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Failure of an Anchored Sheetpile Bulkhead

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SYNOPSIS An anchored steel sheetpile bulkhead was constructed in soft organic silt and clay. The bulkhead failed when the anchors ruptured during dredging in front of the bulkhead. The construction and failure of the bulkhead are described. Analyses were performed to investigate the cause of the failure. The major factors which contributed to the failure were: 1) failure to design for the lowest tide condition, 2) use of design soil strengths which were too high, 3) prestressing of the anchor system which resulted in increased anchor loading due to soil arching, and 4) bending stresses induced in the anchors by settlement and equipment loading. Particular emphasis is placed on the effects of soil arching on the anchor loading.

INTRODUCTION

During construction of the Merrill Marine Terminal facility in Portland, Maine, an anchored steel sheetpile bulkhead failed when the tied-back anchorage system ruptured near the sheetpile bulkhead. The authors were engaged to investigate the cause of the failure. This paper describes the failure and the major factors which contributed to the failure.

SITE AND PROJECT DESCRIPTION

The Merrill Marine Terminal facility is located on the Fore River in Portland, Maine. The facility consists of a wharf for shipping and receiving bulk cargo and storage areas for bulk cargo. The wharf is located on the tidal mud flats along the river bank, as shown in Fig. 1. As originally designed, the wharf was to consist of a 600-ft section and a 300-ft section. The sheetpile bulkhead failure described in this paper occurred in the 300-ft section. The 300-ft wharf was not completed and was eliminated from the project.

The mud flats landward of the wharf were to be filled in and used as a bulk storage area. The mud flats are underlain by 40 to 70 ft of very soft organic clayey silt and sensitive soft to medium stiff silty clay. Wick drains were installed in the area landward of the wharf to accelerate the consolidation of the soft silt and clay under the new fill.

The wharf was designed as a concrete pile-supported deck with a steel sheetpile bulkhead located at the landward edge of the deck, as shown in Fig. 2. The sheetpile bulkhead was to be anchored at the top by batter piles cast into a concrete beam along the edge of the wharf. After installation of the sheetpiles and prior to construction of the concrete wharf, the designers decided to dredge out the organic silt in front of the sheetpile bulkhead and replace it with a sand and gravel berm in order to increase the passive soil resistance in front of the bulkhead. A temporary tied-back anchorage system was installed to support the bulkhead during this operation. The bulkhead failed during the dredging when the tied-back anchorage system ruptured near the bulkhead.

SUBSURFACE SOIL CONDITIONS

The subsurface soil profile at the site consists of the following strata, proceeding downward from the ground surface: very soft organic clayey silt, soft to medium stiff silty clay, stratified silty fine sand and clay, glacial till and bedrock.

The organic clayey silt has a natural water content of 50 to 80%, a liquid limit of 60 to 75%, and a plastic limit of 30 to 40%. It contains varying amounts of shell fragments, organic matter, and occasional lenses of silty fine sand. The undrained shear strength of the organic silt in the mud flat areas prior to filling is in the range of 150 to 300 psf. Figure 3 shows the undrained shear strength profile from UU triaxial tests on samples from the mud flat areas.

The silty clay is a glaciomarine deposit with a natural water content of 25 to 50%, a liquid limit of 25 to 40%, and a plastic limit of 15 to 25%. It contains occasional thin layers of silty fine sand. The upper portion of the clay above about El -40 MLW has been preconsolidated by desiccation. Figure 4 shows the undrained shear strength profile from UU triaxial tests on samples from the mudflat areas throughout the site. The silty clay is

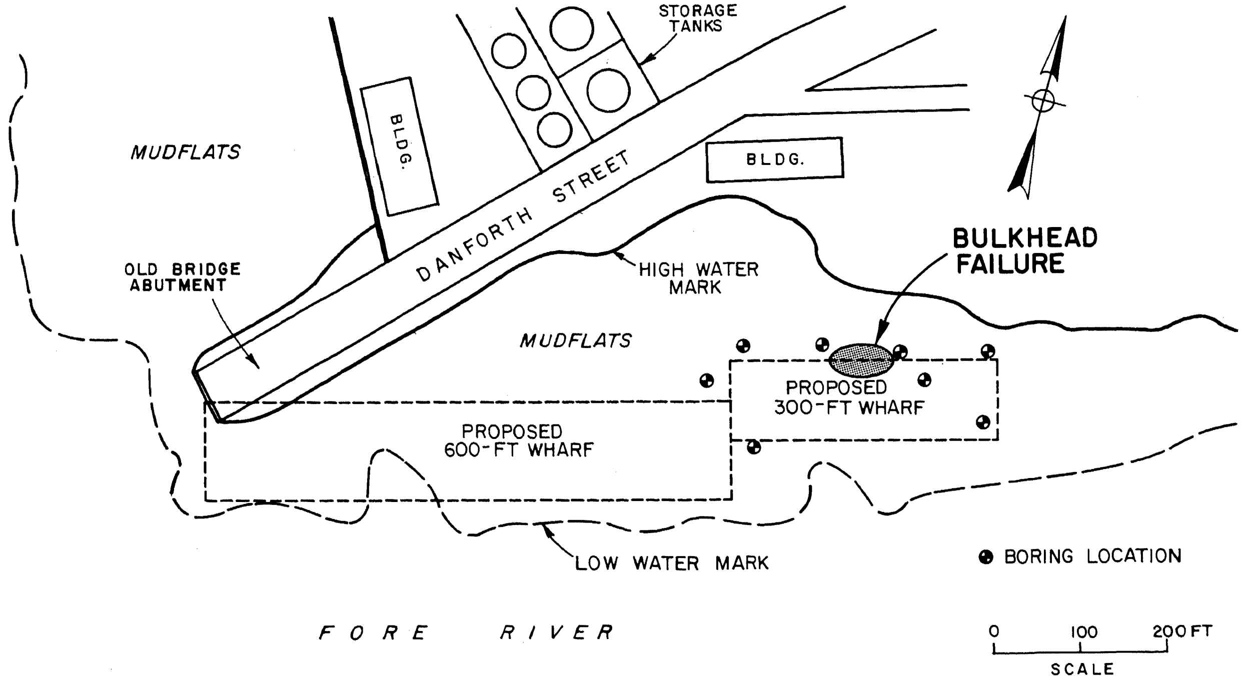

Fig. 1. Project Site

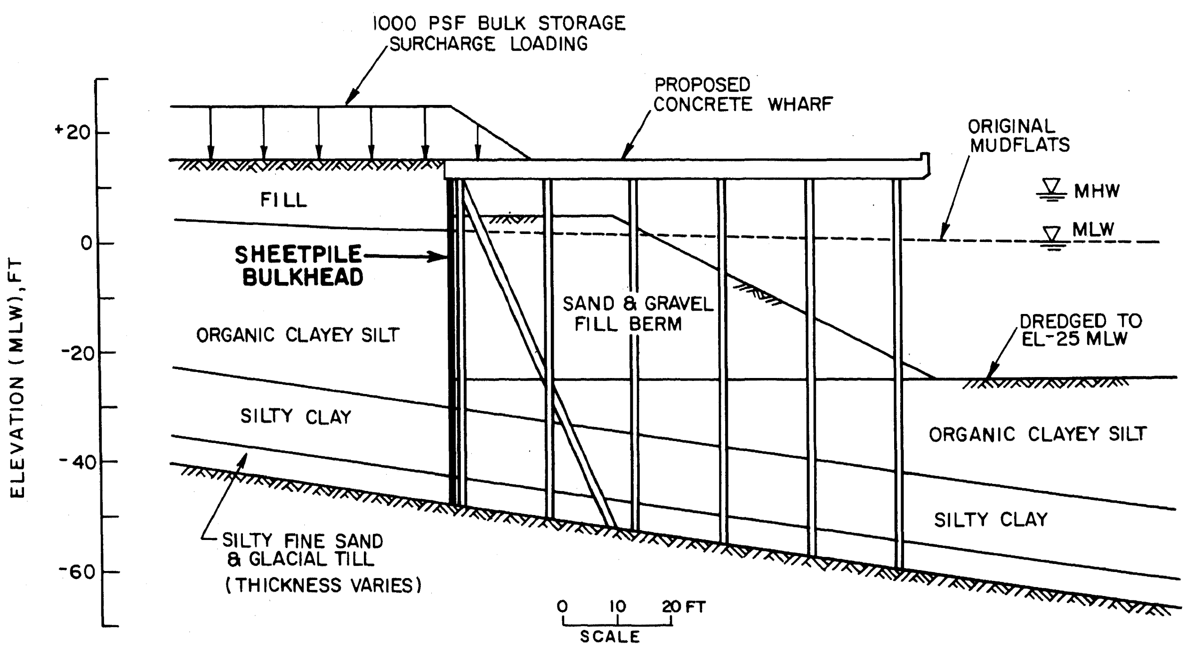

Fig. 2. Cross Section of Proposed 300-ft Wharf

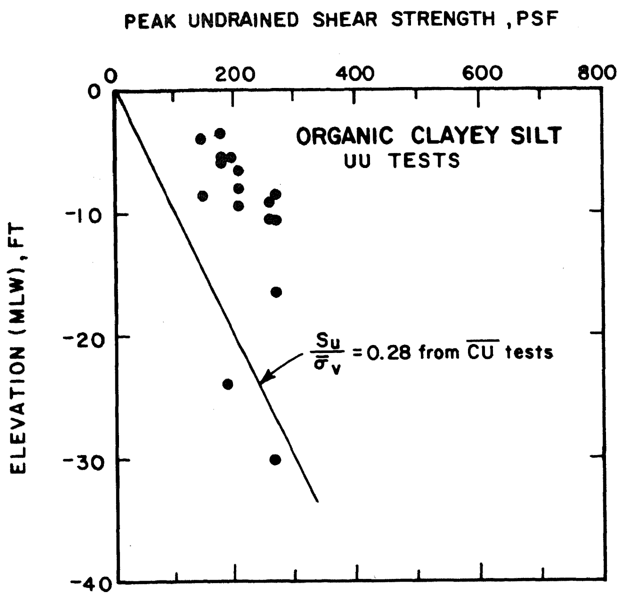

Fig. 3. Organic Silt Undrained Strength Profile

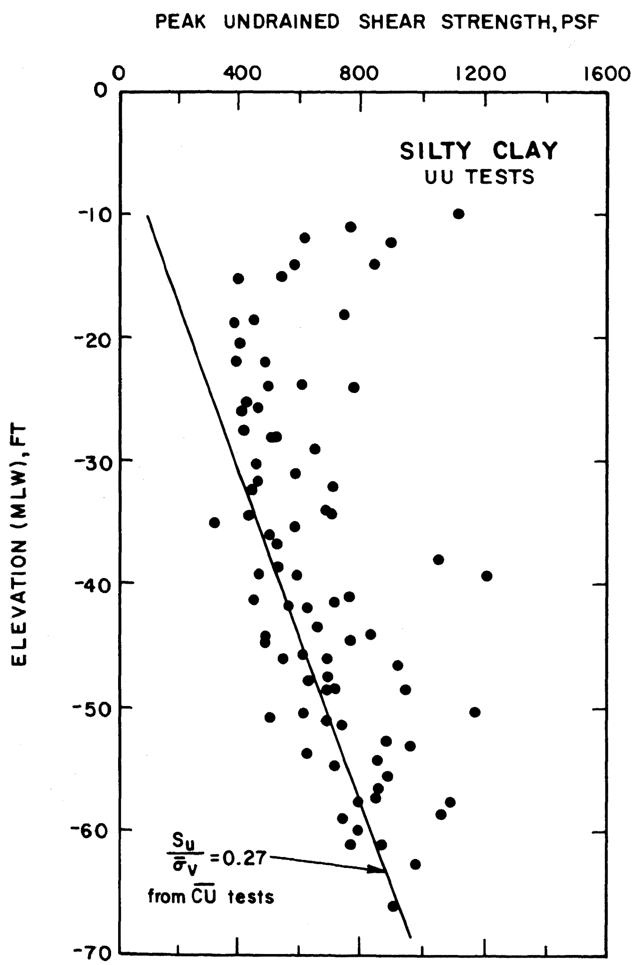

Fig. 4. Silty Clay Undrained Strength Profile

sensitive and is susceptible to significant loss of strength due to disturbance.

The frequency of silty fine sand layers increases toward the bottom of the silty clay stratum, and, in many areas, the lower 5 to 15 ft of the stratum consists of stratified silty fine sand and clay or predominantly silty fine sand. A thin layer of dense gravelly glacial till, typically no more than 5-ft thick, overlies the bedrock in some areas and is absent in others.

Along the sheetpile bulkhead for the 300-ft wharf, the thickness of the organic silt varies from about 25 ft to 30 ft. The thickness of the silty clay stratum below the organic silt decreases from about 30 ft at the west end of the bulkhead to zero at the east end, as the bedrock surface rises from about El -60 MLW at the west end to El -40 at the east end. At the east end of the bulkhead, the organic silt is underlain by about 12 ft of silty fine sand overlying bedrock. The soil profile at the section of the bulkhead where the failure occurred is shown in Fig. 6. The locations of the borings performed in the vicinity of the 300-ft wharf are shown in Fig. 1.

TIED-BACK ANCHORAGE SYSTEM

The temporary anchorage system was tied back to a continuous sheetpile anchor wall located 110 ft behind the sheetpile bulkhead. Beam sections (tiebeams) were used to tie the sheetpile bulkhead to the anchor wall instead of tierods because the beams would have a higher salvage value when the temporary anchorage system was removed. The tiebeams consisted of W8x28 sections at the bulkhead and the anchor wall with larger sections in between. The beam sections were connected by welded splices. The tiebeams were spaced at 18-ft intervals along the bulkhead. The wales consisted of twin HP14x73 sections and were butt welded to form a continuous beam. The sheetpile bulkhead was an Arbed BZ450 section (roughly equivalent to a PZ38).

The construction sequence was as follows: Fill was placed on top of the mud flats out to the bulkhead line up to about El +8 MLW (about 5 to 7 ft above the existing mud flat elevation). The sheetpiles were driven from the edge of the fill. The sheetpiles were fitted with driving tips and driven to refusal in order to toe into the bedrock surface.

The wale at the bulkhead was installed just above the existing mud flats and the tiebeams were installed in shallow trenches in the fill. Weep hole drains were installed at the existing mud flat elevation. Up to 8 ft of additional fill was placed in front of the anchor wall to construct a passive resistance berm extending out about 30 to 40 ft in front of the anchor wall. The anchors were then prestressed to 100% of the 180 kip design load.

DESCRIPTION OF FAILURE

The dredging and backfilling operation proceeded from west to east along the bulkhead. The work along the west 100-ft of the bulkhead was completed with no apparent problems.

During the dredging in the center 100 ft of the bulkhead, the contractor observed some distortion of the tiebeams at the connection with the wale and the dredging was stopped. The tiebeams failed the following morning at a low tide which was approximately 1 ft below mean low tide. At the time of the failure a D-6 bulldozer was operating about 10 ft behind the bulkhead. The organic silt in front of the bulkhead had been dredged to about El -19 when the failure occurred, which is 6 ft above the final dredge depth.

Some of the tiebeams ruptured at the connection with the wale while others ruptured at the first splice behind the bulkhead. The location of the initial tiebeam failure is not known. The tops of the sheetpiles began to lean outward after the tiebeams failed.

After the initial failure, the contractor began excavating fill from behind the sheetpiles and placing it in front of the sheetpiles in order to stabilize the bulkhead. The outward movement of the sheetpiles increased gradually as the failure progressed along the bulkhead in both directions. The sheetpiles initially leaned outward about 3 to 4 ft on the day of the failure and had stabilized at a top deflection of about 8 to 10 ft by the next morning. The failure extended for a length of about 100 ft along the bulkhead. Photographs of the bulkhead after the failure are shown in Fig. 5.

Measurements of the inclination and deflection of the top of the sheetpiles, and later examination of the sheetpiles after they were removed, indicate that the sheetpiles rotated outward about the toe with no movement of the toe and only minor bending of the sheetpiles.

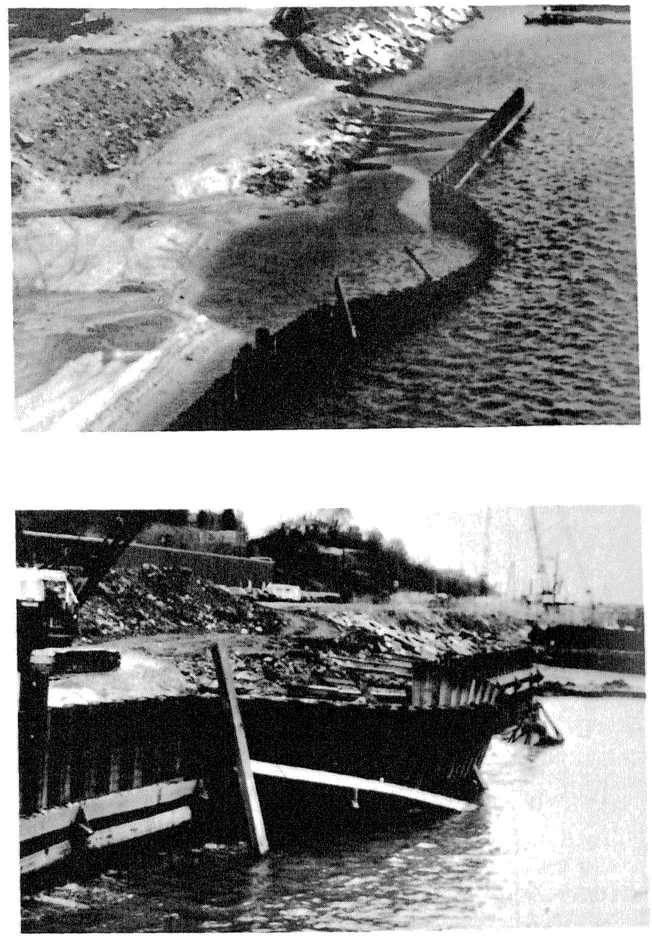

Fig. 5. Photographs of Bulkhead After Failur

ANALYSIS OF FAILURE

The design calculations for the tied-back anchorage system were based on the soil profile at the east end of the bulkhead because that is the critical section for toe stability due to the shallow bedrock at that location. The thickness of the organic silt is about the same at the design section and the failure section, but the organic silt is underlain by sand at the design section and by clay at the failure section.

In order to investigate the cause of the failure, the authors performed calculations of the anchor load based on the soil profile at the failure section and the loading conditions at the time of failure. The soil profile used for these calculations is shown in Fig. 6. The shear strength used for the organic silt includes some strength gain due to consolidation under the new fill (50% consolidation was assumed). The water level in front of the bulkhead was at El -1 MLW (the low tide eleva-

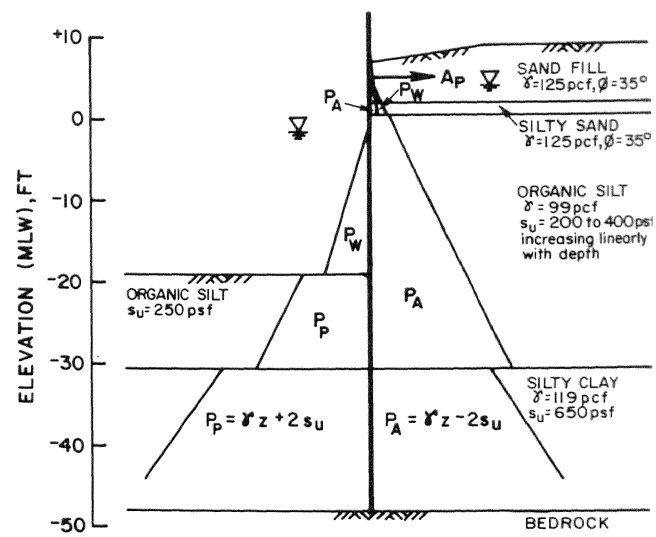

Fig. 6. Soil Profile and Rankine Earth Pressu
for Anchor Load Calculations

tion at the time of failure) and the water level behind the bulkhead was assumed at El +4 MLW (about 2 ft above the weep hole drains). The calculations were performed for the partial dredge depth of El -19 MLW existing at the time of failure.

As a first step, anchor load calculations were performed using conventional design methods based on the Rankine active and passive earth pressures. The Rankine earth pressure distribution is shown in Fig. 6. Anchor load calculations were performed by the Free Earth Support method. These calculations indicate the equilibrium embedment depth is in the lower portion of the clay stratum a few feet above the bedrock toe. Since the lower portion of the clay stratum contains a high percentage of fine sand layers and the sheetpiles were fitted with driving tips to toe into the bedrock, there may have been some fixity near the sheetpile toe. Anchor load calculations were performed by the Equivalent Beam method assuming a point of contraflexure in the center of the clay stratum to evaluate the effect of some fixity of the sheetpiles on the anchor loading. The anchor loads obtained from these calculations are compared to the original design load and the ultimate capacity of the anchors in Table 1. These computed loads do not include any factor of safety on the passive soil resistance and are based on the Rankine active earth pressure without any consideration of the effects of arching on the earth pressure distribution.

The prestressing of the anchors to 100% of the design load results in an essentially unyielding anchorage. In a dredged bulkhead with an unyielding anchorage, soil arching results in a redistribution of earth pressure from the lower portion of the bulkhead to the anchor level, as shown in Fig. 7, and increases the total earth pressure loading from active pressure to a value between active pressure and at rest pressure. The combined effect is to increase the anchor loading above the value computed from Rankine or Coulomb active earth pressure theory. This situation is similar to that encountered in braced excavations and some authors (Peck, et al., 1974; Tschebotarioff, 1962) recommend that the empirical trapezoidal pressure envelopes developed for braced excavations be used in this case instead of the classical design methods based on Rankine or Coulomb earth pressure theory. However, the empirical pressure envelopes are not directly applicable at this site due to the nonuniform soil profile and the large depth of net active pressure loading below the dredge line.

TABLE I. Anchor Loads Computed for Failure Condition

Computed by Free Earth Support method		232 kips
Computed by Equivalent Beam method assuming point of contraflexture at middle of clay stratum		227 kips
Increased by 30% to account for arching		302 kips
Additional load due to D-6 bulldozer		9 kips
Design anchor load:	design	180 kips
	ultimate	300 kips

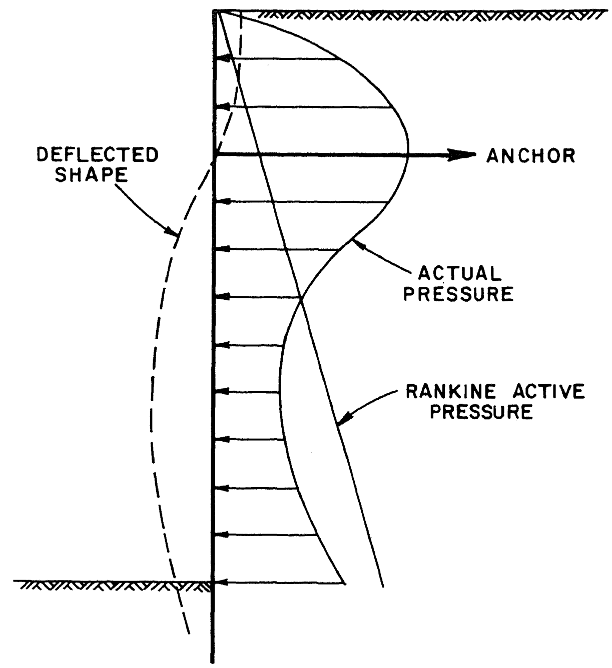

Fig. 7. Earth Pressure Redistribution Due to Soil Arching

Many design references recommend increasing the anchor load computed from Rankine or Coulomb earth pressure theory by an arbitrary factor (or reducing the allowable anchor stress by an arbitrary factor) to account for increases in anchor load due to arching as well as other factors such as unequal loading of the anchors. Table 2 illustrates the range of increase factors published in the literature. These increase factors are recommended for all anchored bulkheads, regardless of type. For a dredged bulkhead with an unyielding anchorage, larger increase factors may be appropriate. In Table 1, the possible magnitude of the anchor load increase due to arching is illustrated by applying a 30% increase to the anchor load computed by the Free Earth Support method.

The anchor load calculations summarized in Table 1 indicate that, even at the El -19 partial dredge depth, the anchor system was loaded beyond its design capacity and approaching its ultimate capacity simply due to the classical earth pressure loading, without including the effects of arching. A review of the original design calculations disclosed that a major reason for this underdesign was that the design calculations were

TABLE II. ANCHOR LOAD INCREASE FACTORS PUBLISHED IN THE LITERATURE

REFERENCE	INCREASE FACTOR
NAFAC DM-7 (1971)	No increase for tierods 20% increase for connections
Terazaghi (1954)	Recommends using reduced allowable stress, but does not specify amount of reduction
Peck, et al. (1974)	20% increase for normal anchorage Use braced sheeting design methods for unyielding anchorage
Tschebotarioff (1962)	25% increase for normal anchorage (80% reduction of allowable stress) Use braced sheeting design methods and reduced allowable stress for unyielding anchorage
USS Steel Sheet Piling Design Manual (1974)	30% increase for tierods 50% to 100% increase for splices and connections
Tsinker (1983)	40% to 70% increase
L. Casagrande (1973)	100% increase

not performed for the lowest tide elevation in front of the bulkhead. In the design calculations the water level was assumed to be at El +3 MLW (the original mud flat elevation) on both sides of the bulkhead. The extreme low tide at the site is El -3.5 MLW, and the failure occurred at a low tide of El -1 MLW. The design calculations also assumed strength values in the organic silt that are somewhat higher than those assumed by the authors. The designer used strength parameters of c = 350 psf and $\phi = 5°$ to compute the Rankine active earth pressure in the organic silt. This is equivalent to using an undrained shear strength of about 370 psf to 450 psf, increasing linearly with depth.

The design calculations did not include any consideration of anchor load increase due to arching. This omission was particularly significant because the system was a dredged bulkhead with an essentially unyielding anchorage as a result of the prestressing to 100% of the design load. The designer used NAVFAC DM-7 as his design reference, which is one of the few references that does not include an increase factor applied to the computed anchor load (Table 2).

The earth pressure loading due to the D-6 bulldozer operating 10 ft behind the bulkhead was evaluated using Terzaghi's modified Boussinesq solution for point loading (Terzaghi, 1954) The calculated increase in anchor load from earth pressure loading due to the bulldozer is shown in Table 1. This calculation indicates that the earth pressure loading due to the D-6 bulldozer was relatively minor. However, the bulldozer may have induced some bending in the tiebeams, as discussed below.

Settlement of the fill behind the bulkhead due to the continuing compression of the underlying organic silt and silty clay may have induced significant bending stresses in the wide flange beam sections that were used instead of conventional tierods or cables. The D-6 bulldozer operating above the tiebeams with only 2 to 3 ft of soil cover may also have induced some bending in the tiebeams. There are too many unknowns to permit computation of the bending stresses actually induced in the tiebeams, but the authors believe that bending of the tiebeams was probably a significant factor contributing to the failure. The use of beam sections instead of tierods or cables made the anchorage system more sensitive to settlement than a conventional anchorage system.

CONCLUSIONS

The major factors which contributed to the failure of the bulkhead anchorage system were:

1) failure to design for the lowest tide elevation in front of the bulkhead.

2) use of design strength values for the organic silt which appear to be too high.

3) prestressing of the anchorage system, which resulted in an essentially unyielding anchorage and increased anchor loading due to soil arching (which was not taken into consideration in the design).

4) use of wide flange beam sections (tiebeams) instead of conventional tierods or cables, which were subjected to bending stresses induced by settlement of the soil behind the bulkhead and operation of construction equipment directly above the tiebeams.

Although several factors contributed to the failure of this bulkhead, the authors would like to place particular emphasis on the fact that the anchorage system was designed for the Rankine active earth pressure loading without any consideration given to the effects of soil arching and other important but indeterminate factors such as unequal loading of the anchors, settlement of the backfill, effects of repeated loading, etc. These factors have been identified and discussed extensively in the literature, and many design references recommend increasing the anchor loads computed by the "conventional" analysis methods to take these factors into account (Table 2). Yet this recent failure shows that their importance has still not been recognized by the entire design profession.

ACKNOWLEDGEMENTS

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