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SYNOPSIS: A 1500-m long anchored bulkhead with a height of 20 m exhibited a localized failure in the form of broken and overstressed anchors several months after construction. The wall had not yet been subjected to its full design loadings. The soil conditions in the failure area differ from those occurring along the rest of the quay wall by the presence of a very soft silt/clay layer, and during construction the wall had been strengthened in this area. Post-failure analysis of the anchored bulkhead indicated that the primary cause of the failure was overly optimistic design assumptions for the strength of the silt/clay layer and mobilization of passive pressure. The effects of certain construction methods employed and the settlement of the silt/clay were contributing factors in the failure. A relieving platform constructed one year after the failure was designed for the original undrained strength of the silt/clay, without taking into account the effects of soil consolidation and strength gains which had occurred.

INTRODUCTION

A large quay wall was planned as the central part of a new harbor and marine development in the northern Arabian Gulf. Three separate geotechnical investigations of the harbor area were conducted in the early 1980's, two of which concentrated on the planned quay wall. Taken collectively, these investigations provided sufficient data for the design of the wall; however, certain design/construction techniques (i.e., staged construction) would have required additional data. The soils occurring over most of the 1500-m length of the wall were competent sands and stiff clays, but a thick layer of very soft silt/clay existed over the last 250 m of the wall.

The original design of the wall consisted of circular sheetpile cells (cofferdam), a conservative (and expensive) approach to the problem of weak soils. However, the contract for construction of the wall was awarded based on an anchored sheetpile bulkhead alternate design submitted by the successful bidder. During installation of the sheetpiles, it became apparent that the final 115 m of the wall required strengthening and a fourth geotechnical investigation was conducted. The design was then modified in this area by adding H-piles driven on the inside of the sheetpiles and enlarging the anchor wall.

Approximately three months after achieving final fill elevation, but before the bulkhead was subjected to the design surcharge and berthing forces, a localized failure occurred approximately 75 m from the end of the wall as indicated on Figure 1. Upon investigation, five tie rods, including three in a row, were found broken. Following an additional geotechnical investigation and load tests of the anchor system, the wall was repaired by constructing a pile-supported relieving platform over the final 184 m one year after failure.

SITE STRATIGRAPHY AND SOIL PROPERTIES

Three geotechnical investigations conducted in the harbor area from June, 1981 to June, 1982 included six borings in the final 350 m of the planned quay wall: four borings spaced at 100-m intervals along the face of the quay wall and two borings 75 and 110 m behind the wall face. A fourth investigation was conducted in September, 1983, immediately following driving of the sheetpiles, and consisted of three borings and five Dutch cone penetrometer tests. The borings were drilled from jack-up barges to depths of 15 to 30 m below the seafloor (elevation -6 ±). The investigations revealed a very soft silt/clay layer along the final 250 m of the quay wall, extending from the seafloor to elevation -11.9 to -12.4 m ISLW, trending slightly deeper towards the end of the wall, as shown in Figure 2. This layer

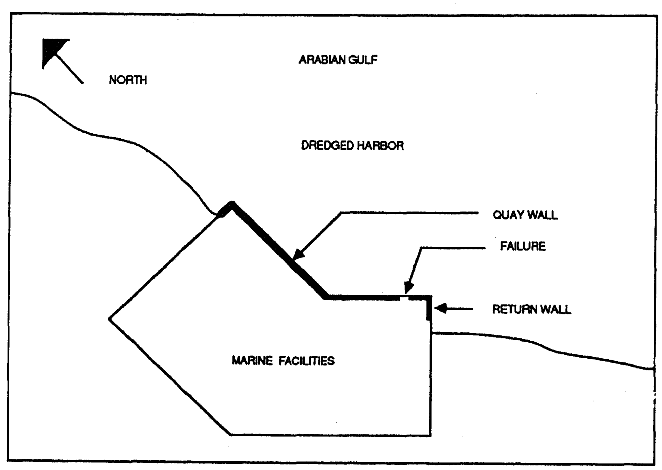

Figure 1. Plan View of Quay Wall

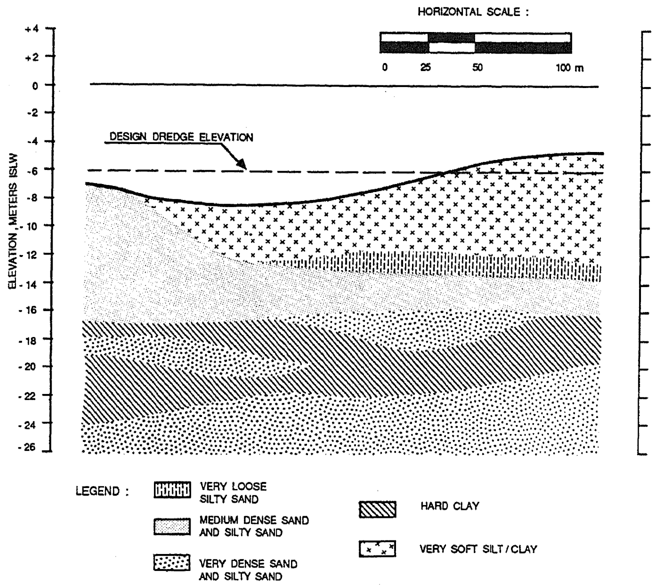

Figure 2. Soil Profile of Final 350 m

is of particular importance to the stability of the wall.

The soil layer was described in the four geotechnical reports as both a silt and a clay with sand pockets and shells. Based on Liquid and Plastic Limits, the soil appears to be a borderline soil, classified as either a silt or lean clay (ML or CL) in the Unified Soil Classification System. The soil exhibited carbonate contents of 25 to 65% and water contents well in excess of the Liquid Limit (Liquidity Index of 2 to 4.8), suggesting that the soil may be sensitive (Wu, 1976).

Results of the laboratory miniature vane and Torvane strength tests and laboratory unconsolidated-undrained triaxial compression tests for the silt/clay layer are plotted as a function of depth on Figure 3. The undrained strength increases from 4 to 5 kPa at the top of the of the layer to 16 kPa at elevation -12 m. These low shear strengths are consistent with field observations that the boreholes were advanced the first 1 to 3 m by the weight of the drill rods, and that during the 1983 investigation the legs of the jack-up barge were pushed into the seabed rather than lifting the barge's deck out of the water. The shear strength is 4 kPa at the surface, increasing with depth corresponding to a c/p ratio of about 0.30, indicating slight, constant preconsolidation.

The soil layers underlying the silt/clay layer are sand and silty sand of increasing density from elevation -12 to -18.6 m; hard (overconsolidated) clay to elevation -21.5 m; and very dense silty sand to the maximum depths explored.

BULKHEAD DESIGN

The quay wall was designed as an anchored bulkhead with a coping beam and facia panel. Two methods for analyzing anchored bulkheads are commonly employed, with the difference being the assumption of the support of the bottom of the sheetpile. The free earth support method assumes that the bottom is simply supported, free to rotate but not translate. The fixed earth support method assumes a fixed support, with no rotation nor translation (Terzaghi, 1943; Tschebotarioff, 1973). Anchored bulkhead failure usually falls into one or more of four types: anchor or tie rod failure, flexural failure of the sheetpiles, toe failure, or a general (slope) failure (Daniel and Olson, 1982).

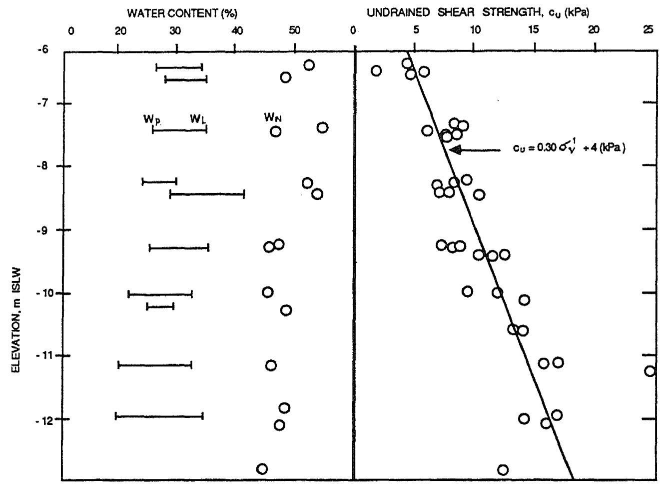

Figure 3. Silt/Clay Properties (After Ladd, et al, 1985)

The design documents indicate that the analysis of the sheetpile wall was conducted using the fixed-earth support method and generally concurring to European industry recommendations (EAU, 1980). The type and depth of sheetpile varied along the length of the wall, governed by the soil conditions. The design loadings included a surcharge of 10 kN/m^2; a mooring load (bollard pull) of 200 kN at ±30° every 16 m; and berthing forces of a 1000 DWT vessel at a speed of 0.3 m/s. Parameters used in the analysis included the harbor bottom at elevation -6.2 m with no consideration for scour or overdredge; low tide at elevation -0.12 m; and the water level behind the wall at elevation +0.84 m. The allowable stress in the steel was 60 percent of the yield strength for both the anchor rods and the sheetpiles.

The design soil profiles for most of the 1500-m bulkhead were in good agreement with the stratigraphy evident from the geotechnical investigations, and the properties chosen for the sands and stiff to hard clay layers were conservative. The performance of the bulkhead was satisfactory except for the final 200 m. The remainder of this paper will concentrate on anchor failure and flexural failure of this portion of the wall.

Original Design

The design soil profile and soil properties for the last 250 m of the bulkhead is shown in Figure 4. The analyses yielded a required sheetpile penetration to elevation -16.26 m to achieve the fixed earth condition, an anchor force of 238 kN/m of wall, and a maximum bending moment of 688 kN-m/m of wall. For anchor rod spacing of 2.55 m, the computed force per anchor rod was 62.9 tons and the Factor of Safety against tie rod failure was 1.82. The Factor of Safety against flexural failure was 1.80.

Modified Design

Following installation of the sheetpiles and the 1983 geotechnical investigation, the design soil profile was changed by extending the depth of the silt/clay layer for the last 75 m of the wall. Reanalysis yielded required tip penetration to elevation -17 to -18 m, indicating that the installed sheetpiles had insufficient penetration to achieve the fixed earth support assumption. The Factor of Safety against flexural failure was reduced to 1.52, less than the required 1.67 safety factory for steel members. The Factor of Safety against tie rod failure of 1.70 was considered adequate.

The design was modified over the final 75 m to include H-piles driven on the inside of the sheetpiles to elevation -20.0 m. It was assumed in design that the H-piles would extend the effective length of the sheetpiles, providing the required fixation, and would carry 21 percent of the bending moment. The Factor of Safety against flexural failure for the sheetpiles was recomputed as 1.82. The Factor of Safety for the H-piles was 1.40, but was recorded as greater than 2.5 due to a calculation error.

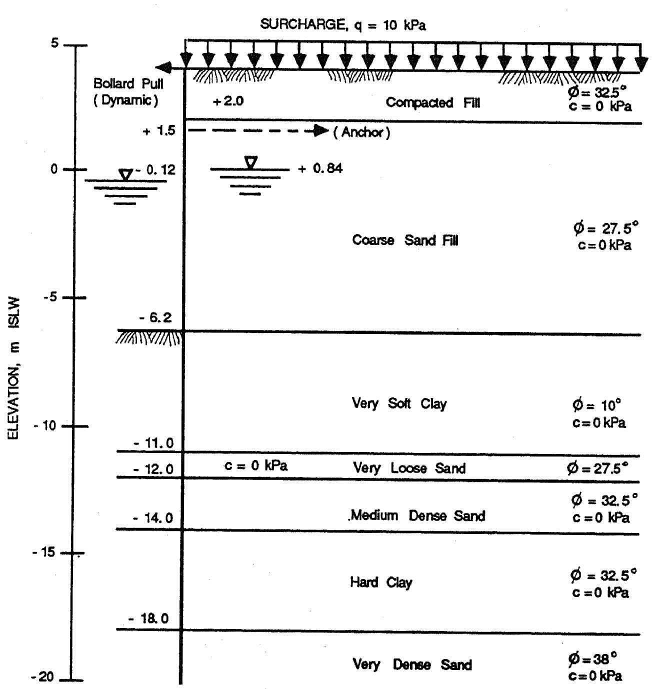

Figure 4. Original Design Soil Profile

QUAY WALL CONSTRUCTION

The harbor had been dredged to elevation -6.2 m, and 20.25-m long British Frodingham FR-5-DR sheetpiles, BS4360 Modified Grade 50B, were vibrated and driven to a tip elevation of -16.25 m. Concurrent with the sheetpile installation, a sand fill (Stage 1) was placed to elevation +1.0 m some 20 m behind the wall, sloping to within 1 m of the dredged bottom as it neared the sheetpile, as shown in Figure 5. A sheetpile anchor wall was installed in this Stage 1 fill, and the anchor rods placed. H-piles were driven between the sheetpiles and the wale beam (on the inside of the sheetpiles) to elevations -19.0 and -20.0 m. Due to material availability constraints, two types of H-piles were used: 254x254x71 piles installed on 0.85-m centers for the first 25 m and W 10x89 piles installed on 1.70-m centers for the remaining 50 m. Additional hydraulic fill (Stage 2) was then placed to elevation +2.25 m, with the remaining sand fill (Stage 3) to elevation +3.8 m compacted by vibratory rollers. Precast concrete mats to elevation +4.0 m provided the final working surface of the quay wall.

The 63-mm diameter steel anchor rods (yield strength of 38.3 kN/m²) were installed at elevation +1.5 m at every third sheetpile, a spacing of 2.55 m. The anchor wall, 34 m behind the main sheetpile wall, consisted of 3.15-m long FR-2N-DR sheetpiles of Grade 43A steel driven to alternating tip elevations of +0.1 m and -0.9. Because the Stage 1 fill sloped downward, the anchor rods had a free suspension of 15 to 20 m and were allowed to sag. The built-in sag varied from anchor to anchor, but was of the magnitude of about 0.5 m.

At the end of the quay, a return wall was constructed perpendicular to the main sheet-pile wall. This return wall was tied back to an anchor wall 34 m away. Thus, the 34-m by 34-m corner of the quay wall was crisscrossed by two anchor systems (walls and rods). The main wall sheetpiles were installed in August, 1983, followed by Stage 1 filling in Sept.,

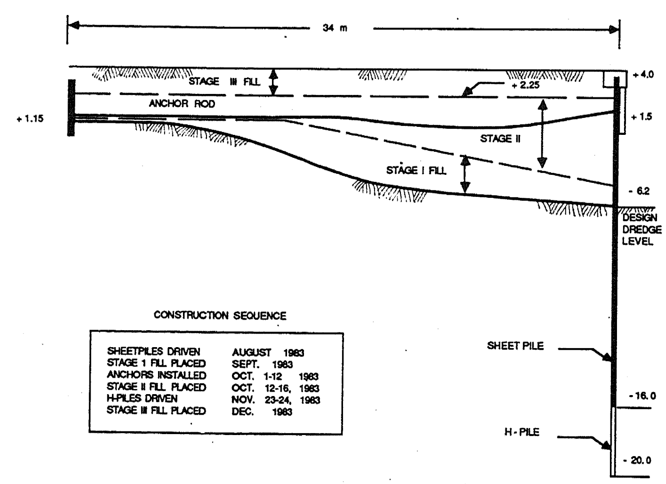

Figure 5. Profile of As-Built Bulkhead

1983. The anchor wall and anchor rods were installed October 1 to 12, 1983. Stage 2 hydraulic fill was placed October 12 to 16, 1983, and some Stage 3 fill was placed October 28 to 31, 1983. The H-piles were driven November 23 to 24, 1983, and placement of fill and concrete mats was completed in late December, 1983.

FAILURE AND INVESTIGATION

On March 3, 1984, before the wall had been subjected to surcharge or berthing forces, a localized failure occurred 75 m from the end of the wall. The concrete pads behind the sheetpile wall dropped approximately 10 cm, and the top of the bulkhead moved outward 50 to 70 mm along a 25-m length of the wall. Project records are incomplete regarding the extent and timing of the settlement of the concrete mats, but it appears that the settlement was confined to the area of failure and coincided with the outward movement of the wall.

Excavation to the anchor rod level revealed that three anchors in a row had failed at the connection to the main sheetpile wall, a result of one of the connection ring plates fracturing in each instance. Ring plate connections of other anchors nearby appeared to be rotated and eccentrically loaded (Wiltsie, 1985). Further investigation revealed that two other anchors had failed at the ring plate connection.

Full-Scale Laboratory Tests

To test the overall anchor capacity, the tie rod/ring plate connection system was duplicated in the laboratory using the same type of materials used in the field. The tested plates came from three sources: new plates, new plates from the site stock, and used plates obtained from the quay wall. The anchor rods were loaded in tension to failure. Four different loading conditions were used on the ring plates such that the anchor rods were either aligned, eccentric, rotated, or catenary. The results of the tests are shown in Table 1 and indicate that the loading conditions had no effect on the yield load but did reduce the ultimate capacity of the anchor system.

Table 1. Full-Scale Anchor Rod/Ring Plate Connection Tests

TEST NO.	PLATE ORIGIN	ALIGNMENT	YIELD LOAD, KN	FAILURE LOAD, KN	FAILED ELEMENT
1	M	ALIGNED	1310	1867	ROD
2	S	ALIGNED	1213	1744	ROD
3	Q	ALIGNED	1310	1647	ROD
4	M	ECCENTRIC	1228	1802	ROD
5	S	ECCENTRIC	1309	1586	ROD
6	Q	ECCENTRIC	1224	1529	CONNECTION
7	M	CATENARY	1340	1860	CONNECTION
8	S	CATENARY	1300	1571	CONNECTION
9	Q	CATENARY	1288	1644	CONNECTION
10	M	ROTATED	-	2006	CONNECTION
11	S	ROTATED	-	1782	CONNECTION
12	Q	ROTATED	-	1599	CONNECTION

M - MANUFACTURER S - SITE STOCKPILE Q = QUAY WALL

In-Situ Measurements of Anchor Rod Forces

Two months after failure, the forces in the anchor rods were measured by means of hydraulic jacking behind the anchor wall. Forces in the 800 to 1000 kN range were recorded at several locations in the final 100 m of the wall, well in excess of the design anchor force. However, as these measurements occurred after system relief (outward wall movement, excavations and tie rod replacements) occurred, the authors believe that the anchor rod forces at failure were probably higher than the measured forces, and that these tests are significnat only in that forces much higher than the design level were measured.

Further Geotechnical Investigation

A fifth geotechnical investigation was conducted in August, 1984 consisting of 5 borings, 12 Dutch cone penetration tests, and 11 Piezocone penetration tests. The results show generally increased strength (maximum of 38 kPa) and decreased water contents in the silt/clay layer. Laboratory consolidation and permeability tests indicated a permeability (k) of 3×10^{-8} cm/sec and a coefficient of consolidation (C_v) of 3×10^{-3} to 8×10^{-4} cm^2/sec.

FAILURE ANALYSIS

Although the Factors of Safety for the wall appeared to be adequate, the design of the wall was based on optimistic assumptions regarding the properties of the silt/clay layer, the development of passive pressures, and the effectiveness of the H-piles.

Properties of the Silt/Clay

The initial strength parameters selected for the silt/clay during design were cohesion (c) of 15 kPa and an angle of internal friction (ϕ) of 10°. Settlement or compression of this layer was apparently not considered in design.

The soil properties in Figure 3 indicate that the soil is cohesive, with $k = 10^{-6}$ to 10^{-8} cm/sec (Abbs, 1985) or C_v = approximately 4×10^{-3} cm^2/sec (Ladd, et al, 1985). The layer would be expected to exhibit an undrained (ϕ = 0) response to loading with initial shear strength (cohesion) of 7 to 10 kPa. The inclusion of a frictional component overestimated the intial strength of the layer by a factor of 5.

Passive Pressures

Earth pressures appear to have been calculated using the Coulomb equations, and included wall friction on both the active and passive sides of the sheetpile. (The inclusion of wall friction increases the passive pressures and decreases the active pressures.) If the angle of wall friction (δ) is greater than $\phi/3$, Coulomb's equations may significantly overestimate the passive pressures (Terzaghi and Peck, 1967). The values for δ used in design ranged from 40 to 62 percent of ϕ.

Development of passive pressure requires outward movement of the wall. To develop full passive pressure in the dense sands near the toe of the wall would require outward movements of 20 to 25 cm, 2 percent of the embedment length (U.S. Navy, 1982; Lambe and Whitman, 1969). However, the fixed earth support method used in design assumes toe fixation with virtually no outward movement. Thus, a factor of safety is commonly applied to the passive forces. The design computations used full passive pressures throughout the effective embedment length. Combined with the inclusion of wall friction, the design used an unconservative estimate for passive pressures (Luscher, et al, 1985).

H-Pile Effectiveness

The modified design assumed the addition of H-piles would extend the effective length of the sheetpiles and resist some of the bending moment. The H-piles were driven on the inside of the sheetpile and had a greater section modulus and penetration depth. Because the H-pile is stiffer than the sheetpile it provides fixation only if on the outside of the sheetpile; otherwise (as in this case), the sheetpile will move away from the H-pile and the fixed earth support assumption will not be achieved (Ladd, et al, 1985).

The H-piles are effective in resisting bending moment by reducing the soil forces acting on the sheetpile at the location of the H-pile. Thus the amount of bending moment reduction is a function of the H-pile spacing and width rather than a ratio of section moduli as assumed in design.

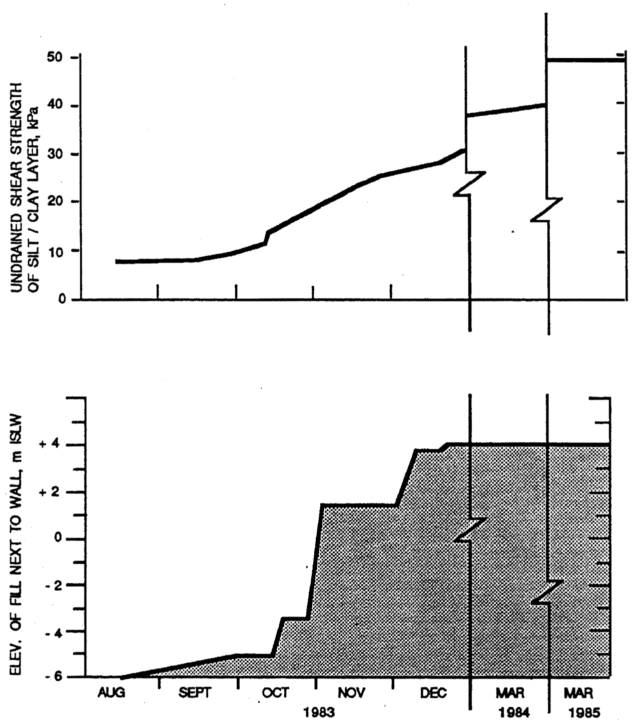

Figure 6. Strength of Silt/Clay Layer versus Time

Table 2. Effect of Strength and Passive Pressure Assumptions

CASE	DESCRIPTION	FS VS ANCHOR FAILURE	FS VS BEND. MOM. FAILURE
1	ORIGINAL DESIGN	1.81	1.76
2	MODIFIED DESIGN	1.69	1.80
3	CASE 2, STRENGTH CORRECTION	1.29	1.07
4	CASE 2, PASSIVE PRESSURE CORR.	0.81	1.23
5	CASE 2, BOTH CORRECTIONS	0.58	0.67

Design Evaluation

The original and modified designs were re-analyzed by the authors using an IBM-PC version of BMCOL (A Program for Finite-Element Solution of Beam Columns with Nonlinear Supports) (Matlock and Haliburton, 1964). The H-pile and the sheetpile were modeled as separate entities whose deflections had to match at certain points.

The designs were analyzed using the original design assumptions and yielded results similar to those in the design calculations. The effects of the strength of the silt/clay and passive pressure/fixed earth support assumptions were also analyzed. The mobilization of passive pressure was modeled by iterative soil-structure interation using Q-W (load-deflection) curves. The results, shown in Table 2, indicate that the wall was underdesigned and would be expected to fail.

Failure Model

Fill was placed behind the quay wall from September to December, 1982, and the strength of the silt/clay layer increased with consolidation, as illustrated in Figure 6. This period is the most critical in determining the anchor rod forces: although the silt/clay continued to increase in strength after filling was complete, the wall deflections

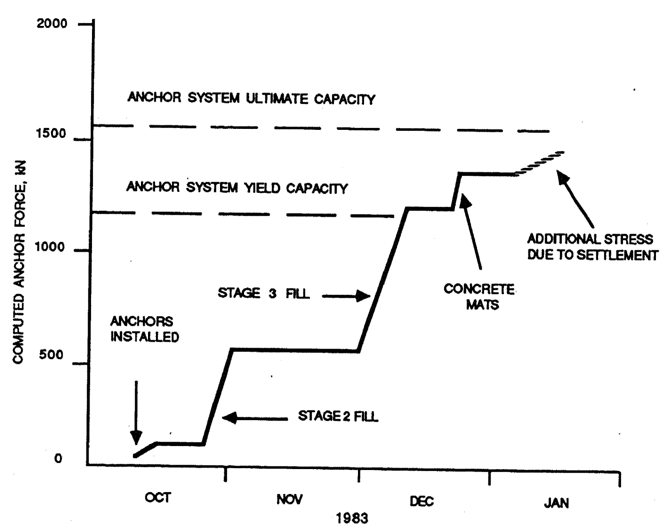

Figure 7. Computed Anchor Rod Forces versus Time

(and thus the anchor rod extensions and forces) could not be reduced due to the sand fill. Figure 7 illustrates the increase in anchor forces as a function of time, taking into account filling levels and strength gains.

The initial tension in the tie rods varied due to construction techniques. The driving of the H-piles which forced the sheetpile wall outward and the initial sag of the tie rods prior to fill placement introduced the greatest degree of variability. Subsequent tightening of rods 190 to 210 after fill placement introduced additional stresses on those rods (the failed rods were 201, 206, 207, 208, 213). Misalignment of the anchor rod connections resulted in variability of the ultimate force each rod could withstand. As opposed to design assumptions, soil properties are not uniform, and therefore pressures on the wall varied as well. As settlement occurred with time, the downward force added additional stress on the rods which were already near their failure threshold, and five broke.

As the wall moved forward, additional passive pressures were mobilized, and pressures acting on the active side of the wall were reduced. The H-piles contributed to stabilizing the wall as they were attached to the sheetpile by the wale beam. However, it was extremely fortunate that a progressive failure did not occur.

RELIEVING PLATFORM

A relieving platform was selected for repair of the wall, with design beginning in January, 1985, and construction from March to August, 1985. The original failure was believed to be primarily due to overestimation of the strength of the silt/clay layer, and therefore, conservative estimates of the initial undrained strength of the clay ($c = 7.2$ kPa, $\phi = 0°$) were used for the relieving platform. The resulting design called for a 184-m long pile-supported structural deck, 15 to 18 m in width, as shown in Figure 8.

This approach failed to recognize the strength increases in the silt/clay due to consolidation under the fill. The shear strength of this layer is estimated to be $c = 45$ to 50 kPa, $\phi = 0°$ one year after the fill had been in place. The overall relieving platform was thus overdesigned, but the structural deck, designed for normal working loads, precluded using this portion of the quay wall for heavier than normal loads.

CONCLUSION

A 1500-m long anchored bulkhead experienced localized anchor rod failures several months after construction. The failure was due to overly optimistic design assumptions regarding the strength of a thick very soft silt/clay layer and the mobilization of passive pressures. Other factors, such as sheetpile toe fixation, driving of H-piles to strengthen wall, and settlement of clay layers were contributing factors. The wall was repaired by constructing a relieving platform one year after failure. The platform was designed for the original undrained strength of the clay layer without taking into account the effects of soil consolidation and strength gains which had occurred.

ACKNOWLEDGEMENTS

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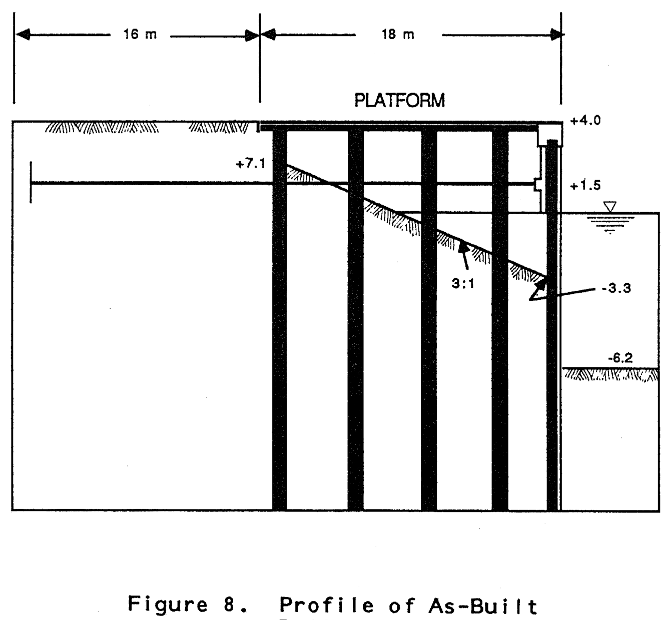

Figure 8. Profile of As-Built Relieving Platform

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