Evaluating the Seismic Capacity of a Newly Designed Wharf at the Port of Oakland

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Abstract

This paper presents the findings of a seismic capacity study of the recently designed Berth 59 wharf at the Port of Oakland (the Port), California. The study was conducted to evaluate whether or not the wharf would collapse in a 2500 year San Francisco Bay Area seismic event. The wharf had been designed in accordance with the Port's seismic design requirements and criteria explained below.

The Port's wharf design criteria require designing for strain limits for the forces resulting from three seismic levels. The three levels are events having 50, 20, and 10 percent probability of exceedance in 50 years. The return periods for these events are approximately 75 years (Level I), 225 years (Level II), and 500 years (Level III), respectively. The wharves are expected to suffer no or little damage and

remain fully functional during and immediately after a Level I event. The wharves are not expected to collapse during a Level III event.

The design criteria are derived from an acceptable risk approach such as is shown in the American Society of Civil Engineers (ASCE), Technical Council on Lifeline Earthquake Engineering (TCLEE) Monograph No. 12, Seismic Guidelines for Ports. "Acceptable Damage" increases as the probability of risk decreases. The original design considered three return periods, 75, 225, and 500 years. The "Acceptable Damage" increased as the return period increased. The criteria were consistent with the acceptable risk approach. This evaluation looked at a much higher return period and allowed greater damage.

The seismic capacity study was accomplished by performing nonlinear pushover analysis and a soil structure analysis to evaluate the effect of slope displacements. In the pushover analysis, the magnitude of static lateral force is increased until the wharf is no longer stable. Ground motions for various earthquake levels were developed using probabilistic seismic hazards analyses (PSHA). The probabilistic analyses were conducted for three return periods (500, 1000, and 2500 years), which are equal or higher than the Level III event.

In addition to the inertial forces and displacements calculated using pushover analysis, the performance of the Berth 59 piling was also evaluated for slope movement (kinematic soil-structure interaction (SSI) effects). The soil deformations were computed using FLAC (nonlinear finite difference program) as well as pseudostatic slope stability analysis combined with Newmark-type sliding block analysis.

It was determined that the wharf at Berth 59 is not expected to collapse due to events with return periods of 2500 years.

Introduction

A performance-based design approach is commonly used for seismic design of modern wharves along the California coast.

The Port of Oakland had a three-level seismic design approach at the time of the Berth 59 design. The first and third levels are events having 50 and 10 percent probability of exceedance in 50 years. The return periods for these events are approximately 75 (Level I) and 500 (Level III) years, respectively. The wharves are expected to suffer little or no damage and remain fully functional during and immediately after a 75 year event. The wharves are expected to suffer damage, but not collapse, during a 500 year event. This study checked the affects of 1000 year and 2500 year events. The wharf structure at Berth 59 was selected for this study, and is presented in Figure 1. The subsurface conditions are presented in Figure 2. As indicated, prior to pile installation and construction of the wharf deck, the liquefiable sand fill and soft young bay mud (YBM) layers have been removed and replaced by a rock dike, thus shoreline instability is not a major consideration at this site.

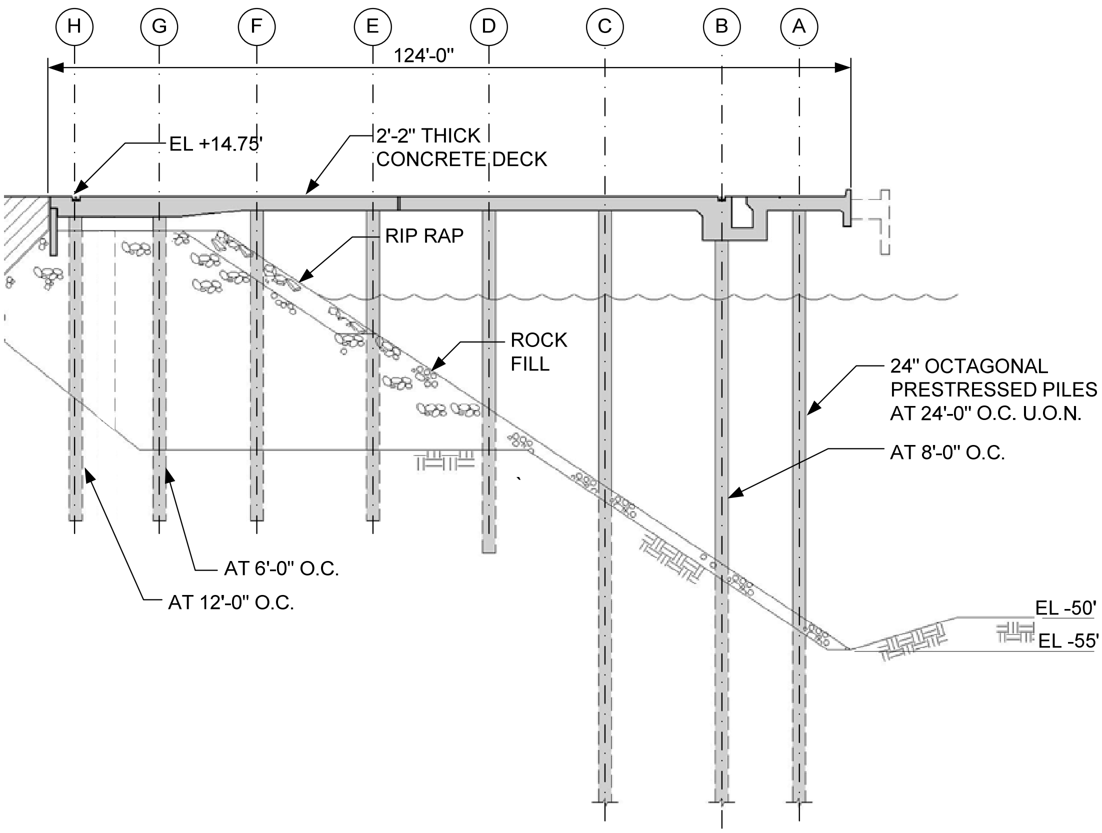

Figure 1. Cross Section of Berth 59 Wharf at Port of Oakland

Layer	Material Type	Top Elevation (ft)	Total Unit Weight (pcf)	Friction Angle (degree)	Shear Strength (psf)	k (pci)	Vs (fps)	G/Gmax - Damping
1	New Fill	15	125	36	0	225	550/450	Fill
2	Mixed Fill	13	125	36	0	225	550/450	
3	Clayey Sand Fill	10	125	30	0	225	550/450	
4	Clayey Sand Fill (below gwt)	6	130	30	0	125	550/450	
5	Bay Mud	-2	100	0	see note	30	350 + 4d	YBM
6	Loose Clayey Sand	-15	130	0	250	30	1000 + 5d	SAF
7	Medium Dense Clayey Sand	-21	135	0	400	30	1000 + 5d	
8	Very Dense Sand	-25	130	45	0	125	1000 + 5d	
9	Dense Clayey Sand	-45	140	38	0	125	1000 + 5d	
10	Very Dense Sand	-60	135	45	0	125	1000 + 5d	
11	Dense Clayey Sand	-65	140	38	0	125	1000 + 5d	
12	OBM	-73	115	0	2500	1000	800 + d	OBM
13	Rock Dike (above gwt)	10	115	42	0	225	600 + 10d	Rock Fill
14	Rock Dike (below gwt)	6	120	42	0	125	600 + 10d	
Note: 354 psf at Elevation -2, then increases at 9.4 psf/ft								

d = depth below surface, OBM= old bay mud
SAF=San Antonio Formation, YBM=young bay mud

Figure 2. Subsurface Conditions at Berth 59

The results of PSHA are presented in terms of five percent damped elastic design response spectra corresponding with events having return periods of 500, 1000, and 2500 years in Figure 3. Near source and directivity effects were accounted for in the probabilistic seismic hazards analyses.

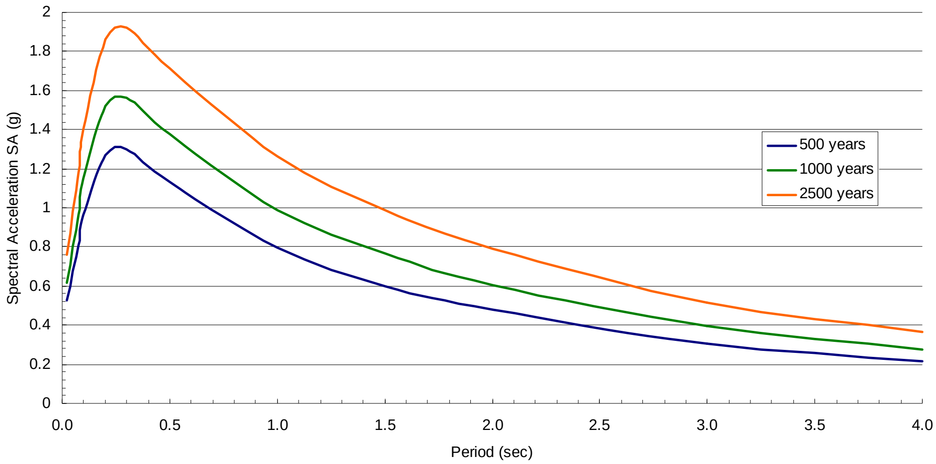

Figure 3. Probabilistic Ground Motion

The 24-inch-octagonal piles used for construction of the wharf are prestressed and have smooth W20 wire spiral reinforcing with a pitch of less than three inches in high moment regions to provide ductility. The pile strain limits criteria used for the events with a 500 year and 2500 year return period are shown below:

Material Strains, in/in	Design 500 year	Study 2500 year
Concrete Pile		
Unconfined	0.004	0.006
Confined-Top	0.02	0.022
Confined-in-Ground	0.008	0.02
Steel		
Mild	0.05	0.15
Prestressing	0.01	0.05

Pile moment curvature relationships were calculated using the computer program XTRACT and followed the recommendations of Mander and Preistley.

Soil resistance against lateral pile loads (p-y curves) were obtained through lateral pile analysis using the computer program LPILE.

Pushover Analysis

The computer program SAP 2000 was used to perform the pushover analysis. The structural model represents a typical three-bay transverse section of the wharf. The piles were modeled using fully nonlinear beam elements capable of accounting for P-delta effects.

The pushover analysis results are presented in Figure 4. As indicated, the lateral load-deflection curve for each pile row is calculated separately. This is reasonable since the wharf deck is much stiffer than the piles. The loads on each pile are independent of the loads on adjacent piles. The load-deflection curve for the wharf structure was then obtained by combining the load-deflection curve for all pile rows. The wharf's load-displacement curve is compared with the seismic demand in Figure 5 for a damping ratio of 15%.

The load-deflection curve indicates the following:

1. Collapse is expected at a lateral wharf deck displacement of approximately 60 inches.

2. A lateral wharf deck displacement of approximately 16 inches is expected for the 2500 year event and 15% damping.

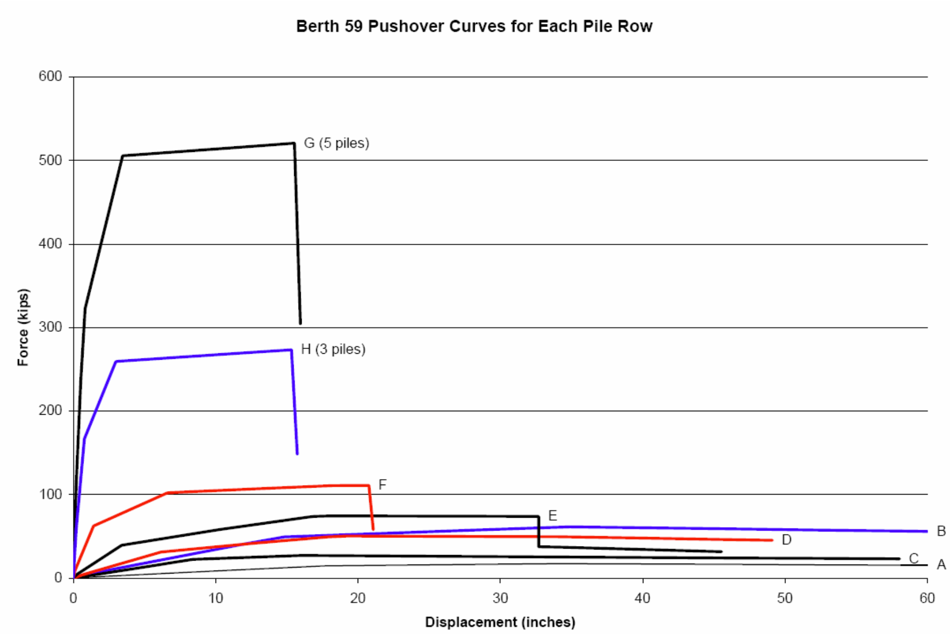

Figure 4. Pushover Analysis Results

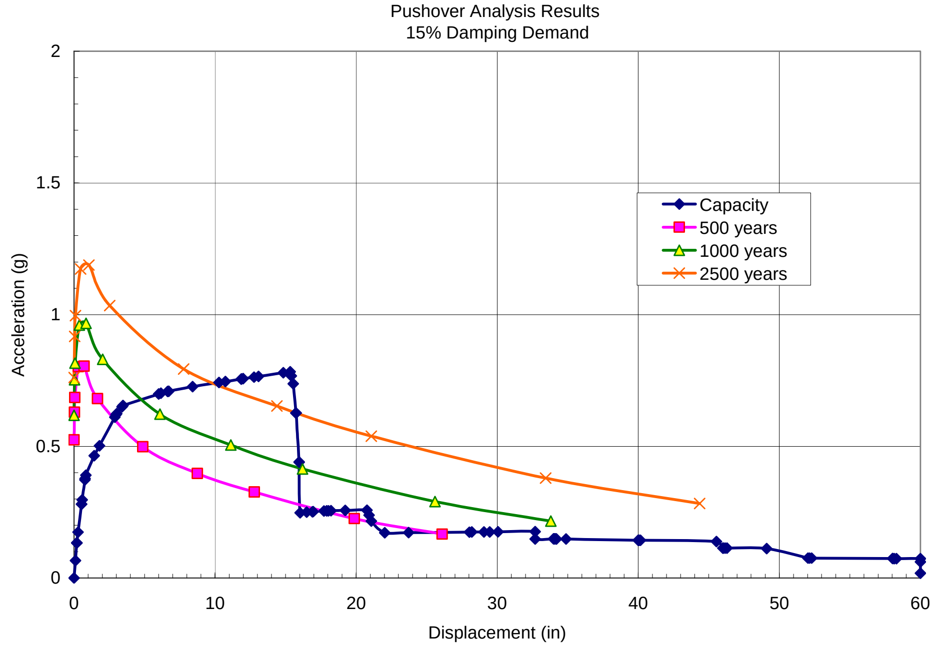

Figure 5. Demand Versus Capacity

It should be noted that the capacity curve accelerations would be 20 to 30 percent larger, and the 2500 year-15% damping displacement would be lower (11 inches as opposed to 16) if the probable material strengths were used instead of design values.

For an explanation of the load-deflection curve, we note that the first plastic hinges occur at the connection of the pile with the concrete deck (for pile rows F, G, and H) at a spectral acceleration of about 0.4g. At a displacement of 15 inches and ground acceleration of 0.75g, plastic hinges are formed: 1) at the connection of the piles in pile rows F, G, and H at the wharf deck; and 2) at the point of maximum moment below the ground surface.

To evaluate the hysteretic damping values, various deflection limits were selected and the hysteretic loops calculated. The results indicate that for large lateral displacements (10 to 15 inches), the hysteretic damping is much larger than the typical damping used in dynamic analysis for these types of structures. The large damping values are reasonable considering the large lateral displacements and highly nonlinear response of the structure at 10 to 15 inches of lateral displacement. However, the authors decided to limit the damping to 15% to be on the conservative side, and to acknowledge the approximate nature of the method used for calculation of damping ratios.

The result of pushover analysis indicates that the structural collapse due to inertial SSI effects is not expected to occur for seismic return periods of 2500 years.

Kinematic SSI Effects

To examine the kinematic SSI effects, lateral permanent displacement of the embankment was calculated for the 2500 year event using two methods: 1) a Newmark-type, decoupled deformation analysis; and 2) a fully coupled, nonlinear finite difference FLAC analysis.

The Newmark-type deformation analysis was performed by 1) selecting three recorded ground motions, spectrally matching them to the 2500 year target response spectra, and obtaining design ground acceleration time histories, 2) performing pseudostatic slope stability analyses to calculate the yield acceleration and to identify the geometry of the critical failure surface, 3) calculating the average induced ground acceleration applied to the sliding mass using the computer code QUAD4M, and 4) double integrating the difference between the yield acceleration and the induced ground acceleration time histories obtained from step three.

The pseudostatic and two-dimensional QUAD4M analyses were performed for various circular and wedge type sliding surfaces. The results are presented in the table below:

Slope Stability		
	Circular Failure	Wedge Failure
Static Factor of Safety	1.8~1.9	2.9~3.0
Yield Acceleration (g)	0.18	0.28

Decoupled Displacements using QUAD4M/newmark (in)		
Ground Motions	Circular Failure	Wedge Failure
500 year	1.5	2.5
1000 year	2.5	7.5
2500 year	6.0	12.0

Fully coupled, nonlinear, finite difference SSI analyses were performed using a FLAC computer analysis. Input time histories were obtained from a site response analysis using the computer program SHAKE. The piles and the concrete deck were included in the FLAC model. Therefore, the soil-pile-structure interaction effects were accounted for in the FLAC analysis.

The FLAC analysis results which indicate the absolute (displacement with respect to the fixed boundary at depth) displacement of the deck and various pile rows are presented in Figure 6 for the Kobe ground motion. The displacements obtained from FLAC SSI analyses were imposed on all pile rows and the induced bending moments due to the kinematic SSI effects were calculated. The results of the kinematic analysis indicate that only piles in Row C had bending moments larger than the plastic moment capacity below the ground surface (Figure 6). Further examination of the kinematic analysis results indicate that damage to the cover concrete below ground surface in Row C piles could occur; however, the strains in the core concrete and the reinforcing steel were within the allowable strain limits. It was determined that the Row C piles would be able to carry the axial loads and that kinematic SSI effects during an event with a return period of 2500 years would not cause the structure to collapse.

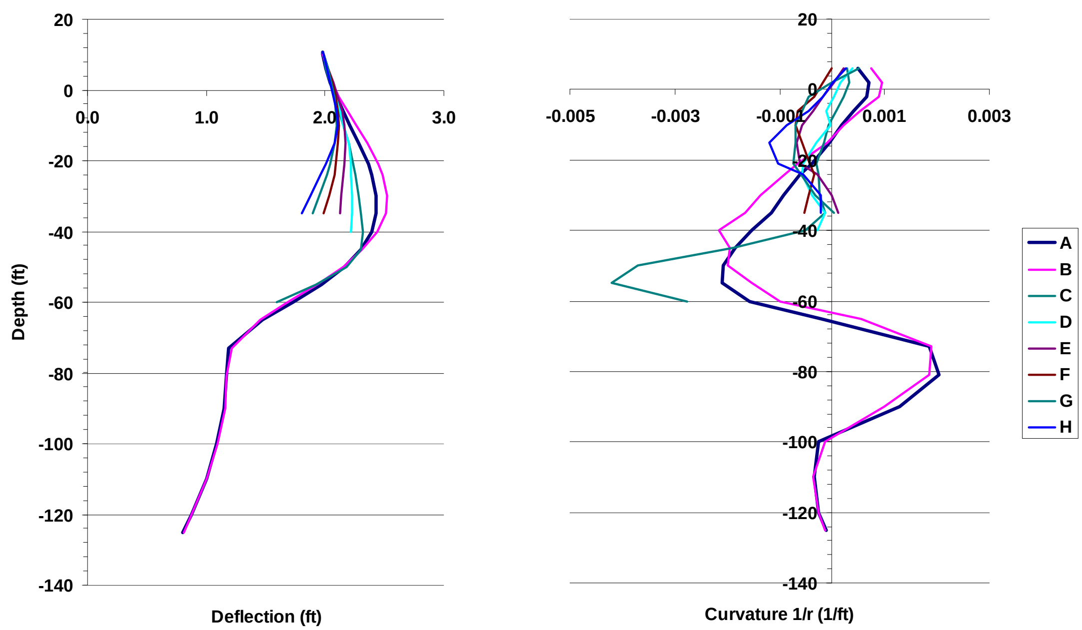

Figure 6. Pile Curvature Due to Kinematic SSI Effects

We note that the point of application of the maximum bending moments from inertia and kinematic interaction effects are well separated both in location along the pile and at time of occurrence. This validates the basic assumptions that these effects could be evaluated separately and that they impact piles at different times during the seismic event and at different locations along the pile.

Conclusions

The seismic response of the wharf structure at Port of Oakland's Berth 59 was calculated through inertial interaction analysis using a pushover analysis method and kinematic interaction analysis using equivalent linear decoupled analysis (Newmark-type) and nonlinear, fully coupled FLAC analysis.

The results of this analysis are conservative and indicate that the structure will not collapse for a seismic event with a return period of 2500 years.

The results confirmed the notion that recently designed and constructed wharves at the Port (e.g. the wharf at Berth 59) will not collapse due to severe ground motion with extremely low probability of occurrence (2500 year recurrence interval).

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