Comparison of 3D Non-Linear Modeling to Recorded Seismic Response for a Pile Supported Wharf

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Abstract

In engineering practice there is ongoing debate concerning the limitations of 3D structural modeling of wharves and piers for seismic analysis. This paper presents the results of an investigation of the seismic response of Berth 24/25 at the Port of Oakland, California. The primary objectives of this project were to evaluate strong motion data from an instrumentation array at Berth 24/25 and to identify the limitations inherent in capturing the complete dynamic character, including soil structure interaction, of a pier or wharf with a structural model. The numerical model was validated using ground motions recorded during the 1989 Loma Prieta earthquake with a twelve channel array placed on and adjacent to the structure. Through a series of simulations, the effect of variation of selected model parameters has been evaluated by comparison to recorded wharf motions.

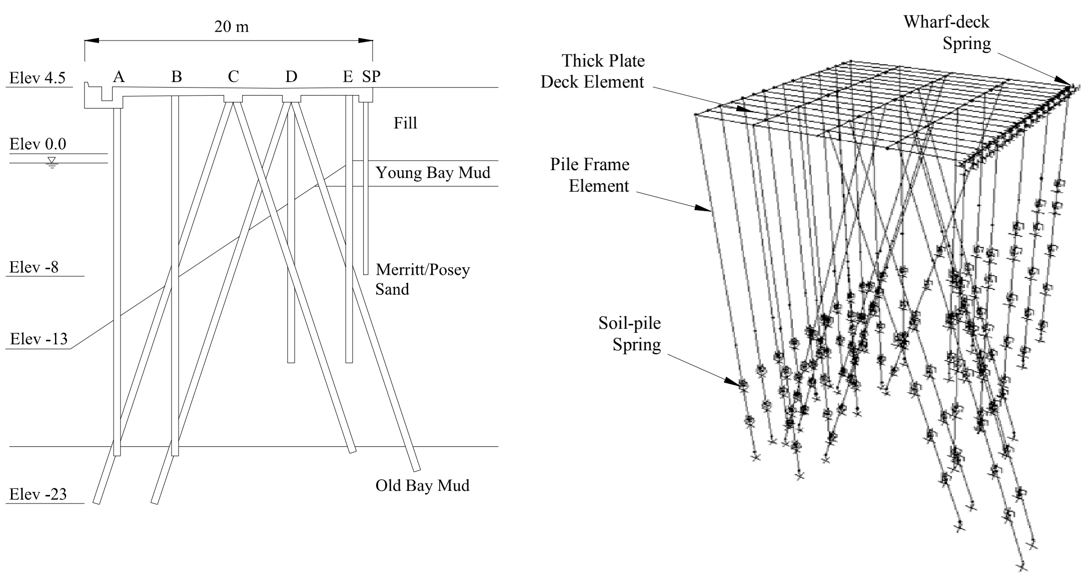
Analyses using design level input motions were performed to evaluate applicability of the full 3D model. The project is expected to serve the professional engineering community by providing guidance in selecting appropriate techniques for seismic analysis and subsequent upgrade of existing port facilities.

Introduction

Construction of Berth 24/25 at the Port of Oakland (Port) was completed in 1979. The facility has overall dimensions of 493 m length by 20 m width. Wharf support is provided by prestressed vertical and batter piles arranged in a pattern that repeats itself every fifteen meters, and a steel sheet pile cut-off wall running the length of the in-shore side of the facility. A cross section of the wharf and typical section of the 3D structural model are provided in Figures 1a and 1b.

On October 17, 1989, Berth 24/25 was subjected to ground motions generated by the Loma Prieta earthquake. Prior to this event the California Seismic Monitoring and Instrumentation Program (CSMIP) had installed a twelve channel array of accelerometers on and adjacent to Berth 24/25. As a result of exposure to strong shaking, this wharf, as well as many other Port facilities, was heavily scrutinized for susceptibility to damage under seismic loading. Previous investigations of the seismic response of Berth 24/25 using 2D non-linear pushover analyses (CH2M HILL and Ben C. Gerwick, 2000) and 3D non-linear modeling techniques (Norris et al., 1991) have been performed. The 2D analyses did not incorporate calibration of model input parameters to strong motion data (SMD) available for Berth 24/25. The 3D non-linear analyses did not report specific soil spring stiffness values resulting from their parametric studies nor did they employ software widely used by the port structural engineering community at large. These factors demonstrate the usefulness and need for a 3D analysis with comparison to recorded seismic behavior and implementation of software that is extensively used in port engineering practice.

The analysis described in this paper was performed using SAP2000 Non-linear version 7.50 (SAP2000) (Computers and Structures, Inc., 2000). Wharf model behavior was primarily governed by values used for springs used to characterize pile-soil and wharf to cut-off wall interaction. Different values for these spring elements were used and the resulting output was compared to recorded SMD. The goal of this comparison is to determine appropriate modeling techniques and values for governing parameters to be used in further 2D and 3D analyses of facilities at the Port.



a) Berth 24/25 cross section **b) SAP2000 repeatable section**

Figure 1 - Wharf schematics

Empirical Evaluation of Strong Motion Data

Figure 2 illustrates the layout of accelerometers installed on and adjacent to Berth 24/25 at the Port. Only the horizontal channels were used for analyses performed in this project. Empirical analysis of recorded SMD for port structures is a useful tool for elucidating structural response of port facilitates and observation of phenomena including torsion and seismic wave passage. Analysis of SMD can also be used to demonstrate shifts in period between comparable backland and structure channels signifying non-linear soil response and soil-foundation-structure-interaction. For Berth 24/25, possible torsion was evaluated by calculating relative displacement-time histories between parallel channels along the wharf. Figure 3 shows the relative displacement in the in-shore/out-shore (i.e. transverse) direction between channels 6 and 9. Given the total wharf length of 493 m, the maximum relative displacement of 4.2 cm between channels 6 and 9 indicates negligible torsion and a low possibility of resulting damage along the wharf.

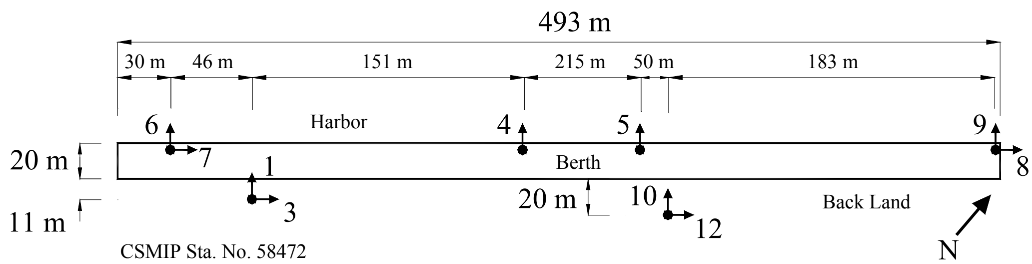


Figure 2 – Berth 24/25 accelerometer layout

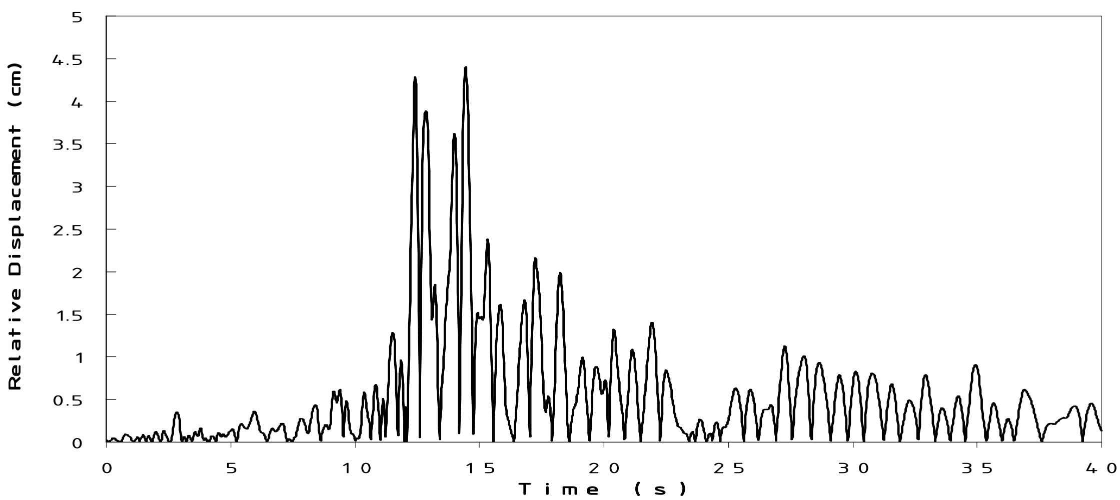


Figure 3 – Relative displacement between channels 6 and 9

The potential for liquefaction of soils below and adjacent to Berth 24/25 is evident from geotechnical data (SPT N-values< 20 blows/30 cm) as reported during subsurface explorations performed for design of the wharf (Port of Oakland, 1979) and from previous investigation of earthquake induced embankment deformations (URS Greiner Woodward Clyde, 2000). Field observations after the Loma Prieta earthquake indicate the occurrence of limited liquefaction (or elevated excess pore pressures not reaching full liquefaction) along Berth 24/25. This is supported by SMD in which absolute displacement-time histories are compared for parallel backland accelerometer channels (Figure 4). At approximately fourteen seconds in the time history, the channel 10 ground motions exhibit a slight shift (i.e. lengthening) of period as compared to that of channel 1. This period shift indicates a change in frequency content of the channel 10 time history which is considered to be an indicator of loss of soil stiffness under significant shaking and therefore a reduced capacity to transmit ground motions (Kramer, 1996). Given the relatively small separation distance between channels 1 and 10 (278 m), it is likely that liquefaction and/or stiffness loss of Northeast backland soils occurred as a result of significant shaking. This assessment of recorded SMD is supported by the pavement settlement and minor crane rail deformations observed by Port personnel after the Loma Prieta event (Serventi, 2003).

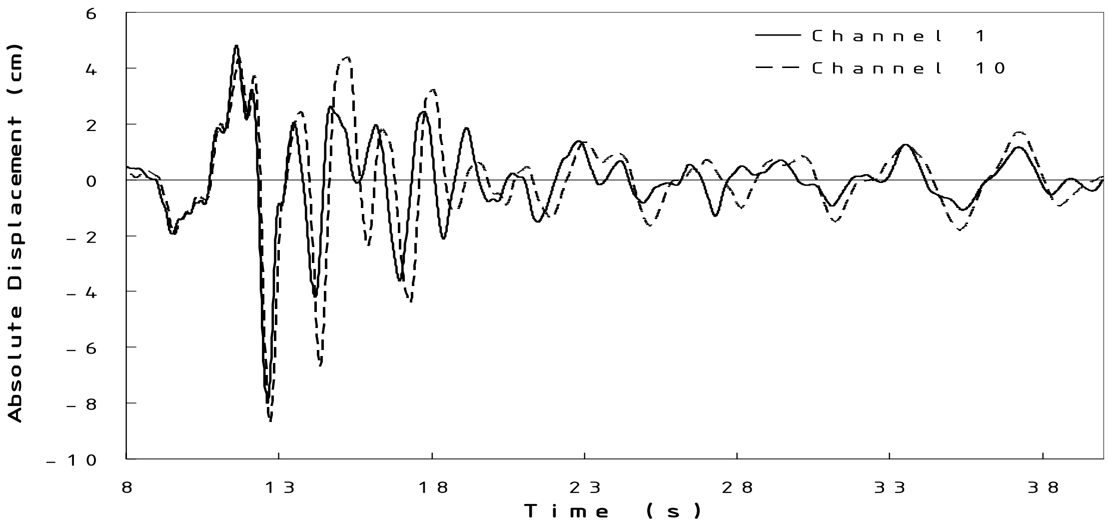


Figure 4 – Comparison of displacement-time histories for Channels 1 and 10

Dynamic Structural Modeling Input Ground Motions

Recorded acceleration-time histories for channels 1 and 3 were deconvoluted to the depth of the lowest pile tip elevation with ProShake (EduPro Civil Systems, Inc. 1999) to generate the input acceleration-time histories for use in the SAP2000 structural model. Channels 1 and 3 were selected as there was no evidence of liquefaction in the southeast wharf backland area as compared to that discussed in the previous section for the area surrounding channels 10 and 11. Table 1 delineates the soil profile used for ProShake analyses including values for plasticity index (PI), average shear wave velocity ($(V_s)_{avg}$), damping ratio (ξ) and unit weight (γ_T) as taken from the WESP report on ground motions (URS Greiner Woodward Clyde, 2000). Non-linear soil stiffness and damping properties were modeled using the well known relationships for sand (Seed and Idriss, 1970 and Seed, et al., 1986) and clay (Vucetic and Dobry, 1991). Given the soft Bay Mud beneath Berth 24/25, ground surface motions were, as expected, deamplified when deconvoluted to a depth consistent with the lowest pile tip elevation of -23 m from mean low water. ProShake input peak ground accelerations (PGAs) for channels 1

Soil Description	Elevation	Depth	PI	$(V_s)_{avg}$	ξ	γ_T
	m	*m*	*n/a*	*m/s*	*%*	*kN/m³*
Artificial Fill	3	2	n/a	200	1.5	19
	2	3		200		
	1	4		200		
Gray Clay (Young Bay Mud)	0	5	50	145	1.5	15
Silty Sand (Merritt/Posey)	-2	6	n/a	260	1.5	21
	-3	8		260		
	-5	9		260		
	-6	11		260		
	-8	12		260		
	-9	14		370		
	-11	15		370		
	-12	17		370		
	-14	18		370		
	-15	20		370		
	-17	21		370		
	-19	24		370		
Blue Clay (Old Bay Mud)	-21	24	50	240	1.5	18
	-21	26				
	-23	27				
Halfspace	n/a			335	1.5	18

Table 1 – Profile for Dynamic Soil Response Analysis

and 3 are 0.28g and 0.20g, respectively. The resulting ProShake output (i.e. SAP2000 input) PGAs for channels 1 and 3 are 0.17g and 0.12g, respectively.

3D Structural Model

Figure 1b gives a 3D view of a portion of the Berth 24/25 3D SAP2000 model. As-built plans (Port of Oakland, 1979) from the original construction of Berth 24/25 were used to gather information on pile and deck geometry and material properties. Concrete compressive strengths (f'_c) were assumed to be approximately 20% greater than the specified 28 day values, consistent with the 12 year age of the structure at the time of the Loma Prieta event. Masses were assigned to the pile nodes based on tributary pile lengths, assigned to deck nodes based on tributary deck area, and calculated with a concrete unit weight of 24 kN/m^3. The same unit weight was used to define the SAP2000 material properties that determine the dead load of frame and thick plate elements. A description of significant parameters used to model SAP2000 elements follows.

Support piles. Each 46 cm square prestressed vertical and batter pile was modeled as a series of frame elements. The lengths of the frame elements for piles in each pile row were determined by unsupported pile length (i.e. region between pile deck and intersection of the design mud line) and selected depths for assignment of soil springs. Soil springs were assigned at depth intervals ranging from every 1.5 to 3 m below the design mud line to represent soil-pile interaction of each soil layer. Concrete material properties for support piles are an f'_c of 49,766 kPa and a modulus of elasticity (E) of 3.34 x10^7 kPa.

Soil springs. Soil-pile interaction was modeled using a series of elastic-plastic NLLink elements. These elements allow for an elastic spring stiffness and limiting failure load beyond which the spring has very nearly zero stiffness. The NLLink elements were created with zero length and connected to fixed and free nodes sharing the same coordinates. Spring stiffnesses were calculated for each pile row at approximate depth intervals ranging from 1.5 to 3m along the pile, resulting in fourteen different NLLink elements. The uppermost pile nodes, representing the pile mud-line interface, were not assigned NLLink elements as soil in this region is not likely to provide much resistance. Secant stiffness (K_{sec}) values were derived from P-y curves formulated using the American Petroleum Institute (API) method (American Petroleum Institute, 1987) by calculating the slope of a straight line drawn from the origin to intersect the upper bound of the curve. This upper bound represents the ultimate lateral soil load as calculated using the API method. K_{sec} values ranged from 5.7 to 224 N/cm^2 for clay and 4.3 to 15.7 kN/cm^2 for sand. These K_{sec} values represent stiffness per unit length along the pile and were multiplied by tributary pile lengths to determine elastic spring stiffness values for each of the fourteen types of SAP2000 NLLink elements. The yield load assigned to SAP2000 NLLink elements determines the point at which the spring behaves plastically and was taken as the ultimate lateral soil load.

Wharf deck. The wharf deck was modeled as an array of thick plate elements 46 cm in depth. Concrete material properties used to model the wharf deck are an f'_c of 33,950 kPa and an E of 2.76 x10^7 kPa.

Pile caps. Piles caps in rows A, C, and D were modeled using frame elements geometrically consistent with the cap portion below the wharf deck. Material properties were the same as those used for the wharf deck thick plate elements.

Steel sheet pile cut-off wall. The steel sheet pile cut-off wall was modeled as a series of linear springs attached to each deck node on the backland side of the wharf. Stiffness values for the in-shore/out-shore and vertical directions were calculated as the flexural and axial elastic stiffness per unit length of wall, respectively. Flexural and axial stiffness were determined to be 25 and 5800 kN/cm, respectively. Given the rigid connection of the individual sheet piles, flexural stiffness in the longitudinal direction along the wharf was assumed to be infinite.

The modeling techniques described above were used to create a base 3D wharf section that repeats itself every fifteen meters. This repeatable section was then replicated to create the entire model. Static dead load and acceleration-time history analyses were then performed. Separate acceleration-time history analyses were conducted using records from the Loma Prieta and 1995 Kobe events each of which was comprised of two orthogonal components. For each event analysis the orthogonal components were applied simultaneously. Loma Prieta records for channels 1 and 3 were deconvolved to the bottom pile tip elevation using ProShake and have PGAs of 0.17g and 0.12g, respectively. Recorded ground motions from the 1995 Kobe earthquake, Amagasaki station, were used as they had been selected as the 10% probability of exceedance in 50 year design event in previous seismic analyses at the Port (URS Greiner Woodward Clyde, 2000). PGAs for both components of the Kobe event were 0.58g.

Discussion of SAP2000 Results

Recorded and modeled displacement-time histories for the Loma Prieta event were compared to evaluate model performance. Maximum pile forces from both the Loma Prieta and Kobe events were examined to estimate the number of piles damaged and/or failed for each level of shaking.

Loma Prieta Displacement-time histories. Figures 5 and 6 compare absolute displacement-time histories for channels 6 and 7 and their corresponding model node (No. 393). All other wharf model nodes showed similarly well matched results to the corresponding recorded data. A number of simulations were run with variation up to ± 20% about the mean K_{sec} values used for the NLLink elements. Results of these analyses showed negligible difference for absolute displacement-time histories. The lack of effect from variation of soil spring stiffness is a result of very large K_{sec} values attributable to Berth 24/25 soil conditions. Analyses were also run by removing the NLLink and linear spring elements from the model to demonstrate their impact on model accuracy. As can be seen in Figure 7 relevant and accurate displacement time histories are not achieved in the longitudinal direction unless the NLLink and linear spring elements are incorporated into the model. However, wharf behavior in the transverse direction is not as dependent on the use of spring elements. Wharf stiffness in the transverse direction is predominantly controlled by batter piles. Therefore, displacement-time histories in this direction showed negligible difference compared to results from simulations with both spring types in place.

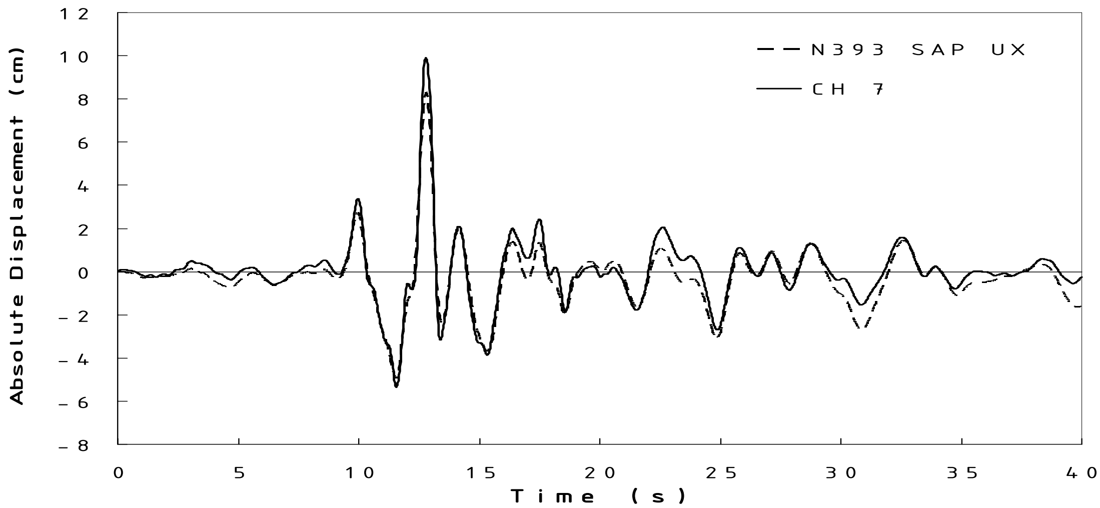


Figure 5 – Comparison of Model results to recorded Channel 7 data

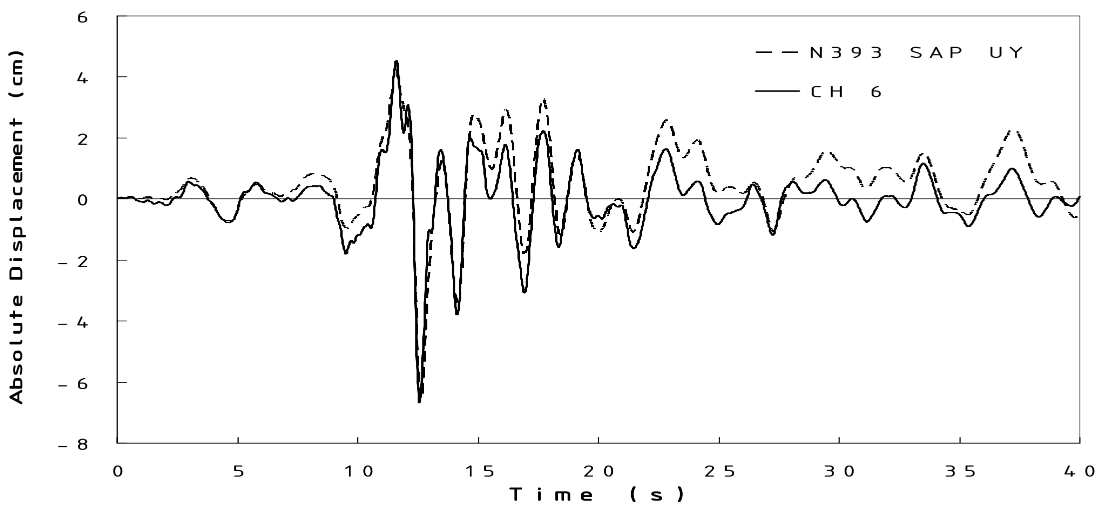


Figure 6 – Comparison of Model results to recorded Channel 6 data

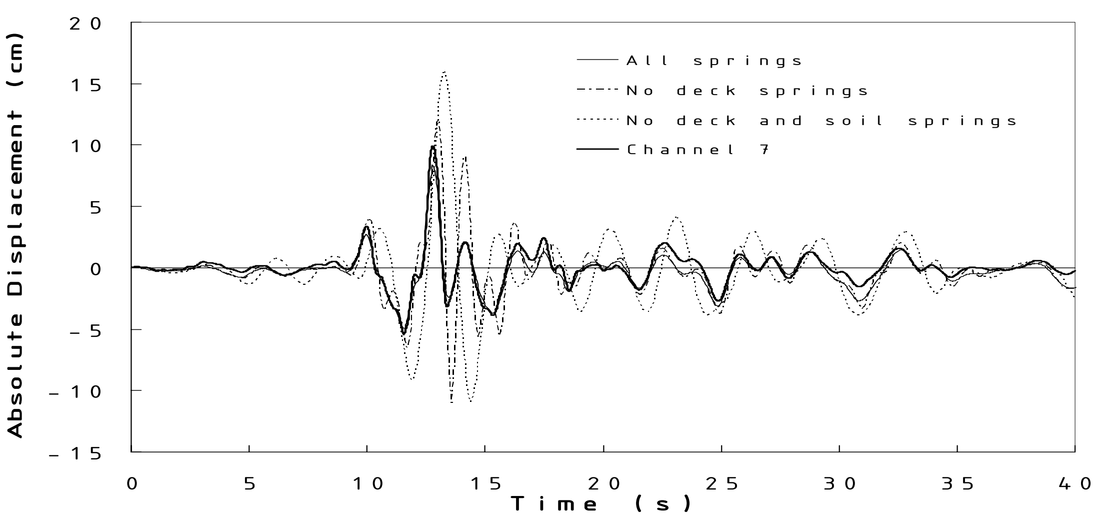


Figure 7 – Comparison of Model results with and without soil and deck springs

Loma Prieta and Kobe pile forces. Three levels of seismic structural wharf performance have been defined by the Port of Oakland. The Level I limit state allows for minor repairable damage under ground motions having a probability of exceedance of 50% in 50 years. The Level II limit state allows for controlled repairable damage without interruption of normal operations under ground motions having a probability of exceedance of 10% in 50 years. The third, or post Level II, limit state allows for unrepairable damage, but prohibits collapse under ground motions having a probability of exceedance greater than those defined for the Level II limit state (CH2M HILL and Ben C. Gerwick, 2000). Loma Prieta ground motions are those associated with the Level I event while Kobe ground motions have been used to define Level II shaking in past analyses at the Port.

Nominal axial, shear and moment capacity and the flexural cracking moment were calculated for the support piles. Calculations were made as described in the Building Code requirements for Structural Concrete (318-02) and Commentary (318R-02) (ACI Committee 318, 2002). Under the simulated Loma Prieta ground motions, none of the modeled piles developed forces exceeding nominal strength or cracking capacities. Time history analyses using the Kobe ground motions showed that 51% of the vertical piles and 15% of the batter piles exceeded flexural cracking loads, but none exceeded any type of nominal strength. The majority of modeled piles exceeding flexural cracking loads are located in pile rows B and E (vertical) and pile row C (outboard batter).

Conclusions

Empirical analysis of SMD gathered from Berth 24/25 at the Port of Oakland has been used to illustrate dynamic structural response. Scrutiny of absolute displacement-time histories indicates that a low amount of wharf torsion was caused by ground motions generated during the Loma Prieta earthquake It is therefore unlikely that below grade components of the wharf were damaged due to torsional motion during the Loma Prieta event. This conclusion is supported by post-event inspections that showed little or no damage to exposed wharf support piles (Serventi, 2003). Other modes of seismic damage to waterfront facilities are due to soil liquefaction and permanent ground deformation. Analysis of absolute and relative displacement-time histories illustrate that liquefaction and/or increased pore pressure is likely to have occurred in the Northeast portion of backland soils. A lengthening of period observable for the channel 10 absolute displacement plot indicates moderate liquefaction of soils in this area and is supported by clear evidence of ground settlement observed in the Northeast wharf backland after the Loma Prieta earthquake (Serventi, 2003). The nature and extent of information gathered concerning wharf performance for Berth 24/25 at the Port of Oakland based on empirical analysis of SMD makes a strong case for the increased deployment of instrumentation at waterfront facilities in seismically active regions of the United States.

Accuracy of 3D model output time histories for Berth 24/25 were found to be strongly dependent on proper inclusion of springs representing pile-soil and wharf-sheet pile wall interaction. For the low level of shaking that occurred at Berth 24/25 during the Loma Prieta event and for ground motions associated with the Level II design event, the non-linear soil springs never exceeded the elastic stiffness range. However, K_{sec} values for the elastic portion of the non-linear springs yielded well matched modeling results. The version of SAP2000 used for this project is not well suited to model post yield behavior of frame elements. Therefore, estimates of response are made using the initial uncracked stiffness. Application of design level ground motions to this type of model showed pile moments exceeding the Level II limit state as defined by the Port. Thus, the analysis under Level II ground motions was only useful in gathering a rough estimate of the amount of damage to be expected during a similar event. It should also be noted that 3D structural models do not incorporate pile loads caused by slope deformations, which have been shown to cause subsurface failure in piles (McCullough et al., 2001). A primary benefit of comparison of 3D model output to recorded data then, is in establishing the initial elastic soil and sheet pile spring values for use in subsequent pushover analyses. Future work should include examination of model behavior once soils springs have been pushed to the non-linear range, and 2D non-linear pushover analyses for comparison to those previously conducted as part of the WESP.

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