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Failure of a Dredged Slope in a Sensitive Clay

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SYNOPSIS During construction of a new wharf facility in Portland, Maine, an underwater slope failed during dredging and subsequent driving of piles through the slope. The construction and failure of the slope are described. The major factors which contributed to the failure were: 1) high sensitivity of the silty clay, 2) placement of riprap on the crest of the slope to 4 to 6 ft above the design elevation, 3) method of dredging which caused high shear stresses and probable disturbed zones near the toe of the slope, 4) dredging slope steeper than design slope, 5) pile driving causing localized disturbed zones with low strength around the piles, and 6) sequence of dredging and pile driving.

INTRODUCTION

During construction of the Merrill Marine Terminal facility in Portland, Maine, a dredged slope failed during the dredging and subsequent driving of piles through the slope. 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 mudflats 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 slope failure described in this paper occurred in the 600-ft section. The 300-ft wharf was not completed and was eliminated from the project.

The mudflats landward of the wharf were to be filled in and used as a bulk storage area. The mudflats 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 silty 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 organic silt and silty clay outboard of the bulkhead were to be dredged on a 2H:1V slope to E1 -35 MLW. The piles for the deck consisted of square prestressed concrete piles spaced at 12 ft on-center parallel to the bulkhead and 16 ft on-center perpendi-

cular to the bulkhead. The piles in Rows A, B and C were 18 in. square and the piles in Rows D, E, F and G were 15 in. square.

The batter piles and the three rows of vertical piles closest to the bulkhead (Rows E, F, and G) were driven before the outboard slope was dredged. The remaining four rows were driven after dredging. The failure in the outboard slope occurred during the dredging and subsequent driving of the four outboard rows of piles. The construction sequence is described in detail below.

SUBSURFACE SOIL CONDITIONS

The locations of the borings, performed before the failure in the vicinity of the outboard slope during the design phase and at the beginning of construction, are shown in Fig. 1. The results of the borings showed that 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 was in the range of 150 to 300 psf.

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 E1 -40 MLW has been preconsolidated by desiccation. In general, the natural water

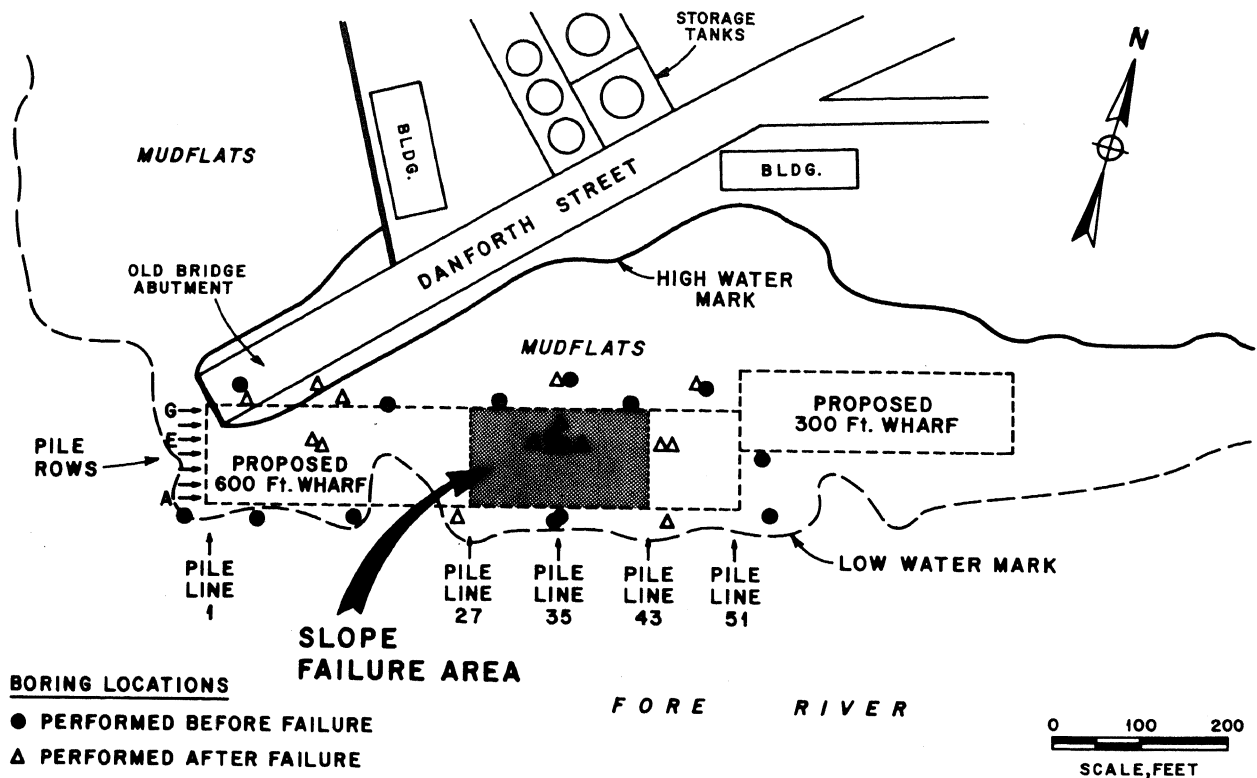


Fig. 1 Project Site

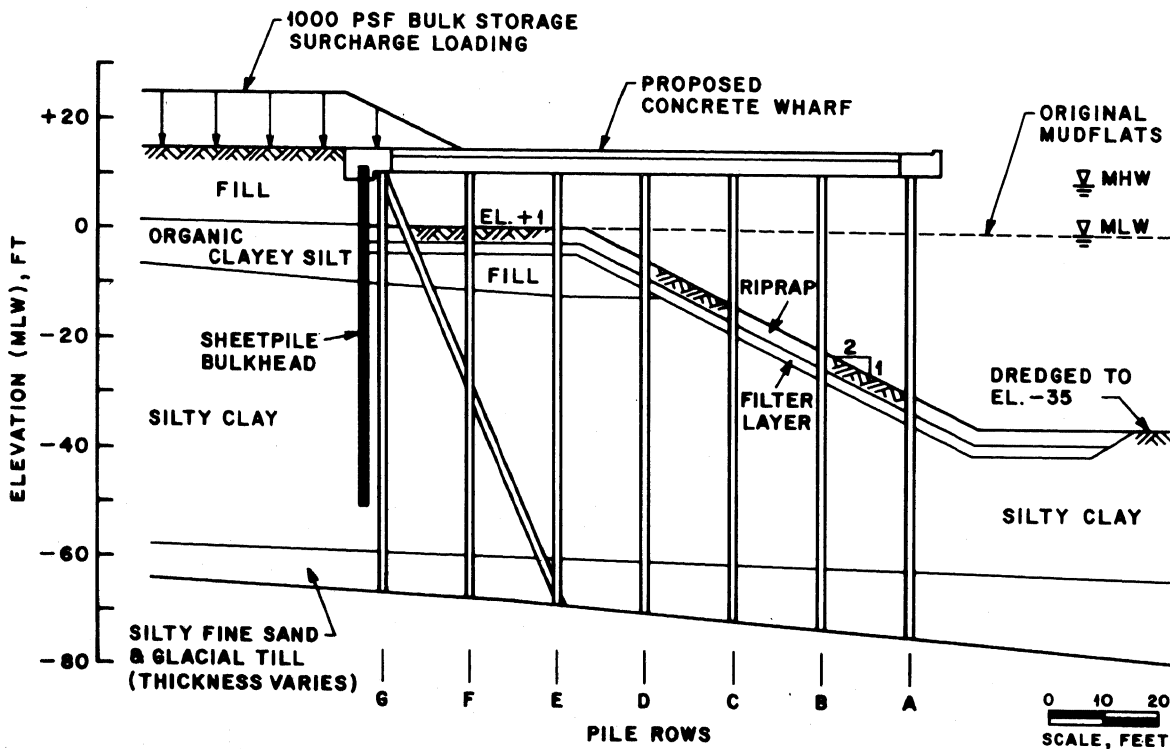


Fig. 2 Design Cross Section of 600-ft Wharf

content of the silty clay is greater than the liquid limit (i.e., liquidity index greater than 1.0). Figure 3 shows the undrained shear strength profile from UU triaxial tests performed during the design phase on silty clay samples obtained from the borings in mudflat areas throughout the site. The $S_u/\bar{\sigma}_v$ relationship based on the results of CU tests performed on the silty clay is also shown. The UU test profile shows the silty clay to be overconsolidated above about El -40 MLW. Figure 4 shows the stress-strain curves from the UU tests. The curves in general show a peak strength at relatively small strains and a subsequent dropoff in strength. Two of the curves are relatively flat, but these samples were probably disturbed during sampling or testing. The water contents of these two samples were greater than the liquid limit indicating that the curves should have been more peaked. The shape of the peaked curves along with the high liquidity index indicate that the silty clay is very sensitive and is susceptible to significant loss of strength if the soil structure is disturbed.

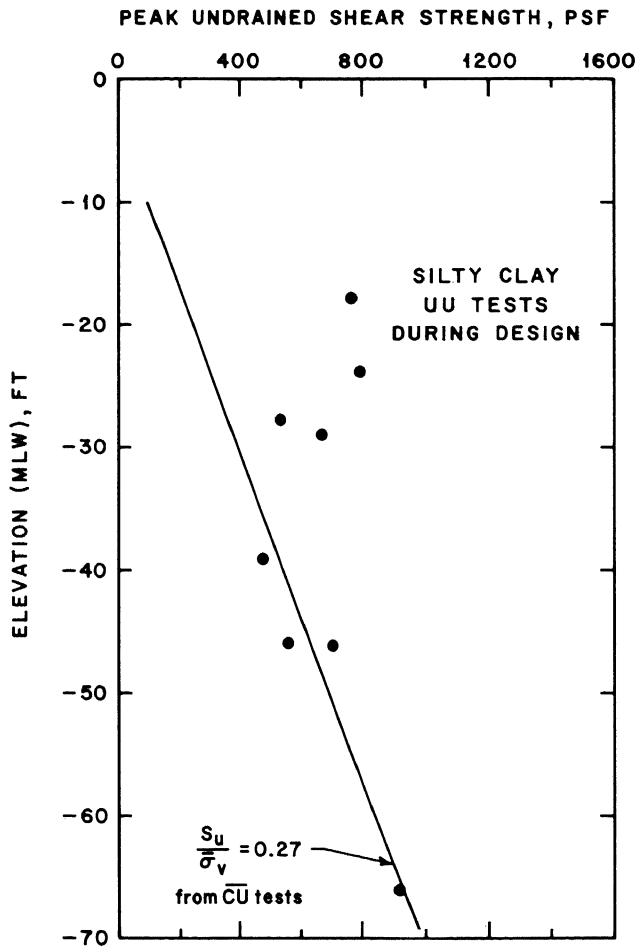


Fig. 3 Silty Clay Undrained Strength Profile From UU Tests Performed During Design

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 600-ft wharf, the thickness of organic silt varies from about 5 ft near the west end to about 20 ft at the east end. The thickness of the silty clay ranges from about 35 ft at the west end to about 50 ft at the east end. The soil profile near the middle of the section where the slope failure occurred is shown in Fig. 2.

DESIGN OF OUTBOARD SLOPE

Stability analyses during the design phase were performed assuming a uniform peak shear strength of 600 psf for the silty clay and assuming the river level at El 0 MLW. The computed factor of safety based on this strength was about 1.6. As part of the analysis of the failure, the authors performed stability analyses for the design geometry using the peak shear strength profile shown in Fig. 3. The computed factor of safety was about 1.8. The unit weights used in the latter analyses were slightly less than the unit

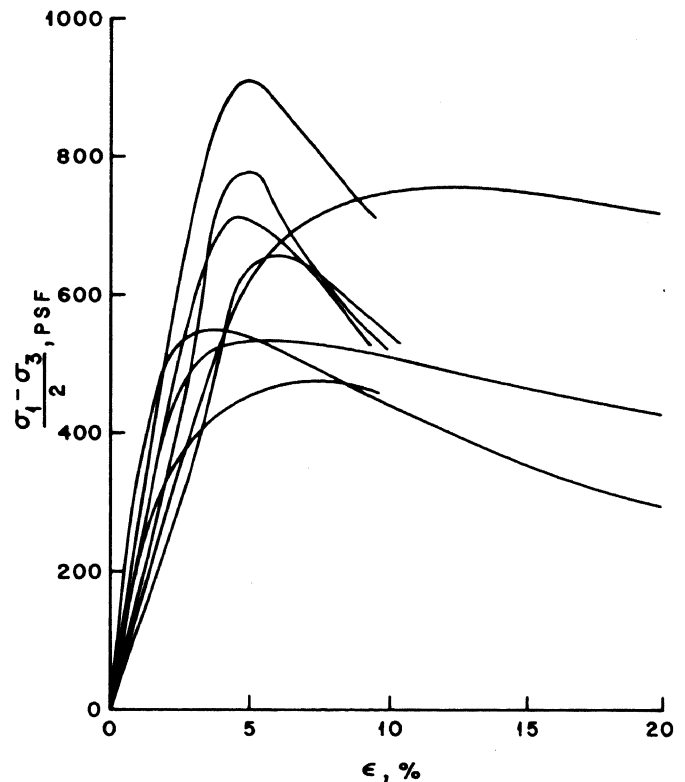


Fig. 4 Stress-Strain Curves from UU Tests Performed During Design

weights used for the design analyses. This difference and the difference in strength assumptions account for the difference in the factors of safety. However, in each case, the computed peak strength factor of safety was greater than 1.5.

CONSTRUCTION SEQUENCE

The construction of the 600-ft wharf was designed to proceed in the following steps. In each construction step, the work progressed from west to east.

- 1) Place backfill on mudflats to El 10 MLW from the existing shoreline out to about the bulkhead line.
- 2) Drive sheetpile bulkhead. The bulkhead was driven to El -40 MLW from Lines 1 to 24, to El -50 from Lines 24 to 44, and to refusal from Lines 44 to 51.
- 3) Drive batter piles and vertical piles in Rows E, F, and G.
- 4) Construct cap beam on top of bulkhead and deck beams out to Row E (construction joint at the middle of top of Pile E).
- 5) Excavate organic silt outboard of bulkhead, replace with gravel fill, and place riprap for crest of outboard slope.
- 6) Construct deck to 7.5 ft inboard of Row E.
- 7) Dredge slope to El -35 MLW in berthing area and El -40 at toe of slope.
- 8) Drive piles in Rows A through D.
- 9) Construct deck beams to outboard edge of deck.
- 10) Place gravel filter and riprap on slope and at toe.
- 11) Complete construction of deck.

CONSTRUCTION PRIOR TO FAILURE

Construction of the wharf began in July 1981. During construction several changes were made to the above construction sequence. The organic silt outboard of the bulkhead was not removed before the gravel and riprap layers were placed. The contractor and engineer anticipated that the gravel and riprap would sink into the organic silt. However, they did not and, as a result, the crest of the outboard slope was at about El 5 to 7 MLW rather than the design elevation of El 1 MLW.

The outboard slope was dredged by the following method. The berthing space outboard of the proposed wharf was first dredged to El -35 MLW resulting in a nearly vertical 35-ft-high dredged slope. The slope was then trimmed by dredging to approximately the design slope of 2H:1V and the toe was dredged to El -40 MLW.

During dredging of the slope at the west end of the wharf, the contractor had difficulty in dredging the toe to El -40. Material appeared to be sloughing from the slope into the excavation. On December 16, 1981, some rotation of the Row E piles and cracking at the construction joint were observed. The designers felt that the pile movement was due to slope movement caused by squeezing out of soft soils below El -25 or slumping at the toe. The movement was considered to be only a local problem. In an effort to prevent future movement, the contractor was instructed to:

- 1) Only dredge to El -35 and dredge at a later time to El -40 for the riprap key.
- 2) Extend deck beam construction joints 5 ft past E line so that the pile was rigidly connected to the deck beam.

After these changes were made, the piles and deck beams from Line 1 to 17 and the deck slab from Line 1 to 11 were completed.

DESCRIPTION OF THE FAILURE

After the west portion of the wharf was completed, the contractor began dredging the slope for the remaining 400 ft of the wharf (Lines 18 through 51). The entire slope was dredged during the period from January 28, 1982 to March 11, 1982. On March 8, hairline cracks were first noticed in the Row E and F piles from Line 29 to Line 51. The cracks were located at the top, inboard side of the piles. The cracking in the Row E piles became more substantial during March 9-10. On March 11, a 1.5-in. gap was observed between the bulkhead and the top of the outboard slope. At some of the piles, several inches of soil was mounded up against the inboard side of the pile, and there was a depression on the outboard side indicating that substantial soil movement was occurring.

Construction was not stopped after the indications of soil movement were noticed. Driving of the piles in Rows A through D started on March 16 beginning at Line 18 and proceeding east. The piles were driven at a rate of about 25 piles every two days. By March 26, the Row C and D piles from Line 18 to 42 and the A and B piles from Line 18 to 37 had been driven. During the pile driving, the cracking in E and F piles became more severe, and construction was stopped on March 26.

The major area of pile cracking was from about Line 27 to Line 43. In this area the Row E piles had cracks up to about 1-in. wide and some were severely spalled exposing the reinforcing steel. The Row F piles in this area had cracks ranging from hairline to about 1/4-in. wide. The Row E and F piles had rotated significantly toward the outboard about the top of the piles. Some of the E and F piles between Line 44 and 51, had hairline cracks.

Figure 5 shows the as-built slope geometry at Line 35. The top of the riprap was at about El 5 MLW rather than El 1 MLW. In addition, the dredged slope was steeper than the design slope,

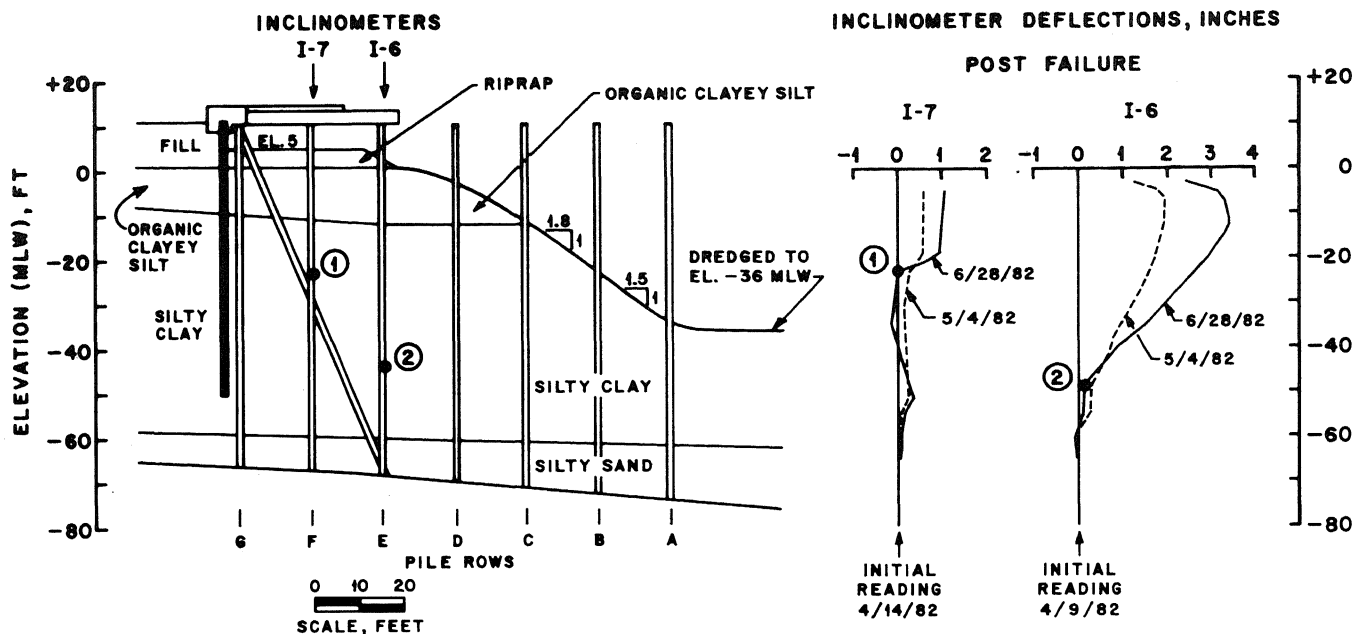


Fig. 5 As-built Cross Section at Line 35

especially near the toe, and was dredged to about El -36 MLW rather than El -35 MLW. After construction was stopped, three slope inclinometers were installed, two in the failure zone at Line 35 and one outside the failure zone at Line 44. The locations of the two inclinometers at Line 35 are shown on the cross section in Fig. 5, along with movements measured at the inclinometer locations.

As shown in Fig. 5, the inclinometer measurements indicated that a deep-seated slope failure was occurring through the silty clay in the outboard slope. It should be noted that a much larger amount of movement occurred before the inclinometers were installed.

At the time in early March when the cracking was first noticed and in late March when the cracking became more severe, the river reached about El -1.5 MLW (1.5 below mean low water) at low tide. The inclinometers also showed large amounts of movement during subsequent lower low tide cycles.

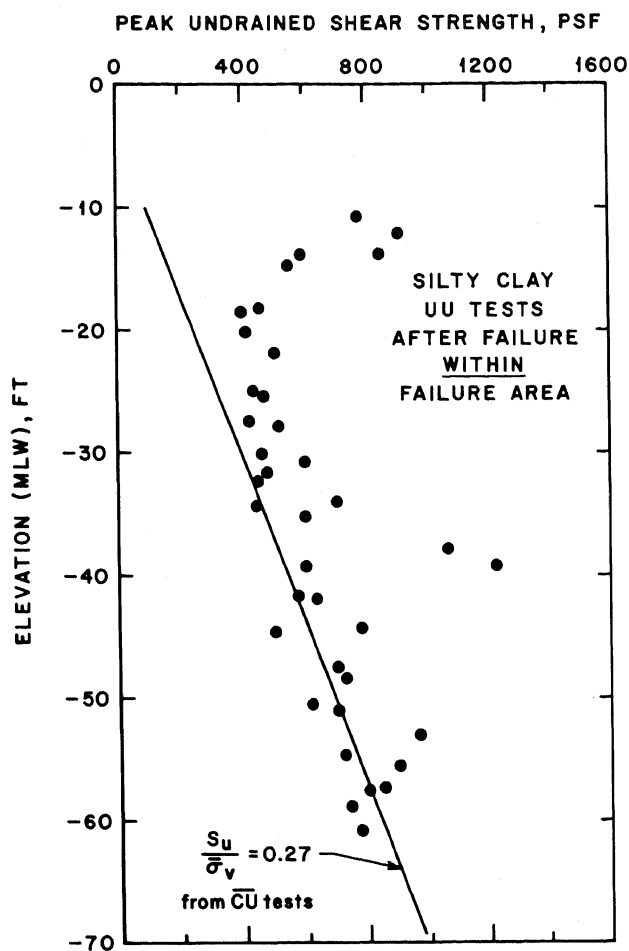
ANALYSIS OF FAILURE

After the construction was stopped, the designer performed a series of borings to investigate the cause of the failure. The locations of the borings performed in the vicinity of the outboard slope are shown in Fig. 1. UU triaxial tests were performed on undisturbed samples obtained in the borings. The results of the UU tests performed on samples of the silty clay from the borings located outboard of the bulkhead are plotted in Fig. 6. Figure 6a shows the results for the borings performed within the failure zone and Fig. 6b shows the results for the borings located outside the failure zone.

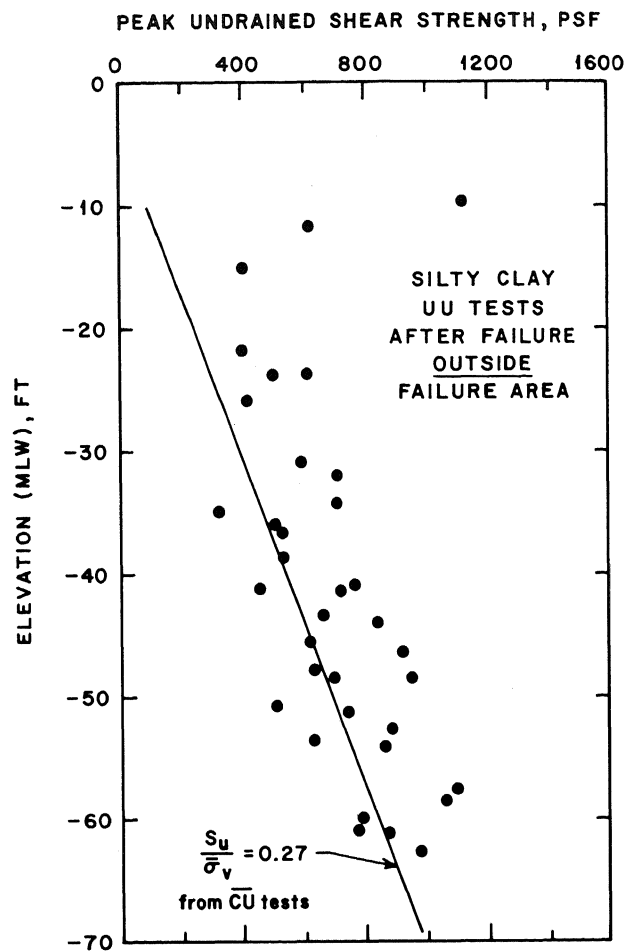
There is no evident difference in the two strength profiles. By comparison with Fig. 3, it can be seen that the strength profiles shown in Fig. 6 are similar to the strength profile measured during the design phase.

Stability analyses were performed to evaluate possible causes for the failure. Analyses were first performed to determine the effect of placing the riprap to El 5 MLW and dredging the slope to a steeper slope and slightly deeper depth than designed. The river was assumed to be at El -1.5 MLW. The strength profile used in the analyses was based on the profile shown in Fig. 6a and was the same used to analyze the design slope geometry. In the first analysis, it was assumed that the riprap was placed to El 5 MLW, but the slope was at the design 2H:1V. The computed factor of safety for this case was about 1.45 as compared to 1.88 computed for the design geometry. Next, the stability was analyzed for the as-built geometry with the riprap to El 5 MLW and the slope dredged steeper and to El -36 MLW as shown in Fig. 5. The computed factor of safety for this case was about 1.25.

These results indicate that the excess riprap and overdredging significantly reduced the factor of safety and increased the shear stresses in the outboard slope. Since the factor of safety was greater than 1.0 based on the peak strengths, the strength of the silty clay must have been reduced in the shear zone for the failure to have occurred. The method of dredging which resulted in a near-vertical face after the berthing area was dredged and the subsequent pile driving probably caused zones of disturbed silty clay with reduced shear strength, especially near the toe of the slope. As a result, the average shear strength along the failure zone was reduced below the average peak strength.



a)



b)

Fig. 6 Silty Clay Undrained Strength Profile from UU Tests Performed After Failure - a) Within Failure Area, b) Outside Failure Area

Analyses were next performed to determine the average value of shear strength required along the failure surface in the silty clay for the factor of safety to equal 1.0. The results indicated that an average value of about 400 psf was required.

The strengths measured in the borings within the failure zone (Fig. 6a) are generally greater than 400 psf. However, these borings were all performed at about Row E. No borings were performed between Rows E and A where the potential for disturbance of the silty clay was the highest due to dredging and pile driving. To evaluate the effect of the sensitivity of the silty clay on the stability of the slope, the post-peak strength measured at 10% strain in each of the UU tests was determined and plotted in Fig. 7. The shear strength at 10% strain was

about 400 psf from about El -20 to El -40 MLW and slightly higher above and below this zone. An analysis was performed assuming the clay outboard of Row E had been disturbed to the degree that the strength could be represented by the profile shown in Fig. 7. The computed factor of safety for this case was about 1.10. Localized zones in the silty clay, especially at the toe of the slope and surrounding the piles, would be probably more disturbed with a lower strength in which case the factor of safety would be closer to 1.0.

Based on the review of the failure, it is felt that the failure occurred as a progressive failure caused by disturbance of the silty clay during construction. The following factors probably contributed to the disturbance of the silty clay and the progressive failure.

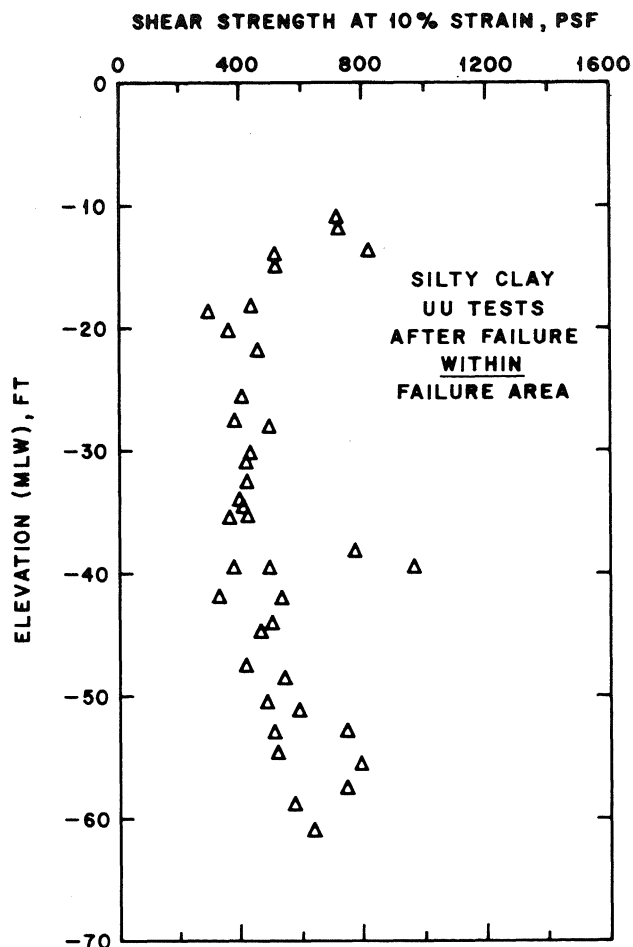


Fig. 7 Silty Clay Strength Profile at 10% Strain from UU Tests Performed After Failure Within Failure Zone

- 1) Sensitivity of the Silty Clay - The silty clay is highly sensitive as evidenced by the shape of the stress-strain curves and its high sensitivity makes the silty clay highly susceptible to disturbance and significant reduction in strength.
- 2) Placement of Riprap Above Design Elevation - Placing the riprap to El 5 to 7 MLW rather than El 1 MLW increased the driving force and shear stresses in the slope and increased the chances for a progressive failure.
- 3) Dredging - When the slope was dredged, the silty clay near the toe of the slope was unloaded significantly. The shear strength of the silty clay will reduce somewhat as it swells under the lower confining stress. The main effect of dredging on the stability of the slope was the method of dredging. The berthing area was dredged to El -35 MLW first, leaving a near vertical slope at the location of the toe of the design slope. The shear stresses and resulting strains were probably very high near the toe of the

slope causing a significant reduction in the strength in the area at the toe. In addition, the slope was dredged steeper, and to a slightly deeper depth than designed.

- 4) Pile Driving - Even though the piles were driven 12 ft and 16 ft on-center the pile driving probably caused disturbance in the silty clay. The disturbance may have been confined to small areas around the piles but may have resulted in very low strengths in these areas. These areas of low strength would have lowered the average shear strength along the failure surface and increased the chance of progressive failures.

The effect of pile driving on the stability of the slope was evidenced during the driving of the remaining deck piles and the replacement piles for the damaged piles after the wharf was redesigned. As part of the required redesign, the crest of the out-board slope was excavated to El -5 MLW before the piles were driven. Movements of the slope were monitored during pile driving using slope inclinometer and tape extensometer measurements. The piles were installed at a controlled rate of 5 to 8 piles per day. Even with these precautions, about 1.3 inches of slope movement occurred during pile driving. This movement reflects the sensitivity of the silty clay and the effect pile driving had on the failure during which the piles were driven at a rate of about 25 piles every two days.

- 5) Sequence of Construction - The piles in Rows A through D were driven very soon after the slope was dredged, even though slope movement and cracking of the Row E and F piles was observed during dredging. If pile driving had been delayed and the cause of the movement investigated, damage to the piles may have been reduced, and it may have been possible to implement remedial measures.

The slope failure at Merrill Marine Terminal illustrates that the designers must consider the sensitivity and the shape of the stress-strain curves of the soils when designing a project. For soils exhibiting peaked stress strain curves with subsequent dropoff in shear strength, applying a generally accepted factor of safety to the peak strength may not insure that there will not be a slope failure. As evidenced at the subject site, construction techniques can cause sufficient shear strains to result in the soils locally to have a significantly reduced strength. If the locally reduced strength causes the average shearing resistance along a potential failure plane to equal the driving shear stresses, failure can occur. As a result, the designer should estimate during design the reduced shear strength the soils may reach during or after construction. The factor of safety selected for design should reflect the strength used in the analysis, the shape of the strength vs strain curve, and the potential for disturbance during construction.

The slope failure at the subject site also illustrates that monitoring and control of construction is especially important when

dealing with sensitive soils. An instrumentation system should be designed to reflect potential movements during construction, and a plan of action should be available should the instruments indicate larger than expected movements. The contractor and the owner should understand the implications of the instruments showing unexpected readings. For example, it should be understood that construction may stop in the area where unexpected movement has taken place. If this issue is carefully explained in the specifications and during construction sequencing, potential conflicts over claimed construction delays may be avoided. After movements were observed at the west 200 ft section of the wharf and during the early stages of dredging of the remaining 400 ft, construction should have stopped and the cause of the movements evaluated.

CONCLUSIONS

The major factors that contributed to the failure of the outboard slope were:

- 1) Not recognizing the importance on design of the high sensitivity of the silty clay.
- 2) Placement of riprap on the crest of the slope to 4 to 6 ft above the design elevation.
- 3) The method of dredging which resulted in high shear stresses at the toe of the slope and, as a result, probable zones of disturbed clay with significantly lower shear strength than the strength used in the stability analysis.
- 4) Local zones of disturbance and low strength in the silty clay caused by driving full displacement piles through the slope.
- 5) Permitting construction to continue very soon after the slope was dredged, even though slope movements and cracking of piles was observed during dredging.

Although several factors contributed to the slope failure, the authors feel that the main reasons for the failure was a lack of appreciation of the high sensitivity of the silty clay and a lack of construction control. The engineer should consider the sensitivity and the shape of the stress-strain curve of the soils when designing a slope or any other structure. In sensitive soils, the construction must be closely monitored and controlled to avoid or detect potential problems and implement remedial measures to correct problems if they occur.

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