TIEBACK ANCHORS FOR BULKHEAD REHABILITATION AT PORT FACILITY

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INTRODUCTION

The project involved rehabilitation of a 9.5-m-high, 250-m-long, steel sheet pile bulkhead at the Port of Everett (POE) in Washington State. Because of its age and exposure to the marine environment, the existing sheet piles were severely corroded and backfill material was washing out through large perforations in the wall. The loss of backfill material was causing areas adjacent to the bulkhead to settle, resulting in a hazardous condition for port operations. The POE was also concerned with failure of the bulkhead, particularly during a strong ground motion earthquake. In addition, the port had long-term plans to provide access for larger ships that required a deeper berth adjacent to the marginal wharf.

SITE AND PROJECT DESCRIPTION

The approximate site location is shown on Figure 1. A Chill Facility building, which is supported on conventional spread footings, is located between Piers 1 and 3, as shown on Figure 2. The bulkhead is located approximately 12 m west of the Chill Facility. The portion of the bulkhead that was repaired extended from Pier 1 to Pier 3.

The existing bulkhead consisted of a steel sheet pile wall that in most areas was tied back with deadman anchors. The existing deadman anchors are supported on timber piles connected to a grade beam running parallel to the shoreline. The bulkhead is backfilled with relatively loose sand hydraulic fill.

Because the height of the bulkhead was over 13.1 m, an anchored sheet pile wall configuration was necessary. However, because the chill warehouse and existing utilities were so close to the bulkhead, passive tieback anchors (deadman) were not feasible. Various repair alternatives were reviewed and evaluated for constructability, cost, and performance. The resulting design solution for the rehabilitation was to install new steel sheet piles slightly offshore of the existing bulkhead and install permanent post-tensioned tieback anchors to provide the required lateral support for the bulkhead. A reinforced concrete waler was designed in between the existing and proposed sheet pile walls to horizontally distribute the wall pressures to the post-tensioned tiebacks.

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Fig. 1. Vicinity map of Port of Everett site.

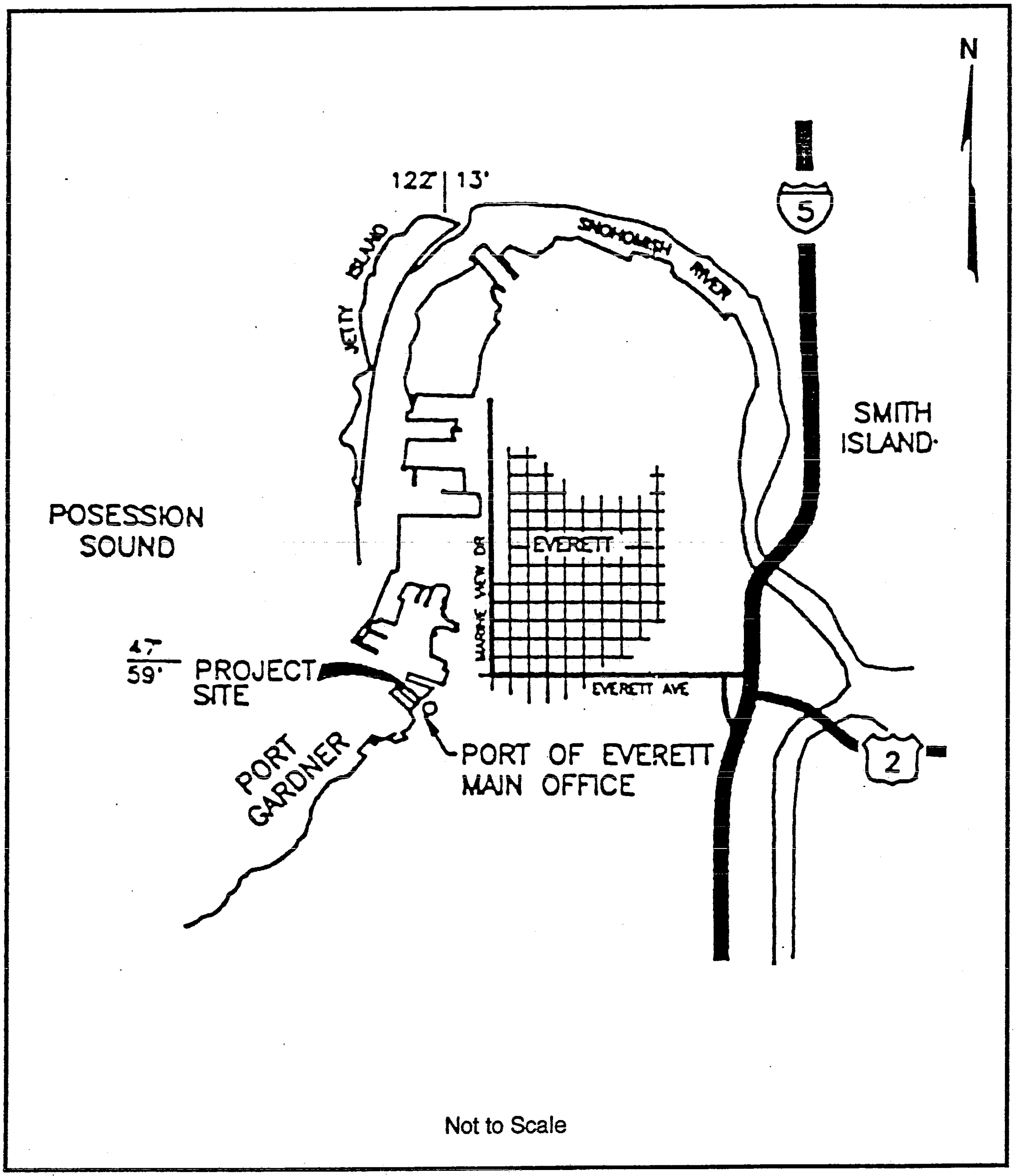
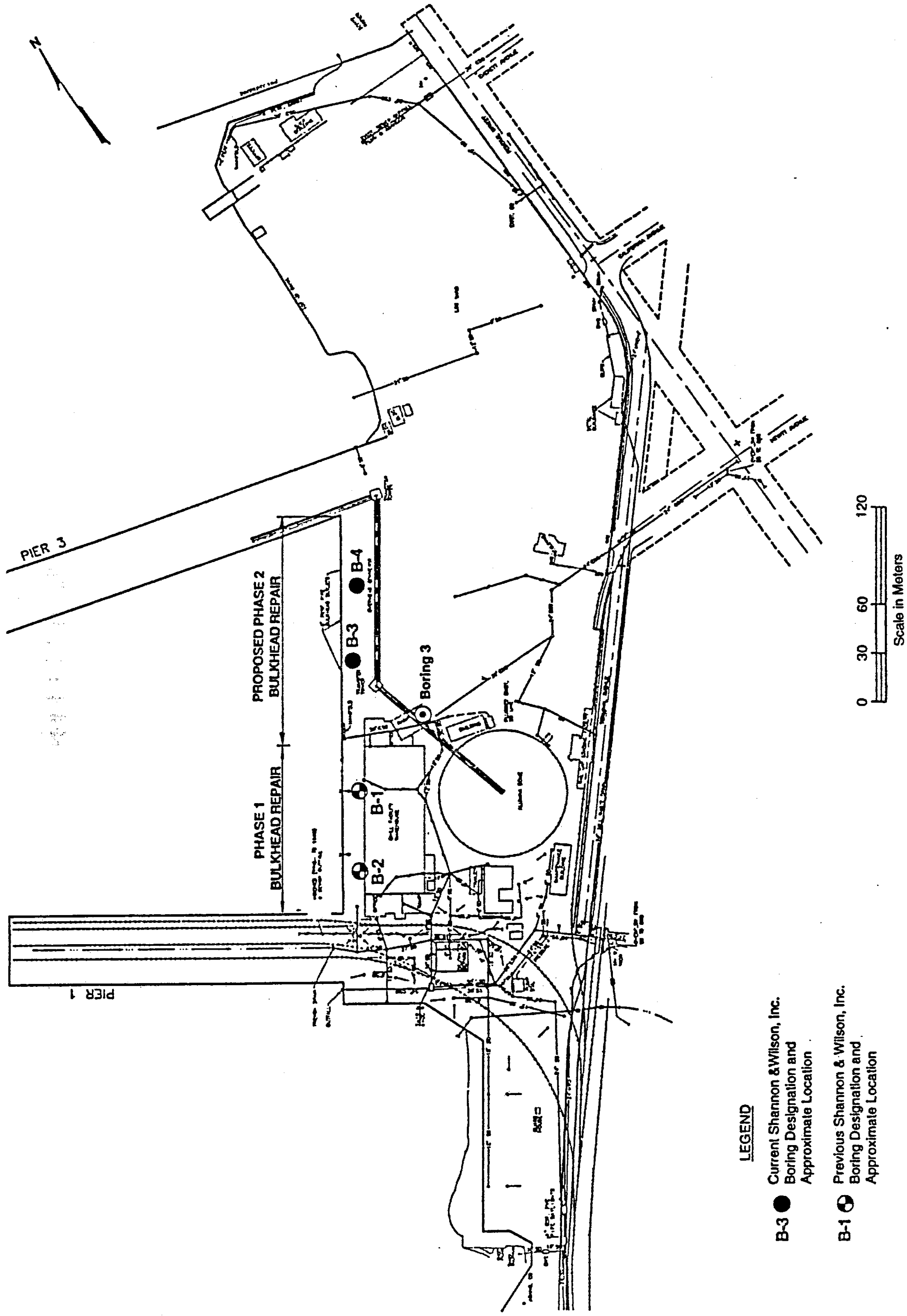


Fig. 2. Site plan showing boring locations.



Lean mix and structural concrete fill was installed between the sheet piles to encase the reinforced concrete waler and provide structural continuity between the existing and new sheet pile bulkhead. Wall drainage was maintained by extending the existing functional wall drain pipes through the new sheet pile bulkhead.

Except for a short section, the new bulkhead was anchored at two locations: at the higher elevation to the existing deadman by extending the existing tie rods to the face of the new sheet pile, and at the lower elevation by installing new tiebacks. The lower elevation permanent tiebacks were 0.2 m below mean lower low water (MLLW). Because of the overall magnitude of the proposed bulkhead repair and the potential disruption it would have on operations, the POE decided to separate the bulkhead rehabilitation design and construction into two phases. Phase 1 scope rehabilitated the approximate southern half of the alignment while Phase 2 included the approximate northern half.

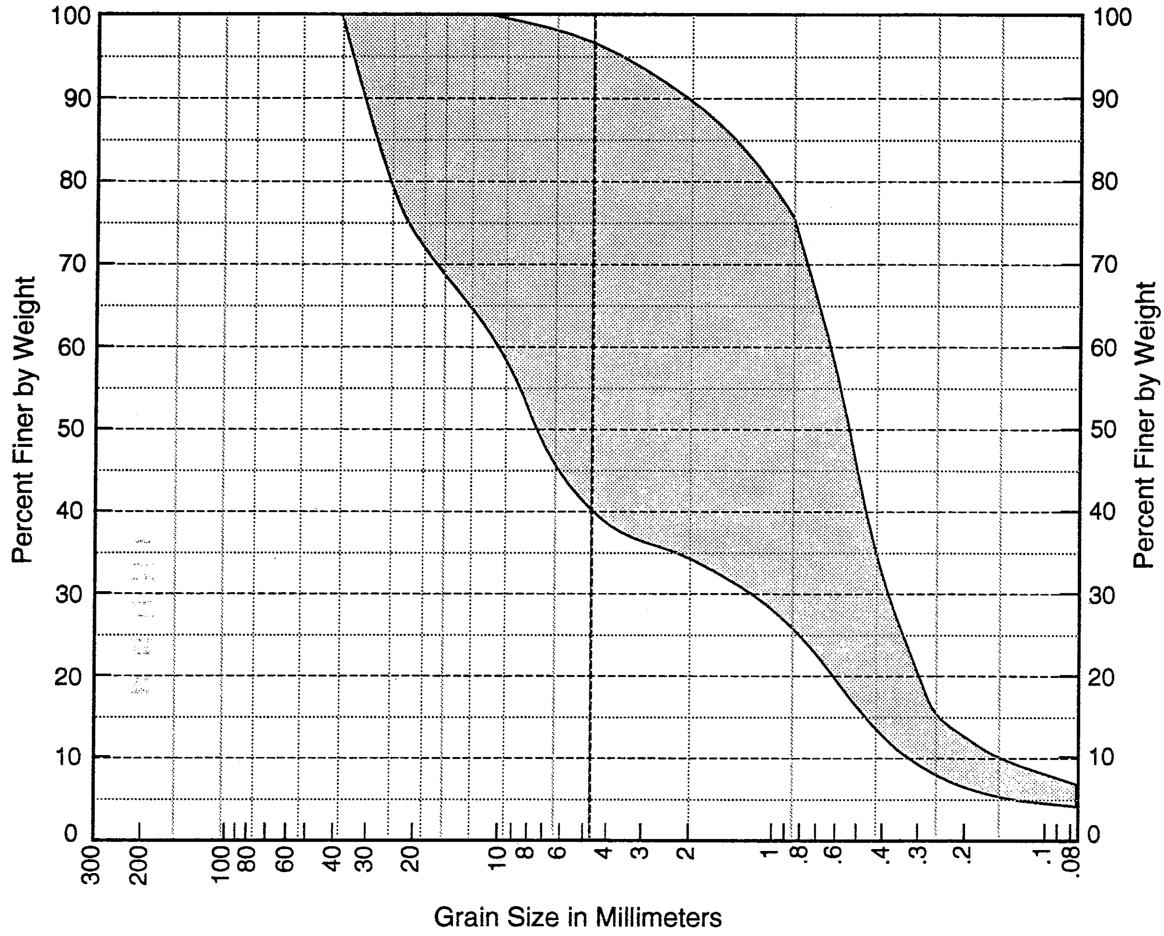
SUBSURFACE CONDITIONS

Four borings, designated B-1, B-2, B-3, and B-4, were advanced to explore the subsurface conditions at the site. The approximate locations of these borings are shown on Figure 2. The borings were advanced using rotary drilling procedures to minimize disturbance. Disturbed samples were retrieved from the borings with a split-spoon sampler in conjunction with the Standard Penetration Resistance Test (N-values).

Soil samples obtained in the field were tested in the laboratory to aid in classifying the soil. The laboratory testing program consisted of visual classification, moisture content determinations, and grain-size analyses. Representative grain-size distributions are illustrated on Figure 3.

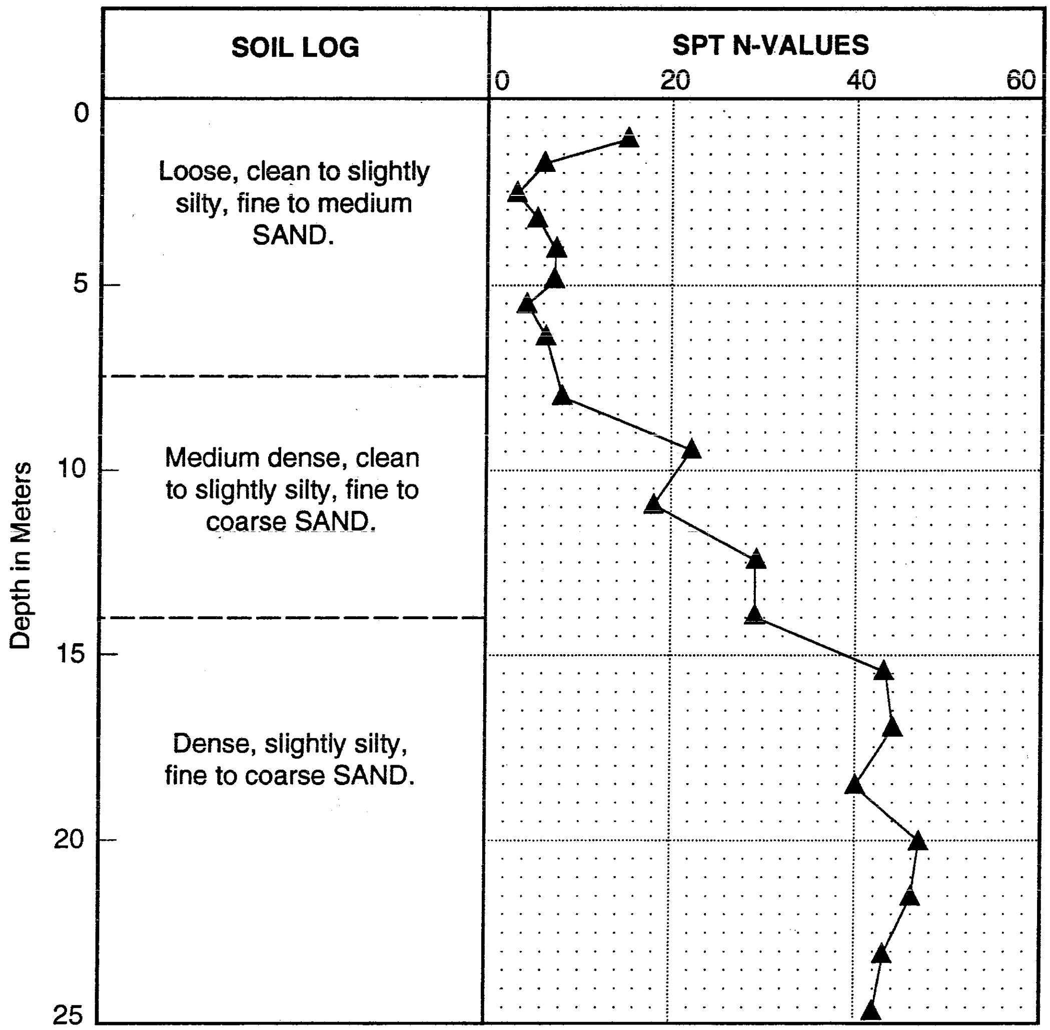
In general, the borings encountered about 8 to 14 m of dredged soils primarily consisting of clean to silty, slightly gravelly to gravelly, fine to coarse SAND with scattered organics and shell fragments. The relative density of this layer varied from very loose to medium dense, generally increasing with depth. These dredged soils were underlain to the boring termination depths by dense to very dense, clean to slightly silty, fine to coarse SAND interbedded with gravel layers. A representative boring log is presented on Figure 4. The groundwater level at the site fluctuates with the tide elevation.

Fig. 3. Range in grain size distribution of dredged backfill material.



Cobbles	Coarse	Fine	Coarse	Medium	Fine
	Gravel		Sand		

Fig. 4. Representative boring log illustrating relative densities.



DESIGN

Lateral Earth Pressures

Based on the subsurface conditions and laboratory test results, lateral pressures for design of the bulkhead under static and dynamic loading conditions were recommended for multiple and single anchored walls. Pressures for the static loading state are presented on Figures 5 and 6. Uniform pressures were used for design of multiple braced walls, and triangular pressures were used for single braced walls. Uniform pressures were determined by reducing the triangular active pressures by a factor of 0.65 (U.S. Navy 1982). For the dynamic (earthquake) loading phase, it was recommended that the active pressures in the very loose to medium dense zone (above elevation -4.9 to -6.1 m) be increased by 30 percent, based on the Mononabe-Okabe method (Seed and Whitman 1970). The increase in lateral earth pressure of the dense to very dense zone for the dynamic loading conditions was considered to be negligible.

The earth pressure diagram presented on Figure 5 was developed for Phase I after drilling borings B-1 and B-2. The diagram was intended for design of the wall with multiple anchors. As shown, the passive earth pressure coefficient ($K_{p\beta}$) values were reduced above elevation -11.5 m to account for sloping ground conditions. Dynamic pressures were considered only above elevation -6.1 m. Later, however, the POE decided to dredge to elevation -8.2 m, which required an increase in anchor design capacity in order to maintain the same embedment depth. Figure 6 presents lateral earth pressures for a single row of anchors and the final dredge line elevation.

Analysis indicated that the soils between elevations +3 m and -5 m had a potential for liquefaction under an earthquake with a maximum ground acceleration of 0.27 times gravity. Liquefaction potential was evaluated using procedures suggested by Seed and Harder (1990). This value of maximum ground acceleration is commonly used for a design earthquake for the Puget Sound region (AASHTO 1992). Recommended earth pressures for the liquefaction case are presented on Figure 7. As shown in this figure, the earth pressure in the liquefied zone was estimated to be equivalent to a fluid density of 4.71 kN/m^3 in addition to the overburden pressure. The philosophy for wall design was to apply static, dynamic, and liquified soil pressures to the bulkhead. During a strong ground motion earthquake, the wall would first react to dynamic loads. After enough load cycles occur, the dredged sand would liquefy and result in a fluid-type loading condition. The wall was then designed for the maximum loading condition using appropriate safety factors.

Fig. 5. Design pressures for static loading condition with multiple anchors.

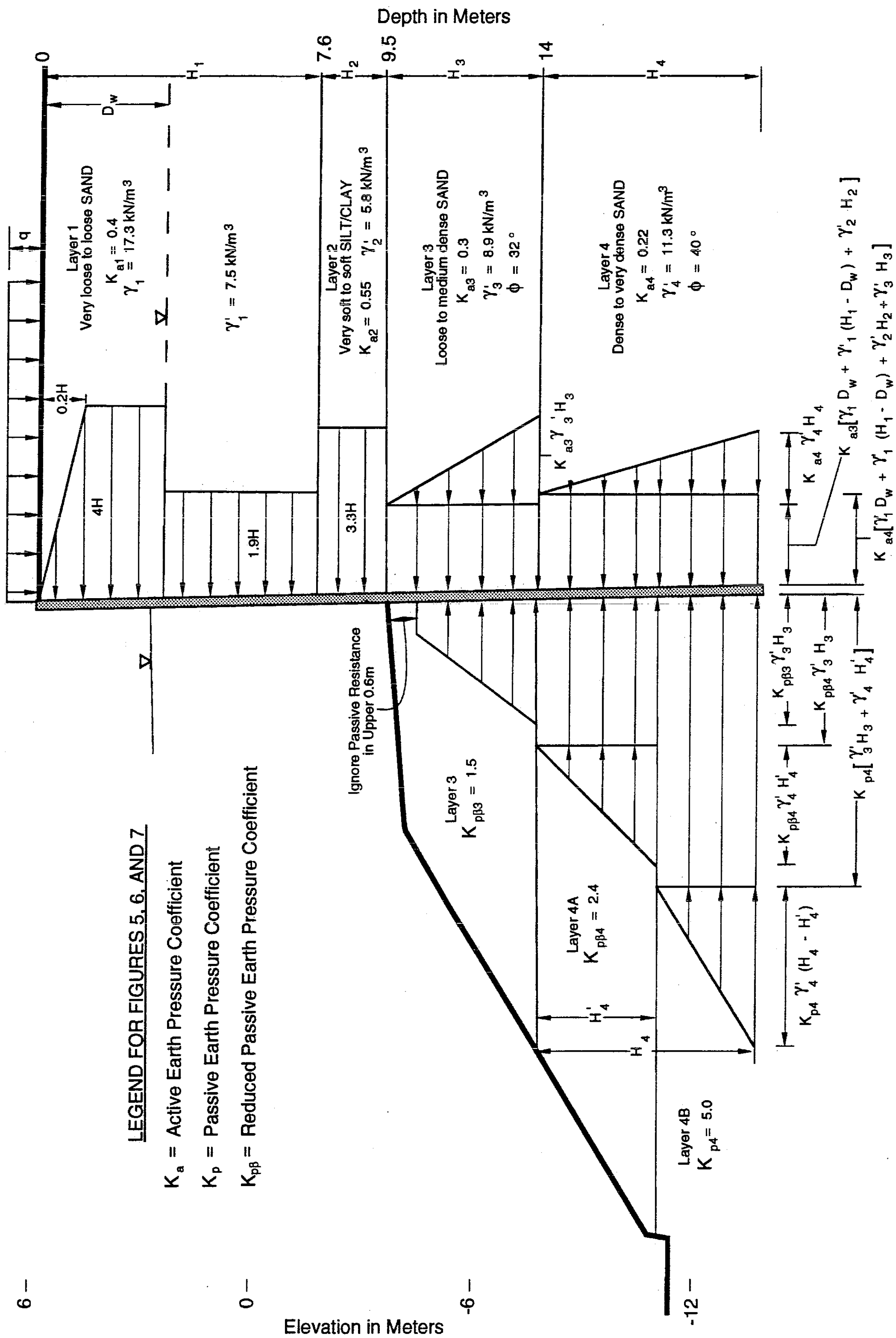


Fig. 6. Design pressures for static loading condition with a single row of anchors.

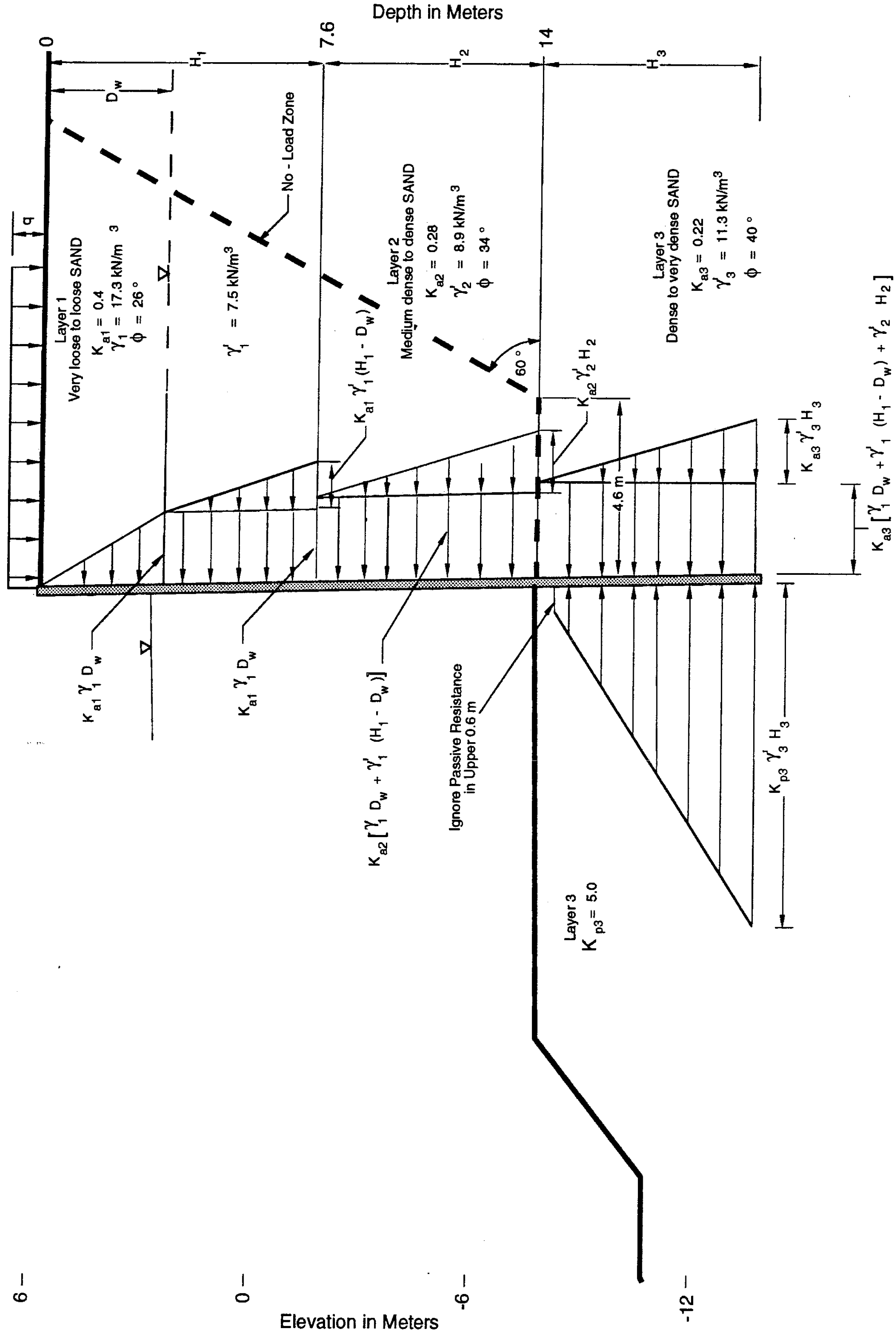
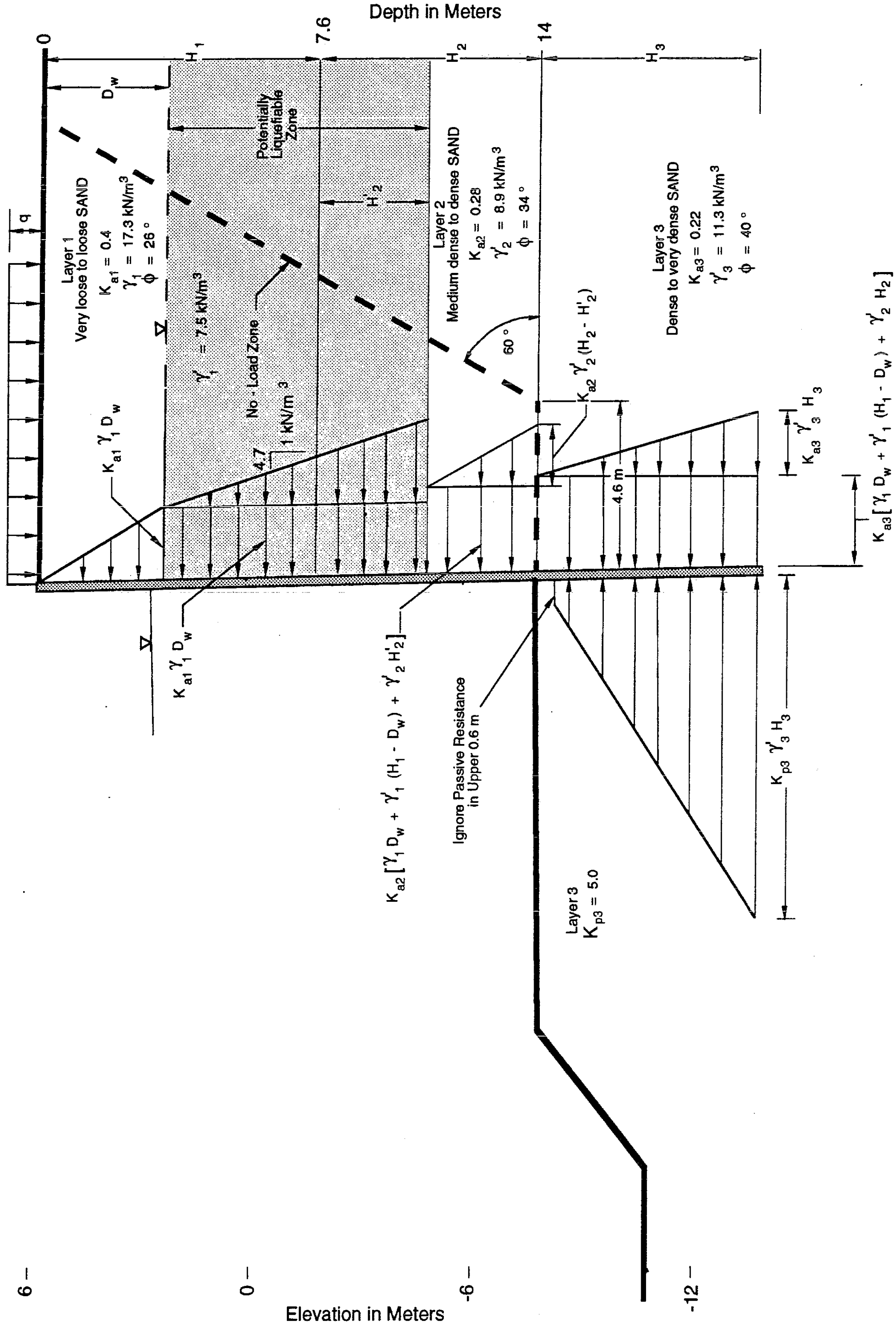


Fig. 7. Design pressures for liquefaction case with single row of anchors.



Adequate drainage was provided through the bulkhead. The bulkhead design, however, considered differential hydrostatic loading to account for tidal and groundwater fluctuations. Lateral pressures from anticipated surcharge loads on the bulkhead were also included in the design.

Capacity of Existing Deadman Anchors

As mentioned previously, the new bulkhead was designed to be anchored at two locations. At the higher elevation, the tie rods from the existing deadman anchors were extended to the face of the new sheet piles, using couplers. Before designing the new tieback anchors, however, it was necessary to determine the capacity of the existing deadman anchors. It was estimated that the anchors could resist up to 218 kN per tie rod spaced approximately 1.22 m apart.

Lower (New) Anchor Loads

Based on the recommended lateral earth pressures and the estimated capacities of the existing deadman anchors, the structural engineer determined the design load for the lower tieback anchors. The sheet pile wall was modeled as a beam element with the embedded portion of the wall modeled as a beam-on-elastic-foundation. The horizontal subgrade modulus of reaction was used to develop horizontal spring constants for the embedded portion of the sheet pile. Three different load cases were used in the design: static, dynamic (earthquake), and liquefaction. The bulkhead was analyzed, and the governing sheet pile bending moments, shears, axial forces, and tieback forces were used in the design.

The passive earth pressure coefficients (Kp) presented on Figures 5, 6, and 7 included a factor-of-safety of 1.5 to limit bulkhead deflections. For a soil-structure interaction analysis, however, ultimate Kp values were used.

Spring constants to model the passive pressure were estimated based on the following equation (Terzaghi 1955):

$$K = N_h Z \Delta H \text{ (kN/m)}$$

where:

K = Spring Constant (kN/m)

N_h = Constant of Horizontal Subgrade Reaction (kN/m^3)

Z = Depth of spring below mudline (m)

ΔH = Vertical spacing of springs (m)

The N_h values ranged from 0.9 x 10^{-3} kN/m^3 in loose to medium dense sand to 3.6 x 10^{-3} kN/m^3 in dense sand.

A linear, elastic, finite element method (FEM) program was used to design the bulkhead. Calculated spring displacements of the embedded portion of the sheet pile were reviewed, the horizontal spring constants (subgrade modulus of reaction) were revised, and the analysis was rerun to approximate non-linear soil behavior. Adequate sheet pile embedment was checked by comparing the resultant force of the soil spring constants with the allowable passive pressure resultant force. Classical static equilibrium methods were also used as a second check on the design embedment depth. Finally, global wall stability was evaluated utilizing Bishop's Method for analysis of circular failure surfaces, using the computer program PCSTABL5M.

With the exception of a few, anchors were installed at an inclination of 30 degrees to the horizontal and normal to the plane of the bulkhead. For these conditions, it was determined, based on the beam-on-elastic model, that the anchors should be designed for loads of 610, 663, and 672 kN for the static, dynamic, and liquefaction cases, respectively.

Anchor Capacities

Based on the anticipated construction conditions, it was recommended that the tiebacks be installed at a minimum inclination of 30 degrees to the horizontal for the following reasons:

(a) To increase the bond length in the more competent dense sand below the potentially liquefiable soils.

(b) To reduce the potential for soil piping out through the anchor borehole during and after installation due to fluctuating water levels.

The load-carrying capacity of soil anchors depends on the type of anchor selected, the method of installation, and the soil conditions around the anchor. The allowable bond stress values used for design of the pressure-grouted anchors for Phase 2 are presented in Table 1.

Table 1 Allowable Bond Stress (kN/m)

Zone	Load Case		
	Static	Dynamic	Liquefaction
Above elevation -1.8 m	NA	NA	NA
Between elevation -1.8 and -4.9 m	50	67	NA
Below elevation -4.9 m	80	107	107

The allowable bond stress values were based on a factor-of-safety (FS) of 2 for static loads and 1.5 for dynamic and liquefaction loads. The no-load zone is as shown on Figures 6 and 7.

Anchor Lengths

Determination of anchor lengths needed for sufficient capacity was different from the conventional because of the different tieback loads and allowable anchor capacities for the static, dynamic, and liquefaction load cases. Table 2 presents the anchor lengths that were determined from the allowable bond stress values presented in Table 1 for each loading case.

Table 2 Anchor Lengths

	Tieback Load (kN)	Unbonded Length (m)	Bonded Length Needed (m)	Total Length (m)
Static	610	9.1	9.1	18.2
Dynamic	663	9.1	7.6	16.7
Liquefaction	672	13.1	6.4	19.5

As indicated in Table 2 above, the liquefaction load case was most critical for determining the total anchor lengths, and it became the basis for the design. Although a portion of the bond zone (approximately 3 m) extended into the potentially liquefiable layer, the FS against pullout was estimated to be greater than 1.5.

Based on construction drawings for the old bulkhead, there was concern that some anchors might hit obstructions during installation and may have to be installed at a steeper horizontal angle of inclination or at a skew to the plane perpendicular to the bulkhead. To cover this possibility, an increased tolerance for the installation angles was allowed in the plans, and the total anchor length was increased to 19.8 m with an unbonded length of 9.1 m and a bonded length of 10.7 m.

Anchor Testing

Seven anchors were performance tested: four in Phase 1 and three in Phase 2. All remaining anchors were proof tested. Anchor installation, testing, and acceptance was in accordance with the recommendations of the following publications:

(a) Permanent Ground Anchors, Federal Highway Administration (FHWA) Report No. FHWA-DP-68-14, 1984.

(b) Recommendations for Prestressed Rock and Soil Anchors, Post Tensioning Institute (PTI), 1986.

Determining the loads for anchor proof and performance tests was different from the conventional methods. In a conventional permanent tieback anchor testing program, anchors would be performance tested to 200 percent of the design load and proof tested to 130 percent of the design load. For this project, the question was, "What is the design load?" Since the purpose of a proof test is to determine if the anchor has an FS of at least 1.3 for the maximum anticipated load, it was decided to proof test each anchor to 130 percent of the maximum 672 kN load (liquefaction case). With respect to the performance test load, it was decided to load the anchors to 200 percent of the allowable static bond stress times the bond length. To limit the bulkhead deflection, it was recommended to lockoff the anchors at the static design load.

CONSTRUCTION

Construction was completed in two phases. Phase 1 was constructed during the period from November 1994 to May 1995. Phase 2 construction lasted from August 1995 to May 1996.

The general construction sequence was as follows:

1. Install new sheet piles on the seaward side of the existing bulkhead.
2. Extend existing tie rods through the new sheet piles.
3. Place concrete between the old and new sheet piles.
4. Install and stress new tiebacks.

A typical section through the bulkhead is presented on Figure 8.

Tieback Anchors

Encapsulated strand tendons, also known as double-corrosion protected (DCP) anchors in the United States, were used to provide protection against corrosion in the harsh marine environment. Other corrosion protection measures that were followed are discussed in the following section. The DCP strand tendons consisted of two levels of corrosion protection. The tendons were encapsulated in a grout-filled, corrugated plastic tube. For ease of installation, the tendons were grouted inside the encapsulation after the tendons had been placed inside drill holes.

Anchor Head Assembly

Galvanized, steel thread bar couplers were installed on the existing deadman anchor bars, and the tie rods were extended using epoxy-coated thread bars. Galvanized anchor nuts and plates were used at the face of the new sheet pile for corrosion protection. The couplers and thread bars were embedded in a steel form box filled with concrete.

For the installation of the new tiebacks, holes were cut in the old sheet pile to match the outside diameter of a pipe sleeve. The annular space between the pipe sleeve and the old sheet pile was plugged to prevent soil from washing out. After installation of the new sheet piles, holes were cut in the valley of the sheet piles to provide access for anchor installation. A 20-cm-diameter, extra strong steel pipe sleeve was installed through the two holes to sufficient length to prevent existing backfill from washing out. Restressable anchor head assemblies with fully sealed and welded grease caps were used for corrosion protection. In addition, a steel form box was welded to the face of the sheet pile, and the box was filled with concrete to provide corrosion protection for the anchor head, as shown in Figure 9. A concrete bond breaker was used on the anchor head cover and inside the form box to allow future access to the restressable anchorage without damage.

Fig. 8. Typical section through the bulkhead.

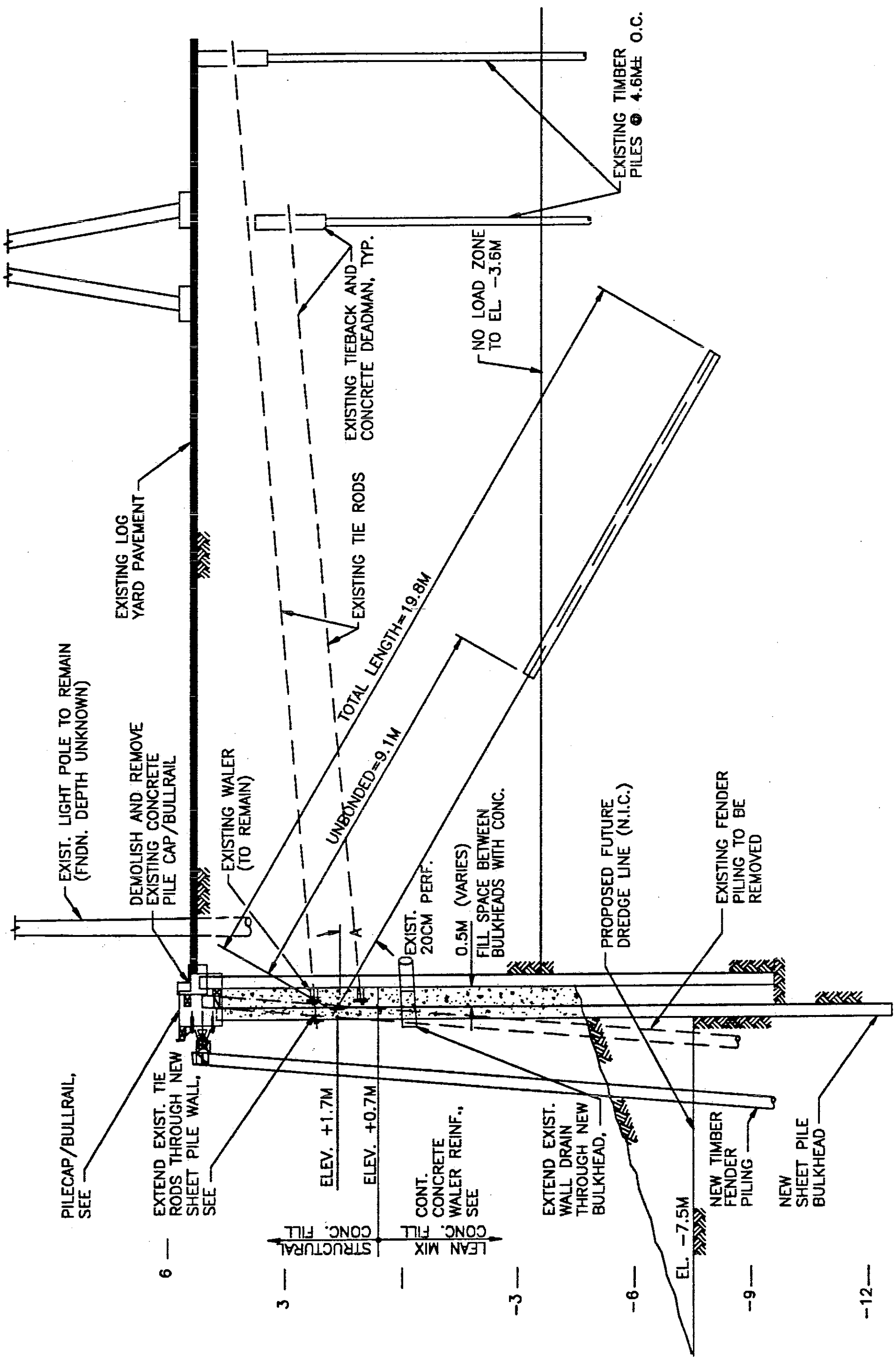
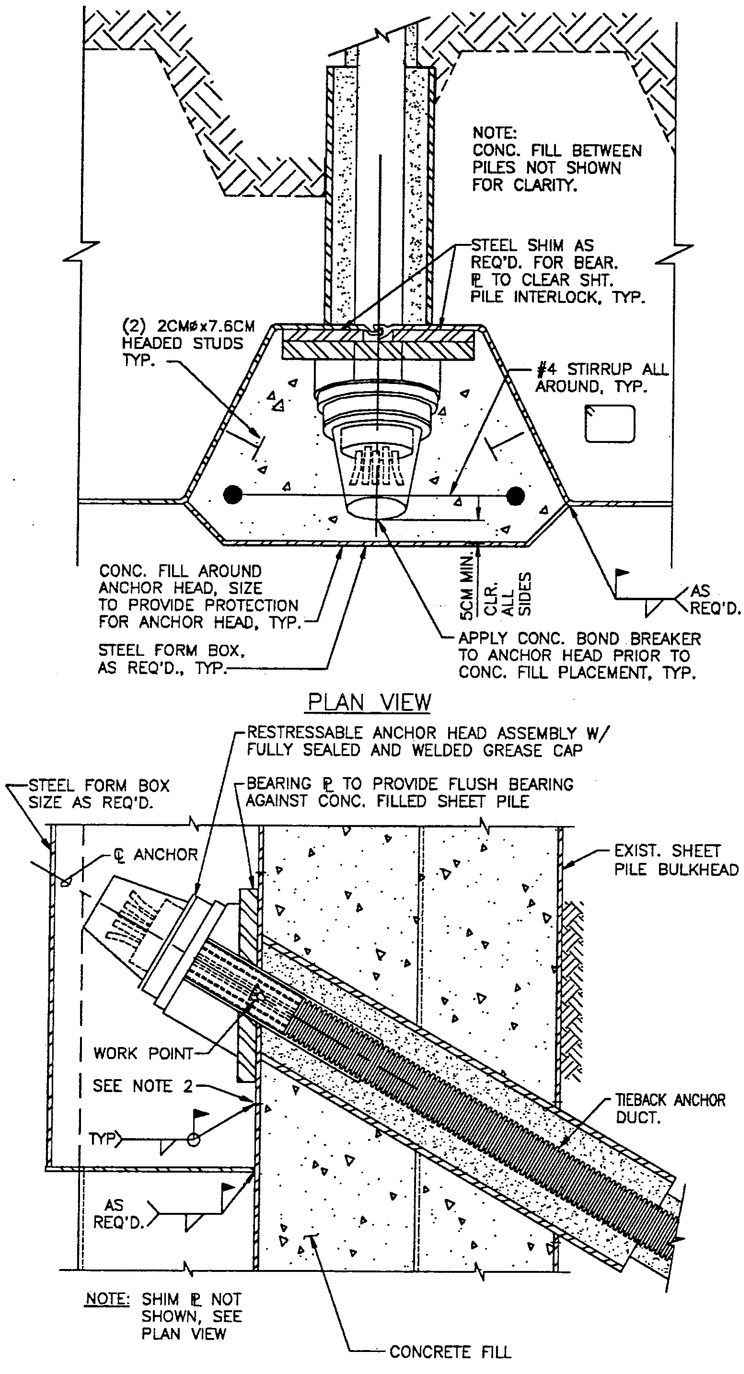


Fig. 9. Anchor head assembly at the bulkhead.



Anchor Installation and Testing

A total of 41 anchors were initially installed for Phase 1; however, 12 anchors failed the proof test and were subsequently replaced with supplemental anchors. The allowable bond stress in dense sand for Phase 1 was estimated to be 102 kN/m. The allowable design bond stress was reduced to a more conservative value of 80 kN/m for Phase 2. The reason for the relatively high failure rate is uncertain, but was partially due to anchor installation and grouting difficulties. For Phase 2, 63 anchors have been installed. Approximately 75 percent of the anchors have been tested to date (April 3, 1996) and all have passed the proof and performance testing.

Innovative installation techniques were used by the contractors for Phases 1 and 2 to eliminate the need for a barge. The anchors were drilled using a Klemm MR704 hydraulic mini drill rig that was mast-mounted on the digging arm of a hydraulic track hoe. The drill casing was advanced into the soil using the hydraulic rotary method. As the drill hole advanced, 1.5-m-long segments of 18-cm-diameter, threaded casing were added. The end of the casing string was equipped with teeth that allowed the casing to advance while directing cuttings up and outside of the drill casing. This process was facilitated by using water under relatively high pressure for flushing the cuttings. After the drill hole had reached the required depth, the casing was filled with grout, tendons were inserted, the encapsulation tube was tremie filled with grout, and the hole was pressure-grouted as the casing was pulled out. After the casing was pulled up to the top of the bond length, pressure grouting was discontinued. Anchor installations could be accomplished only at night and during periods of low tide.

SUMMARY AND CONCLUSIONS

An existing deteriorating steel sheet pile bulkhead was successfully rehabilitated with a composite steel/concrete wall, retained with permanent DCP pressure grouted tieback anchors. The restressable anchorages were located near MLLW elevation, and encased in a steel form box filled with removable concrete for corrosion protection. The wall was designed to retain cohesionless hydraulic fill that is subject to liquefaction during a strong ground motion earthquake.

The wall was designed for single and multiple levels of anchors and to resist static, dynamic, and liquefied soil loads. The dynamic (earthquake) load was determined by taking 30 percent of the static force and distributing the load increment uniformly. The liquefaction load was applied as a fluid density in addition to the overburden pressure.

The anchors were proof tested to 130 percent of the maximum load, which was the greater of either the static, dynamic, or liquefaction load. Performance test anchors were loaded to 200 percent of the allowable static bond stress times the bond length, and all tiebacks were locked-off at the static load. The anchor ultimate bond value in dense sand (160 kN/m) corresponded to the value recommended by the PTI.

Innovative techniques were used to install the anchors which eliminated the need for a barge. The anchors were drilled using a Klemm MR704 hydraulic mini drill rig that was mast-mounted on the digging arm of a hydraulic track hoe. After the drill hole had reached the required depth, the casing was filled with grout, tendons were inserted, the encapsulation tube was tremie filled with grout, and the hole was pressure-grouted as the casing was pulled out. Numerous methods were attempted in order to prevent the sand from heaving into the casing. The selected procedure involved internally flushing the cuttings with a large capacity water pump and high air pressure.

Rehabilitating the bulkhead with concrete encased permanent tieback anchors was an innovative and cost-effective solution under conditions of limited access and because a conventional deadman anchor system was not possible.

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4-10-96/TIEBACK.ANC/TMG-lkd/dgw