

BERTHS 57, 58 AND 59 CONTAINER WHARF AT THE PORT OF OAKLAND

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Introduction

Berths 57 to 59 will extend Berths 55/56 so that the entire Berths 55 to 59 wharf and yard will increase the size of container handling facilities at the Port of Oakland by more than 120 acres, or 25%.

B57-59 extend from the eastern end of B56 in the middle harbor channel, meeting the western tip of B60 3,600 feet away at a fifteen degree angle. In addition to the design of the wharf, this project includes the design of dredging and embankments under the wharf and at the UP Mole west of B55/56. All excavated material will be reused on Port property, either as fill in the container yard or in the area northwest of the wharf designated as the North Cell. The North Cell will later become part of a public beach and park.

Originally, this project included only the design of a 3,000-foot long wharf and the design of the excavation and embankment for the full 3,600-foot length of B57 to B59. The remaining 600 feet were intended to become a future tugboat wharf. The designs were expanded to include the remaining 600 feet of Berth 59, after the Port secured a tenant requiring a 3,600-foot container wharf. Berth 59 will be built as a change order to the B57/58 contract.



Fig. 1 – Aerial View of Project Site

When built, the channel in front of the embankment will be dredged to El. -50 feet. The elevation at the wharf crane rails is El. +15.25 feet. The embankment is designed assuming the channel is dredged to -55 feet. In the first 3,000 feet of the wharf, the embankment is stabilized using a combination of Cement Deep Soil Mixing (CDSM) and rock dike. CDSM is a method of soil strengthening where multiple two foot plus diameter augers drill into the earth, injecting and mixing a cement slurry. Continuous walls of this strengthened material are built in a grid pattern to stabilize the soil embankment. A small rock dike is built waterside of the CDSM walls to protect the embankment and to stabilize a small mud area that remains in front of the walls after the CDSM installation.

A rock dike design is used in the remaining 600 feet of wharf in B59. The rock dike is used for this section because it simplifies the transition between the end of B59 and the existing B60 wharf. A rock dike also allows greater flexibility in design of a future wharf.

The project includes a soil management program for tracking, stockpiling, and testing the upper 15 feet of excavated material prior to reuse as fill. The excavation of the upper soil layer was required to conform to extensive project environmental requirements dictated by the Corps of Engineers and the Regional Water Quality Control Board.

Site

Typically, on the B55 to B59 site, there is an upper fill layer consisting of sandy material from elevation +13' to elevation 0'. Beneath, there is a soft mud layer about 25 feet thick down to elevation -25'. Below this are layers of dense cemented sands referred to as Merritt Sands.

The site conditions for this project are generally similar to those at the adjoining B55/56, with a few significant differences. In an approximately 600 foot long section in the middle of B57, the mud layer is up to 60 feet thick and extends down to elevation -60°. Also, a layer of liquefiable soil was identified underneath the mud layer. The presence of both the soft Bay Mud and liquefiable sands significantly influence the shoreline stability during a seismic event and were major factors affecting the shoreline stabilization schemes developed for this project.

The deep mud section required an entirely different embankment design to insure stability in an earthquake.

Concept Study

A concept study was undertaken to select an optimal design for the site and project requirements. The study considered five alternatives for stabilizing the embankment and providing lateral support to the wharf in the shallow mud section and five different alternatives for the deep mud section.

The alternatives included various arrangements of rock dike, CDSM, relieving platforms, and 24-inch and 48-inch piles. A relieving platform is a concrete slab behind the wharf supported on a dense grid of piles. The closely spaced piles provide lateral support to the embankment and support to the wharf.

As discussed above, CDSM is a method of stabilizing a soil mass by drilling into it with large mixing type augers, injecting a cement mixture, and mixing the cement with the existing soil to create a stronger material. One advantage of this method is that it does not displace the existing material and therefore results in less material requiring off site disposal.

The evaluation criteria for the alternatives included construction cost and schedule, excavation quantities, constructability, structural reliability, analysis reliability, interface with adjacent berths, seismic performance, durability, and ease of repairs. An evaluation of these criteria reduced the number of viable alternatives to three schemes, each with a solution for the shallow mud and deep mud sections.

Differences in cost and construction schedule between three final alternatives were small and were not a significant factor in selecting a final scheme.

The Port selected a CDSM stabilized embankment for Berths 57/58 and a rock dike embankment for Berth 59. CDSM was selected primarily to minimize the amount of Bay Mud to be excavated and disposed of. Rock dike was selected for B59 because it provided flexibility to build several types of facility. At the time of the decision, the Port planned a tug boat wharf for this location, but wisely kept its options open.

Embankment Design and Pile Spacing

Large diameter cylinder piles were used on this project because, compared to a rock dike embankment, a CDSM embankment provides less direct lateral pile support. The large piles were sized to provide a substantial connection between the wharf deck and the dense Merritt sands below, independent of the CDSM and the rock on top of it. Because these piles do not rely on rock to transfer loads, the thickness of the rock layer on top of the CDSM could be reduced and the height of the CDSM walls increased. This reduced the amount of excavation and rock fill required. The large diameter piles also increase embankment stability.

The selected design is equally suitable for the deep and shallow mud sections. In the deep mud section, the width of the CDSM walls is substantially increased, but the pile design remains the same. See Figures 2 and 3.

Twelve-foot spacing is required between the CDSM walls to provide acceptable clearance to the 48-inch piles. To work with this module, the typical pile spacing on the wharf is 24 feet. The row spacing used on the adjacent B55/56 wharf was continued resulting in typical 18- by 24-foot bays for the piles and deck.

The two CDSM walls running parallel to the wharf are 31 feet apart with the landside wall centered 6.5 feet landside of the landside edge of the wharf. The landside CDSM wall is located to provide additional stability to the embankment, which has an underlying layer of liquefiable soil. Having the CDSM further landside gives the added advantage of additional containment for the soils behind the wharf, reducing long term settlement in the container yard.







Fig. 3 – Wharf Section, Deep Mud

Deck design/pile spacing

The crane rail girders are designed for the ultimate factored crane wheel loads of 325 kips per wheel at waterside and 240 kips at landside. The factored wheel loads with the crane in the operating and stowed positions are similar.

Standard ACI factors of 1.4 on dead load, 1.7 on lifted load, and 1.3 on wind load were used to calculate the factored crane wheel loads.

The deck is designed for a uniform service live load of 1000 psf. Both uniform and skip loads were considered. The piles are designed for a uniform service live load of 800 psf. The deck is also designed for HS20-44 truck loading and from a container handler with a maximum service load of 285 kips on the drive axle and 158 kips on the steering axle.

Independent finite element and beam models were made of the deck. Reinforcing was determined based on the distributed loads and seismic loads. The capacity of the deck for equipment loads was verified using yield line analysis.

The maximum moments at the piles were averaged over the pile strips and uniform reinforcing provided. An economic and functional system was achieved with these assumptions.

The 600 feet of wharf at the end of Berth 59, which were added after the start of construction, have a rock dike embankment. The most efficient structural design with this type of embankment is a system that uses all 24 inch piles. Instead of the row of 48 inch piles, 24 inch piles six feet on center are provided in row G.

Lateral Load Design - Large diameter piles

The design earthquake motions used by the Port of Oakland are similar to those used by other major California Ports, except that an additional design level is added. This design level is identified as Level 2. The table below gives an overview of the three design motions and corresponding performance levels.

Design Earthquake Motions and Performance Criteria		
Level	Probability of Exceedence	Performance Goal
1	50% in 50 Years	No Damage – Minimal Embankment
		Deformation
2	20% in 50 Years	Minor Damage –
		Embankment Deformation < 6"
3	10% in 50 Years	No Collapse – Repairable Damage –
		Embankment Deformation < 12"

To track the expected structural damage under various seismic events, the performance based analysis method was applied using allowable stresses and strains for the various structural components. Port of Oakland wharf design consultants have typically used the force reduction or R-factor method. A comparison with this method was therefore made in order to demonstrate the differences and help the decision making process. UBC, AISC and the ATC-32 Guidelines supplemented the design basis.

Only one row of 48-inch hollow prestressed precast concrete piles is required to limit the displacement of the wharf deck to acceptable levels. Two rows of traditional 24-inch octagonal prestressed concrete piles placed on each side of the 48-inch piles and spaced 12 feet and 24 feet apart, respectively, make up the remainder of the lateral load carrying pile system. The 48-inch piles are placed away from the landside crane rail to avoid congestion of rebar and rail anchorage bolts and to provide more efficient distribution of moments in the deck.

Headed reinforcement dowels are used to transfer seismic moments and shears from the pile to the wharf deck. A typical detail of the pile/deck connection is shown below.



Fig. 4 – 48 inch Pile Connection Detail

In addition to the inertial loads on the upper part of the piles and the pile/deck connection, the embankment soil deformations create significant forces on the piles deep in the ground. If this condition is not accounted for, it can lead to severe damage of the piles in the ground. Based on the expected soil displacement demands established by the geotechnical consultant, a nonlinear finite element analysis was conducted to identify the stress and strain levels in the piles from such soil movement. Sufficient shear reinforcement was placed in the piles to assure proper performance during a seismic event.

Shear keys

The shear keys for this project are designed to provide energy dissipation through the ductile shear failure of wide flange steel beams. Three 16 foot long and 3 feet deep steel beams are provided at each wharf expansion joint. The beams sit in reinforced slots in the wharf accessible from above.

Shear key forces were calculated using a simplified nonlinear model of the wharf structure, including the stiffness of the piles, the soil at the back of the wharf deck, and the cutoff wall. The controlling shear key forces occur from the eccentricity between earthquake forces along the length of the wharf and the resisting force in the seismic piles at the landside of the wharf. The moment from this eccentricity is resisted, in part as lateral load in the piles, and in part by the couple between the shear keys at the ends of each wharf segment.

A maximum service load of 2,077 kips was calculated across the expansion joints for the Level 1 earthquake. The three steel shear key beams are designed to transfer this force across the expansion joint using eccentrically braced frame theory with a load factor of 1.4 on the service load. Damage to the shear key steel beams is expected in Level 2 or 3 earthquakes. The beams are designed so that they can be replaced from the top of the deck after a major earthquake.

This design is more expensive than the standard interlocking reinforced concrete shear keys used at most facilities. This is because, in addition to the cost of the wide flanges, a large amount of reinforcement is required in the concrete around the steel beams. In return, the shear keys are designed to prevent seismic damage to other parts of the wharf and to be easily replaceable. This results in minimal down time to wharf operations after a significant earthquake.



Fig. 5 – Shear Key

Crane rail support

The wharf uses a continuous sole plate rail support system with a continuous rail pad. This system was selected after an evaluation of the performance, cost, and installation procedures for continuous plate and intermittent plate systems. Installation and alignment of the continuous sole plate system is simpler and more accurate. The continuous plate provides more uniform vertical support to the rail. An intermittent plate system resists lateral load through plates less than one foot in length. The continuous plate system resists lateral load on the 10-foot plates installed with eight pairs of bolts. This significantly reduces local stresses from lateral loads. Reduction in vertical and lateral stresses results in an increased service life.

The continuous plate system uses more plate and requires a greater amount of epoxy grout because the entire width under the plates must be grouted. With the discontinuous system, the space under the individual plates is grouted, but where the rail sits directly on the pad and grout, the grouted width is less. Therefore, the continuous plate system has a higher material cost.

To obtain a similar material cost, we specified a narrower rail clip with adequate lateral load capacity: 20 kips per clip. The narrower clip allows the use of a narrower base plate and rail trench, thereby reducing the total amount of material required. The continuous plate system is less labor intensive and therefore less costly to install. With the narrower clips, plate, and trench, the cost of the continuous and discontinuous plate systems is comparable.

Cut-off wall

Wharves on vertical piles typically sustain earthquake damage just below the deck in the upper two feet of the piles. Therefore, access for inspection and repair was an important consideration in the design of this facility. A two-foot crawlspace is provided between the bottom of the wharf deck and the top of the support rock to give access for inspection.

The cut-off wall panels are designed with a maximum length of 24 feet and are bolted to the back of the deck. Repairs to the two back rows of piles can be accomplished by removing the cut-off wall. By making a small excavation behind the wharf and unbolting the panels, the panels can be lifted out of the way to give full access for pile repair.

Fender and mooring system

An analysis was made of the mooring forces developed with 1200, 900 and 500 foot long vessels. Currently, the largest container ships in the world are approximately 1100 feet long. It is interesting to note that it is the smallest design ship that produces the largest fender force because it contacts as few as two fenders on impact when berthing.

Wind load on the largest ship controls the bollard design. The highest design load of 330 kips occurs on the bollard between two large ships berthed end to end. The analysis showed that a fender and bollard spacing of 72 feet is adequate for this facility.

The wharf structure is designed for twice the manufacturer's recommended fender design load or a minimum of 800 kips combined with orthogonal friction loads. This is to account for the vessel imparting a higher force on the wharf in case the fenders get damaged from overload. The coefficient of friction is taken as 0.25 and orthogonal loads are combined with the direct load in both the vertical and lateral directions.

Crane stops

Crane stops are provided at the end of the crane rail runway to prevent the cranes from running past the end of the wharf. The design load for the crane stops is 600 kips ultimate load. This is the highest force the crane can impart without tipping over at impact. This loading could theoretically occur in an extreme windstorm if the crane breaks loose from the stowage pins and runs down the wharf.

Each wharf crane is equipped with two hydraulic buffers at each rail, each capable of absorbing the energy required to stop a container crane gantrying at its maximum speed of 150 feet/sec. The cushioning effect of the buffer is dependent on the speed of the collision. At higher speeds, the buffer will deflect only a small amount before reaching its ultimate capacity and failing in a ductile mode by bulging of the walls.

The crane-mounted hydraulic buffers are adequate to protect the cranes against damage from accidents under normal operations. No number of buffers will be effective in protecting against damage in the case of a run-away crane. Therefore, it was decided not to put hydraulic buffers at the crane stops.

Utility Box Covers

It is not uncommon to see badly corroded or bent hinge pins on utility covers on wharf facilities. The pins are often a weak point in the design because their connections trap water resulting in corrosion. The hinge mechanisms are often complicated to build and are structural weak points in the design.



Fig. 6 – Utility Box Cover

Two types of utility box covers are provided on this facility. Boxes that will be opened infrequently are designed to be lifted out of their slot. Covers that will be used regularly by ships calling on the wharf are designed to tilt open through a hinge. The tilting covers can be either tilted open, as shown in Fig. 6, or lifted out of the slot.

The pins are 1 1/4 inch high strength round bars welded to the cover frame. The cover has curved slot that the pin slides into as the cover is dropped into position. When seated, the cover can be either tilted or lifted out.

Wharf Wearing Surface

A study was made to evaluate the cost and practical advantages of different wearing surfaces on the wharf. It was decided that adding two inches to the deck concrete is the most economical and practical solution over time. The high strength concrete used in the deck will provide a durable wearing surface far more resistant to damage than an asphalt surface.

Conclusion

Various schemes are investigated to stabilize the liquefiable soft Bay Muds. The two finalists injecting the soil with Cement Deep Soil Mixing (CDSM) technique and using a rock dike were close in cost and construction time. The CDSM was selected because of the reduced excavation quantity and for environmental reasons. The CDSM requires specialized equipment and expertise. A rock dike may be a better alternate for facilities where reliable CDSM expertise is not available.

The deck structure is a moment resistant frame consisting of the deck, hollow 48 inch diameter concrete piles, and 24 inch octagonal prestressed concrete piles. The design philosophy of using different earthquake motion levels and corresponding deformations provides a more economic structure with acceptable performance. The eccentric brace concept for the shear key design provides a ductile energy absorbing fuse that will mitigate damage to the rest of the structure. The damage to the shear key in a large earthquake is expected to be limited to the shear key beams, which are easy to repair or replace.

The CDSM design requires the embankment to be within two feet of the deck structure. It is important to be able to inspect earthquake related structural damage to the underside of the deck. The removable cut-off wall provides access in such an event.

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