

## SEISMIC DESIGN OF AN IRREGULAR WHARF UTILIZING ASCE-61

A. Bergman<sup>(1)</sup>, R. Toro Pavez<sup>(2)</sup>, S. Story<sup>(3)</sup>

<sup>(1)</sup> Associate, P.E., KPFF Consulting Engineers, adam.bergman@kpff.com

<sup>(2)</sup> Associate, P.E., S.E., KPFF Consulting Engineers, rodrigo.toro@kpff.com

<sup>(3)</sup> Design Engineer, P.E., KPFF Consulting Engineers, sean.story@kpff.com

#### Abstract

ASCE/COPRI 61-14 Seismic Design of Piers and Wharves (ASCE 61), (ASCE (2014)), published in 2014, offers a uniform methodology for multi-level performance-based seismic design that relies heavily upon the Substitute Structure methodology. Prior to this document, marine structural engineers have drawn from a collection of reference documents when performing seismic design.

The publication of this document coincided with the design of the Pier 4 Reconfiguration Project at the Port of Tacoma, Tacom, Washington (USA), and was adopted as the criteria document for seismic design. The project is a pilot project for the Port of Tacoma for use of this standard.

A specific challenge that was present in the design was the extreme stiffness irregularity along the length of the wharf. The southern portion of the wharf is designed as a typical wharf structure, with shorter exposed pile lengths inboard, and longer pile lengths outboard. Due to the existing slope configuration, the northern portion of the wharf structure is located in an area where all of the piles have an exposed length above the mudline of approximately 68 feet. This configuration creates slender piles at the northern end of the wharf. To control deflections during vessel mooring/berthing, the northern portion of wharf utilizes batter piles in the transverse direction.

To provide a more uniform seismic response, lead rubber bearings (LRBs) were incorporated into the design of the portion of the structure utilizing batter piles to reduce response differentials along the length of the wharf. A comparative nonlinear, direct integration, time-history analysis was completed in select transverse locations to evaluate crane-wharf interaction and to develop displacement amplification factors for use in a nonlinear, direct integration, time-history analysis of a simplified spine model for the entire pier. This paper discusses how ASCE 61-14 was used in the design and how the methodology was adapted for use in this complex structure.

Keywords: ASCE/COPRI 61-14, Lead rubber bearings, Wharf design, Time-History Analysis, Hysteresis



## 1. Introduction

The Port of Tacoma intends to reconfigure Pier 4 in alignment with the adjacent Pier 3, which has recently undergone an upgrade to accommodate 100-foot gage container cranes. The new Pier 4 structure will be a 1,725 foot long concrete structure supported on concrete piles. A portion of the existing Pier 4 structure will remain in place to provide access at the northern end of the new pier to the upland portion of the terminal. Isolation joints will be provided between the existing portion of Pier 4 to remain, the new Pier 4 structure, and Pier 3 to decouple their seismic responses.

Due to the existing Pier 4 slope configuration, the northern portion of the new Pier 4 structure will be located in an area where all of the piles have an exposed length above the mudline of approximately 68 feet. This configuration creates a low transverse stiffness at the northern end of the new pier. To control deflections during vessel mooring and berthing and provide a ductile seismic response, the northern 225 feet of Pier 4 will utilize LRBs beneath the transverse pile caps. These bearings will be supported on batter pile subcaps with batter piles battered in the transverse direction. The southern 1,500 feet of the pier will be constructed entirely on vertical piles similar to other traditional wharf designs. See Fig. 1 for a plan layout of the new Pier 4 structure. In addition, Fig. 2 and Fig. 3 show cross section views at the plumb pile and LRB supported portions of the pier, respectively.



Fig. 1 – Pier 4 Plan

The seismic lateral force resisting system of the southern plumb pile portion of the wharf will be provided by the formation of plastic hinges in the vertical octagonal concrete piles. The seismic lateral force resisting system of the northern portion of the wharf will be provided by the LRBs and batter piles. The batter piles have been designed to remain elastic for combined, bi-directional, axial and moment loading while allowing the plumb piles to yield in bending at the caps and below ground if required. The concrete deck of the structure acts as a nearly rigid diaphragm to transmit the seismic loads to the lateral force resisting systems.





Fig. 2 – Typical Section at South End with Plumb Piles



Fig. 3 - Typical Section at North End with LRBs



## 2. Seismic Design Goals

Per ASCE 61 a displacement-based, nonlinear, direct integration, time-history design approach was utilized for this design. The goal of the displacement-based design approach is to limit structure damage for operational and contingency level events and provide life safety protection for a design level event. The performance requirements are based on three levels of ground motion and seismic performance as required by ASCE 61, and described below.

#### 2.1 Operating Level Earthquake (OLE)

The OLE event represents ground motions with a 50 percent chance of exceedance in 50 years (72-year return period). The target response of the structure for the OLE event is that the structure should experience only minimal damage, should exhibit a near-elastic structural response with minor residual deformation, and should result in no loss of serviceability of the structure.

#### 2.2 Contingency Level Earthquake (CLE)

The CLE event represents ground motions with a 10 percent chance of exceedance in 50 years (475-year return period). The target response of the structure for the CLE event is that the structure should experience only controlled and repairable damage, should respond in a controlled and ductile manner, and should experience limited inelastic deformations at locations where repair is possible within several months after the event.

#### 2.3 Design Earthquake (DE)

The DE event is currently the same as that required by ASCE 7-05 and represents ground motions with 2/3 of the hazard associated with a 2 percent chance of exceedance in 50 years (2/3 of the hazard of the 2,475-year return period). The target response of the structure for the DE event is that the structure should continue to support gravity loads (including crane loads), should allow egress of the structure post-event, and should protect life safety.

In order to achieve these performance targets ASCE 61 sets concrete and steel allowable strain limits for each of these event levels. A site specific response spectrum and a suite of seven ground motions for each of the events above was developed by the project Geotechnical Engineer and utilized in the analysis. The ground motions were utilized in the nonlinear time-history analysis of the full wharf and of portions of the wharf to determine crane-wharf interaction. The response spectra were utilized to run bent stiffness and damping checks utilizing the Substitute Structure approach to lock down local wharf behavior with and without crane influence.

A pushover failure analysis was also performed on select bents of the portion of Pier 4 to remain and the adjacent Pier 3 in order to conservatively size seismic joints between the new portion of Pier 4 and the existing portion of Pier 4 and Pier 3. The pushover failure analysis allowed existing batter piles and short plumb piles to fail successively, allowing demand to redistribute until it was deemed that complete vertical load carrying capacity was lost. Joints were sized utilizing these calculated displacements along with the averaged displacements of the new Pier 4 structure drawn from the time-history analysis. This was done in order to minimize the risk of pounding without having to perform a full time-history analysis on the adjacent structures.

#### 3. New Pier 4 Structure Displacement Based Design

#### 3.1 Substitute Structure Method

For structures that are anticipated to undergo inelastic deformations during seismic events, a two-dimensional, nonlinear, static demand analysis is utilized to establish a lower-bound limit on the displacement demand in accordance with ASCE 61. For simple, two dimensional structures, this analysis method is also known as the "substitute structure method." For a simple structure, such as an individual bent, the analysis can be performed with a single nonlinear static pushover analysis by determining the nonlinear static pushover curve, and iterating



effective secant stiffness values to arrive at the appropriate displacement demand and level of damping. This is achieved by estimating the structural damping and adjusting the response spectrum appropriately. However, this process is not easily modified for use with a three dimensionally irregular structure.

The substitute structure analysis was utilized at each bent as a check of stiffness and damping levels and to bound the displacement demands in the transverse direction. The demands were developed utilizing a nonlinear time history analysis and are described below. Per ASCE 61, the transverse displacement demands utilized were always at least 2/3 of the transverse values obtained utilizing the substitute structure analysis.

The substitute structure method used to bound and check the time history analysis values involves an iterative process that uses an effective secant stiffness at the response displacement demand, assuming a single degree of freedom system, and thus a single mode, for each event. The secant stiffness is then used to calculate the period of the structure, the approximated damping level, the associated spectral acceleration, and the design displacement from the appropriate acceleration displacement response spectrum (ADRS). This process is then repeated until the design displacement is within 3% of the previous iteration and utilizes the appropriately damped spectrum.

#### 3.2 Wharf and Crane Mass Considerations

The mass considered for the dynamic analysis included the self-weight of the structure and 10% of the design uniform live load. Building off of the work performed by Jaradat and Cheng (2013), a separate study utilizing a time history analysis on individual bents with cranes was performed to determine the level of participation provided by the cranes in the transverse direction. The crane to wharf interaction was modeled utilizing friction pendulum elements so that crane tipping could be considered. Damping parameters were calibrated to reflect the hysteretic behavior of soil and structural elements as described later in this document. It was determined that the full mass of the container cranes does not participate, but that the response is amplified by up to 40% in some cases. This is higher than traditionally assumed and is due to the relatively long period of the flexible structure and the relatively short period of the large cranes that are proposed for use at this facility. Displacement amplification factors were determined utilizing this analysis for different zones on the wharf depending on the location and response of the bents in question. These were applied to the transverse displacement demands that were determined from either the time history or substitute structure methodologies. Amplification factors were not utilized in the longitudinal direction because the crane period is greater than twice the period of the wharf in that direction. ASCE 61 Section 8.5.2 further describes when to include container crane mass.

#### 3.3 Lead Rubber Bearings Used to Reduce Stiffness Irregularities

As mentioned above, the primary use of the Substitute Structure method for this structure was to bound the results from the time-history analysis at each bent and to develop an understanding of the response of each bent to the localized mass demand. This analysis was also used as a way to "tune" the parameters of the LRBs such that the northern bent response could be made to approximate the response of the southern bents. The LRBs' lateral force-deformation response can be accurately characterized by their initial stiffness, yield strength, and post-yield stiffness. Through the use of approximate equations (Christopoulos and Filiatrault (2006)) and manufacturer-supplied properties, bearing dimensions were selected such that the lateral pushover response of the bents with LRBs reasonably matched the pushover curve used for the southern portion of the pier (see Fig. 4)





## **Bent System Pushover Comparisons**

Fig. 4 - Bent System Pushover Comparisons

As seen in the bent system pushover comparisons of the different bent configurations in Fig. 4, the initial and post-yield stiffness of the non-isolated batter pile bent with ductile sleeved dowel connections (Harn (2004)) was much larger than that of the typical southern bent supported on plumb piles. The lateral displacement capacity of the non-isolated batter pile bent was also significantly less than the typical southern bent supported on plumb piles, indicative of a sudden, brittle failure mechanism at its ultimate displacement capacity. Comparing the pushover curves of the batter pile system isolated from the deck structure with LRBs to the typical plumb pile system, it is evident that the lateral response of these two configurations is much more similar, effectively smoothing out the lateral response of the structure along its full length. This bent specific analysis was used to pre-tune the LRBs before performing the full nonlinear, direct integration, time-history analysis in order to reduce the number of design iterations that would require running the full suite of time-histories.



## 4. Nonlinear Dynamic Time-History Analysis

Though the substitute structure method on individual bents provides a good check and allows the solution to be bounded and tuned, the irregular nature of the structure requires a more sophisticated analysis. The seismic analysis for this structure was performed using a plan-oriented, two dimensional (2D), nonlinear, time-history analysis on a representative spine model (see Fig. 5). Bent and longitudinal foundation stiffness and damping was modeled with transverse and longitudinal nonlinear springs utilizing parameters based on pushover curves developed for the structure at various bents and for various longitudinally loaded pile groups. The individual bent models utilized to develop the springs for the 2D spine model analysis contained effective section properties for the concrete elements and included pivot hysteretic models (Dowell et al. (1998)) for the pile hinges. Nonlinear soil springs were modeled to capture the soil-pile interaction below the mudline including utilizing multi-linear plastic links to approximate soil hysteretic behavior using a gapping Takeda hysteretic model. Separate models were constructed for each soil profile, and strength degradation due to liquefaction was also considered. Upper and lower bound soil springs were modeled and up-slope and down-slope effects were included by utilizing bi-directional pushover models that incorporate hysteretic elements.



Fig. 5 – Spine Model Schematic at North End

The ultimate goal was to accurately model the transverse and longitudinal stiffness, hysteretic behavior, and mass distribution along the full length of the wharf in order to capture an accurate response to the timehistory analysis. The process of developing the spine model started with developing an accurate twodimensional, nonlinear, static pushover analysis curve for each bent spring and each longitudinal pile group spring in the spine model. These curves also effectively set the basis for determining the displacement capacity of individual elements. In the tranverse bent and longitudinal group models, nonlinear soil springs were modeled to capture the pile interaction below the mudline and the effective, cracked structural properties of the piles were used where applicable. The expected material properties were used in hinge development per ASCE 61. The following steps were followed to determine the displacement capacity of the structure and create the backbone for the multi-linear plastic links used in the spine model. Hysteretic damping behavior was not considered in the capacity/backbone models so that moment axial interaction hinges could be utilized thus accounting for the hinge response under changing axial load. Identical models with moment only hinges were developed to determine the hysteretic behavior and will be described below. The following steps were utilized in determining the backbone for the multi-linear plastic links and capacity curves.



- 1. Step one was to determine the nonlinear soil springs along the depth of the pile using LPILE (ENSOFT, INC., 2013) and input from the geotechnical engineers. Upper and lower bound soil stiffness was used, and strength degradation due to pile group effects and liquefaction was also considered.
- 2. Step two was to determine the moment curvature curves for the pile hinge regions per ASCE 61 Section 6.6.2 at various axial load levels. The material properties and stress strain curves used were per Section 6.5. The material strain limits for the various events were per ASCE 61 Tables 3.1, 3.2 and 3.3, for the OLE, CLE, and DE, respectively. Calculation of the plastic hinge length was per Section 6.6.4. X-tract (TRC, Inc.) was utilized to develop the hinge properties which were suitably bilinearized per ASCE 61. These curves allowed the development of the axial-yield moment interaction curve and post yield behavior required to develop the moment axial interaction hinges in SAP 2000 (CSI, 2014).
- 3. Step three was to create a two-dimensional, nonlinear, static pushover model using the effective section properties, hinge properties, and soil springs along the length of the pile. Effective (cracked) section properties for elastic components were modeled using the modifiers shown in ASCE 61, Table 5.2. Effective section properties for inelastic components were determined from axial-moment-curvature analysis.
- 4. Step four was to run the nonlinear pushover analysis using the strain limits specified to determine the displacement capacity for each event and the overall pushover backbone curve form.

Once the backbone of the springs was developed utilizing the steps above, the hysteretic behavior was determined. The same models utilized to develop the backbone of the spring curve were utilized with moment only hinges developed at average axial load levels for each hinge in question so that pivot hysteretic models could be utilized in SAP2000. In addition Takeda hysteretic damping was added to the soil springs and the following steps were utilized to develop overall hysteretic behavior for each spring and perform the time-history analysis on the spine model.

- 1. Step one was to determine the overall hysteretic behavior of each representative spring type by cycling the structure back and forth with stacked pushover analysis cases. The models utilized moment only concrete hinges with pivot damping hysteretic behavior, and soil springs with Takeda gapping hysteretic behavior. The multi-linear plastic links ultimately used in the spine model included the basic shape of the curve and utilized pivot hysteretic damping with pivot damping parameters selected to match the overall hysteretic behavior observed in the cycled pushover models. In essence, the pivot damping parameters for the multi-linear plastic links were modified such that the hysteretic behavior of the link is curve matched to the overall hysteretic behavior observed in the cycled pushover models of the whole bent. Fig. 6 and Fig. 7 demonstrate this curve matching.
- 2. Step two was to construct a two-dimensional spine model of the pier for nonlinear time-history analysis. The spine model accurately represented the center of mass, which varies along the length of the pier. In the longitudinal direction, the springs were modeled at groups of piles, at the center of rigidity of each group to accurately capture torsional effects. The longitudinal stiffness of the sheet pile wall in the southern portion of Pier 4 was also included.
- 3. Step three was to run the time-history analysis on the spine models and extract individual principal pile displacements from the analysis. The displacements were then amplified based on the crane amplification factors and compared with single pile and bent displacement capacity values as determined above. The displacement demands were enveloped at each demand level based on the upper and lower bound soil stiffness assumptions and the average of the seven ground motions for each event level.



# **North End Typical Hysteresis**



Fig. 6 - Typical Hysteresis at North End

Fig. 6 and Fig. 7 show a typical push-pull analyses for the full bent model and the multi-linear plastic link model. The figures pertain to the North and South ends of the pier respectively. As mentioned above, pivot damping was utilized for the multi-linear plasic links and the parameters were adjusted to curve fit the link hysteresis to the hysteresis observed in the full bent models. Matching the parameters of the Northern bents with the LRBs proved to be fairly simple, while matching the pinched hysteretic behavior of the Southern bents was slightly more challenging. However, in all cases including at the South portion, it was possible to achieve a link hysteresis that matched closely with the hysteretic behavior observed in the push-pull of the individual bents or pile groups. Ultimately, these links could then be utilized in the spine model and represented "backbone" stiffness and hysteretic behavior observed in the more complex bent and group models.





# **South End Typical Hysteresis**

Fig. 7 – Typical Hysteresis at South End

### 5. Seismic Displacement Demand and Capacity Analysis – Lead Rubber Bearings

Lead rubber bearings were utilized at the Northern end of the structure to provide adequate elastic stiffness for mooring, berthing and wind loading (strength loads) while providing good post yield performance for seismic demands. The bearings were modeled in the individual bent models as rubber isolator link elements based on specified isolator properties that transfer no tension loading. The bearings have been detailed with loose bolt connections to prevent them from going into tension. Bearing properties were developed based on potential manufacturer recommendations and have been specified with testing requirements suitable to achieve a reasonable match between actual and modeled properties.

Due to the extreme level of displacement capacity in the bearings, the batter piles and batter pile caps have been designed for 125% of the calculated displacement demand rather than for the capacity of the bearings with an overstrength factor. The batter piles and caps have been designed to remain elastic up to this demand level. Significant reserve capacity exists in the plumb piles as the bearings allow the piles to remain almost elastic to the calculated displacement demand.



## 6. Conclusions

The new Pier 4 structure has been designed and analyzed in accordance with ASCE 61 and has utilized the displacement-based design approach outlined in that standard. Due to the unique irregular geometry of the structure and the use of LRBs and batter piles at the north end, additional displacement analysis was required in the form of a nonlinear, direct-integration time-history analysis.

Kinematic analysis was also performed on the new Pier 4 structure in accordance with Section 4.7 of ASCE 61. This analysis captured the effects of permanent lateral ground deformations on the foundation elements. Section 4.7 states that simultaneous loading of the inertial forces and kinematic forces should be considered in design. In order to do this, two-dimensional, finite difference, dynamic soil structure interaction analysis of the deck, pile, and soil was performed at appropriate bents using PLAXIS. A description of this analysis is beyond the scope of this paper, but the reader can review De la Torre et al. (2016). For a complete description of the project as a whole the reader can reference Kuebler and Thornsly (2016).

## 7. References

The following list contains some of the references that were used when performing this analysis.

- [1] ASCE American Society of Civil Engineers (2014): Seismic Design of Piers and Wharves, Standard ASCE/COPRI 61-14.
- [2] Jaradat, O., Lai, Cheng, Elsadek, A. (2013). Crane-wharf Interaction Nonlinear Time History Analysis for Pier E Wharf at the Port of Long Beach, Ports 2013, ASCE 2013.
- [3] Christopoulos, Constantin C., Filiatrault, Andre. (2006): Principles of Passive Supplemental Damping and Seismic Isolation, IUSS Press 2006.
- [4] Harn, R..(2004). Displacement Design of Marine Structures on Batter Piles, 13th World Conference on Earthquake Engineering, Vancouver B.C.
- [5] Dowell, Robert K., Seible, Frieder, Wilson, Edward L. (September-October 1998). Pivot Hysteresis Model for Reinforced Concrete Members, ACI Structural Journal.
- [6] ENSOFT, INC., LPILE v2013, 2013.
- [7] TRC Bridge Design Software, X-tract.
- [8] Computers and Structures, Inc. (CSI), SAP 2000v17, 2014.
- [9] De la Torre, Chris, Lindquist, Doug D., Horvitz, Garry, Bergman, Adam. (2016): Geotechnical Components of a Seismic Soil-Structure Interaction Model at the Port of Tacoma, Ports 2016, ASCE 2016.
- [10] Kuebler, Scott, Thornsly, Trevor. (2016) Big Ship Ready in Tacoma The Reconfiguration of Pier 4, Ports 2016, ASCE 2016.