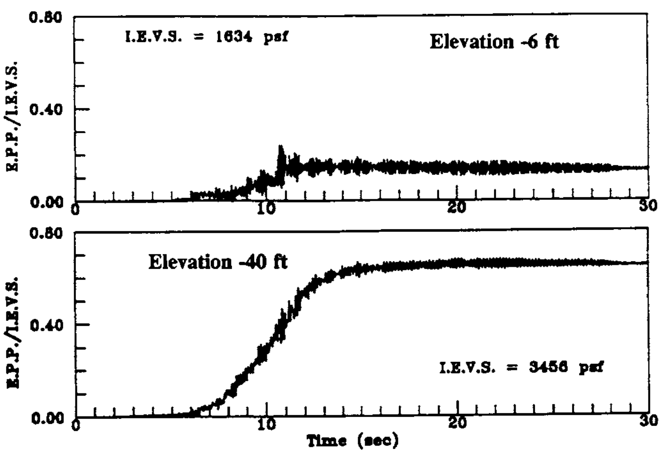

Fig. 7. Excess Pore Pressure Ratio-Time Histories Predicted by 1-Dimensional DYSAC2 Analysis for Berths 121-126 Site (E.P.P. = Excess Pore Pressure, I.E.V.S. = Initial Effective Vertical Stress)

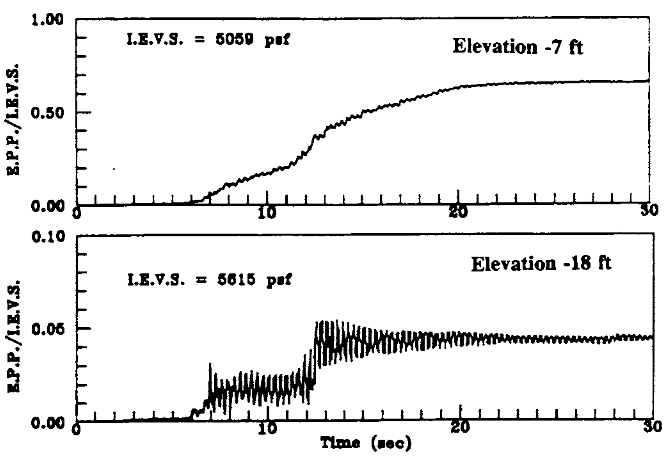

Fig. 8. Excess Pore Pressure Ratio-Time Histories Predicted by 1-Dimensional DYSAC2 Analysis for Pier 300 East Dike Site (E.P.P. = Excess Pore Pressure, I.E.V.S. = Initial Effective Vertical Stress)

Maximum accelerations of 0.30g and 0.12g were predicted by DYSAC2 on top of landfills for Berths 121-126 and Pier 300 East dike, respectively. Most of the amplification of the Berths 121-126 motion was contributed by the top most sand layer. This could have been due to the natural frequency of this sand layer matching the predominant frequency of the motion underneath it. Since no pre-earthquake information was available for this landfill, bounding surface model parameters of other similar landfills were used in the analyses. These assumed properties may also have contributed to the high predicted acceleration value of 0.30g.

TWO-DIMENSIONAL DYSAC2 ANALYSIS OF PIER 300 EAST DIKE CROSS-SECTION

The finite element mesh used to represent the Pier 300 East Dike cross-section is shown in Figure 9. The total lateral displacement of the crest of the East Dike predicted by DYSAC2 is approximately 6 inches. A settlement of approximately 1 inch was also predicted at the crest of the dike. These values approximately match the observed values during the Northridge earthquake. Maximum horizontal and vertical accelerations of 0.15g and 0.07g, respectively, were predicted at the crest of the dike. Selected excess pore pressure ratio-time histories underneath the dike and within the landfill are shown in Figure 10.

The average pore pressures generated in the foundation soils immediately below the dike and landfill are on the order of 40 to 50 percent of the effective vertical stresses. Average pore pressure ratio within the landfill plastic silt layer (layer containing Element 139) is approximately 70 percent. These average pore pressure values were subsequently used in the pseudo-static slope stability analyses in calculating the yield accelerations.

Considerably higher pore pressures are predicted underneath the dike and landfill in the 2-dimensional DYSAC2 analysis compared to the 1-dimensional analysis. This increase is most likely due to the fact that in a two-dimensional analysis, soils underneath the dike and landfill are subjected to cyclic loading superimposed on monotonic loading caused by the outward movement of the dike, compared to only cyclic loading in the 1-dimensional analysis. The observed sand boils immediately behind the dike during the Northridge earthquake also support the possibility of high pore water pressures underneath the dike.

SLOPE STABILITY AND NEWMARK SLIDING BLOCK ANALYSES

Two-dimensional slope stability analyses were performed using the computer code PC STABL 5M (Achilleos, 1988). These slope stability analyses were used to obtain the static factors of safety and yield accelerations of various cross sections. Yield acceleration is the horizontal acceleration that results in a pseudo-static factor of safety of 1.0. Once the yield acceleration is estimated, lateral displacement of the slope during any given earthquake motion can be estimated by assuming that the slope moves as a rigid body under applied

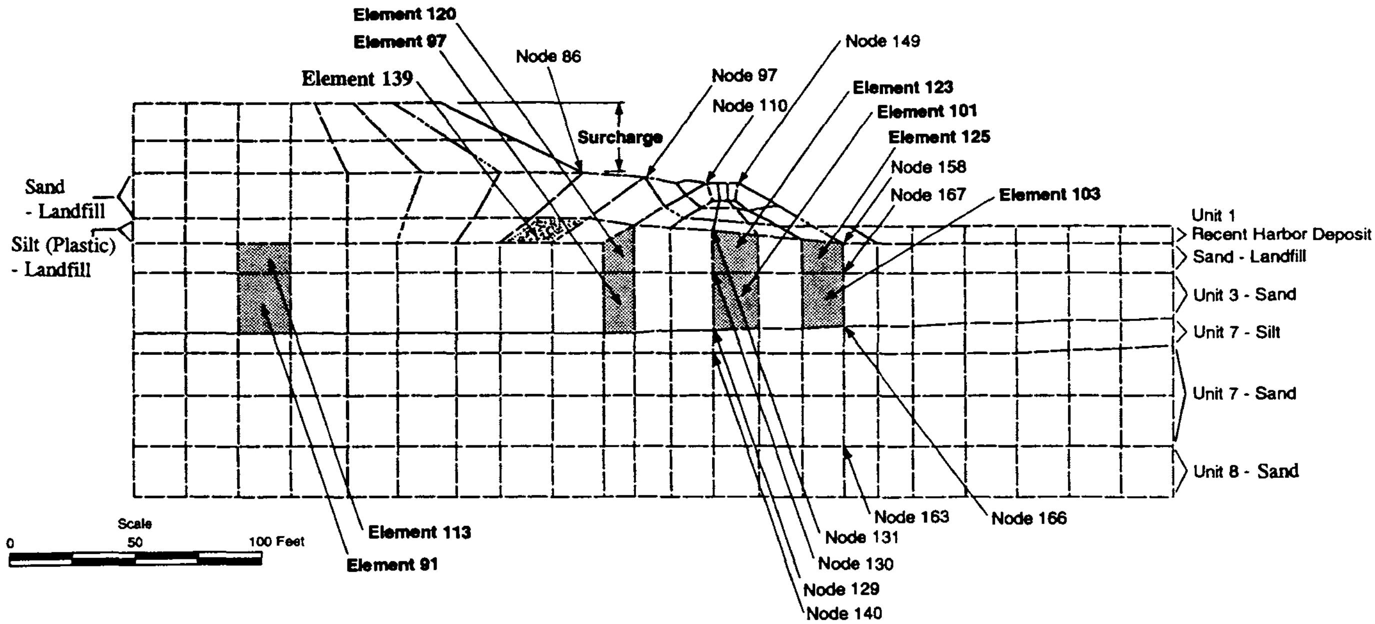

Fig. 9. Finite Element Mesh Used in the 2-Dimensional DYSAC2 Analysis of Pier 300 East Dike Site

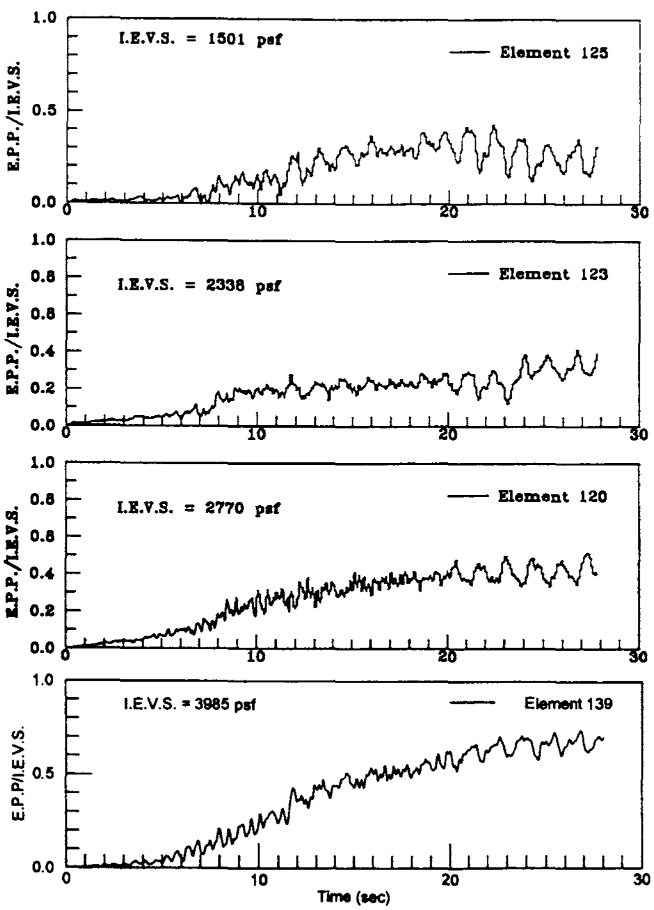

Fig. 10. Excess Pore Pressure Ratio-Time Histories Predicted by 2-Dimensional DYSAC2 Analysis for Pier 300 East Dike Site (E.P.P. = Excess Pore Pressure, I.E.V.S. = Initial Effective Vertical Stress)

acceleration in excess of the yield acceleration (Newmark, 1965).

Results of Newmark's sliding block analyses for Berths 121-126 dikes and Pier 300 East and South Dikes are shown in Figure 11. Anticipated lateral displacements are plotted as functions of yield accelerations. The lateral displacements were determined using acceleration-time histories obtained at the bottom, middle, and top of the landfill from the SHAKE analyses of the simplified soil profiles and averaging the three displacements values. For comparison purposes, the displacements based on the firm-ground outcrop motion is also included in Figure 11. Sophistication provided by SHAKE analysis was considered sufficient in calculating the displacements using Newmark's method.

As the DYSAC2 analysis indicated that the pore pressure buildup is likely during the earthquake, yield acceleration values were calculated from the pseudo-static analyses using various excess pore pressure ratio values for the landfill sand and plastic silt layers. For example, for a cross-section through Berth 121, when an excess pore pressure ratio values of 0.0 was used in all the layers a yield acceleration value of 0.19g was obtained. For the same section when an excess pore pressure ratio of 0.55 was used in Layer Numbers 4 and 5 (see Figure 2) a yield acceleration value of 0.03g was calculated. From Figure 11, yield acceleration values of 0.19g and 0.03g correspond to lateral displacements of 0.0 and 2.4 inches, respectively. Similarly for a cross section through Pier 300 East Dike, a lateral displacement value of 0.0 inches was calculated when using an excess pore pressure ratio of 0.0 in all the layers and a value of 2.9 inches was calculated when using an excess pore pressure ratio 0.45 in Layer Numbers 3 and 5 and 0.70 in Layer Number 4 (see

Figure 3). As it can be seen from the above calculations when average pore pressures obtained from the 2-dimensional DYSAC2 analysis were used together with Newmark's analysis, predicted lateral displacements are in the range of observed values during the earthquake.

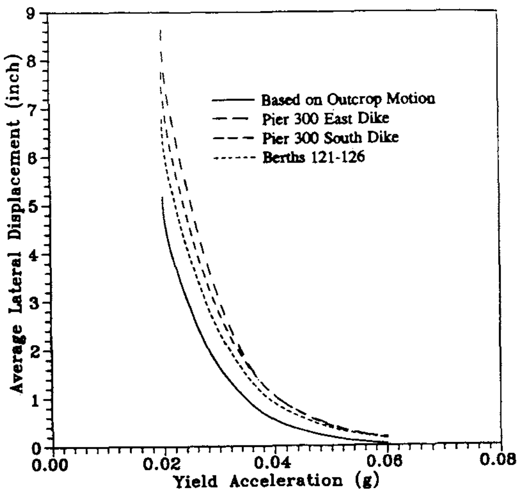

Fig. 11. Results of Newmark's Sliding Block Analyses

CONCLUSIONS

Based on the results of the analyses performed during this study, the following conclusions can be made:

1. Observed performance of the Pier 300 East Dike during the Northridge earthquake is predicted reasonably well by the two-dimensional DYSAC2 analysis.
2. Utilizing the average excess pore pressures obtained from the two-dimensional DYSAC2 analysis in pseudo-static slope stability analysis and Newmark's deformation analysis, observed lateral deformations are predicted well. This methodology provides a problem-specific, intermediate practical analysis procedure having a level of sophistication between simplified SPT-based methods and fully-coupled analysis procedures. This intermediate analysis procedure is a cost-effective design tool to analyze various design scenarios of a project.
3. Due to the fact that soils underneath the dikes are subjected cyclic loading superimposed on monotonic loading can cause significantly more pore pressures in these soils compared to a similar soil far away from the dike which is only subjected to monotonic loading. Evidence of liquefaction just behind the dike at the Pier 300 site seems to support this conclusion.

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