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ABSTRACT

The combined San Pedro Bay, California Ports (Long Beach and Los Angeles) have been developed mostly by placing dredged material behind rock dikes to create useable land and the wharfs constructed over the rock dikes. An overview of the stability analysis of the dikes was presented in a 1991 paper that summarized slope stability and seismic criteria prior to 1991. Since that time, deeper channel depths, higher seismic criteria, and higher seismic survivability expectations by the users have resulted in higher levels of analysis. This paper provides an update of a paper presented in 1991 and presents data regarding slope stability finite element/difference method (FEM) analysis completed by different investigators that included the contribution of the wharf piles that extend through the rock dikes to slope stability and reduction of deformation.

The conclusions reached and statements made in this paper are solely those of the authors and do not necessarily represent the opinions of other parties, firms, or ports in any of the projects referenced.

KEYWORDS

slope stability, soil-structure interaction, rock embankment slopes, finite element analysis, deformation, seismicity, wharf

BACKGROUND AND SITE CONDITIONS

The San Pedro Bay contains both the Ports of Long Beach (POLB) and Los Angeles (POLA) as shown in Fig. 1. The POLA is west of the POLB. The channel depths have been increasing for container terminals and are near -13.5 to -16.5 meters mean lower low water (MLLW). The container terminal ground surfaces were near +4.5 meters MLLW.

Typical Port construction procedures have consisted of building containment rock dikes and filling behind the rock dikes, usually using hydraulic methods to create new land. Several different types of containment rock dikes and filling methods were previously discussed (Yourman & Diaz, 1991). The POLA Pier 400 (Fugro West, Inc., 1999), POLB Pier J (Dames & Moore, 1961), Pier J Expansion (Geofon, 1987) and portions of the POLB Pier A (Leighton & Associates, 1996) projects were constructed using this technique. Typical sections and the cross sectional areas of these fill rock dike sections are shown in Fig. 2. The buttress section at the toe of the 1961 Pier J rock dike was added when the channel was deepened in 1990. (Harding Lawson Associates, 1990A). The amount of quarry run rock per meter of length of rock dike is shown in parenthesis on Fig. 2 and Fig. 3.

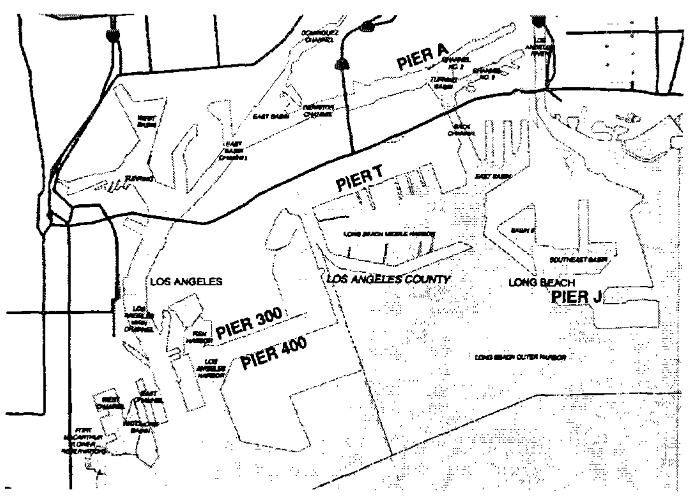

Fig. 1 - San Pedro Bay

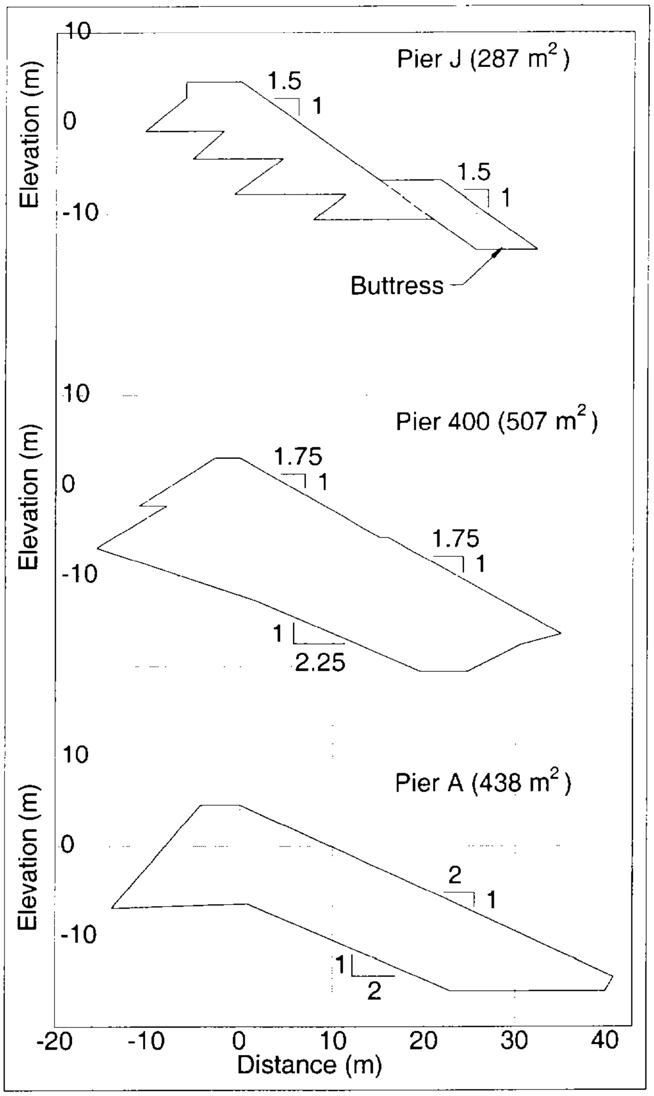

Fig. 2 - Fill Rock Dike Sections

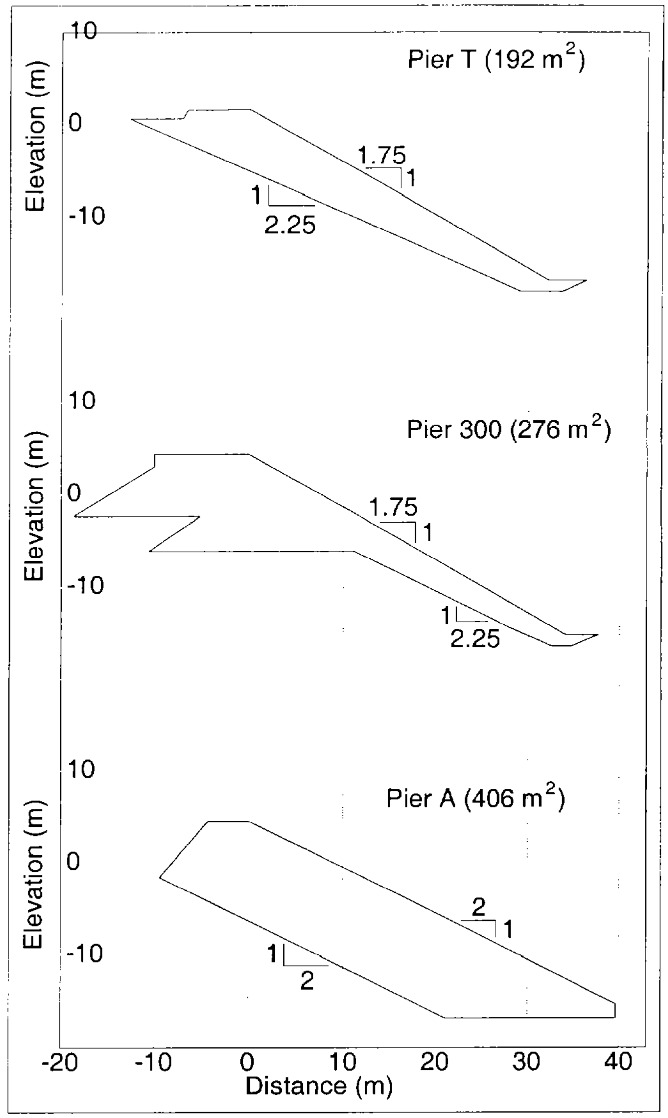

Fig. 3 - Cut Rock Dike Sections

For other projects, existing fill and/or natural ground and rock dikes were excavated and a new rock dike/revetment was placed to provide the grade between the channel and upland. The POLB's Pier T (Diaz•Yourman & Associates, 1997), portions of Pier A, and POLA Pier 300 (CH2MHill, 1992) projects were constructed in this manner. Typical sections and the cross sectional areas of these cut rock dike sections are shown in Fig. 3.

Before the mid 1980s, the inclination of the rock dikes was 1H:1.5V (horizontal to vertical) such as for Pier J and Pier J expansion. However, since the POLA Berths 212-215 project (Harding Lawson Associates, 1987), the rock slopes were inclined at 1H:1.75V. (A 1H:1.75V slope with the channel depths of 13.7 meters allows for a 30-meter gauge container handling crane to be fully supported on a wharf.)

For the recent projects, the wharves were usually constructed with an open field of 610 mm octagonal 32-meter long (approximate) precast prestressed concrete piles supporting cast-in-place concrete decks. The pile bents were typically 4.6 to 5.5 meters apart and the piles within a single bent had similar spacing. To resist lateral loads, the POLB wharves used batter piles combined with a seismic fuse while the POLA used vertical piles in bending.

GEOLOGIC AND SEISMIC SETTING

San Pedro Bay is in a highly active seismic area. The primary faulting system is the 1000-kilometer-long (km) San Andreas Fault system, located approximately 90 km east of San Pedro Bay. Of more importance to the Ports are the Newport Inglewood fault system located 6 km east of POLB and the Palos Verdes fault located under the POLA. Probabilistic estimation of peak ground acceleration (PGA) within the Ports has been performed since 1975 and for most of the major projects since then. Plots of some of the results of these analyses, shown on Fig. 4, indicate progressively higher estimates of ground accelerations for the operating level earthquake (OLE, 50 percent exceedance in 50 years) and contingency level earthquake (CLE, 10 percent of exceedance in 50 years) since 1990. Prior to 1991, the OLE and CLE had PGA of approximately 0.19 g and 0.3 g, respectively. However, since 1991, the estimated OLE and CLE PGA were approximately 0.23 g and 0.53 g, respectively.

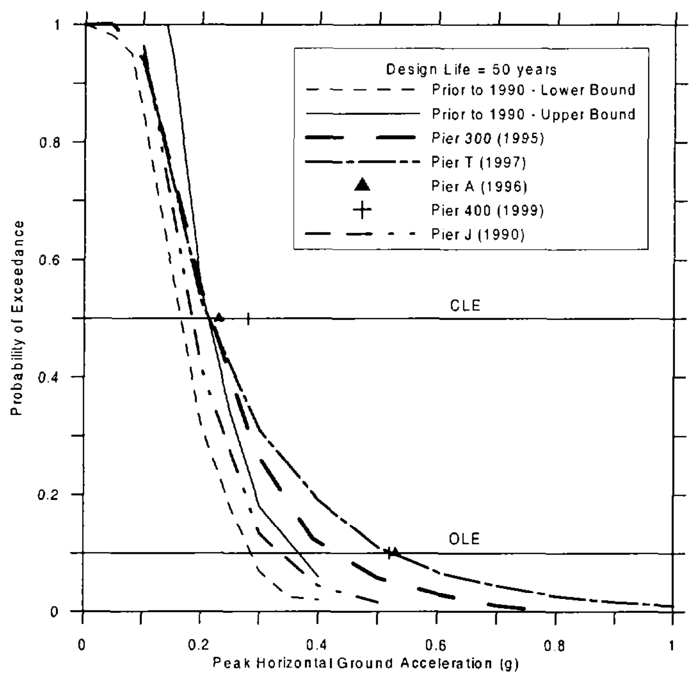

Fig. 4 - PGA vs. Probability of Exceedance

ANALYSES

Prior to 1991, slope stability analysis and liquefaction analysis were generally performed separately. The slope stability analyses were performed using limit equilibrium computer programs such as PCSTABL or STABR, for static and pseudostatic seismic cases and the liquefaction analysis was performed separately using simplified methods recommended by Seed and his co-workers (Yourman & Diaz, 1991). Deformations were estimated using Newmark's method such as that recommended by Makdisi & Seed (1978). The more recent projects used similar approaches for initial evaluations but, because of deeper channel depths, higher seismic criteria and higher seismic survivability expectations, FEM analysis were completed by different investigators which included the contribution of the wharf piles which extend through the rock dikes to slope stability and reduction of deformation. A summary of the FEM analysis techniques is presented in Table 1.

POLB Pier T

This project started with a 790-meter-long wharf that is currently being extended to 1200 meters (Diaz•Yourman & Associates, 1997). The wharf was designed for a water depth of -16.8 meters MLLW. Environmental restrictions required the slope to cut into an area that was filled in the 1940s (Fig. 3). A double row of batter piles was used together with a seismic fuse to provide lateral resistance as shown in Fig. 5. A conventional slope stability and simplified deformation analysis were performed by Diaz•Yourman & Associates (1997) as summarized in Table 2. FEM deformation analysis was performed by Woodward-Clyde Consultants (1996), as summarized in Table 3. A summary of the results of simplified deformation analysis is presented on Fig. 6. A comparison of the FEM slope stability is presented on Fig. 7 and Fig. 8 for OLE and CLE, respectively.

POLB Pier A

The POLB Pier A development required both cut and fill sections using rock dikes for a 1100-meter long wharf (Leighton & Associates, 1996). Typical fill and cut rock dike sections are shown on Fig. 2 and Fig. 3, respectively. The existing dikes were constructed at least 20 to 50 years before the proposed wharf construction. Petroleum extraction caused approximately 5 meters of subsidence that caused the rock dikes to be constructed. This wharf incorporated a seismic fuse (Fig. 5). The results of conventional limit equilibrium slope stability analysis were not reported. The results of a FEM analysis performed by Dames & Moore (1996) is summarized in Table 3 and was based on the parameters presented in Table 2. A comparison of the results of this and other analysis are also shown on Fig. 6 through Fig. 8.

POLB Pier J

Pier J was designed and constructed in the early 1960s (Dames & Moore, 1961). A series of multi-lift rock dikes (Fig. 2) were slowly placed over soft soils to allow for shear strength gain by consolidation. In the late 1980s, the channel in front of the rock dike was deepened by 3 meters, from elevation -10.7 to -13.7 meters MLLW for a 790-meter-long wharf. Batter piles were planned for lateral resistance for the wharf. Conventional limit equilibrium slope stability analysis and simplified deformation analysis were performed by Harding Lawson Associates (1990A). Those analyses showed that the rock dikes (and wharf) would not perform satisfactorily (pseudostatic and post earthquake factors of safety [FS] less than 1 and deformation greater than 3 meters) because of earthquake loading. A FEM deformation analysis was performed by Dames & Moore (1990) using the computer program DSAGE2.1. The results of the Pier J FEM study developed a seismic fuse and used the pinning effect of the wharf piles in the FEM deformation analysis. The fuse consisted of a pendent wall extending downward from the wharf that failed in bending. A schematic of the fuse is shown on Fig. 5. The FEM analysis indicated acceptable slope and wharf performance.

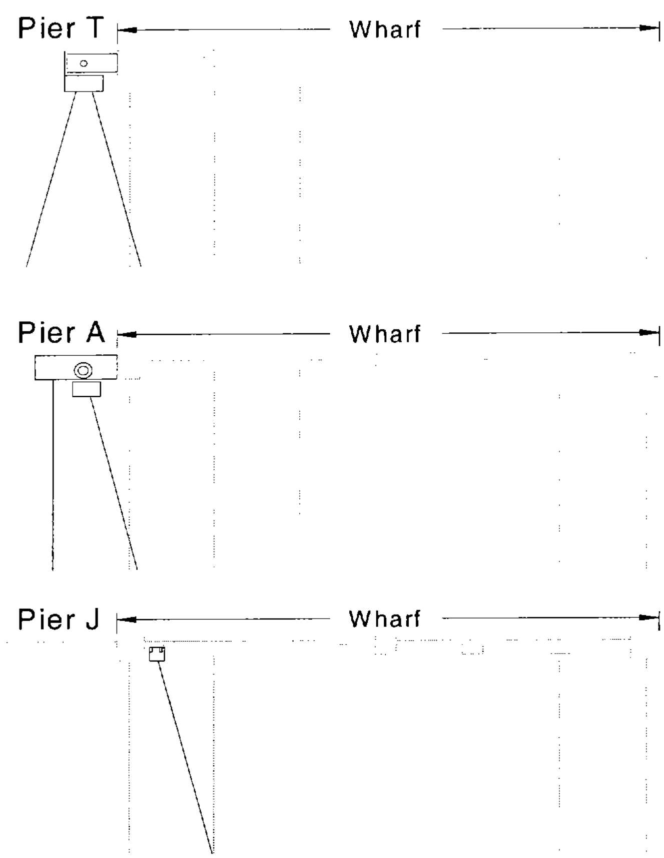

Fig. 5 - POLB Fuse Design

<u>POLA Pier 400</u>

The Pier 400 project recently created approximately 1.4 million square meters in the harbor. New rock dikes (Fig. 2) were used to retain hydraulic fill. The first wharf will be 1600 meters long and be designed for a water depth of -16.8 meters MLLW. Resistance to lateral loads was supplied by vertical piles in bending. A comprehensive investigation was performed by Fugro West, Inc., (1999). FEM analyses were performed by Earth Mechanics, Inc. (1999) and the results are summarized in Table 2 and Table 3. The analyses performed for this project were more comprehensive than performed for the other projects noted herein. The analyses considered various zones of ground improvement behind the rock dikes. The piles are free headed so the analysis did not consider the restraining effects of the wharf structural system. The rock dikes were considerably larger than that used for other projects.

<u>POLA Pier 300</u>

For the POLA Pier 300 project, an all-vertical pile supported wharf approximately 1200 meters long was constructed along an existing two-lift rock dike. The rock dike was constructed in the late 1970s by the US Army Corps of Engineers to retain dredge spoils from the deepening of the POLA from -10.7 to -13.7 meters MLLW. The conventional slope stability analysis was performed by CH2MHill (1994) and is summarized in Table 2 and Table 3. FEM deformation analysis was performed by Earth Mechanics, Inc. (1992) as summarized in Table 3 and outlined in Table 1. A comparison of the simplified FEM deformations is presented in Fig. 7 and Fig. 8.

RESULTS

The FEM analyses used are summarized in Table 1 the results of the analyses are summarized in Table 2 and Table 3. The lateral deformations are plotted in Figures 6 and 7.

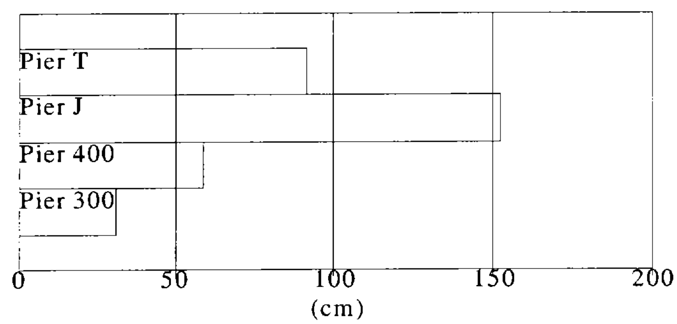

Fig. 6 – Simplified Analysis Deformations without Piles

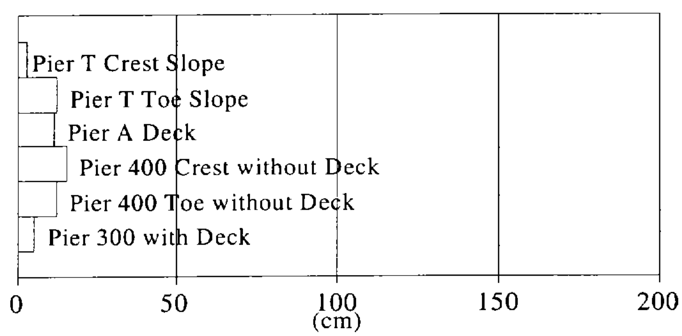

Fig. 7 - OLE Deformations with Piles

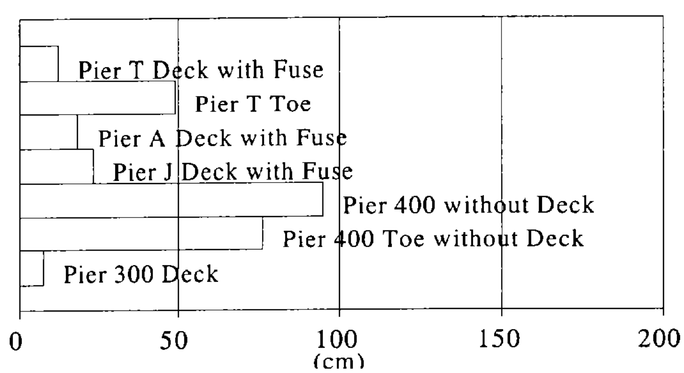

Fig. 8 - CLE Deformations with Piles

Project	Program	Numerical Method/Constitutive Model	Other
Pier T	FLAC-3.3	Finite difference - elasto-plastic, Mohr-Coulomb, parameters obtained from conventional laboratory tests and correlations	Pre-degraded strengths used (liquefaction already occurred at start of analyses)
Pier A	FLAC-3.3	Finite difference - elasto-plastic, Mohr-Coulomb, parameters obtained from conventional laboratory tests and correlations	Loosely coupled, pore pressure generation calculated separately based on simplified procedures and input into the constitutive model
Pier J	DSAGE-2.1	Finite difference - elasto-plastic, Mohr-Coulomb, parameters obtained from conventional laboratory tests and correlations	Loosely coupled, pore pressure generation calculated separately based on simplified procedures and in put into the constitutive model
Pier 400	DYNAFLOW	Finite difference - clays modeled as pressure insensitive material having a Von Mises type yield function Other soils modeled using nested multi-yield elasto-plastic model	Fully coupled, pressure-volume relationship for water with 2 degrees of freedom
Pier 300	LINOS	Finite difference - elasto-plastic, Von-Mises type model for rock dike and non-liquefiable soils. Bounding surface Drucker- Prager for liquefiable soils. Parameters obtained from conventional laboratory tests, correlations, and curve fitting	Loosely coupled, pressure-volume relationship for water with 1 degree of freedom

Table 1 - FEM Computer Program Summary

PROJECT	SLOPE INCLINATION 1:X	SLOPE HEIGHT (m)	ROCK FRICTION ANGLE (degrees)	STATIC FS	PSEUDOSTATIC			POST EQ FS	SIMPLIFIED DEFORMATION (cm)	
					g	FS	YIELD ACCEL.		OLE	CLE
PIER T	1.75	21.3	45 to 50	1.43 to 1.58	0.15	0.86	0.1	1.27 to 1.48	0	49 to 91
PIER A	2	19.8	45	-	-	-	-	-	-	-
PIER J	1.5	18.3	46	1.14 to 1.45	0.1 to 0.2	0.59 to 0.79	-	0.70 to 1.08	-	152
PIER 400	1.75	21.3	45	1.42 to 1.71	-	1.19 to 1.50	0.084 to 0.115	-	2 to 5	34 to 59
PIER 300	1.75	20.4	43	1.5	-	-	0.89	1.3	10	31

Table 2 - Static/Pseudostatic Slope Stability and Deformation Summary

PROJECT	FEM PROGRAM	OLE (g)	CLE (g)	DEFORMATION LATERAL (cm)			
				CREST		TOE	
				OLE	CLE	OLE	CLE
PIER T	FLAC 3.3	0.22	0.53	3	12	12	49
PIER A	FLAC 3.3	0.23	0.53	9 to 11	15 to 18	-	-
PIER J	DSAGE 2.1	0.19	0.38	-	23	-	-
PIER 400	DYNAFLOW	0.28	0.52	15	95	12	76
PIER 300	LINOS	0.24	0.38	5	8	-	-

Table 3 - FEM Deformation Summary

CONCLUSIONS

Based on the results of our review we conclude that the results of the different analyses for the different projects are not directly comparable because the cases studied are different for each. However, some general observations were made.

The estimation of seismic exposure of San Pedro Bay has increased by approximately 20 and 60 percent for the OLE and CLE, respectively, in the last 10 years.

The deeper channel depths and the higher seismic criteria require that the positive effects of piles on the rock slope stability must be considered to demonstrate an acceptable level of seismic deformation caused by the CLE. This situation requires FEM analysis, which was performed for each of the new container terminals.

The FEM analyses estimated deformations for CLE that were within the designer's allowable limits.

Two-dimensional analyses were used to solve a three-dimensional problem.

The difference between the CLE deformation of the free head piles of Pier 400 and the wharf deck of Pier 300 shows the importance of including the combined pile and deck in the analytical model. Without the deck in the Pier 400 analysis, the free pile head deformation were more than 10 times the deformation of the Pier 300 deck even though the Pier 400 rock section is substantially greater.

The Pier 300 (all vertical piles) deck deformation is substantially less than that of Piers T, A, and J (which include batter piles with fuses). This seems counter intuitive and the reasons are unclear but likely has to do with the assumptions that the investigators made in their parameter selection, the contribution of the fused connections, and durations of shaking. The limited information presented in the reports detailing FEM analysis does not allow a reviewer to duplicate the analysis reported. Also, none of the investigators completed a comparison of the two support schemes (batter versus vertical piles).

None of the FEM programs noted herein have been calibrated to a full-scale deformation of a rock slope with an open field of piles.

The combined San Pedro Ports offer a perfect opportunity to compare the performance of the different wharf and dike designs in the same seismic setting. This should be done analytically and seismic instrumentation should be installed to learn from future earthquakes.

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