DISPLACEMENT-BASED SEISMIC DESIGN OF A LARGE NAVAL PIER

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ABSTRACT

This paper discusses the displacement-based seismic design of the newest naval pier at Naval Station Bremerton in Bremerton, Washington. It is 150 feet wide by 1310 feet long and is capable of simultaneously berthing two aircraft carriers. Displacement-based design is the state-of-the-art in seismic design and its basic premise is to determine what displacement will occur during the design earthquake and detail the structure to accommodate its displacement. This is a significant change from most current codes, which are force based. The project enjoyed a 50 percent reduction in piling by using the displacement-based when compared to the force-based method. Soil-structure interaction, push-over analysis, material overstrengths, and site-specific response spectra are used to determine the displacement response of the pier under Levels 1 and 2 earthquakes, the "displacement demand." A push-over analysis is then performed to the displacement demand and the expected capacity of individual members and connections are evaluated and compared to the displacement demand on each member and connection.

Keywords: Seismic Design, Precast Concrete Piling, Displacement Based Design, Innovation, State of the Art, Waterfront Structure, Soil Structure Interaction, Precast Design

INTRODUCTION

Pier Delta was designed and built using the design/build method of procurement. A photo of the completed pier is shown in Figure 1. The new Pier Delta replaced the existing pier, which was 100 feet wide and 1,200 feet long. The new pier consists of a 108-feet-wide center section constructed with a reinforced concrete topping slab (used to embed conduit), supported on precast-prestressed concrete haunched deck panels spanning 30 feet to cast-in-place concrete pile caps supported on (467) 24-inch (in) solid prestressed concrete piles and (164) 36-in steel pipe piles (631 total piles). Utilities are routed in 24.6-feet-wide by 26.9-feet-long precast concrete utility tunnels on both sides of the pier, each weighing approximately 210 kips. The precast elements also contain an integral trench drain

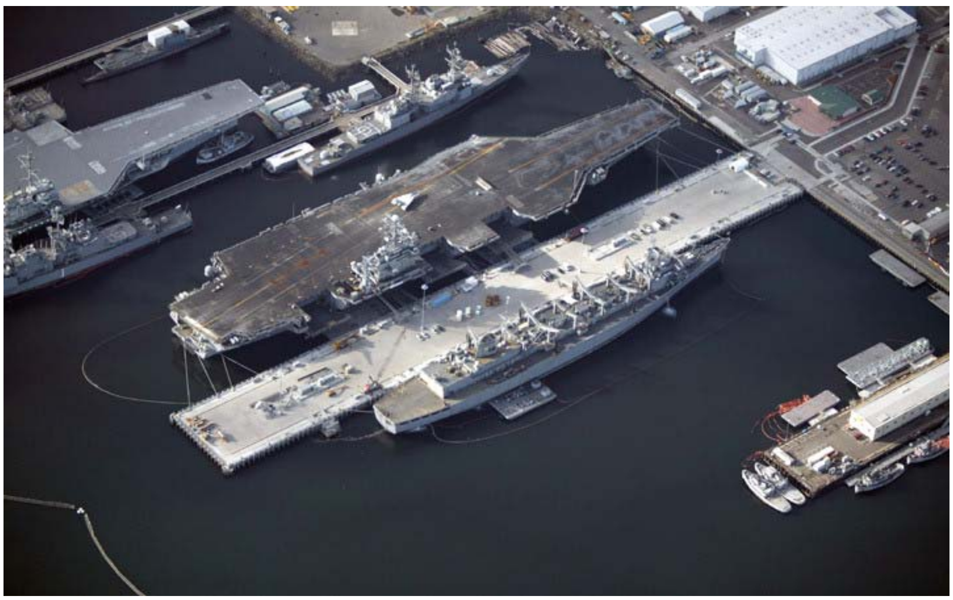

Figure 1: Completed Pier Delta

that provides drainage and stormwater retention, completely eliminating the need for storm drain piping on the pier. In addition to the pier, the project included a fleet recreation facility, substantial upland utility improvements, and (2) 15-kilovolt (kV) substations on the pier. The pier is of heavy-duty construction, capable of supporting a uniform live load of 800-pounds-per-square-foot (psf) and 140-ton mobile cranes with 234-kip outrigger loads operating anywhere on the pier. Mooring hardware consists of 100- and 200-ton bollards at 60-feet on center with cleats between. The total lateral mooring load was 5,080-kips. A pier cross section is shown in Figure 2. Photos showing the typical elements described above are shown in Figures 3 and 4.

INTRODUCTION TO DISPLACEMENT DESIGN

Most current seismic codes use a force-based design method as follows. A base shear (force) is determined and applied to the structure in accordance with the code. That force (static and/or dynamic) is reduced from the actual earthquake force by a ductility factor “R”, for that type of structure. The “R” factor is a measure of how well a structural system is expected to perform in an earthquake. The higher the “R” value the greater the assumed ductility and consequently the better the performance in an earthquake. Typically the various codes have applied “R” factors to various structural types based on testing and actual performance in earthquakes. However, piers do not fit in neatly with the structural systems used in building codes and so criteria specific to piers has been developed, such as the U.S. Navy Technical Report TR-2069, J.M. Ferrito, 1997. One of the drawbacks to force-based design is that the engineer only analyzes the undamaged, elastic structure and consequently does not have a complete understanding of the inelastic behavior that occurs in an earthquake. Although this approach has proven to be effective in preventing collapse, it has not prevented significant damage and the resulting economic loss to the owner from occurring. Displacement-based design involves determining the expected displacement of the structure during the design earthquake and then designing the structural elements for that displacement. The main benefit of the displacement-based design method is the ability to better predict and control where inelastic behavior will occur in a structure, resulting in less uncertainty about how a structure will behave in an earthquake. In most cases, the method will result in a more economical structure.

DESIGN CODE

The design of Pier Delta was performed in accordance with “Seismic Criteria for California Marine Oil Terminals, Technical Report TR-2103-SHR,” John Ferrito, et al, 1999 (TR-2103). This technical report represented the state-of-the-art in seismic design in waterfront construction in 2002. It expands on Technical Report TR-2069 developed by the U.S. Navy to provide design criteria for waterfront construction. The California State Lands Commission, Marine Facilities Division, has now codified this technical report into “Marine Oil Terminal Engineering and Maintenance Standards,” California State Lands Commission. It was adopted into law by the State of California in 2005 as Chapter 31F. All marine oil terminals within the State of California are governed by this document. The United States Navy’s Military Handbook 1025/1, 1987, for the design of Piers and Wharves has been superseded by Unified Facilities Criteria UFC 4-152-01, 2005, for the design of Piers and Wharves. UFC 4-152-01 requires using displacement based design methods for significant structures on Navy facilities.

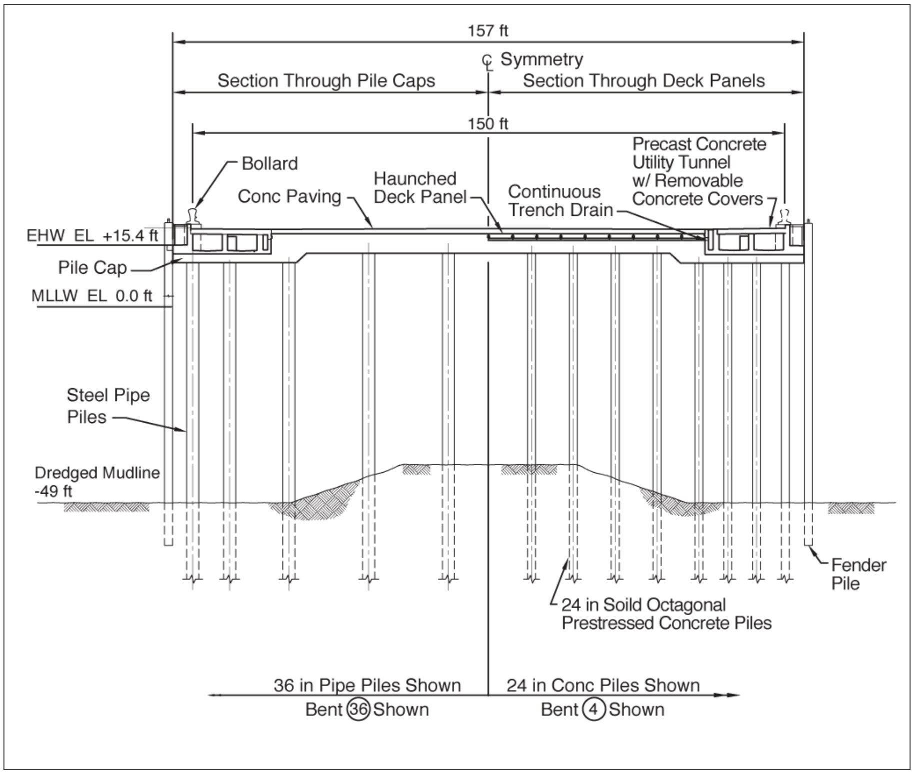

Figure 2: Cross Section of Pier Delta

Figure 3: View of Deck Panels, Pile Caps, and Piles under Construction

Figure 4: View of Precast Concrete Utility Tunnel Being Erected

TR-2103 requires designing for Level 1 and 2 earthquakes. The Level 1 earthquake has a 50 percent probability of exceedance in 50 years (72-year return period) and is considered a serviceability event. The structure should resist the Level 1 earthquake without sustaining damage. The Level 2 earthquake has a 10 percent probability of exceedance in 50 years (475-year return period) and is considered a damage-control event. The structure should resist the Level 2 earthquake without collapse but with repairable damage.

DESIGN CRITERIA

Pier Delta was designed to criteria established by the U.S. Navy in the request for proposal (RFP). Included in the RFP was geotechnical data, such as soil parameters, minimum pile capacities, and Levels 1 and 2 site-specific seismic response spectra. The Level 2 spectrum is shown in tripartite format in Figure 5 with a damping of 5 percent. The tripartite logarithmic plot uses one curve to plot the displacement, velocity, and acceleration relative to the period. The plot is a simplified representation intended to envelope the actual motion indicated by the jagged lines. For short periods, the spectral acceleration is equal to the peak ground acceleration (pga) or 0.31g. The acceleration is amplified as the period increases, reaching a peak of 1.0g between a period of 0.3 and 0.6 second. The spectral velocity remains constant and acceleration decreases between periods of 0.6 and 2 seconds. The spectral displacement is taken as constant at 11.8 inches for periods beyond 2 seconds for this response spectra. This is commonly referred to as a displacement cap; and beyond this point, the structure is flexible enough that it is effectively base isolated. In other words the structure essentially remains still while the ground moves beneath it. The displacement between the structure and the ground is equal to the maximum ground motion. Because Pier D has a period of 4 seconds, it is effectively base isolated. Recognition of the displacement cap in the design criteria was key to the success of the Pier D project. The cap helped produce a simple, yet elegant plumb pile system that used no additional piles to resist seismic loads beyond what was required for gravity (dead plus live) loads. This resulted in a savings of approximately 690 piles when compared to a batter pile system studied during preliminary design. The response spectrum shown in Figure 5 is in metric as it was provided in the RFP. The english values for the constant velocity and displacement are shown in parenthesis after the metric values.

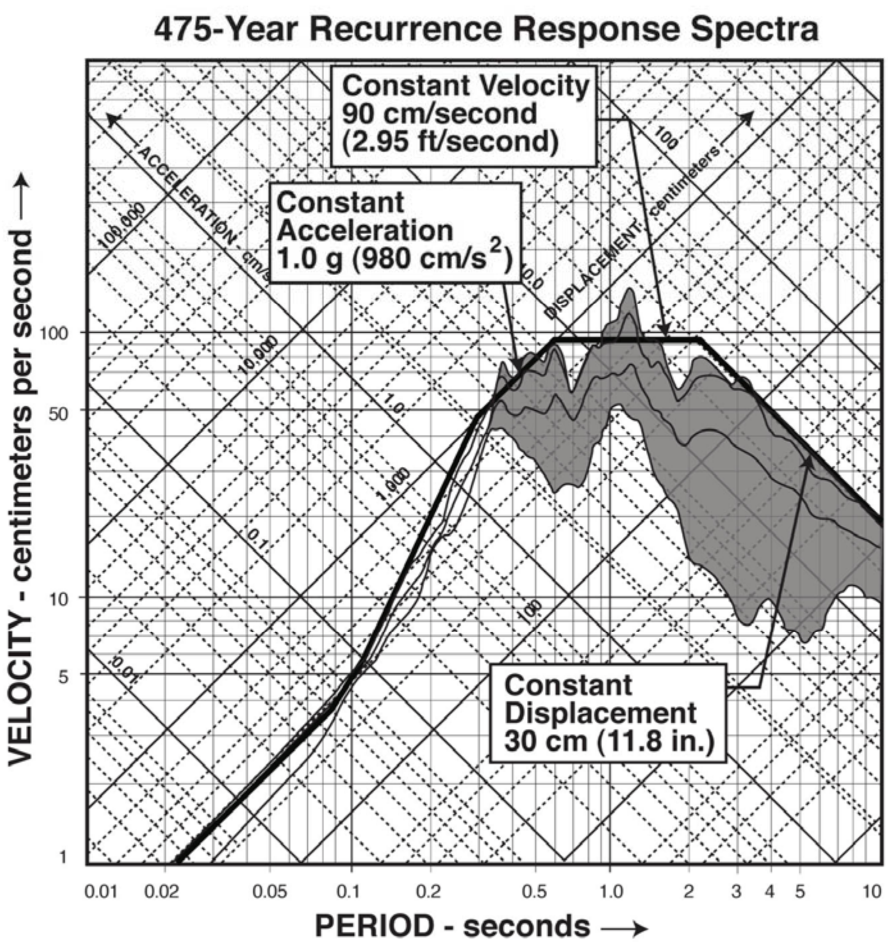

Figure 5: Response Spectra for Pier D (pga = 0.31g)

DESIGN APPROACH

In concept, displacement-based design is relatively simple. Determine what displacement will occur during the design earthquake and design and detail the structure to accommodate that displacement. The difficulty comes in determining the expected displacement and detailing the structure for that displacement. The displacement cap in the site-specific response spectrum for Pier D made the design simpler because the global response of the structure was obvious when examining the spectrum.

Other lateral loads included in the design were very substantial mooring loads (5,080 kips) distributed approximately uniformly along the pier length from the two carriers. The seismic and mooring load cases required designing a structure with the ductility desired for seismic loads and the stiffness and strength desired for mooring loads. The final solution used 24-in concrete piles for Bents 1 through 5, 36-in steel pipe piles for Bents 6 through 12, 24-in concrete piles for Bents 13 through 35, and 36-in pipe piles for Bents 36 through 46. This pile layout provided a structure that was flexible enough for seismic design and with plenty of stiffness and strength to resist the mooring loads.

DISPLACEMENT DEMAND

Although determining the overall displacement demand was simplified by the displacement cap, a detailed analysis was required to accurately determine the displacements at the landside interface and along the pier to design the seismic joints for the utilities and the deck elements. This analysis included the effects of soil/structure interaction that was complicated by the range in mudline elevation and geotechnical properties throughout the length of the pier.

The project site was dredged prior to construction of the previous Pier Delta in 1946. A mound of granular material was placed under the pier after dredging and prior to driving the piles to consolidate and densify the remaining native material. This 1946 mound remains under the replacement Pier Delta as shown in Figure 2. The geotechnical report divided the pier into 11 different geotechnical segments to account for the variation in soil parameters along the pier. These segments can be seen in the longitudinal cross-section of the pier in Figure 6.

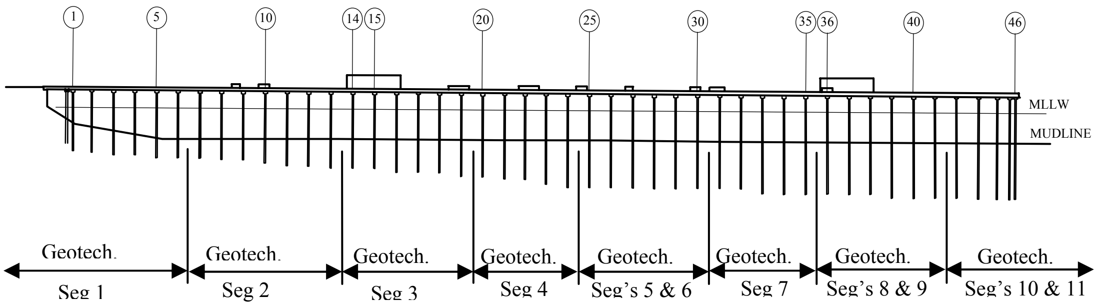

Figure 6: Geotechnical Segments for Design

The software "L-Pile" was used to accurately model the soil structure interaction. Variables included the range of mudline elevations, the different geotechnical properties for each segment, the type and size of pile, and the moment capacity of the pile-to-cap connection. A force-displacement or push-over curve was developed for each bent by combining the soil-structure interaction results with the moment curvature characteristics of the pile-to-cap connection to determine the stiffness for each bent. The bent stiffness ranged from 17 to 168 kips/in.

A multimodal spectral analysis was run with a two-dimensional spine model to determine the seismic displacement at each bent along the pier. In the model, seismic masses were lumped at nodes along the pier centerline at each bent location. The stiffness matrix developed from the push-over analysis was used for the support condition at each node. The spine section properties consisted of the deck section and utility tunnels on each side of the pier. The initial modal analysis indicated a significant difference between the displacements at each end of the pier due to the large variation in bent stiffness inshore to offshore. This created an eccentricity between the center of mass and center of rigidity, generating significant torsion for pier motions transverse to the long axis of the pier. This torsion resulted in displacements at the ends of the pier during seismic events that were up to 50 percent larger than the global seismic displacement of 11.8-in. The initial calculated displacement at the inshore end exceeded the capacity of the seismic joints in the pipes and would have caused the pipes to hit the side walls of the utility tunnel. In addition, the displacement would have caused the short concrete piles at the inshore end to fail in shear. The innovative solution was to pin the tops of the concrete piles in Bents 1 through 3 to reduce the stiffness and associated moments and shears in the piles by approximately a factor of 4. A partially pinned connection was used in Bents 4 and 5 to reduce their stiffness, moment and shear forces. The resulting loss of stiffness from pinning the landside concrete piles was offset by the addition of 36-in steel pipe piles in Bents 6 through 12. The net effect was to produce a pier with a relatively uniform distribution of mass, strength, and stiffness resulting in calculated displacements that ranged from 11.7-in at the landside end to 13.3-in at the outer end. Figure 7 illustrates the final pile layout and the tuning of the structure.

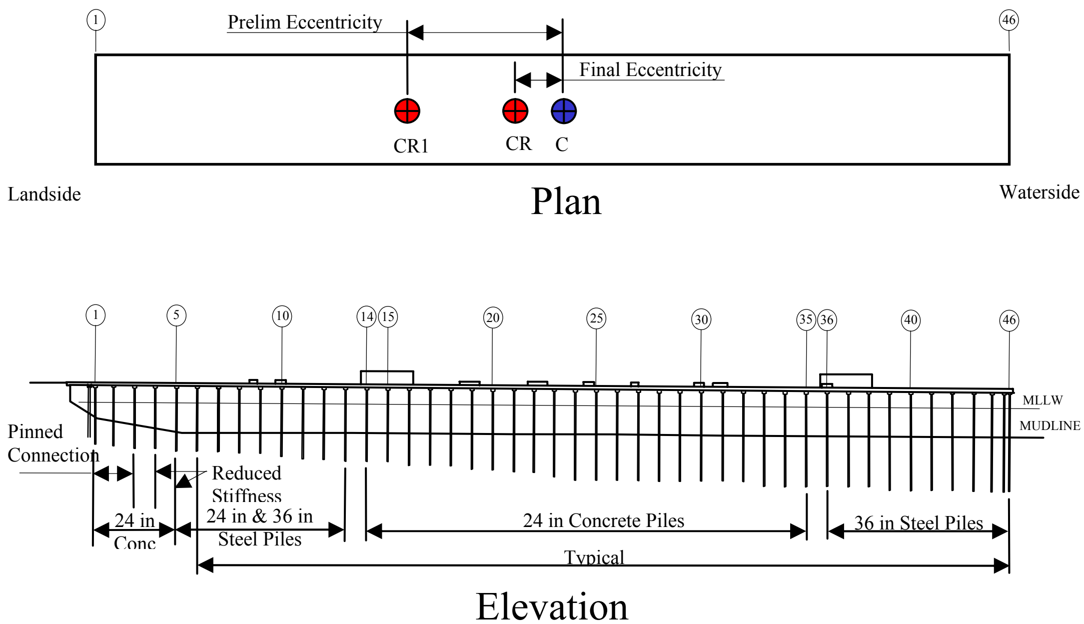

Figure 7: Tuning the Structure

DISPLACEMENT CAPACITY

After determining the displacement demand on the structure, all the elements and connections in the structure must be designed for that displacement. TR-2103 provides for dependable inelastic action by specifying that the inelastic behavior occurs in carefully detailed plastic hinges located in the piles. This means that the piles are detailed to undergo that inelastic behavior without loosing their load-carrying capacity to prevent collapse of the structure. Shear failures of piles and inelastic actions of deck members are prevented by ensuring that the dependable strength of these members exceeds the maximum feasible input corresponding to the plastic hinging in the piles.

A moment curvature analysis was used to determine the strains in the pile plastic hinges. A moment curvature plot for a typical concrete pile below mudline is shown in Figure 8. The analysis takes into account the strength of the concrete, the amount and location of the reinforcing, and is performed using the probable lower bound estimates of the constituent material strengths. These strengths are higher than the minimum specified material strengths to reflect the reality that actual strengths of the materials are higher than the minimum specified strengths. The lower bound estimates are used because strength is less important to seismic resistance than is displacement capacity, i.e. ductility. Material strengths used for the lower bound analysis are f'_{ce}= (1.3) (f'_c) for concrete, f_{ye}= (1.1) (f_y) for mild reinforcement, and f_{pe}= (1.0) (f_{pu}) for prestress strands. The concrete extreme fiber compression and prestressing strand strains in the concrete piles and the structural steel strains in the steel pipe piles were checked for the Levels 1 and 2 earthquakes using the moment curvature analysis. The strains in the plastic hinges located below mudline were very low because of the flexibility of the piles. The initial analysis indicated strains were very high at the pile-to-cap connection. As a result, two innovative connections were developed to reduce them. As previously mentioned, Bents 1, 2, and 3 were pinned at the pile to cap connection to reduce the moment and shear forces and preventing a shear failure. This effective pinned connection is shown in Figure 9. This type of connection is often used in bridge and building construction. Initially, the strains in the reinforcement exceeded the criteria even with the pinned connection. A portion of the bar was debonded at the connection by taping to increase the yield length and reduce the strains. Foam was provided to isolate the outer portion of the pile and allow the rotations to occur.

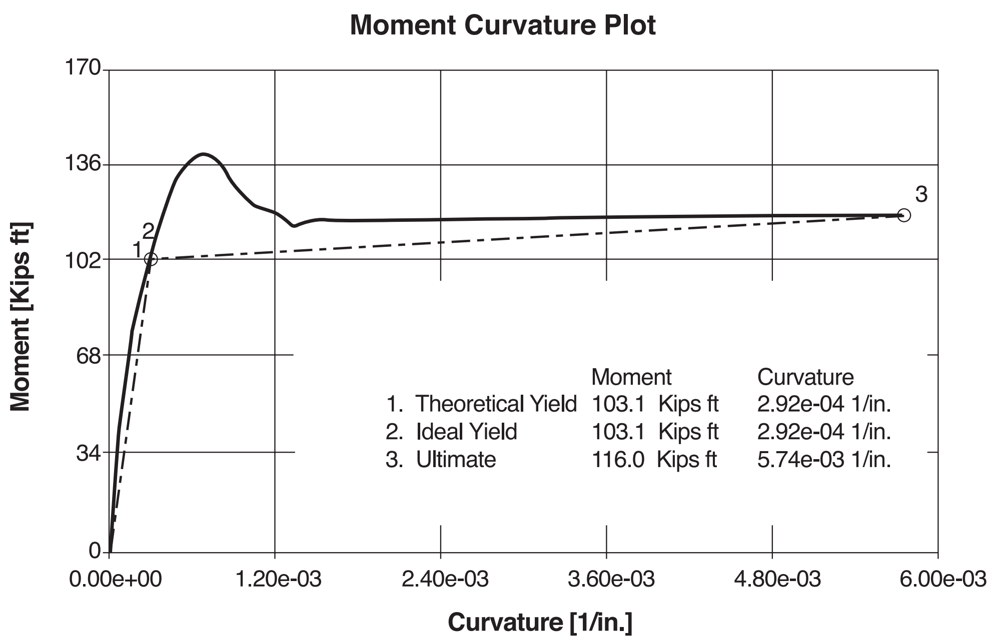

Figure 8: Typical Moment Curvature Plot

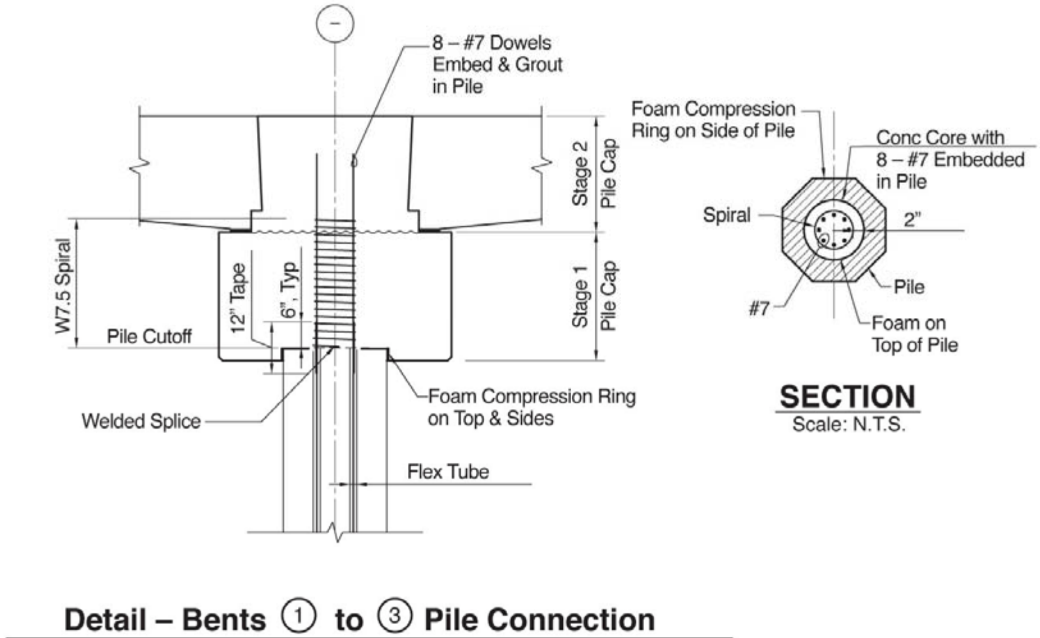

Figure 9: Typical Pinned Connection Bents 1 to 3

The typical connection used for the remainder of the concrete piles is shown in Figure 10. A similar connection was used for the pipe piles. To keep strains below the limit set in TR-2103, a 12-in length of the dowels was debonded by taping as indicated.
The connection to the pile cap always has less strength than the pile below. Consequently, the rotation is concentrated at the joint that will cause a gap to open on the order of ¼-in to 3/8-in during the Level 2 earthquake. This creates a condition where commercially-available moment curvature software is not valid because it typically assumes that there are many small cracks distributed over the length of the plastic hinge zone rather than being concentrated in one location. In recognition of this, a length of the bar was taped to debond it, and a special analysis was done to determine the strains associated with this connection. The strains were checked against the specified maximum strain.

CAPACITY PROTECTION

Shear failures of piles and inelastic actions of deck members are prevented by ensuring that the dependable strength of these members exceeds the maximum feasible input corresponding to the plastic hinging in the piles. They are designed using the moments and shears associated with upper-bound estimates of material strengths. These upper-bound material strengths are f'_{cm}= (1.7) (f'_c) for concrete, f_{ym}= (1.3) (f_y) for mild reinforcement, and f_{pum}= (1.1) (f_{pu}) for prestress strands. Alternatively, the forces resulting from the initial moment curvature analysis can be increased by a factor of 1.4 and used for this portion of the design. Phi factors are used in computing the moment and shear strength to ensure that the dependable strength exceeds the maximum feasible demand.

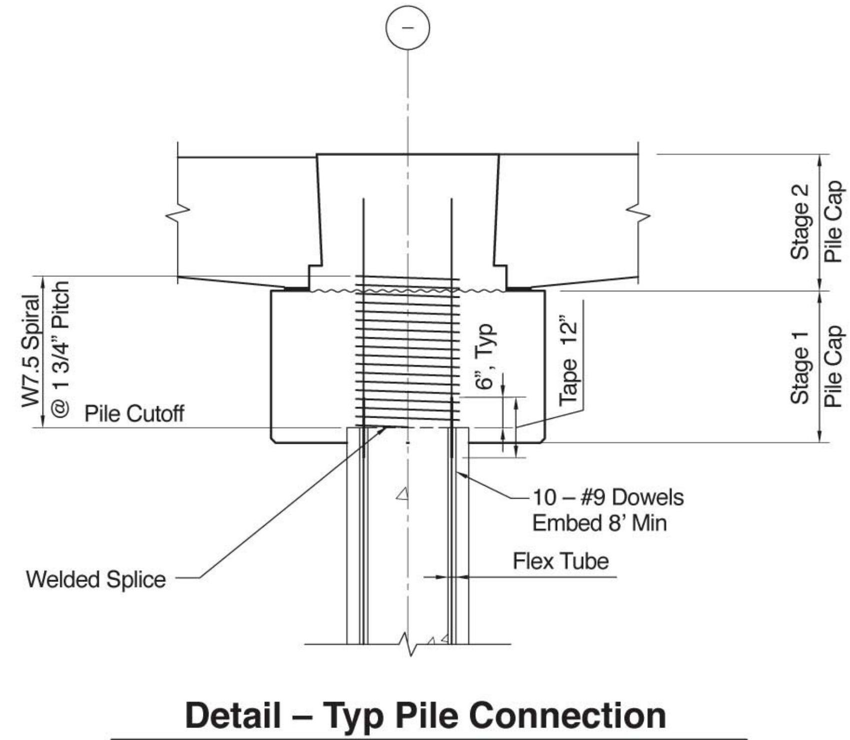

Figure 10: Typical Pile Connection with Debonded Dowels

PRECAST ELEMENTS

The innovative and extensive use of precast elements in the design of this pier created efficiencies in erection during construction and reduced the use of over-water concrete formwork. Precast elements used for this pier included prestressed piling, prestressed haunched deck panels, a utilidor surrounding the pier, utilidors on land and on the marginal wharf, channel beams on the marginal wharf, and lids for the utilidors. A brief description of selected precast elements, how they were integrated into the design of the pier, and photographs of connections are discussed in this paper. The first precast element to be installed during construction is precast and prestressed concrete piling. The design of the piling for the seismic loading condition is discussed in detail earlier in this paper and will not be described here. Concrete piling is very resistant to damage during installation, and is very commonly used for pier and wharf construction. The typical connection between the pile and the pile cap is shown in Figure 10. There were a few piling that could not be driven to tip elevation during installation. An alternate connection using the prestress strand was used for piling, extending a significant distance above the soffitt form. The cover and core concrete were carefully removed to expose the strand and the strand was embedded in the pile cap. Guidelines were provided to the contractor requiring the addition of grouted dowels if an excessive amount of damage to the strand occurred during removal of the cover and core concrete. Figure 11 is a photograph taken during construction of Pier Delta of the typical pile connection. The dowel tubes need to be trimmed and the top of the pile cleaned of debris prior to grouting of the dowels.

Figure 11: Photograph of Typical Pile Connection with Dowels

Figure 12 is a photograph taken during construction of Pier Delta of the alternate pile connection. The cover concrete has been removed and the strand trimmed to the correct length. The core concrete still needs to be removed.

Figure 12: Photograph of Alternate Pile Connection with Strand

The cast-in-place pilecap that follows the pile installation is the only cast-in-place structural element on the pier except for closure pours and a topping slab. Precast, prestressed haunched deck panels rest on top of the pile cap and support the vertical and horizontal dead and live loads. These deck panels are designed for a 30-foot bent spacing, precast offsite, brought to the site by barge and set in place by a floating crane. The long bent spacing combined with the rapid erection of precast elements combine to create a large amount of deck space in a short amount of time. The joint between the pilecaps and the deck panels and the deck panels to its matching deck panel must be designed for the moments and shears resulting from dead, live, wind, berthing, and earthquake loads. The top bars of the panels are welded to their matching top bar in the adjacent panel with the use of an angle creating a full-strength connection for negative moment capacity. The strand at the bottom of the panels is extended beyond the end of the panels and across the width of the pile cap to lap with its matching strand from the adjacent panel. The embedment length of the strand is not sufficient to fully develop the strand, but it is sufficient to develop the positive moment required in the joint, primarily resulting from frame action during seismic events. Strand chucks can be used on the ends of the strand, if desired, to increase the moment capacity of the connection. Figure 13 is a photograph of the welded top bar connection between the deck panels on Pier Delta.

Figure 13: Photograph of Deck Panel Top Bar Connection

A concrete utilidor for dirty and clean utilities, a large diameter steam line, and a trench drain is at the perimeter of the pier. We designed a one-piece concrete utilidor to support all utilities and provide drainage that spans between bents at 30 feet on-center. Each section is 24.6 feet-wide, 26.9 feet-long ,and weighs approximately 210 kips. The utilidor was site cast by the contractor at another terminal in Puget Sound, loaded onto barges, transported to the site and placed in position by the floating crane. The utilidor was not designed to be part of the lateral resisting system of the pier but it is designed to support the 234-kip outrigger loads resulting from the 140-ton mobile cranes. The utilidor is also designed to span 60-ft between bents, with dead load only, just in case an exterior support pile is completely broken. Reinforcing is provided across the pilecap for the load case by doweling into the ends of the precast utilidors and lapping the bars across the width of the pilecap. Precasting the utilidors eliminated overwater formwork, saving money, and allowed for parallel construction of elements, shortening the overall construction time for Pier Delta. The utilidors were cast in a continuous pour without any cold joints to reduce the possibility of leakage. Figure 14 is a photograph of the reinforcing cages for the utilidors ready for erection of forms at the casting location. Five utilidors were cast at a time. Note the casting bed under the reinforcing that was constructed to provide a flat surface for the bottom of the utilidors.

Figure 14: Site Precasting of Utilidors

SUMMARY

In addition to the extensive use of precast concrete elements, including; precast prestressed piling, long-span precast deck panels and precast concrete utilidors, three innovations in particular helped make the seismic design of Pier Delta a success. First and most important was the recognition of the significance of the displacement cap in the site-specific spectrum that made the plumb pile concept feasible and resulting in a savings of 690 piles. The second innovation was the use of the 36-in steel pipe piles in Bents 6 through 12 in conjunction with the effective pinned connection in Bents 1 through 3 to tune the strength and stiffness of the bents, thereby reducing the torsion and differential motions between the ends of the piers.

Third was debonding the dowels in the pile-to-cap connection. This maintained the stiffness and strength of the pile-to-cap connection, resulting in lower service load deflections while at the same time reducing the strains to allowable limits in the Level 2 earthquake.

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