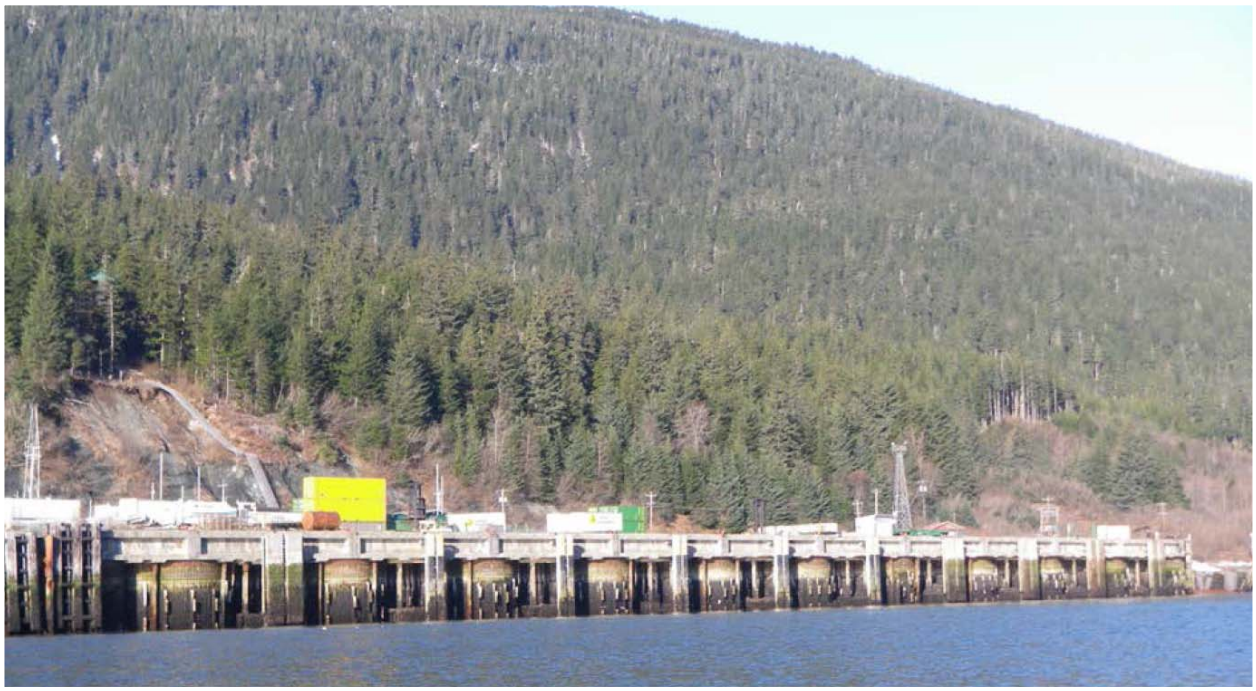
Lutak Dock Structural Assessment

Haines, Alaska



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1. EXECUTIVE SUMMARY

The Haines Borough has commissioned this report to document results of a structural assessment of the Lutak Dock located in Haines, AK. The effort was directed to reviewing existing documentation for both maintenance and repairs, and to determine the probable remaining service life under static (non-seismic), and seismic conditions. This structural assessment has been based on the guidance established by the United States Army Corps of Engineers (USACE) for cellular structures as identified in manual EM 1110-2-2503 "Design of Sheet Pile Cellular Structures, Cofferdams and Retaining Structures".

Notwithstanding the maintenance and repairs/modifications to the structure since the original USACE design and construction in 1953, it is the opinion of PND Engineers, Inc. (PND) that the structure has reached the end of credible 60-year service life. Further utilization is effectively on "borrowed time." The presence of sink holes in the working surface of the structure is consistent with loss of fill arising through observable gaps between the main cells and the Z-sheet pile sections utilized for repairs. Per Ref. #4, this repair work is assumed to have been completed around 2003. It could be argued that these gaps and material loss can be repaired by re-sealing/backfilling the closure arcs.

A source of significant risk to the structure arises from failure of the "tee" connection between the lower closure arc(s), and main arcs themselves. This risk is in addition to the damage at 5 of 11 closure arcs that arose after the 2002 repair, and the splitting failure at Closure Arc 7.5. Other structural weaknesses were revealed in this assessment though none as historically troublesome as the welded connection between the main cell and the closure arc. In 1965 the connection detail utilized for the original Lutak Dock was specifically prohibited by the USACE after a series of arc connection failures on temporary cellular structures utilized on the island river system. Given the USACE design background, the current corrosion loss, and the results of this assessment, it is the view of PND that failure conditions exist at all other closure arcs.

Notwithstanding the strong evidence of distress at closure arcs, the results of this assessment demonstrate that the Lutak Dock does not meet current USACE minimum factors of safety for cellular structures for the classic failure mode of vertical shear under the conditions of dead load plus operating live load, and for dead load plus phreatic water pressure. While it can be said that absent the failure of Closure Arc 7.5 the structure has remained serviceable, the calculations by PND demonstrate that the facility is "near the edges" and that in our view it is prudent to begin the process to replace the structure to meet current minimum standards under operating conditions and potential seismic loading.

The seismic forces arising in Haines are modest, and the structure meets USACE critera for low intensity earthquake with ground acceleration of .072g (50% likelihood of occurrence in 50 years). Above this level of ground shaking the structure cannot withstand earthquakes at the current "design event" level criterial mandated by building codes, waterfront design guides, or departments of transportation manuals.

2. BACKGROUND

The Lutak Dock is an 1100' long bulkhead located approximately four miles north of downtown Haines. The bulkhead was designed by the United States Army Corp of Engineers (USACE) and constructed in 1953. Specific data regarding the original construction of the bulkhead came from a set of "As-Built" construction drawings (Ref. #11).

Figure 2-1. Lutak Dock - 2004 (Photo From Google Earth)

Ownership of the bulkhead is split between the Haines Borough (HB) and the Alaska Marine Highway (AMH)[KI1]. Lutak Dock is comprised of 15 interlocking circular closed cells; the Borough's portion of the structure is the 11 western-most cells. The 15 main cells are approximately 66'-8" in diameter and are spaced approximately 69' apart. The closure arcs have a radius of approximately 16'-4". The top of the dock is at El. +28.5'. Figure 2-2 shows the typical closed cell bulkhead configuration.

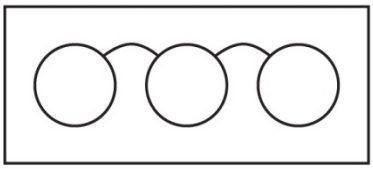

Figure 2-2. Typical Closed Cell Configuration.

The cells are built from interlocking flat web sheet piles. The sheet piles in the closure arcs have web thicknesses of 3/8". The main cells have sheet piles with web thickness of 1/2". PND was not able to ascertain the original manufacturer of the sheet piles. For the purpose of this structural assessment, PND has assumed that these sheet piles have an interlock strength of 16,000 pounds per inch; this is consistent with piles of the vintage of this bulkhead. The tip elevation of these piles is approximately El. -46.5' along the face of the bulkhead. The tip elevation of the sheet pile tip tapers up to approximately El. -36.5' at the rear of the bulkhead (Ref. #1).

A concrete facing beam is mounted on the top of Lutak Dock. This facing beam is partially supported by the closed cell sheet pile and partially by driven H-Pile. This facing beam is approximately 8'-6" tall and runs the full length of the bulkhead, wrapping around the ends of the bulkhead on the east and west sides. The top of this facing beam coincides with the top of the bulkhead, El. 28.5'.

3. CONDITION

The Dock has experienced significant corrosion loss of the base metal in the sheet piles over the last 61 years. Corrosion of the sheet piles has been well-documented through periodic inspections between 1976 and 2014. The inspections include:

1976 – Dock Inspection – R&M Engineers – (Ref. #7)

1988 – Dock Inspection – PND Engineers – (Ref. #6)

2003 – Dock Inspection – Echelon Engineers – (Ref. #5)

2014 – Dock Inspection – Echelon Engineers – (Ref. #4)

Each of these inspections document the substantial growth of corrosion over the life of the dock. The most recent documentation indicates corrosion loss of sections of approximately 0.16 inches. This represents deterioration of between 30% and 46% of the original section. Note that the main cell sheet piles have a thicker web than the closure arc sheet piles.

Lutak Dock was most recently modified sometime around 2002, according to the Shannon and Wilson Report (Ref. #1) by an unidentified contractor. The documentation of these repairs indicates that z-sheet pile closure walls were driven 13'-4" behind the bulkhead, as well as supporting H-Piles were driving through the concrete facing beam. Once these additional supporting piles were installed, the closure arcs were cut down to El. 0'. Figure 3-1~~Figure 3-1~~ is a detail excerpted from the 2002 repair drawings (Reid Middleton) depicting repairs to the closure arcs (Ref. #8).

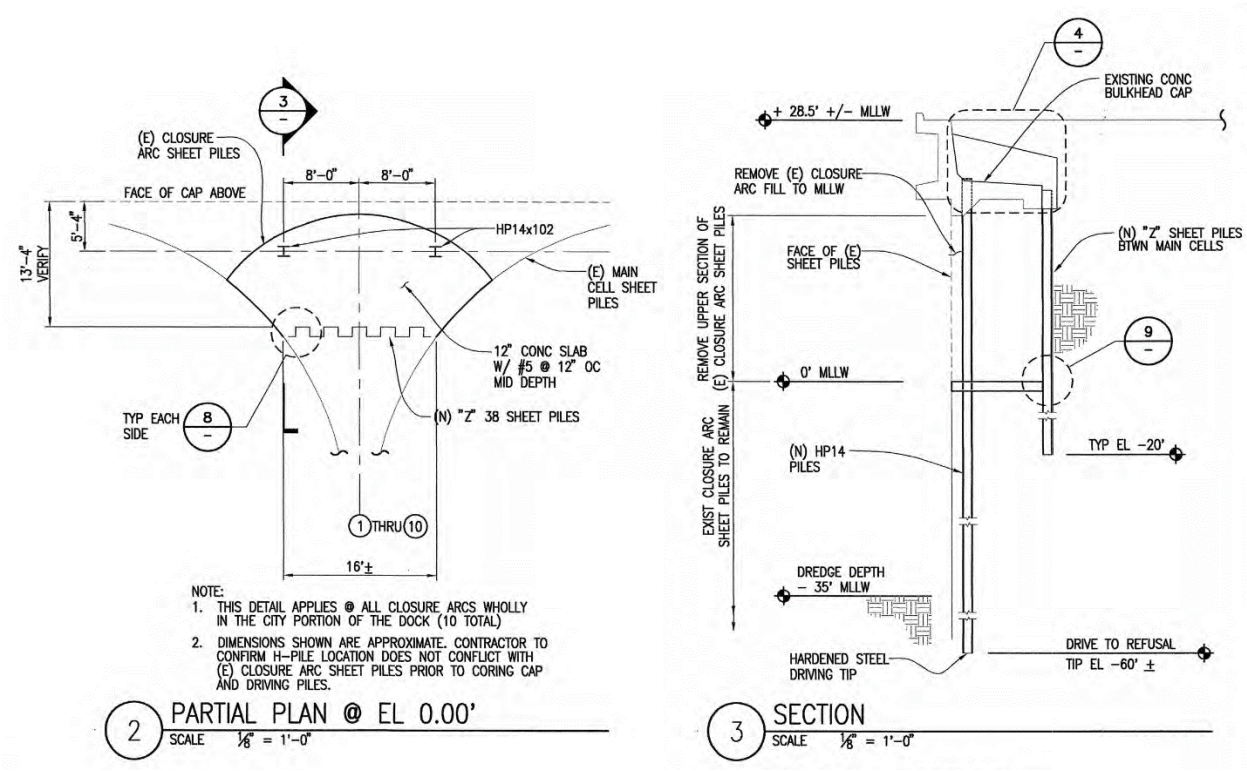

Figure 3-1. Closure Arc Repair Detail

After completion of these repairs, sink holes formed behind the z-sheet pile bulkhead (Figure 3-2). The cause of these holes is believed to be soil escaping through gaps between the z-sheet wall and the main cells. Some attempt was made to utilize geotextile fabric along with the H-pile closure detail to ensure soil was retained through flushing effects of successive tide cycles. The reported frequency and magnitude of the sinkholes suggests that significant volumes of material have been lost, which de-stabilizes the working surface of the dock. Highly loaded vehicles may suddenly fall into an undetected hole with potentially severe consequences to persons, and equipment and property.

Figure 3-2. Sink Holes Observed at Lutak Dock

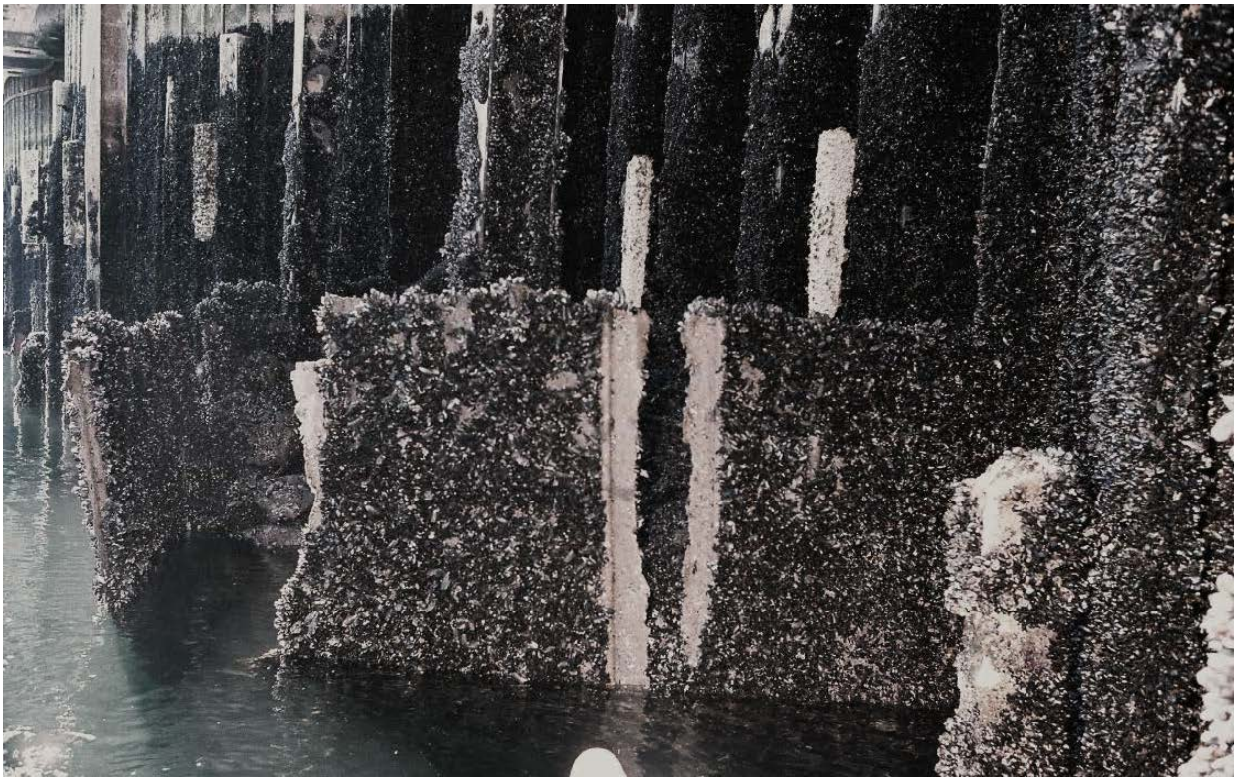

Figure 3-3. Failure at Closure Arc 7.5

Recent inspections indicate Closure Arc 7.5, located roughly at the centerline of the bulkhead, has failed. Figure 3-3 depicts the damaged closure arc. It is not known to PND precisely when this failure occurred. However, failure of Closure Arc 7.5 must have occurred approximately around 2003 as the sheet piles in the closure arc have clearly been cut down.

3.1. LOWER CLOSURE ARCS AND CONNECTING "TEES" TO THE MAIN CELL

In 1965, the connection detail utilizing a 90-degree "tee" between the main and closure arc was specifically prohibited by the USACE after a series of "tee" failures on temporary cellular structures utilized for lock and dam construction (Figure 3-4)(Ref. #14). [Note: the USACE uses the terms Primary Cell and Intersecting Arc, our terminology is amended to match earlier inspection reports.]

5-2. Failures.

a. Failure Modes. The primary reported causes of cofferdam failures are:

(1) Structural.

(a) Fabricated Tees and Wyes. Numerous failures have involved welded connector piles. Such failures in welded tees normally occurred in the web of the main sheet pile, the web often rupturing on both sides of the tee stem and separating the tee into three pieces. Weakness in these tee members is attributed to improper welding of steel with a high carbon content and laminations in the steel sheet piles used in fabricating the tees.

Figure 3-4. Excerpt from Ref. #12, USACE Design Manual EM 1110-2-2503, *Page 5-1*

Figure 3-5 depicts the current USACE endorsed configuration for the connections between the main cell and the closure. The geometry of the connection utilized at Lutak in 1953 (Figure 3-6) is identical to those subsequently prohibited by USACE. In 2007, PND investigated options for a private cement company to repair damage resulting from failure of an identical "tee" joint between closure arc and main cell at a structure constructed in 1964. Photographs and observations of the cement company failure are consistent with the mechanism observed by the USACE at other cellular structures.

Figure 3-5. Endorsed USACE Connection Post-1965

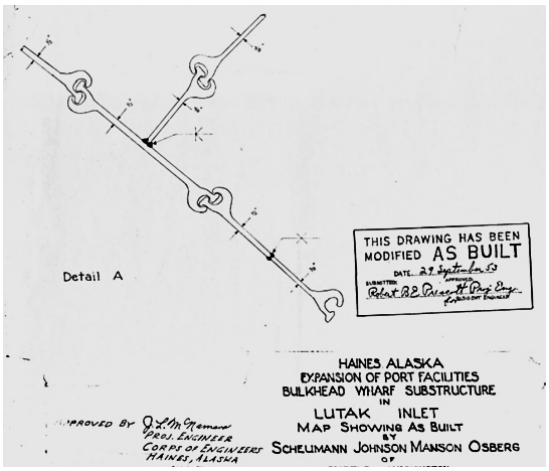

Figure 3-6. Connection Detail at Lutak Dock, 1953

Significant repairs have been made to the structure since original construction of the bulkhead in 1953 by Scheumann Johnson Manson Osberg Company (now Manson Construction). Sometime around 2003, Z-sheet piles where driven between all Main Cells, behind the Closure Arcs to relieve a portion of the closure arc load consistent with Ref. #1. This same report specifically points out

that as a result of the repairs, structural demand to support the lower portion of the Z-sheet pile wall would be transferred onto the remaining portion of the lower arc. An assumption was made that this would be adequate (Ref. #1, pg. 8).

A splitting failure of lower Closure Arc 7.5 has occurred since 2003, based on inspection reports prepared by Echelon (Ref. #4). It is uncertain whether the failure of Closure Arc 7.5 is due to structural loading to support the lower end of the Z-sheet pile wall installation, or from damage arising during initial pile driving. Some damage has occurred at Closure Arc numbers 1.5, 5.5, 6.5, 8.5, and 10.5 as a result of driving of H-piles. The piles sliced into the thinner corroded webs of the lower closure arcs. Condition of the lower arcs is stated in Ref. #4 to be from fair to poor, with heavy corrosion loss and pitting noted at the testing/sample locations.

It is evident from the recent inspection that 6 of 11 closure arcs that comprise the HB-owned portion are compromised, and that the connecting tee between the main cell and closure arc are equally vulnerable. The failure mechanism at (Closure Arc 7.5) is different from this "classic" failure articulated though it is probable that distress is significant at the location where the lower arch joins the main cell. It is the view of PND that each remaining closure arc is at, or near, a condition of failure as a result of corrosion loss, structural loading on weakened section, and as a result of damage during the 2003 pile driving efforts.

4. GEOTECHNICAL CONDITIONS

The dock site is located in a fjord formed via glacial carving. The terrain around the site consists of steeply sloping mountains on both sides of Lutak Inlet.

No specific borings were collected in the preparation of this report. Geotechnical conditions were assessed using borings from previous subsurface investigations. (See Ref. #1 through Ref. #3.)

Generally, the site is known to consist of the following layers:

Fill (El. 28.5' to El. -20')	Fill material (per Ref. #1), consists of granular-type material with SPT blow count ranging from 10 to 57 blows per foot.
Native Soil (El. -10' to Bedrock)	Native material (per Ref. #1), consists of a granular-type material. SPT blow count for native material was approximately 30 blows per foot.
Bedrock	Bedrock elevations were estimated using a sub-bottom profiler. The profiling was performed by Apollo Geophysics (per Ref. #1). The data collected indicated an irregular bedrock (or hard layer) varying between El. -40' and El. -70' along the face of the bulkhead.

For the purposes of evaluating the bulkhead in its present condition, PND has assumed the following soil properties inside the cells:

Unit Weight of Soil:	125 pounds per cubic foot
Angle of Internal Friction:	30°

These properties are consistent with the lateral pressures depicted in Figure 8 of Ref. #1.

5. USAGE AND LOADING

PND understands that the bulkhead is currently used by AML as a container terminal, where the primary load results from the operation of a container forklift on the bulkhead surface. Design loading on the bulkhead from the Lutak Dock Rehabilitation Project in 2003 (Ref. #8) list loads of 130 kip vehicle axial load as well as a 1000 psf uniform load.

6. ENVIRONMENTAL CONDITIONS

6.1. WATER SURFACE ELEVATIONS

PND has estimated the tidal conditions at the Lutak Dock by referencing tidal information available on National Oceanic and Atmospheric Administration (NOAA) currents and tides website (Ref. #9). The nearest source of tidal information was Skagway, Alaska which is approximately 12 miles away. The following data is recorded for Skagway, Alaska.

Mean Higher High Water (MHHW): 16.73'

Mean Sea Level (MSL): 8.81'

Mean Lower Low Water (MLLW): 0.0'

Ground water elevations are estimated from borings available in the geotechnical reports (see Ref. #1 through Ref. #3.). Geotechnical reports indicate that the ground water elevation varies between El. 15' and El. 5'. These ranges are generally consistent with the tidal elevation from Skagway. In particular, the median groundwater elevation coincides roughly with the MSL recorded in Skagway.

6.2. EARTHQUAKE

Ground motion parameters for the Lutak Dock were estimated using a Java® applet written by the United States Geologic Survey (Ref. #10). This applet allows the user to compute the seismic parameters of a variety of different earthquake return periods. This computer program accepts as input the latitude and longitude of the site in question as well as the type of seismic ground motion desired and the return period of interest. Seismic parameters summarized in Table 6-1~~Table 6-1~~ were computed for the Lutak Dock site.

The seismic forces arising in Haines are modest, and the structure meets USACE criteria for low intensity earthquake with ground acceleration of .072g (50% likelihood of occurrence in 50 years). Above this level of ground shaking the structure cannot withstand earthquakes at the current "design event" level criterial mandated by building codes, waterfront design guides, or departments of transportation manuals. The .072G represents the "maximum tolerable ground acceleration" for Lutak Dock.

Table 6-1. Seismic Parameters

	99% PE in 50 years (10-year EQ)	50% PE in 50 years (72-year EQ)	10% PE in 50 years (475-year EQ)	2% PE in 50 years (2475-year EQ)
Peak Ground Acceleration	0.016 g	0.072 g	0.200 g	0.494 g
0.2 sec Spectral Acceleration	0.036 g	0.0162 g	0.459 g	1.159 g
1.0 sec Spectral	0.010 g	0.070 g	0.188 g	0.438 g

Acceleration				

7. SUMMARY OF 2014 ECHELON ENGINEERING INSPECTION (REF. #4)

As part of the task of preparing this report, PND tasked Echelon Engineering (Echelon) with performing an underwater inspection of the condition of the bulkhead. Echelon inspected the condition of the Dock as well as the level of corrosion present. Their findings can be grouped into the three separate components of the structure: Main Cells, Closure Arc Repair and Remaining Closure Arc.

During their inspection, Echelon photographed areas where cell damage was observed. PND has attached to this report, as Ref. #4, a plan view of the bulkhead with the notations of where the damage depicted in the photographs has occurred.

7.1. MAIN CELLS

Echelon rated Main Cells 1 through 11 as in "fair condition" with regard to sheet pile corrosion. The average measured thickness of the sheet pile base metal was 0.316" against an original material thickness of 0.500". "Heavy Pitting" was observed at "virtually all test sites" (Ref. #4), with pit-depths ranging from 0.060" to 0.200".

During the inspection, water was observed to be retained behind the Z-sheet sheet pile bulkhead. Figure 7-1, excerpted from the Echelon's inspection report, depicts water flowing out of a weep hole several feet above the outboard water surface elevation. Based on the scale of the photograph, the fall height of the water could be between 6' and 11'. Further, the water surface elevation behind the sheet pile would have to be higher than the elevation of the weep hole.

Figure 7-1. Retained Water Behind Main Cell

There is evidence, shown in Echelon's inspection report, of the tilting of the bulkhead. Figure 7-2 depicts a full depth crack in the bulkhead facing beam, as the beam wraps around the western side of the dock. This crack is located near the mid-line of the cellular structure.

Figure 7-2. Cracking in Facing Beam near Mid-Line of the Bulkhead

7.2. Closure Arc Repair

The Closure Arc Repair section was rated in "good condition" with regard to sheet pile corrosion. Inspection revealed that these repairs were not sealed against the main cells. Echelon observed that a visible gap between the main cells and the repair sheet piles at five locations. Echelon also reports that the geotextile was not intact at any locations where gaps had developed. There was also evidence that the fine-grained material behind the repair piles has washed away.

7.3. Remaining Closure Arcs

Echelon rated the Remaining Closure Arc section as in "poor condition" with regard to sheet pile corrosion. During the inspection, perforations in sheet piles were observed in Closure Arcs numbered 1.5, 5.5, 6.5, 7.5, 8.5, and 10.5. This represents half of all of the closure arcs. Two closure arcs were observed to have been punctured by pile driving during the repair of the closure arcs (cells 8.5 and 10.5) in approximately 2003. Closure Arc 8.5 was observed to have partially failed as indicated by the lowering of the backfill material.

Echelon observed indication of tensile yielding failure at Closure Arc 7.5. Figure 7-3 depicts necking of the sheet pile web prior to failing.

Figure 7-3. Tension Necking and Rupture of Sheet Pile Web

8. ANALYSIS

PND compared the dock structure against the current industry standard for the design of closed cell bulkheads. The current design standard is the United States Army Corp of Engineers Manual EM 1110-2-2503 (Ref. #12). This document calls for the following limit state checks to be made on the structure during design phase (Table 8-1):

Table 8-1. Cellular Structures Limit States w/ Factors of Safety

USACE – Cellular Structures Limit States with Factor of Safety				
Limit State	Normal – Cond.	Temporary – Cond.	Seismic – Cond.	Internal/External
Sliding	1.5	1.5	1.3	External
Overturning	Inside Kern	Inside Kern	Inside Base	External
Rotation	1.5	1.25	1.1	External
Bearing Capacity	2.0 (Sand)	2.0 (Sand)	1.3 (Sand)	External
Interlock Tension	2.0	1.5	1.1	Internal Check
Vertical Shear	1.5	1.25	1.1	Internal Check
Horizontal Shear	1.5	1.25	1.1	Internal Check
Sheet Pile Pullout	1.5	1.25	1.1	Internal Check

The terms Normal, Temporary and Seismic are assumed to mean the following (based on provisions in EM 1110-2-2504 [Ref. #15]):

Normal: (Defined as Usual loading in EM 1110-2-2504 [Ref. #15]) Loads which are associated with frequent use of the facility's primary intended function. Loads associated with primary function would include Dead Load, Live Load and Hydrostatic Load.

Temporary: (Defined as Unusual Loading in EM 1110-2-2504 [Ref. #15]) Construction or Maintenance operations which produce infrequent loading of a short duration which exceed loads define in the usual condition.

Seismic: Extreme condition of short duration loading. Extreme condition loading would include Earthquake Loading.

Limit state checks are generally divided into internal and external checks. In general, external limit states are checks involving modes of failure outside of the cellular structure. For example, a bearing capacity failure would be a failure of the soil beneath the driven sheet pile; therefore, it would be considered an external check.

For the purposes of this study only internal checks are considered; this is for two reasons. First the bulkhead has been in service since 1953. If there were issues pertaining to the stability of the soil beneath the structure, these would have already become manifest in the performance of the structure since constructed. Secondly, the present issue with the bulkhead stems from ongoing corrosion of the sheet pile. Damage to the sheet pile would not tend to destabilize limit states which are independent of

the structure. Therefore, this study checks the factors of safety of the structure against the limit states of interlock tension, vertical shear and horizontal shear/tilting.

Sheet pile pullout is an internal stability check. However, this mode of failure is independent of the structural performance of the cells. Unlike vertical shear or horizontal shear, this limit state does not rely upon interlock tension in the cells to resist load. Therefore, given that the wall has remained stable since 1953, and the variables affecting this limit state are not time dependent, it is reasonable to conclude that this limit states does not control the design.

8.1. INTERLOCK TENSION

The soil fill inside the cellular structure applies radial pressure against the sheet pile. This pressure is resisted via interlock tension perpendicular to the vertical axis of the sheet pile. Typically peak interlock tension occurs at approximately the bottom quarter point of the cell.

The most direct consequence of section loss is a reduction of the amount of steel resisting arc tension. In PND's experience, the most likely cause of failure for cellular structures is the corrosion rupture of sheet pile at the closure arc/main cell joint.

Assessing the remaining capacity of the sheet pile is difficult to do accurately. For the purposes of investigating the condition of the bulkhead, PND reduced the interlock capacity by the ratio of remaining metal to original metal. On this basis, the sheet piles have lost 30% to 46% of their capacity.

For the Lutak Dock there are three areas where interlock tensions are estimated. The first is in the main cells away from the closure arcs. The second area where interlock tension is computed is at the connector between the main cell and the closure arc. This area carries higher tension than the main cell due to the geometry of the connection.

The third area considered is the interlock stresses in the remaining sections of the closure arcs. The repairs in approximately 2003 required that the toe on the replacement sheet pile bear against soils which in turn bear on the remaining sections of the closure arc sheet pile. This toe bearing pressure exerts substantial lateral pressures on this remaining closure arc sheetpile. Table 8-2 summarizes the results of the analysis of interlock tension checks.

Table 8-2. Interlock Tension - Limit State Capacities/Demands

Analysis Results – Limit State Capacities/Demands						
Limit State	Computed Capacity	Dead Load	Live Load (Surcharge)	Live Load (Axle Load)	Hydrostatic (WSE = 0')	EQ (72-yr Return)
Interlock Tension (Closure Arc)	9.2 kips per Inch	6.4 kips per Inch	1.3 kips per inch	~ 0 kips per inch	~ 0 kips per inch	--
Interlock Tension (Main Cell)	10.9 kips per inch	6.9 kips per inch	1.3 kips per inch	~ 0 kips per inch	2.0 kips per inch	0.5 kips per inch
Connector Tension	9.2 kips per inch	10.7 kip per inch	--	--	--	--

Industry standard for interlock tension factor of safety is 2.0 for normal conditions and 1.5 for temporary conditions. The factors of safety for other tension tests are summarized in Table 8-3.

Table 8-3. Interlock Tension - Factors of Safety

Factors of Safety				
Limit State	DL - Only	DL + LL(Sur.)	DL + Hydro	DL+ EQ (72-yr Return)
Interlock Tension (Closure Arc)	1.44	1.19	1.44	1.33
Interlock Tension (Main Cell)	1.58	1.33	1.22	1.47
Connector Tension	0.86	--	--	--

Table 8-3 indicates that under normal conditions the bulkhead does not meet either the normal or temporary factor of safety requirements.

Further, interlock tension at the connector between closure arc and main cells has a factor of safety less than 1.0. This indicates that these elements should be either about to fail or are in the process of failing. This fact is supported by field observations made by Echelon and noted in their report that 6 of the 10 closure arcs have perforations in their sheet pile.

8.2. VERTICAL SHEAR

Cellular structures achieve stability via the effects of the mass they possess. As a result, applied horizontal loads are resisted via cantilever action of the cellular structure. The amount of lateral load which can be resisted by the closed cell is governed by two primary limit states: vertical shear and horizontal shear/tilting. These two limit states are critical to stability and represent the "classic" structural check for a cellular structure.

The vertical shear check is analogous to the shear checks performed when sizing a timber beam. Mobilized shear stresses at the neutral axis are checked against an estimated shear capacity. A cellular structure resists vertical shears through interlock friction and soil shear resistance which act together to produce total resistance.

The equations for structural/geotechnical check for vertical shear are provided in Ref. #12. The primary mechanism to mobilize vertical shear is the confining pressure on the retained soil resulting from the exterior ring of flat-web sheet piles. This confining pressure enables the soil to resist vertical shears via inter-soil friction. The soil friction and the interlock friction are converted into moment resistance by multiplying these mid-line friction by 2/3 the effective width of the cell.

Table 8-4 summarizes the vertical shear capacity and loading demand.

Table 8-4. Vertical Shear - Limit State Capacities/Demands

Analysis Results – Limit State Capacities/Demands						
Limit State	Computed Capacity	Dead Load	Live Load (Surcharge)	Live Load (Axle Load)	Hydrostatic (WSE = 0')	EQ (72-yr Return)
Vertical Shear (Moment Cap.)	4065 k-ft	2580 k-ft	938 k-ft	204 k-ft	930 k-ft	475 k-ft

Table 8-5 depicts selected factors of safety for the bulkhead checked for vertical shear.

Table 8-5. Vertical Shear - Factors of Safety

Factors of Safety				
Limit State	DL - Only	DL + LL(Sur.)	DL + Hydro	DL+ EQ (72-yr Return)
Vertical Shear	1.57	1.16	1.16	1.33

Table 8-5 indicates factors of safety which do not meet current industry standards. Current industry standard require factors of safety of 1.5 for normal conditions and 1.25 for temporary conditions.

There are indications that the Lutak dock is susceptible to vertical shear. Structures which have low factors of safety for vertical shear would be expected to tilt outward. The 1976 inspection report (Ref. #7) indicated that the bulkhead was leaning outward several inches confirming that a slow "creeping" of the structure is occurring. Further, photographs from the 2014 Echelon report (Ref. #4) depict that the facing beam is cracked near the mid-line of the end cell. This may be an indication of shear deflection occurring about the mid-line of the main cells.

8.3. HORIZONTAL SHEAR/TILTING

The horizontal shear check investigates the stability of the upper portion on the cell against rotation. This check computes the moment resistance of both the sheet pile interlocks (see 8.2 Vertical Shear) and the moment resistance of a wedge of soil at the bottom of the closed cell. If there is insufficient horizontal shear to resist applied loading, the bulkhead will tilt outward.

The moment resistance of the soil wedge is estimated by assuming a wedge of soil, inclined at the friction angle of the soil, bears against the leading edge of the bulkhead. The magnitude of the bearing force against the side of the leading edge of the bulkhead is equal to the amount of shear on the incline plane of the soil wedge.

The moment resistance from the soil wedge is added to the moment capacity from sheet pile friction and then compared to moment demand to compute a factor of safety for horizontal shear/tilting.

As-built drawings provided to PND (Ref. #11) indicate that Lutak Dock sheet pile tip elevations taper upward toward the rear of the cells. The sheet pile tips at the back of the cells are estimated to be 10' higher than at the front (Ref. #1). Since this is the case, estimates of capacity and demand are computed for the tip elevation of the rear piles at El. -36.5'.

Table 8-6 summarizes the vertical shear capacity and loading demand.

Table 8-6. Horizontal Shear/Tilting - Limit State Capacitates/Demands

Analysis Results – Limit State Capacities/Demands						
Limit State	Computed Capacity	Dead Load	Live Load (Surcharge)	Live Load (Axle Load)	Hydrostatic (WSE = 0')	EQ (72-yr Return)
Horizontal Shear (Moment Cap.)	4504 k-ft	1739 k-ft	703 k-ft	See Surcharge	609 k-ft	357 k-ft

Table 8-7 depicts selected factors of safety for the bulkhead for various limit states.

Table 8-7. Horizontal Shear/Tilting - Factors of Safety

Factors of Safety				
Limit State	DL - Only	DL + LL(Sur.)	DL + Hydro	DL+ EQ (72-yr Return)
Horizontal Shear (Moment Cap.)	2.64	1.87	1.95	2.19

The results in Table 8-7 demonstrate that horizontal shear does not control the capacity of the bulkhead. Further, the factors of safety computed for horizontal shear satisfy current industry standards.

8.4. Liquefaction Analysis Results

PND investigated the potential for liquefaction of the soils at the Lutak Dock site. PND considered the data from two boreholes taken during the 2003 repairs (Ref. #1). These were borings B-2 and B-6. These boreholes as well as the seismic data for the project site were entered into the computer program Shake 2000. This software estimates the Factors of Safety against liquefaction versus elevation in the soil column/stratification. This analysis indicated that the soils would not liquefy during the maximum earthquake the bulkhead appears able to withstand (72-year return period). The analysis did indicate that some soil layers are susceptible to liquefaction at earthquake events of higher intensity and prolonged ground shaking. The effect of an earthquake at magnitude levels specified by national design codes would most likely demonstrate that the bulkhead structure would fail under prolonged ground shaking. Elaborate analysis would be necessary to demonstrate the earthquake effects, and to reflect the weakening effects of corrosion on the various structural elements.

8.5. Condition of Main Cells

The main cells retain a higher percentage of their Z-sheet pile section than the closure arcs. The bulkhead would not meet the current industry standard of care. Interlock stress exceed allowable factors of safety for even temporary conditions. Further, the bulkhead does not meet the standard of care for vertical shear capacity. Field observations of the structure may indicate that tilting, potentially due to vertical shear, has been an on-going problem. Photographs taken of the Lutak Dock by Echelon may indicate shear deflection of the bulkhead at the mid-line.

8.6. Condition of Closure Arcs

The effects of corrosion loss of section at the closure arc have reduced the steel thicknesses substantially. The modifications recommended in Ref. #1, to install a z-sheet wall has resulted in redistribution of wall loading downward into the remaining lower portion of the closure arc. These modifications require the toe of the Z-sheet pile walls to bear against soils behind the closure arc and exert considerable load. Calculations reveal that applied loads are sufficient to cause the Z-sheet pile wall to fail at the closure arc/main cell connector. This finding is confirmed by photographs from the 2014 Echelon survey (Ref. #4). The report noted that 6 of 10 of the closure

arcs have significant perforation in sheet piles. Two of the 10 closure arcs have either failed or partially failed.

The consequences of the closure arc failure are significant. Structural support to the lower embedded portion of the z-sheet piles is immediately compromised or eliminated, with corresponding outward movement of the soil mass and potential loss of material between the main cell and the z-sheets. This failure would also manifest itself by the formation of sink holes on the dock surface behind the face of the bulkhead as the retained soils behind the z-sheets move outward and down. This appears to be occurring (refer to Figure 3-2), based on the observed formation of sink holes and by evidence, noted in Echelon's report, that fine grained soil has been lost from behind the closure walls.

The Shannon and Wilson geotechnical report (Ref. #1) indicates that the repair wall sheet pile are intended to provide vertical support to the rear of the facing beam. Loss of fill would erode at the vertical load carrying capacity of the repair sheet pile potentially leading to local collapse of the face beam.

Soils confined by the closure arcs stabilize the new columns supporting the face beam. Without lateral support from the closure arc sheet pile and surrounding soil the H-Piles supporting the facing beam may have insufficient structural capacity to support applied load.

The presence of yielding at Closure Arc 7.5 indicates high tensile forces in the closure arc sheet pile. There is the potential that these high interlock forces could begin to compromise the main cell sheet pile. Typically, the main cells in cellular structures are most susceptible to tension stresses at the connection between the closure arc and the main cell. Given the extent of marine growth (Figure 3-3), it is not known if these connector piles have been inspected for perforation or damage.

9. RECOMMENDATIONS

Given that the bulkhead is euphemistically working on "borrowed time" it is PND's primary recommendation that planning for full replacement begin as soon as credibly possible. The bulkhead does not meet required factors of safety for normal operating conditions of self-weight dead load with surface live load and operating vehicles, and cannot withstand a design level earthquake. Therefore, due to the age of the structure and the inspected deterioration, PND does not believe repairing the existing facility is a viable option.

Given the failure of two closure arcs, and the poor conditions of four others, it is prudent to halt vehicle operations in areas defined by the closure arc z-sheet pile walls and the primary cells. These areas can be marked to prevent personnel and equipment from being inadvertently placed in areas where sink holes have developed or where a latent void may collapse.

Failure of the primary cells is less likely based on the condition assessment of Ref. 4 though keeping a watchful eye on the connecting tee between the closure and primary cell is wise. Rupture of the connecting tee between the closure arc and primary cell occurred on two projects (for which PND was retained to provide repairs) and in each case there was no obvious triggering event or advance warning. Marking the locations where damaged closure arcs exist (consistent with Ref. #4) is recommended given the potential of these locations to fail with limited or no warning. Monitoring

identified areas of distress (on a monthly basis) is prudent so see if there is discernable degradation of the bulkhead as time moves forward.

10. REFERENCES

The following references were used to prepare this report.

Ref. #1: Geotechnical Report Lutak Dock Improvements, Haines, AK – 2002. Prepared by Shannon and Wilson.

Ref. #2: Geotechnical Report for Haines Ferry Terminal, Haines, AK – 1983. Prepared by Alaska Department of Transportation and Public Facilities.

Ref. #3: Geology Data Report, Haines Terminal Improvements. Dated 2005. Prepared by Alaska Department of Transportation.

Ref. #4: Echelon Engineering Report – Inspection and Assessment of Lutak Dock, Haines, AK. Prepared May 2014.

Ref. #5: Echelon Engineering Report – Underwater Inspection Lutak Dock, Haines, AK. Dated April 2002.

Ref. #6: Condition Survey Lutak Dock – Prepared by PND Engineers. Date October 1988.

Ref. #7: Haines-Lutak Inlet Port Facility Engineering Condition Report – Dated April 1976. Prepared by R&M Consultants.

Ref. #8: Lutak Dock Rehabilitation Project Drawings. Prepared by Reid Middleton. Dated November 2003.

Ref. #9: NOAA Tides and Current. Data from Skagway, AK recording station.

Ref. #10: United States Geologic Survey Applet.

Ref. #11: 1953 Haines Expansion Port Facilities Project. As-Built drawings of the Bulkhead.

Ref. #12: United States Army Corps of Engineers Design Manual EM 1110-2-2503 – Design of Sheet Pile Cellular Structures, Cofferdams and Retaining Structures.

Ref. #13: “Cellular Cofferdams – Developments in Design and Analysis,” Clough and Martin, 1988.

Ref. #14 Grayman, R. (1970), Cellular structures failures, Proceedings, Conference on Design and Installation of Pile Foundations and Cellular Structures, ed. H. Y. Fang and T. D. Dismuke, Envo Publishing Co., Lehigh Valley, pp. 383–391.

Ref. #15 United States Army Corps of Engineers Design Manual EM 1110-2-2504 – Design of Sheet Pile Walls.

REF #1

Geotechnical Report Lutak Dock Improvements, Haines, AK – 2002. Prepared by Shannon and Wilson.

Geotechnical Report
Lutak Dock Improvements
Haines, Alaska

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GEOTECHNICAL REPORT
LUTAK DOCK IMPROVEMENTS
HAINES, ALASKA

1.0 INTRODUCTION

This report presents the results of our subsurface explorations, laboratory testing, and geotechnical engineering studies for proposed Lutak Dock improvements in Haines, Alaska. The purpose of this geotechnical study was to explore subsurface conditions and provide stability analyses and foundation design recommendations for the proposed improvements. More specifically, the proposed improvements will replace the corroded portion of the existing sheet pile structures that constitute the face of the dock. To characterize the subsurface conditions for geotechnical design of the improvements, seven soil borings were advanced within the proposed area of development. Soil samples recovered from the borings were tested in our geotechnical laboratory. Presented in this report are descriptions of the site and project, subsurface exploration and laboratory test procedures, an interpretation of subsurface conditions, and conclusions and recommendations from our studies.

Authorization to proceed with this work was received in the form of a written contract between Shannon & Wilson and Reid Middleton on March 18, 2002. Our work scope was conducted in general accordance with our December 2001 proposal. The total cost of our contract was reduced by 4 percent to accommodate project budget restraints.

2.0 SITE AND PROJECT DESCRIPTION

The project area (Lutak Dock) is located approximately 4 miles north of Haines, Alaska. Lutak Dock is located on the coast of a sheltered inlet connected to the Lynn Canal. The portion of the dock that comprises this study is adjacent to and northwest of the Haines Ferry Terminal. Generally undeveloped land and more shoreline is located to the northwest. Directly upland or southwest of the dock is the staging area for the dock and aforementioned ferry terminal. A vicinity map is included as Figure 1 that shows many of the features described above.

According to technical drawings of the site provided to us, the dock was constructed with a single row of circular cellular cofferdams (approximately 67 feet in diameter). The upland areas behind the cells were backfilled to create a continuous, level dock surface with an elevation of

about 28 to 30 feet (MLLW). The seaward, wedge-shaped voids or gaps between the circular cells were bridged with additional sheet pile walls to create a relatively flat dock face. Figure 2 shows the general geometry of the existing structures at the dock. According to the City of Haines, segment of the sheet piles closing the wedge and in the tidal fluctuation zone (above elevation 0 feet) are corroding at the face of the dock and fill from behind the walls is being lost to the ocean. From diving surveys, we understand that the sheet piles forming the main cells and those pile segments below MLLW filling the wedges are not in need of repair/replacement.

We understand that repair of the corroded face area will take place as five general steps:

1. cut hole in the cap beam and drive vertical H piles to firm bearing to provide intermediate support of the cap beam in the wedge area.
2. Drive replacement Z sheet piles inside and upland of the existing corroded sheets and H piles to form a new, smaller, soil filled wedge between the circular cells.
3. Tie in the new H and sheet piles into the concrete cap beam.
4. Cut away corroding sheets to MLLW and remove soil outside new sheet piles.
5. Install intermediate horizontal supports to new sheets, if necessary.

These proposed improvements are also identified in Figures 2, 6 and 7.

The focus of exploration effort was to identify the nature of the existing subsurface conditions at the site. This includes subsurface soil conditions, especially classification of fill and native soils and their approximate thicknesses. In addition, geophysical work was conducted to attempt to define offshore subbottom conditions and determine the depth to which the existing sheet piles were driven. As-built drawings of the sheet pile lengths are (to date) not available for this facility. This report also presents the information that will be needed to design the proposed above repair including lateral earth pressures, allowable pile capacities, and embedment depths. Additionally, stability results are presented to assess dredging feasibility along with recommendations for upgrading the staging area with a higher quality unpaved surface.

3.0 SUBSURFACE EXPLORATIONS

The field explorations for this project were conducted on February 10 through February 13, 2002. Subsurface explorations consisted of advancing and sampling seven soil borings, designated Borings B-1 through B-7, to characterize the subsurface conditions. The approximate locations of the borings are identified on Figure 2. The depth of the borings ranged from 21.5 to

71.5 feet below the ground surface (bgs) to yield a total drilling effort of approximately 278 feet. Borings B-1 and B-5 were advanced over the seaward edge of the dock while the remaining borings were advanced through the fills behind the face of the dock.

The soils encountered were visually classified in the field according to the Soil Classification System that is presented in Figure 3; field classifications were later verified through laboratory analysis. Frost classifications were determined for the soil types based on visual and laboratory evaluation and are shown with grain size classification results on the boring logs. The frost classification system is presented in Figure 4. Detailed logs of the borings are presented in Appendix A.

Drilling services for this project were provided by Discovery Drilling, of Anchorage, Alaska, using a truck mounted CME-75 drilling rig. The borings were advanced with 3-$^1/_4$-inch inner diameter (ID), continuous-flight, hollow-stem auger. An experienced engineer from our office was present continuously during the fieldwork to locate the borings, observe drilling operations, recover soil samples, and log the subsurface conditions encountered in each boring.

As the borings were advanced, disturbed samples were typically recovered with a 2-inch outside diameter (OD) split spoon sampler using Standard Penetration Test (SPT) Procedures. In this test, samples were recovered by driving the sampler into the bottom of the advancing hole with blows of a 140-hammer free falling 30 inches onto the drilling rod. The number of blows required to advance the sampler the final 12 inches of a total 18-inch penetration is termed the Standard Penetration Resistance, which was recorded for each sample. These values are shown graphically on the boring logs, adjacent to the sample depth. The values give a measure of the relative density (compactness) or consistency (stiffness) of cohesionless or cohesive soils, respectively.

Upon completion of the borings, they were backfilled as much as possible with cuttings removed during drilling. As part of this backfilling, 2-inch, blank PVC pipe was inserted into the entire length of Borings B-2 and B-6 for later use by the geophysics subcontractor. The deeper borings (B-2, B-3, and B-6) did not produce enough cuttings and backfill material was borrowed from the City of Haines fill stockpile to completely fill these borings.

Two geophysical studies were conducted on the site by Apollo Geophysics. First, a bathymetric and subbottom over water survey was performed along the length of the existing dock (about 1,000 feet long) extending out approximately 50 feet from the dock face. Contours from the

marine survey were used to construct sea floor profiles of the apparent bottom (top of the mineral soils) and apparent sub-bottom (bedrock or other bottom reflectors).

For the second effort, a conduction probe was lowered into the PVC pipes in Borings B-2 and B-6. The intent of this survey was to determine the bottom of the sheet piles as the conductivity of the ground near the metal piles should drop off as the probe extends below the bottom of the sheet piles. The methods and results of the bathymetric, subbottom and in-hole surveys by Apollo Geophysics (and their report in its entirety) are included herein as Appendix B.

An engineer from our office positioned the boring locations in the field. Representatives from the USKH Juneau office surveyed the boring locations and elevations after explorations were complete. These surveyed locations are shown on Figure 2 and the surface elevations of the borings are presented on the boring logs in Appendix A.

4.0 LABORATORY TESTING

Laboratory tests were performed on selected samples recovered from the borings to confirm our field classifications and to determine the index properties of the typical materials encountered at the site. The laboratory testing was formulated with emphasis on determining the materials gradation properties, in situ water content, and frost characteristics. This data plus estimated strength and density properties determined from Standard Penetration tests provided information used in formulating our recommendations.

Water content tests (45 total) were performed on samples collected from the borings. Water content tests were generally conducted according to procedures described in American Society for Testing and Materials (ASTM) D-2216. The results of the water content measurements are presented graphically on the boring logs in Appendix A.

Seven grain size classification tests were conducted to confirm the field classifications of the typical soils at this site and to estimate permeability characteristics and frost susceptibility. The gradation testing generally followed procedures described in ASTM D-422. The grain size testing results are presented in Figure 5, and are indicated on the boring logs as percent gravel, percent sand, and percent silt. In addition, three tests were conducted on selected samples from our borings to determine the amount of fines passing the Number 200 (P-200) Sieve or 0.075mm. This testing was conducted in general accordance to ASTM D-1140. P-200 results are summarized on the boring logs in Appendix A.

5.0 SUBSURFACE CONDITIONS

5.1 Soil and Groundwater Conditions

The subsurface conditions encountered at the site are depicted in detail in the boring logs in Appendix A. Two soil profiles (Profile A-A' and B-B') were also created and are included as Figures 6 and 7, respectively. In general we encountered varying amounts of fill material (in the borings advanced upland of the dock face) overlying native sands and gravels, and what is believed to be bedrock at depth.

Fill material encountered by our borings was relatively variable in thickness and composition. In general, it consisted of greenish gray, silty, sandy gravel to silty, gravelly sand. As shown in the profiles, the fill appears to be thickest near the face of the dock in Borings B-2 and B-6 with thicknesses of approximately 36.5 and 42 feet, respectively. Penetration resistance values in the fill generally ranged from 10 to 57 blows per foot, averaging around 20 blows per foot. This suggests the fill has an average medium dense consistency. This material is also classified as slightly to moderately frost susceptible (F1 to F2) due to the amount of silt or fines shown in Figure 5. From cutting returns and relatively rough drilling action, the presence of cobbles to 6 inches in diameter could possibly exist in the fill materials, however, this larger material is probably not significantly persistent through the fill soils.

Native soils, primarily slightly silty, gravelly sand, and slightly silty, sandy gravel, encountered by our borings tended to be similar to the fill material, however, according to our boring logs and laboratory testing, it appears that they are somewhat cleaner, containing a smaller fine grained fraction. During sampling, the native soils exhibited minor heaving at the bottom of the auger during drilling. This is another indication of low soil cohesion and less silt content. The native material encountered by our borings was generally dense with an average blow count value of 30 blows per foot. Average moisture content of native soils was around 10 percent.

In the two borings advanced over the face of the dock, similar native soil conditions were encountered to those found in the upland borings. The only significant difference was a layer of soft, black, decayed organic "muck" or organic debris on the sea floor. This layer was approximately 9 and 3 feet thick in Borings B-1 and B-5, respectively. A similar organic layer, likely representing original ground, was also found at the interface of the fill and native soils in Boring B-2, though greatly compressed and only about a foot thick. Where it is offshore, it is

likely bark and timber slashing dumped off the dock to clean up the upland storage area when it was used as a timber/log loading facility.

Possible bedrock was encountered at the bottom of Borings B-5 and B-6 in Figure 7. Auger refusal was reached on each of these borings. Additionally, samples attempted at each location met strong sampler refusal. With each hammer stroke, the rods and sampler appeared to be bouncing on a solid surface. Unfortunately, had both these borings been drilled early in the program where the possibility of bedrock existing at relatively shallow depths would have been recognized, other borings could have been carried deeper to better define these conditions across the site.

Groundwater conditions in our borings varied across the site and are dependent on the fluctuating tides. As shown in the profiles in Figures 6 and 7, water levels in the borings ranged from 17 to 23 feet bgs in the vicinity of the cofferdams. It should be noted that, because of the proximity of this site to the tidal waters of the Lynn Canal, the groundwater level can be expected to fluctuated as much as 6 to 10 vertical feet at the dock face with the progression of the tide cycles, but probably much less (less than 5 feet) at greater distances back from the face.

5.2 Geophysics Results

The findings of geophysical studies performed by Apollo Geophysics can be found in their report in Appendix B. In general, the study concluded that the existing sheet piles near Boring B-2 and B-6 were driven to 64 and 55 feet below the level of the dock, respectively. According to the bathymetric survey results, the sea floor elevation at the face of the dock ranges from around -35 to -8 feet. The subbottom profiles generated by the overwater survey show a strong reflective layer (possibly bedrock) continuously across the face of the dock. The elevations of this layer fluctuated from -70 to -35 feet.

6.0 ENGINEERING RECOMMENDATIONS

We understand that the rehabilitation of the dock structure will consist of several major adjustments: 1) a new bridging sheet pile section is to be driven approximately 12 feet upland of each of the existing closure arcs, 2) h-piles are to be driven through the existing concrete pile caps at the face of the dock for cap support between the cofferdam and existing closure arcs are to be removed along with the soil behind them to an elevation of approximately 0 feet, 3) a pile supported fender system is to be constructed in front of the dock face, 4) the working surface of

the dock is to be reconstructed. In addition to these structural improvements, the sea floor in front of the dock is to be dredged to an elevation of approximately -36 feet. The following sections provide geotechnical engineering recommendations regarding each aspect of the project described above.

6.1 Pile Recommendations

6.1.1 Sheet Piles

New sheet pile walls are to be constructed approximately 12 feet behind each of the existing closure arcs at the dock face. It is our understanding that the tops of the new Z sheets will be attached to the existing concrete cap beam as an upper restraint. After the sheet pile walls are constructed, the existing closure arcs (and the soil behind them) will be removed down to an elevation of 0 feet. This will create a free-standing height of the new Z sheets of between 20 and 25 feet. It is our understanding that, in addition to lateral loading from soils in the new wedge, the sheets will also support a vertical load of approximately 12 kips per linear foot of wall.

The sheet pile wall will derive vertical capacity primarily through tip bearing and as friction in the embedment soils below Elevation 0 feet. According to our analysis, the piles should be embedded to a tip elevation of -15 feet. This allowable capacity contains a factor of safety of 2 on the ultimate vertical capacity.

The recommended lateral earth pressures for the design of the new sheet pile wall segment between the circular cells are presented in Figure 8. These pressures assume that the free standing portion of the Z sheets retaining the soil will be tied into the cap beam and driven to a depth that will resist toe kickout. This lateral pressure also recognizes that part of the soil in the active wedge behind the wall will be supported within the circular cells and thus will not develop on the face of the new wall. Based on tidal fluctuations, water will collect behind the wall and at low tides will add hydrostatic forces on the wall. In adding hydrostatic pressures over the drawdown zone, the buoyant weight of the soil should be assumed in this region. Also any surcharge or live loads likely to occur from storage of equipment or supplies should also be added to the above pressures.

To resist toe kickout, the bottom of the sheet piles must be driven far enough below Elevation 0 feet to mobilize enough passive earth pressure from the soil immediately below the top of remaining closure arc steel not cut away, and into deeper soils that are completely confined. For

design, we recommend that this passive pressure be taken as an equivalent fluid pressure of 200 pcf in the top 6 feet below Elevation 0 feet and 400 pcf below this. These pressures include a factor of safety of 1.5 or more, but assume that the steel in the existing closure arc is adequate to resist these loads recognizing that dredging to Elevation -36 feet is well below the bottom of these sheet piles. Our preliminary calculations suggest that the embedment depth is about 10 feet, or 5 feet less embedment than required to support the 12 kip vertical line loads. The added embedment depth for vertical loads will thus control in the design, meaning the increased embedment is more than adequate to satisfy lateral load requirements or resistance against toe kickout.

6.1.2 H Piles

In determining the vertical capacities of the new H piles supporting the concrete cap beam and edge live loads, we assumed that the piles were driven through fill and native soils behind the existing dock face to an elevation of -55 feet. Since the H piles will be driven behind the existing closure arcs and derive support starting at the cutaway portion of the existing closure arcs (Elevation 0 feet), the calculated capacity must rely on the continued hoop support of the closure arcs below Elevation 0 feet. According to our analysis, the allowable vertical capacity of the proposed HP14x89 piles is 70 kips if driven to -55 feet elevation. This value is low compared to the nearly 400 kips that is desired. In addition to the HP14x89 piles, analyses were run on two larger H piles, 18 and 24-inch, to determine if the desired capacities could be achieved at the desired embedment depth. Allowable capacities for a 22-inch, closed-ended pipe pile were also estimated for this purpose. The allowable pile capacity curves for each pile size are shown on Figure 9. While the capacities were increased somewhat using larger piles, none of them approached 400 kips.

In order to achieve the desired higher pile capacities, the depth of embedment of the piles will need to be extended further than our deepest boring where bedrock or other soils exist. It is our opinion that if piles are carried to bedrock, an allowable capacity of 400 kips could be used for design if the steel stresses are within tolerable limits.

Profile C-C' shown on Figure 1 was prepared in an attempt to estimate the possible depth to bedrock. The location of this profile is shown in Figure 2. Our borings show rock, defined by auger refusal, to be about Elevation -45 feet at the dock face in Profile B-B' and greater than Elevation -60 feet at Profile A-A'. Superimposing an average band of 5 feet for organics on the Apollo Geophysics subbottom thickness data indicates that bedrock, or a dense reflector is

continuous across the dock face and may lie between Elevations -35 and -70 feet. This data is in general agreement with our drilling data at Profiles B-B' and A-A'.

Even though these geophysical data shows reasonable agreement between borings, geophysics is an interpretive tool for planning and should not, in our opinion, be used to estimate pile lengths for final design without more solid information to confirm continuity between borings. To confirm this continuity for estimating pile lengths, we recommend that probe holes be drilled at 5 additional locations along the dock face to determine refusal. At two of these locations, the nature of the bedrock should confirmed by drilling spot rock cores. Spot cores are 5-foot core runs to determine that rock is present and strong enough to accommodate the 400 kip loads and not a boulder.

6.1.3 Pipe Piles for Fender Systems

According to plan drawings provided to us, a new fender system is proposed for the face of the dock to accommodate horizontal ship loads. Each fender generally consists of two 22 ½ inch adjacent piles (with conical driving tips) driven off of the dock face. The plan shows the piles connected to each other via a durable, rigid plastic face and the top of each pile will be connected to the dock with a large shock absorber. Our analysis of these piles takes into account that a limited amount of horizontal deflection will be allowed by the shock absorbers at the top of each pile. It also accounts for the fact that total force on the piles is shared by both shock absorbers and foundation soils at the top and bottom of the piles, respectively. Estimated deflections and anticipated lateral forces were provided by Reid Middleton for our analysis.

Because these fender piles will be rigidly attached to the top of the dock face, we believe some degree of fixity is needed at the bottom of the pile to accommodate toe kick-out. To analyze this condition, we assumed a maximum deflection of 6 inches at the top of the pile (elevation 28.5 feet), and a shear force acting on the piles of 10 kips at Elevation 20 feet. Under these boundary conditions, we recommend that the piles be driven a minimum of 15 feet into the dense, sandy gravel to gravelly sand soils encountered by our borings in front of the dock. This depth does not achieve complete pile fixity, however, it is our opinion that it will allow enough resistance so that the pile tips will not lose lateral resistance at their tips under repeated loading.

According to the provided plans, the ocean floor in front of the dock is to be dredged to an elevation of -35 feet. Assuming that this dredging completely removes the very soft, decaying organic layer encountered by our two off-shore borings, a 15-foot embedment into the

underlying granular soils will make the elevation of the pile tip approximately -50 feet. Based on Figure 10, it is apparent that some areas over the dock face may have bedrock conditions shallow enough to prevent the above embedment.

If rock is too shallow, we recommend that the driven piles be seated in or anchored to the bedrock to create additional lateral resistance. This may be accomplished by several possible methods including:

1. Predrilling the rock and physically anchoring them in.
2. Installing an h pile stub on the tip with a rock driving show and drive the pile until some embedment is achieved.
3. Drilling through the pipe pile into the rock and installing rock anchors below the tip.

As stated previously, we recommend that additional probe holes be advanced off of the dock face to confirm the depth and nature of the bedrock. Once cores of the rock are obtained, it should be possible to determine which of the above anchoring methods are best suited for this project.

6.1.4 Pile Driving

For the above piles, we recommend that a pile driving hammer having a rated energy of about 50,000 foot pounds be used for the above piles. Hammers with greater or less than the above value could be used provided the contractor can demonstrate that he can drive the piles to the desired embedment depths without damage to the piles. Piles driven to bedrock will meet refusal once the pile tip contacts and penetrates the upper foot or more of bedrock. Refusal on or within bedrock should be considered to occur when driving resistance exceeds 20 blows per inch.

As shown in Figure 10, the elevation of the bedrock surface varies from -40 feet to -65 feet. For this reason, we recommend that the previously described probes be advanced behind the dock face to confirm the bedrock elevations assumed in this report. We also recommend that a continuous driving record (in blows per foot) should be taken for the entire depth of all constructed piles. Shannon & Wilson is available to perform these observations or to provide on-call consulting services as conditions are encountered that need to be evaluated.

6.1.5 Pile Settlements

According to our borings, the embedment soils are largely dense and granular and many of the piles driven for the proposed improvements will be driven to bedrock. Therefore, it is our opinion that total statically induced settlements of piles supported in the native sands and gravels would be small, generally on the order of ½-inch or less. Because of the dense, granular nature of the native soils, the differential settlements for piles founded on soils driven adjacent to piles founded on bedrock should be about half of the above total value. Piles driven to bedrock should be subject to settlements less than ¼-inch. Most of these settlements will occur nearly as fast as the loads are applied such that long term static settlements should be negligible.

6.2 Dock Resurfacing

It is proposed that the unpaved surface of the dock is to be resurfaced with the new gravel section to provide a smooth, well drained staging area for containers, dock machinery, etc.... We understand that the current dock surface performs poorly during spring breakup and periods of high rainfall. Water collects in large puddles causing soft surface conditions and deep rutting. We believe that the poor dock performance is due to the generally low quality fills penetrated by our borings in the upper 2 to 3 feet under the dock surface. Our borings encountered a relatively silty, sandy fill material with small amounts of gravel. It is our opinion that the upper levels of the existing fill should be replaced with a more gravelly fill with less fines.

6.2.1 Subgrade Preparation

Before reconstructing the dock surface, steps should be taken to prepare the site to receive the new structural fill material. Assuming that the design grade of the new dock surface is to be approximately the same as the existing surface, the upper 2 feet of existing material should be excavated and disposed of off site. The exposed subgrade should then be examined and probed to detect possible loose or soft zones that may be present.

Special attention should be given to identification of potential areas of high organic contamination in the existing fills. We understand that the dock was used extensively in the past for logging purposes and that the potential for related detritus (wood chips, sawdust, timber, bark/slash, etc...) is relatively high. If persistently loose or organic rich zones are detected at the subgrade level, we recommend that these areas be excavated locally and replaced with compacted Select Material Type A structural fill. A description of the recommended structural fills and compaction procedures are presented in Section 6.2.3.

Once the subgrade is exposed and repaired as described above, it should be sloped at a 2 percent grade to allow for drainage to the edges of the dock to prevent ponding in the structural fills to be placed on the subgrade. The final subgrade surface should be "proof-rolled" with several passes of a vibratory roller to provide a firm surface to receive the structural fill material. This process should compact the existing fill soils that may have been disturbed during the preparation processes described above.

6.2.2 Structural Section and Drainage

We recommend a 24-inch structural fill section to be placed above the prepared subgrade. In descending order, this section should consist of 6 inches of surface course (C-1 Aggregate) over 18 inches of base course (Select Material Type A). The recommended structural fill material along with compaction procedures are described in Section 6.2.3. We believe that this section will provide adequate support for the anticipated loads imposed by dock equipment, vehicles, and containers.

To provide for adequate drainage of surface water on the new dock surface, we recommend that the final grade of the dock surface be constructed to maintain the drainage paths set by the prepared subgrade toward the edges of the dock. In other words, all points of the surface should be sloped to a 2 percent grade to allow water to shed away from the dock surface toward the edges of the dock. Doing so should prevent possible ponding and soil softening during spring break-up and periods of high rainfall.

As long as this unpaved surface does not become contaminated with silt or organics with use, the above design section, in our opinion, is suitable for future direct paving with asphalt. If the surface becomes contaminated with use, it should be removed locally and upgraded to the above section before paving.

6.2.3 Structural Fill and Compaction

Structural fill will be required for the reconstruction of the dock surface. Classified structural fill that is placed should be clean, granular soil to provide drainage and frost protection. These soils should contain less than about six percent (by weight, based on the minus 3/4-inch portion) passing the No. 200 sieve. Both C-1 Aggregate and Select Material Type A mentioned in the recommended structural section meet these requirements. The gradation requirements for each of these materials are shown on Figure 11. A detailed description for Select Material Type A

can be found in Paragraph 1 of Section 703-2.07 of the ADOT&PF's Standard Specification for Highway Construction. A detailed description of the C-1 material is also specified in Section 703-2.03 of the above reference. According to our borings, the on-site silty sands and silty gravelly sands do not meet the requirements for either the base course or surface course described above.

Base course to be used in the reconstruction of the dock surface should be placed in lifts not to exceed 10 inches loose thickness, and compacted to 95 percent of the maximum density as determined by the Modified Proctor compaction procedure (ASTM D-1557). To construct the surface course layer, the material should be placed in two 3-inch lifts. Compact the first lift so that the coarse aggregate partly embeds its way into the sandy subbase materials to develop a strong bottom lift. Place the second lift and compact to 95 percent of the maximum density as determined by the Modified Proctor compaction procedure (ASTM D-1557).

6.3 Cofferdam Stability

To determine the existing dock stability and the stability of the dock after the proposed improvements are constructed, a generalized profile was analyzed using a Windows compatible computer-aided program, GSTABL7, Version 2. GSTABLE7 is a two dimensional, limit equilibrium slope stability program used with STEDwin, a graphical user interface to evaluate and print out the results. The summarized results for each static analysis are included as Figures 12 and 13, respectively. Stability results under dynamic (earthquake loading) are exhibited on Figures 14 and 15, respectively.

Several assumptions were made to carry out the stability analysis. Approximate soil parameters were determined for the soil conditions depicted in our borings. These parameters are listed on the attached stability output graphs on Figures 12 through 15. In addition to soil parameters, the bedrock profile was assumed given the limited data provided by our field investigation and geophysical results. It was assumed that the existing sheet piles for the cofferdams were driven to a tip elevation of -46.5 feet at the dock face and -36.5 feet at their upland extents. Final dock geometry after the proposed renovations was interpreted from a set of preliminary plan sheets provided by Reid Middleton. Lastly, a seismic loading coefficient of 0.1g was assumed for stability analysis under earthquake (dynamic loading) conditions.

Because of the existence of shallow bedrock and the presence of the cofferdam structures, we believe that if this structure were to fail, it would do so as a block confined by the closed cell

cofferdam. In defining the case parameters, failure zone initiation points were designated at both the toe and head of the dock, and a probable failure zone beneath the sheet pile cells was assumed. When initiated, the computer program randomly searches for a failure surface (in this case, through the predetermined failure zone beneath the cofferdam) and calculates the factor of safety for each possible failure surface given the above listed assumptions. The ten most critical failure surfaces produced are then detailed in the program output. Static analyses output are displayed on Figures 12 and 13 and dynamic loading analyses results are included as Figures 14 and 15. The most unstable surface generated for each analyses are marked "a" and are drawn in red on each figure. The resultant factors of safety for critical surfaces are presented in the table in the upper left hand corner of each of the figures. It should be noted that (due to limitations in the analysis program) the elevation scale on the right side of each figure have been adjusted so that none of the point or line coordinates would have a negative value. Therefore, MLLW (Elevation 0 feet) appears as Elevation +70 feet on these four figures.

The results shown from the stability analyses clearly indicate that the existing dock structure is stable under the assumed static and dynamic loading conditions. According to our analysis, the change in the stability factor of safety, from the existing structure to the structure after improvements are made, is very small. We believe structural enhancements including additional structural support gained by new H and pipe piles and some unloading of the head of the slope (removal of closure arcs and soils) counteract some of the stability lost due to dredging at the toe of the cofferdams.

Because the bedrock elevation in the area was observed to be somewhat variable, the model was altered to determine the sensitivity of stability with varying bedrock conditions. This sensitivity analysis revealed that the stability of the structure did not significantly change as the bedrock elevation was lowered. In addition, the factor of safety of the structure against sliding increased as the bedrock elevation increased. When the top of the bedrock approached the bottom elevation of the existing sheet piles, the factor of safety (under dynamic loading conditions) increased to over 2.0.

6.4 Dredging Considerations

As mentioned previously, dredging of the seafloor directly in front of the dock will take place as part of the improvements to the existing dock. According to the available preliminary drawings, the proposed elevation to which the floor will be lowered to is approximately -35 feet. This elevation has been marked on the soil profiles on Figures 6, 7 and 10. On preliminary drawings,

the elevations of the existing sheet piles are represented as -46.5 feet. For the purposes of performing the above stability analysis on the proposed improvements, we assumed that the elevations of all of the sheet pile wall tips are -46.5 feet. It is important the note, however, that if this assumed elevation is not correct, and the existing cofferdams are in fact undermined by dredging, the stability analysis performed above would not apply. Undermining could cause significant subsidence in the dock surface and capacities of the new piles would be adversely affected especially lateral resistance in the new sheet piles. To assure that dredging below the front tips of the existing sheet piles will not occur, we recommend that initial dredging be carried out in a few local areas to Elevation -40 feet at the face. This should be followed by diving to confirm that the sheets are present at those depths. Once confirmed, wider spread dredging may take place.

According to our borings and the geophysical results summarized in Figure 10, the material to be removed by dredging will consist of mostly sandy gravel to gravelly sand. Soil classifications indicate that these soils are relatively clean with a low silt content (estimated at less than 10 percent). In addition to the granular soils, the soft organic layer encountered by both offshore borings will be dredged from the bottom. Using several cross sections provided by Reid Middleton, and the cross sections in Figures 6 and 7, we estimate that, of the total amount of dredged material, 15 to 25 percent will be this soft, organic matter.

Figure 10 shows that the rock surface may also be encountered at a few locations. Since this rock surface only rises a few feet above Elevation -35 feet, we expect that dredging in these areas will stop when and if rock is encountered in these areas. The recommended five probes, described previously, should help further provide confidence as whether bedrock will or will not be encountered during future dredging.

7.0 CLOSURE AND LIMITATIONS

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they presently exist. It is assumed that the exploratory borings are representative of the subsurface conditions throughout the site, i.e., the subsurface conditions everywhere are not significantly different from those disclosed by the explorations.

If, during construction, subsurface conditions different from those encountered in these and prior explorations are observed or appear to be present, Shannon & Wilson should be advised at once so that these conditions can be reviewed and recommendations can be reconsidered where necessary. If there is a substantial lapse of time between the submittal of this report and the start

of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, it is recommended that this report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

We recommend that we be retained to review those portions of the plans and specifications pertaining to earthwork and foundations to determine if they are consistent with our recommendations. In addition, we should be retained to observe construction, particularly the driving of piles at this site.

Unanticipated soil conditions are commonly encountered and cannot fully be determined by merely taking soil samples or advancing borings. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. Therefore, some contingency fund is recommended to accommodate such potential extra costs. Shannon & Wilson has prepared the attachments in Appendix C "Important Information About Your Geotechnical/Environmental Report" to assist you and others in understanding the use and limitations of the reports.

Sincerely,
SHANNON & WILSON, INC.

Prepared by,

Kyle Brennan
Geotechnical Engineer

Reviewed by,

Fred R. Brown, P.E.
Senior Vice President

GRADATION REQUIREMENTS

SELECT MATERIAL TYPE A

U.S. STANDARD SIEVE SIZE		PERCENT PASSING BY WEIGHT
4 in.	100 mm	100
2 in.	50 mm	85 - 100
No. 4	4.75 mm	20 - 55
No. 200	0.075 mm	0 - 10

C-1 AGGREGATE

U.S. STANDARD SIEVE SIZE		PERCENT PASSING BY WEIGHT
1.5 in.	37.5 mm	100
1 in.	25.0 mm	70 - 100
3/4 in.	19.0 mm	60 - 90
3/8 in.	9.5 mm	45 - 75
No. 4	4.75 mm	30 - 60
No. 8	2.36 mm	22 - 52
No. 40	0.425 mm	8 - 30
No. 200	0.075 mm	0 - 6

NOTE:
Gradation requirements referenced from Paragraph 1 of Section 703-2.07 of the ADOT&PF's Standard Specification for Highway Construction.

Ingra/Gambell Street Rail Overpass
Anchorage, Alaska

GRADATION REQUIREMENTS

February 2001 | 32-1-01384

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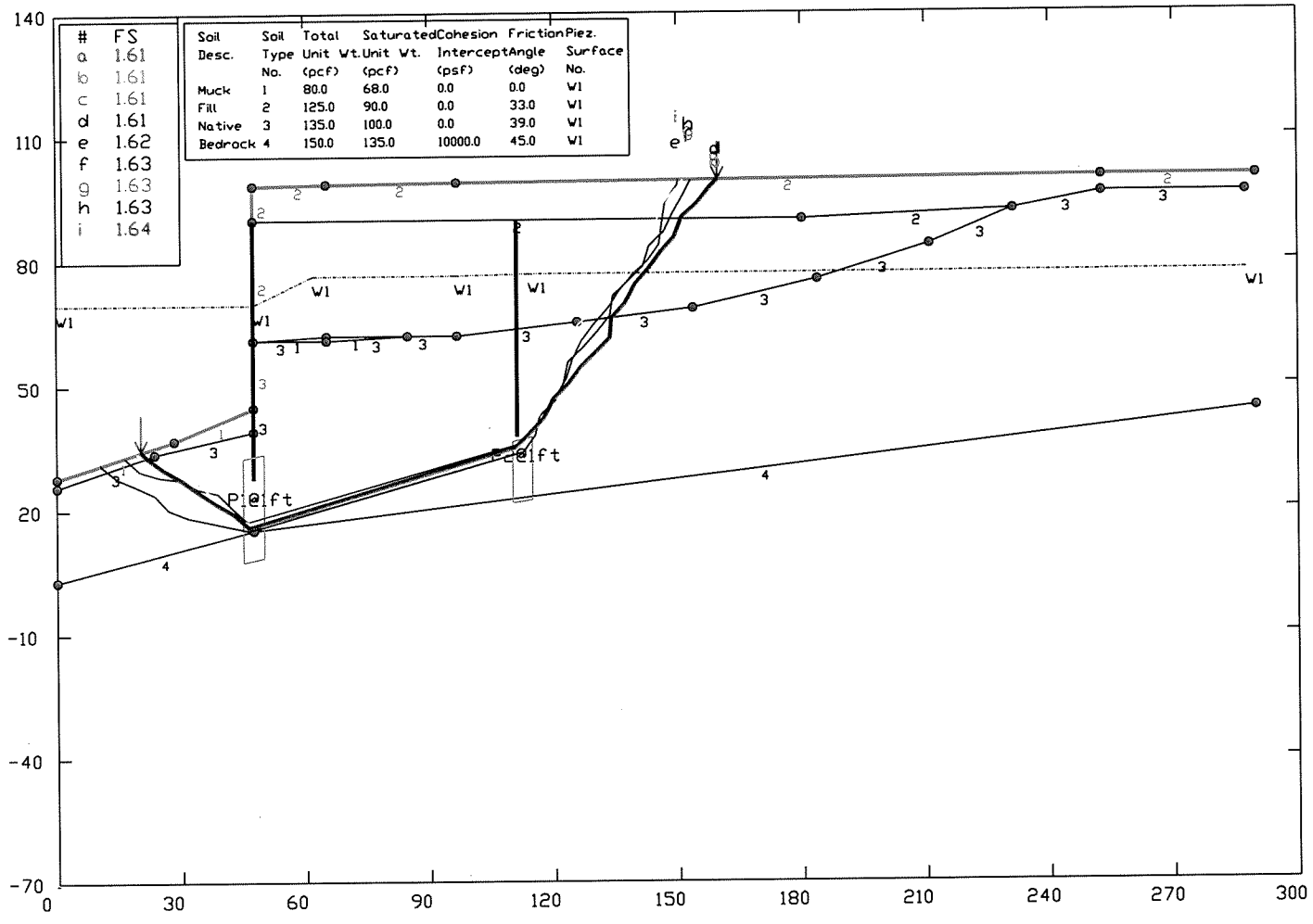
Fig. 11

Static Stability -- Existing Structure

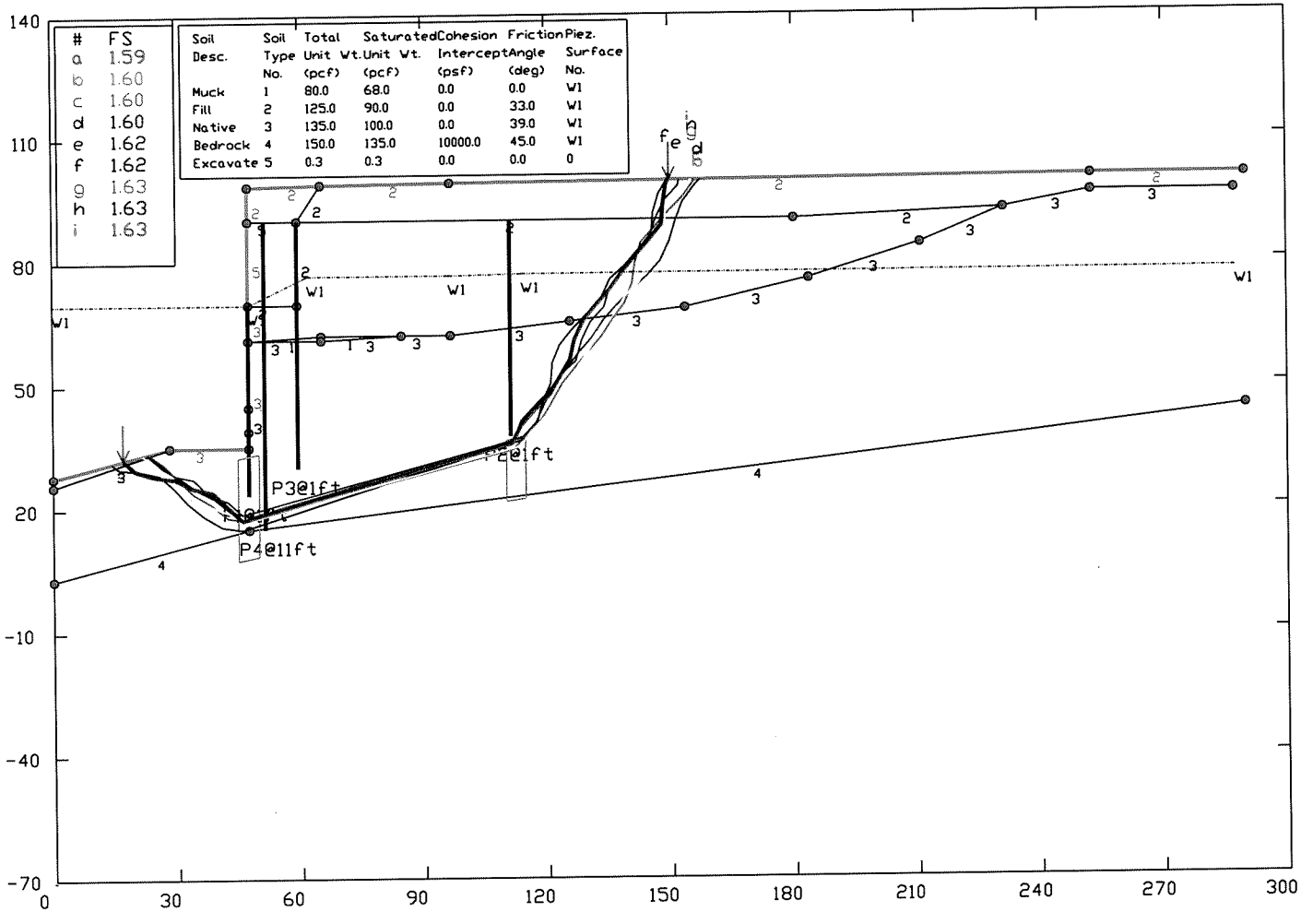


Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7 v.2 FSmin=1.61

Lutak Dock Improvements Haines, Alaska	
STATIC STABILITY -- EXISTING STRUCTURE	
April 2002	32-1-01482
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants	FIG. 12

Static Stability -- After Improvements



Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7 v.2 FSmin=1.59

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STATIC STABILITY -- AFTER IMPROVEMENTS

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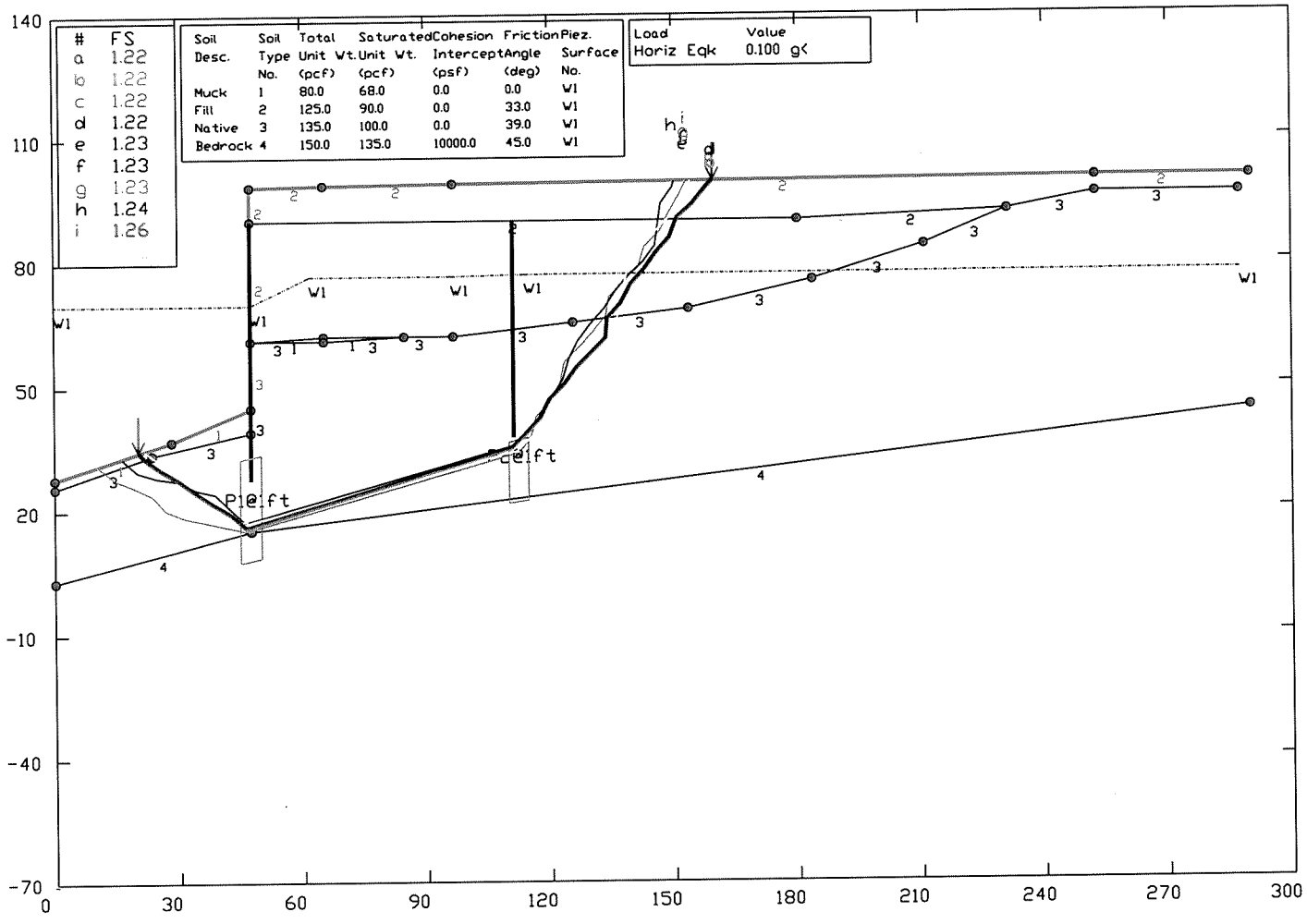
FIG. 13

Dynamic Stability -- Existing Structure

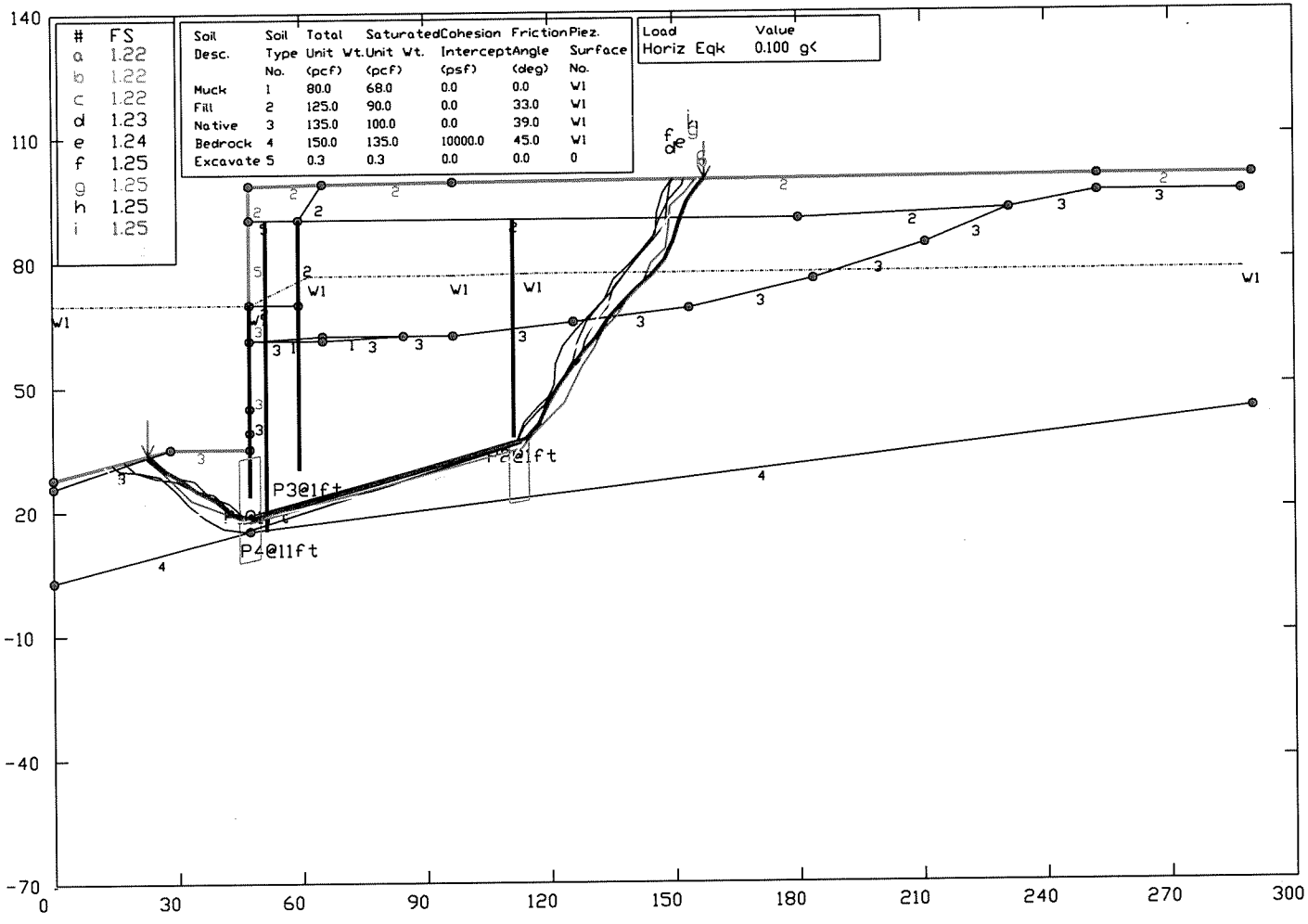


Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7 v.2 FSmin=1.22

Lutak Dock Improvements Haines, Alaska
DYNAMIC STABILITY -- EXISTING STRUCTURE
April 2002 — 32-1-01482
SHANNON & WILSON, INC. Geotechnical & Environmental Consultants — FIG. 14

Dynamic Stability -- After Improvements



Safety Factors Are Calculated By The Simplified Janbu Method for the case of c & phi both > 0

GSTABL7 v.2 FSmin=1.22

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DYNAMIC STABILITY -- AFTER IMPROVEMENTS

April 2002 32-1-01482

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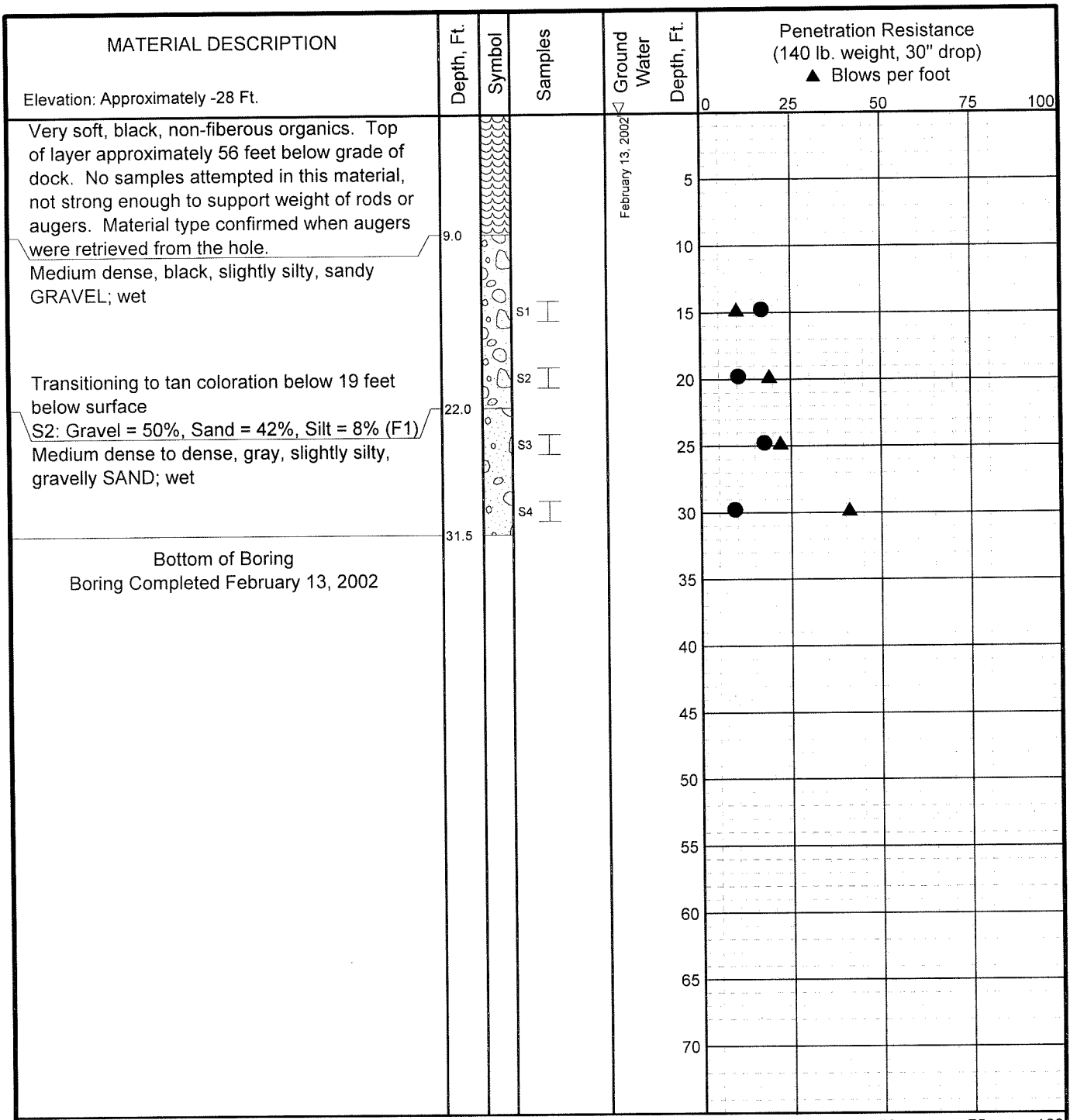
FIG. 15

APPENDIX A

BORING LOGS



LEGEND

* Sample Not Recovered

2" O.D. Split Spoon Sample

Ground Water Level At Time Of Drilling

Static Water Level

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

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Haines, Alaska

LOG OF BORING B-1

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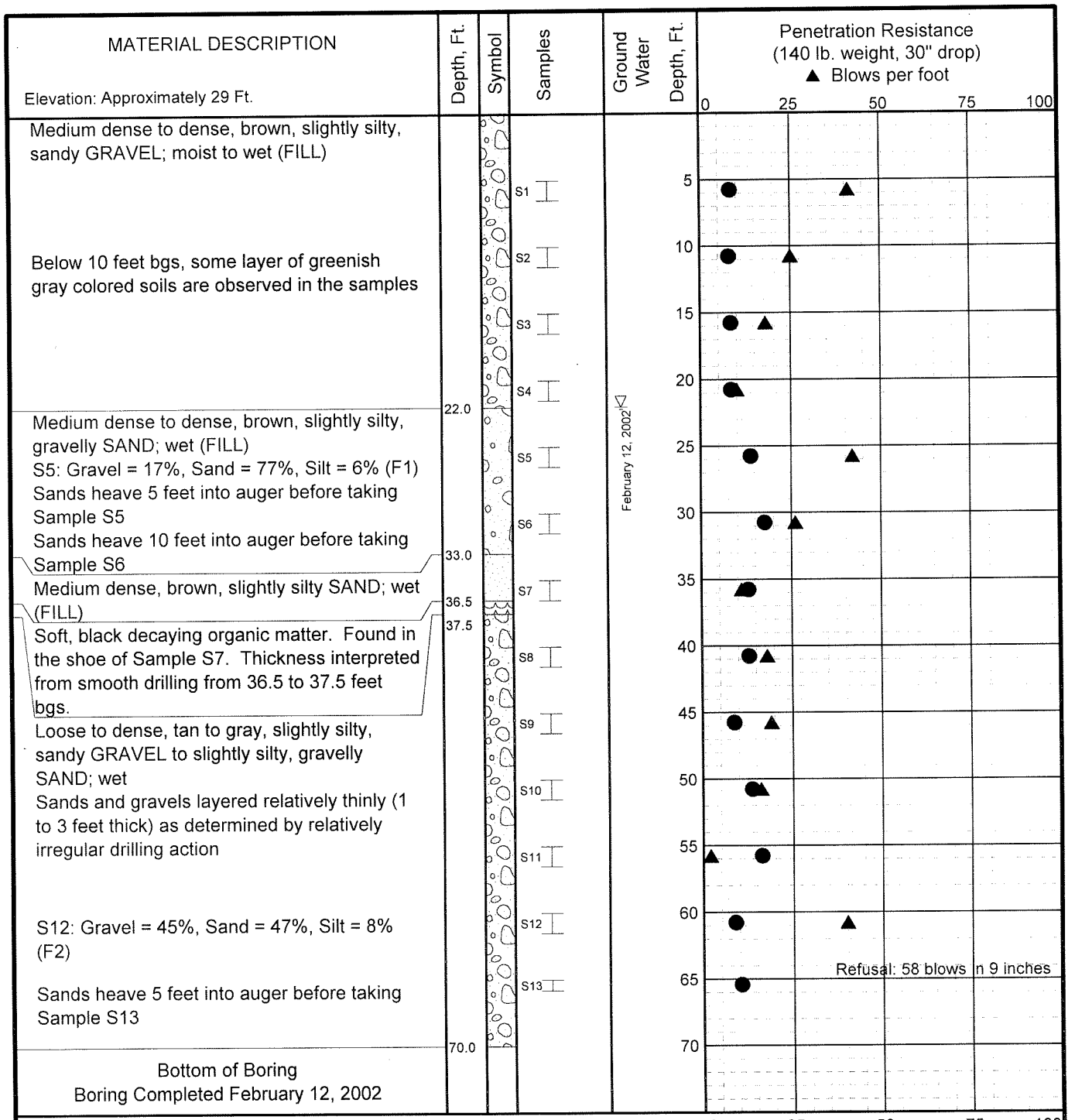
Fig. A-1

S&WGEO1_LOG_A1482.GPJ S&W_GEO1.GDT 4/12/02



LEGEND

* Sample Not Recovered

2" O.D. Split Spoon Sample

Ground Water Level At Time Of Drilling

Static Water Level

● % Water Content

Plastic Limit ⊢●⊣ Liquid Limit

Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

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Haines, Alaska

LOG OF BORING B-2

April 2002 32-1-01482

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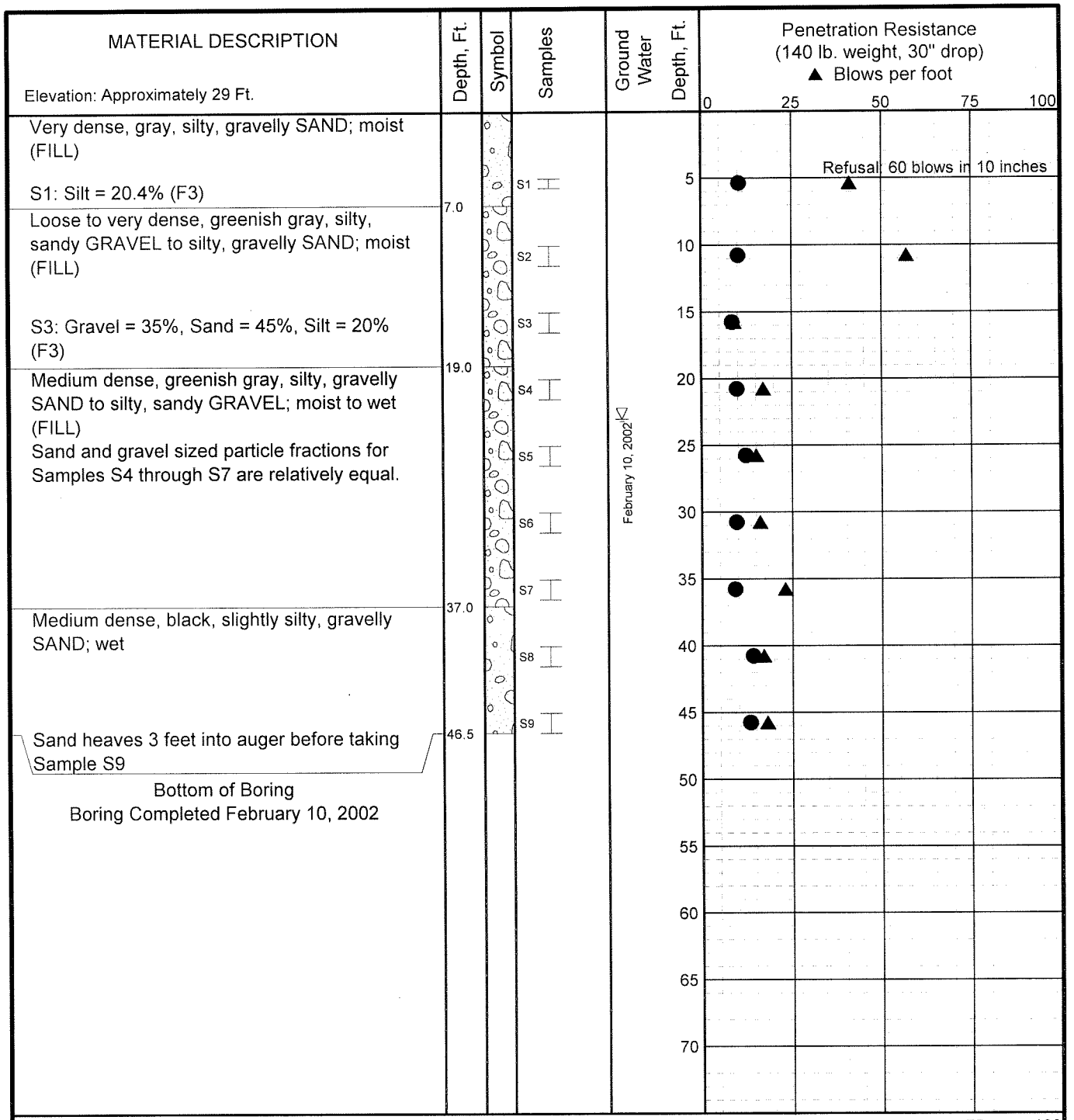
Fig. A-2

S&WGEO1 LOG A1482.GPJ S&W GEO1.GDT 4/12/02



LEGEND

* Sample Not Recovered

2" O.D. Split Spoon Sample

Ground Water Level At Time Of Drilling

Static Water Level

● % Water Content

Plastic Limit ⊢●⊣ Liquid Limit

Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

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Haines, Alaska

LOG OF BORING B-3

April 2002 32-1-01482

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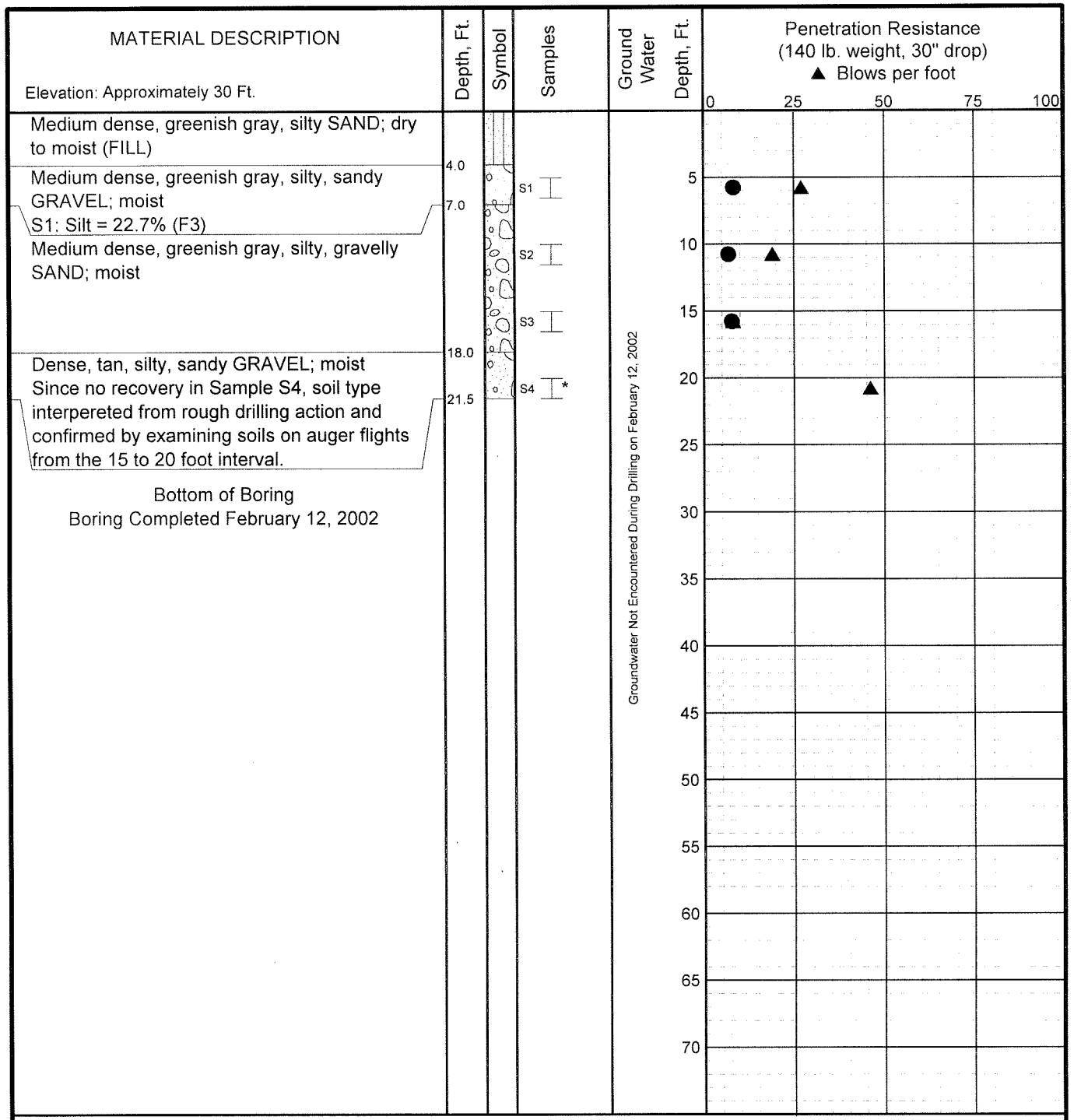
Fig. A-3

S&WGEO1_LOG_A1482.GPJ S&W_GEO1.GDT 4/12/02



NOTES

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2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

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Haines, Alaska

LOG OF BORING B-4

April 2002 32-1-01482

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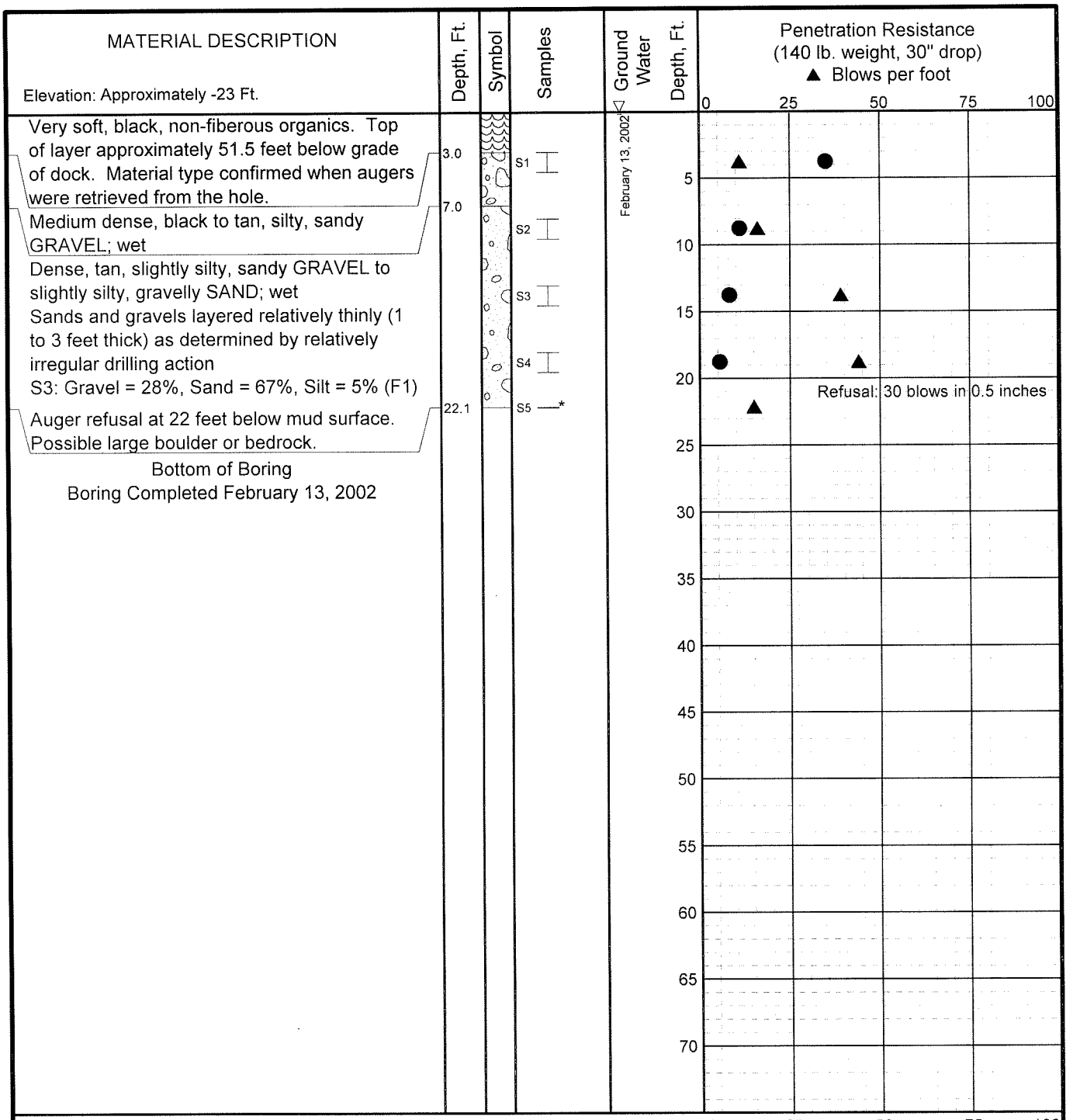
Fig. A-4

S&WGEO1_LOG_A1482.GPJ S&W GEO1.GDT 4/12/02



NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

Lutak Dock Improvements
Haines, Alaska

LOG OF BORING B-5

April 2002 32-1-01482

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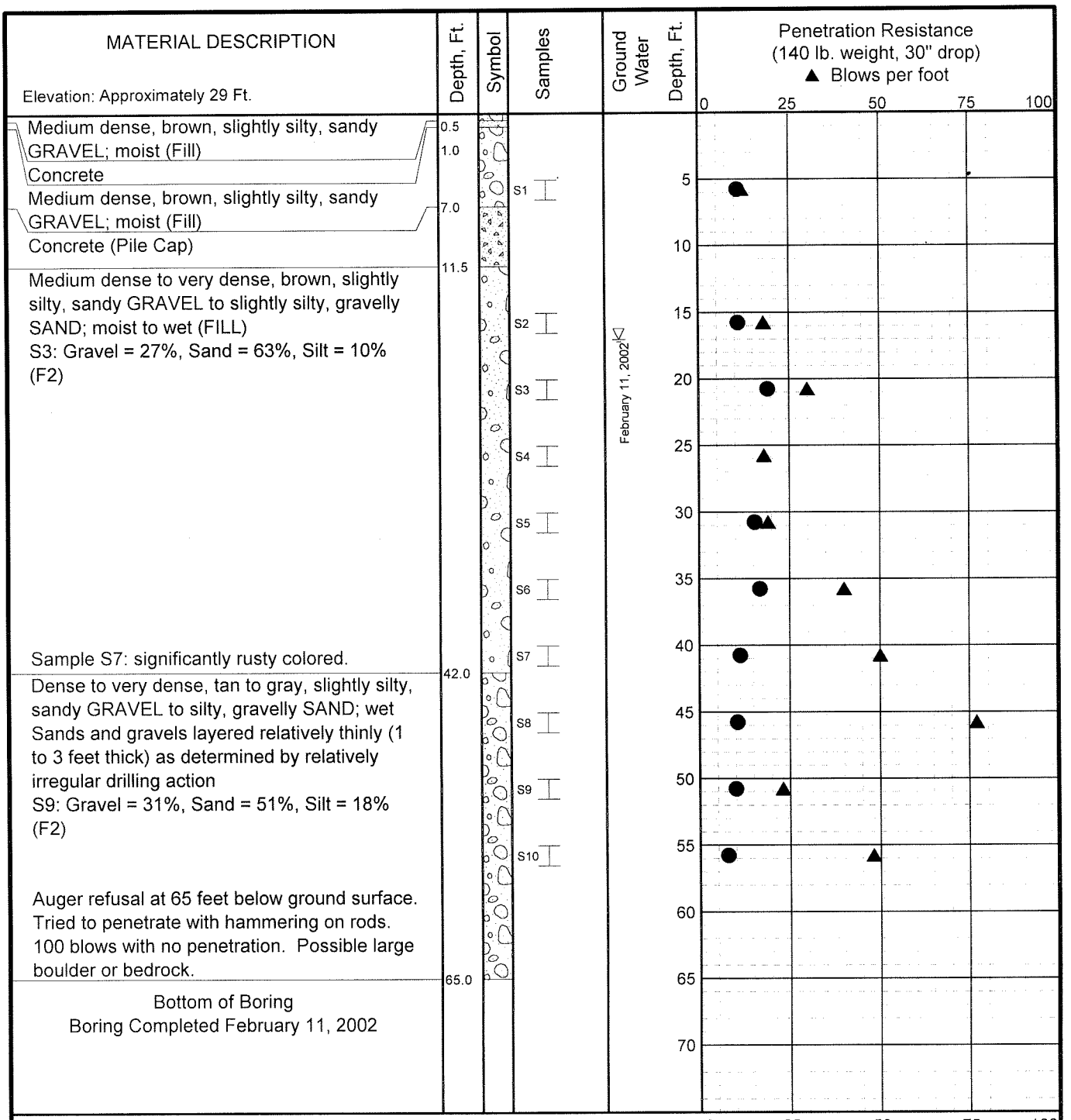
Fig. A-5

S&WGEO1_LOG_A1482.GPJ S&W_GEO1.GDT 4/12/02



LEGEND

* Sample Not Recovered

2" O.D. Split Spoon Sample

Ground Water Level At Time Of Drilling

Static Water Level

● % Water Content

Plastic Limit ⊢●⊣ Liquid Limit

Natural Water Content

NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

Lutak Dock Improvements
Haines, Alaska

LOG OF BORING B-6

April 2002 32-1-01482

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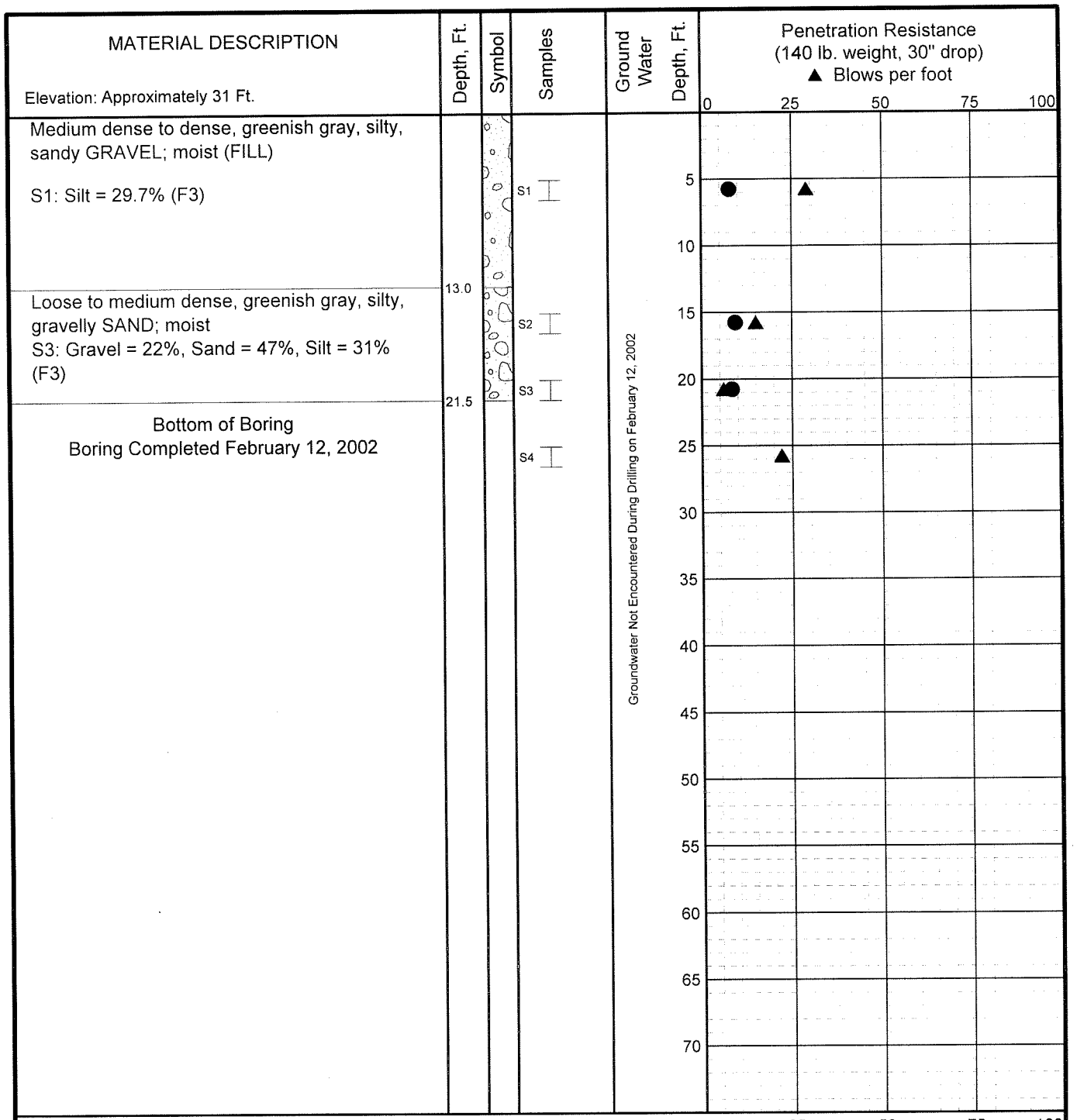
Fig. A-6

S&WGEO1_LOG_A1482.GPJ S&W_GEO1.GDT 4/12/02



NOTES

1. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
2. The discussion in the text of this report is necessary for a proper understanding of the nature of subsurface materials.
3. Water level, if indicated above, is for the date specified and may vary.

Lutak Dock Improvements
Haines, Alaska

LOG OF BORING B-7

April 2002 32-1-01482

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Fig. A-7

S&WGEO1_LOG_A1482.GPJ_S&W_GEO1.GDT 4/12/02

APPENDIX B

APOLLO GEOPHYSICS CORPORATION REPORT

APOLLO GEOPHYSICS CORPORATION
Engineering, Hydro-Geology, Environmental, & Construction

Monday, April 15, 2002

Fred Brown
Shannon & Wilson, Inc
5430 Fairbanks Street
Anchorage, Alaska 99518-1263

AGC File No.: **02.1003**

Re: **Borehole Conductivity & Marine Geophysical Exploration**
Lutak Dock Rehabilitation
Haines, Alaska

Dear Mr. Brown,

This report presents the results of our Borehole Conductivity measurements and Marine Geophysical Exploration located at Lutak Dock in Haines, Alaska. A two-person field crew from **APOLLO GEOPHYSICS** completed the field exploration on February 13 -16, 2002 and February 24 - 25, 2002. This revised report supercedes a previous submittal dated March 29, 2002.

BOREHOLE CONDUCTIVITY MEASURMENTS

Borehole conductivity measurements were completed in wells 2 and 6. The data from Well-2 shows a slight dipole at a depth of approximately 64 feet. This may be an indication of the base of the sheet pile, but the data is not conclusive. An increase in conductivity occurs in the data between depths 37 to 44 feet. This zone corresponds fairly well to an area in the boring log that contains organics and may relate to the natural bottom materials. The conductivity plot in relation to depth for Well-2 is shown in Figure 1.

The data from Well-6 shows a strong dipole at a depth of approximately 55 feet that may correspond with the base of the sheet pile. There are a few minor variations in the data above 55 feet that are probably associated with the local geology. The most notable variation would be a slight increase in conductivity between depths 34 and 39 feet. This zone corresponds fairly well to an area in the boring log containing rust colored soils that lie directly above a dense to very dense unit. This area may relate to the natural bottom materials. The conductivity plot in relation to depth for Well-6 is shown in Figure 2.

The instrumentation utilized for this effort consisted of an industry standard downhole logging system with an electromagnetic sensor tool. The tool senses changes in conductivity within the adjacent borehole materials approximately 2 to 6 feet out from the tool location.

MARINE GEOPHYSICAL EXPLORATION

Data from five sub bottom profiles were collected running parallel to the dock in a 10-foot grid to the northeast, Profiles 10, 20, 30, 40 and 50. Profile 10 is located approximately 10 feet northeast from the dock, Profile 20 is located approximately 20 feet northeast from the dock and so on. Location control for the profiles was obtained using a mobile GPS unit attached to the survey boat. Vertical control is based upon bathymetry measurements and referenced to MLLW. The approximate location of the Profiles is shown in Figure 3, Profile Location Map.

The sub-bottom information was obtained with an Edgetech X-Star Chirp Sub Bottom Profiler and recorded on optical disk media. The assumed velocity of the overburden soil (bottom material) was 5800 feet per second. The bottom and sub bottom data from Profiles 10 through 50 is presented in topographic format in Figure 4, Bathymetric Map and Figure 5, Sub-Bottom Elevation Map. The obtained data corresponds to soil borings located in the vicinity of the survey. Based upon correlation to the soil borings, we believe the first interpreted interface below the bottom represents the local bedrock. The interface was continuous throughout the data. A review of the data was performed

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at your request, late in the analysis, and did not reveal credible data with regard to the location of potential organic debris on the bottom. In addition to the data presented for each surface an interpreted isopach map of the bottom material thickness is shown in Figure 6, Bottom Thickness Map.

The bathymetric data was obtained utilizing industry standard equipment and methods. A bar check was completed at a known water depth for calibration of the instrumentation. The bathymetric data is presented as a topographic map (Figure 2). It should be stressed that the bathymetric data is presented to assist your analysis, but is not of sufficient density or quality for design purposes. The primary purpose of the bathymetric map was to provide a basis by which to determine interpreted thickness of the bottom materials above the bedrock as applied to the collected field data.

WARRANTY OF SERVICES

All geophysical information presented is based upon geophysical measurements made by generally accepted methods and field procedures and **APOLLO GEOPHYSICS'** interpretation of these data. The geophysical results are, therefore, interpretative in nature and are considered to be a reasonably accurate presentation of existing conditions within the limitations of the methods employed. Services performed by **APOLLO GEOPHYSICS** under this agreement are conducted in a manner consistent with, but no less than, that level of care skill ordinarily exercised by members of the profession currently practicing under similar conditions. We cannot guarantee the accuracy or correctness of any interpretation, and we shall not be liable or responsible for any loss, cost, damages or expenses incurred or sustained by the Client resulting from any interpretation made by any of our officers, agents or employees. No other warranty, expressed or implied, is made. **APOLLO GEOPHYSICS** recognizes that subsurface conditions may vary from those encountered at the location where geophysical or other explorations are made. The data interpretations and recommendations made by **APOLLO GEOPHYSICS** are based solely on the

information available to them at the time of performance; and **APOLLO GEOPHYSICS** shall not be responsible for the interpretation, by others, of the information developed.

We trust this will complete your requirements for this portion of the project and look forward to working with you on the next portion of this project. If you have any further questions or need further assistance, please don't hesitate to call.

Sincerely,

APOLLO GEOPHYSICS CORPORATION

Lynn M. Ringstad
Project Geologist

Clyde A. Ringstad, RG, CEG
Senior Geophysicist

Figure 1 - Borehole Conductivity Well 2

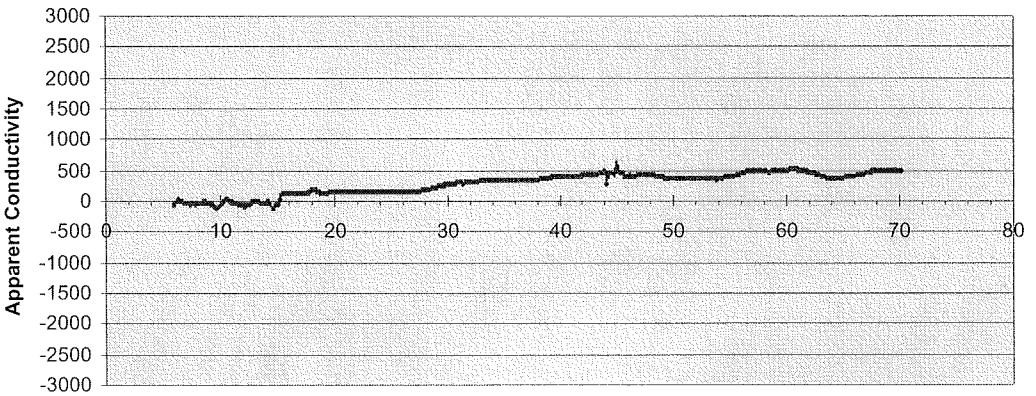
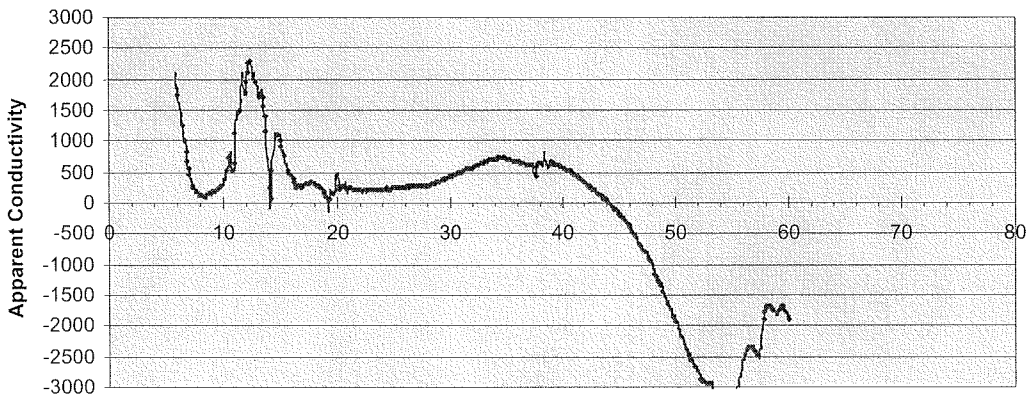
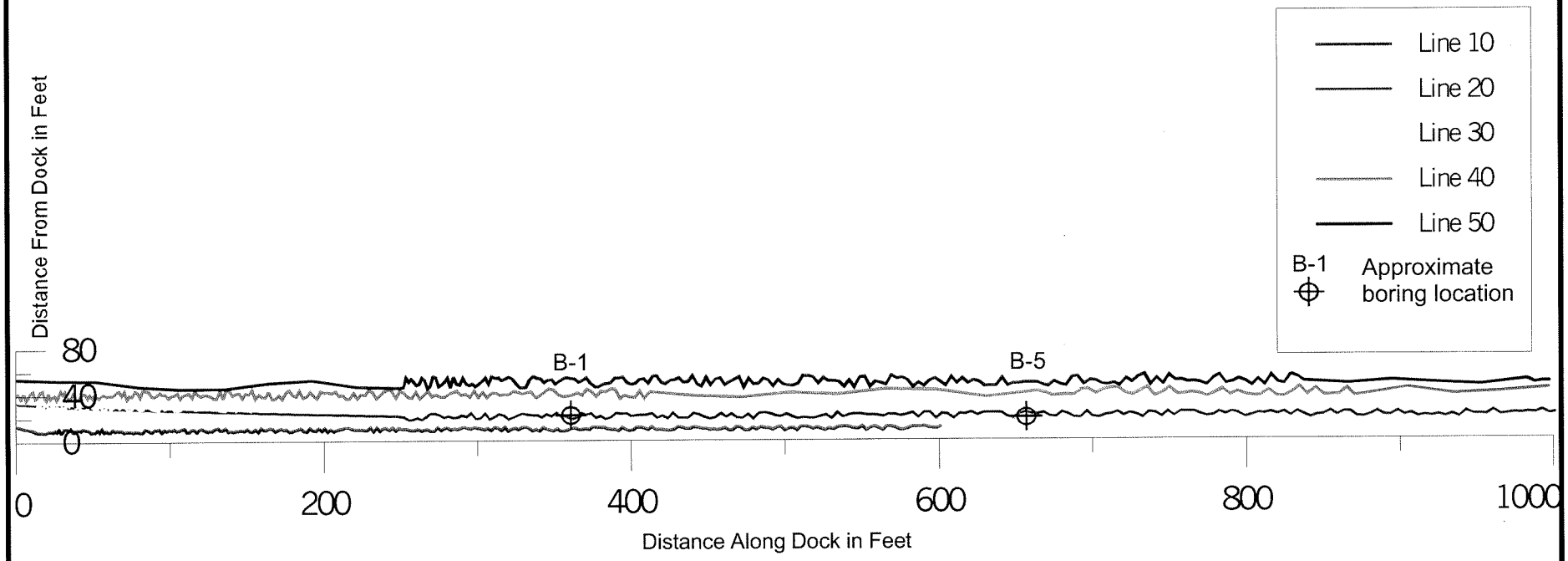


Figure 2 - Borehole Conductivity Well 6



Profile Location Map



0 100 200

Approximate Scale 1" = 100'

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Lutak Dock Rehabilitation
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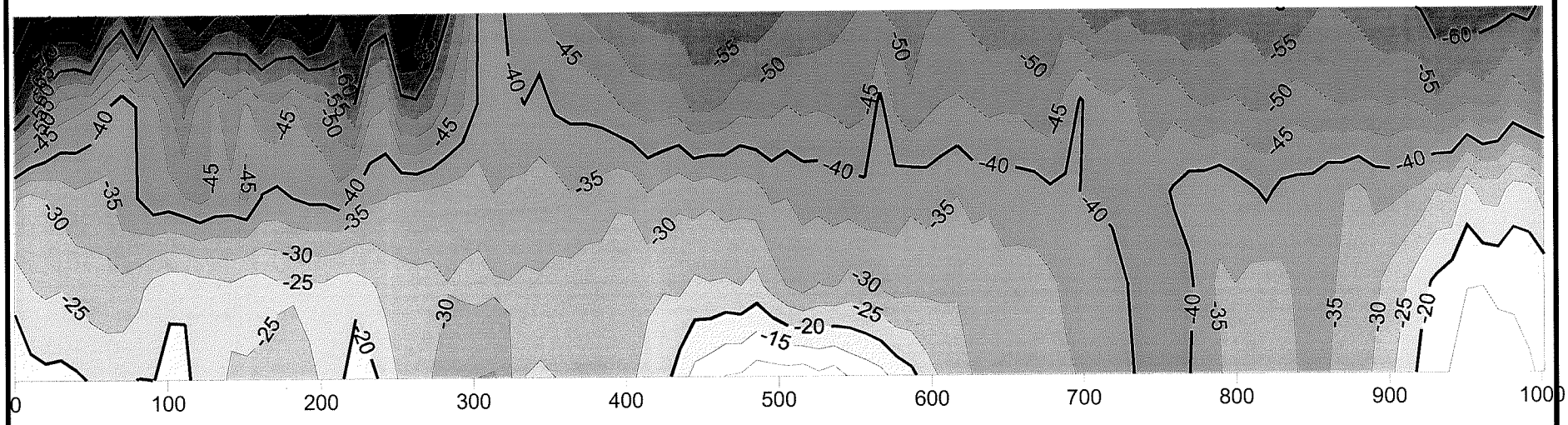
FIGURE 3

FILE NO. 02.1003

DATE April 2002

Geophysics\2002 Project Reports & Data\02.1003\02.1003Line Map

Bathymetric Map



0 100 200

Approximate Horizontal Scale 1" = 100'

0 20 40

Approximate Scale 1" = 20'

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Lutak Dock Rehabilitation
Haines, Alaska

FIGURE 4

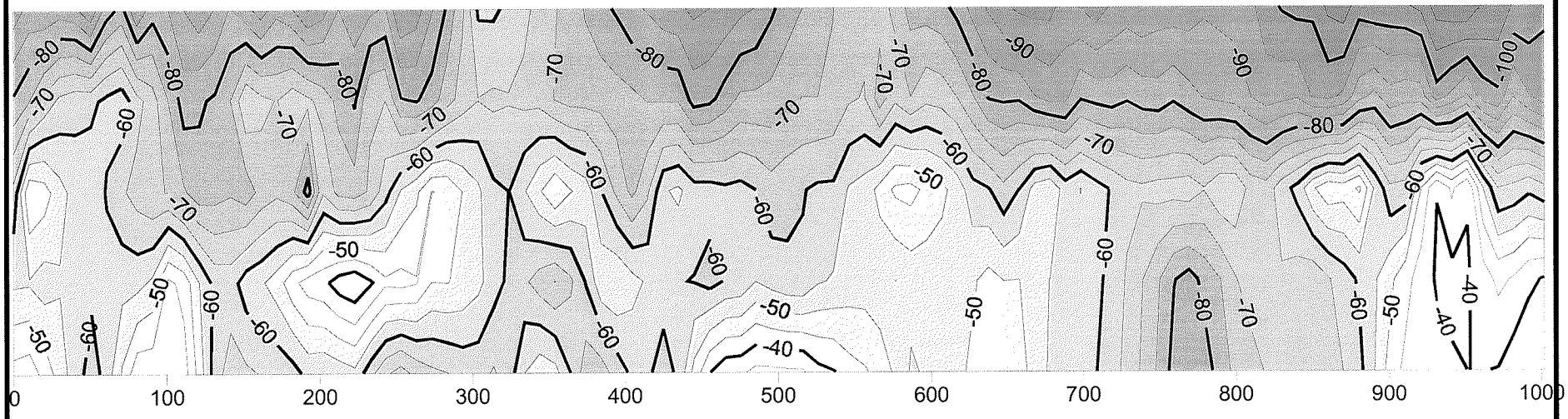
FILE NO. 02.1003

DATE April 2002

www.apollogeophysics.com

Geophysics\2002 Project Reports & Data\02.1003\02.1003 bathymetry

Sub-Bottom Elevation Map



Dock

0 100 200

Approximate Horizontal Scale 1" = 100'

0 20 40

Approximate Scale 1" = 20'

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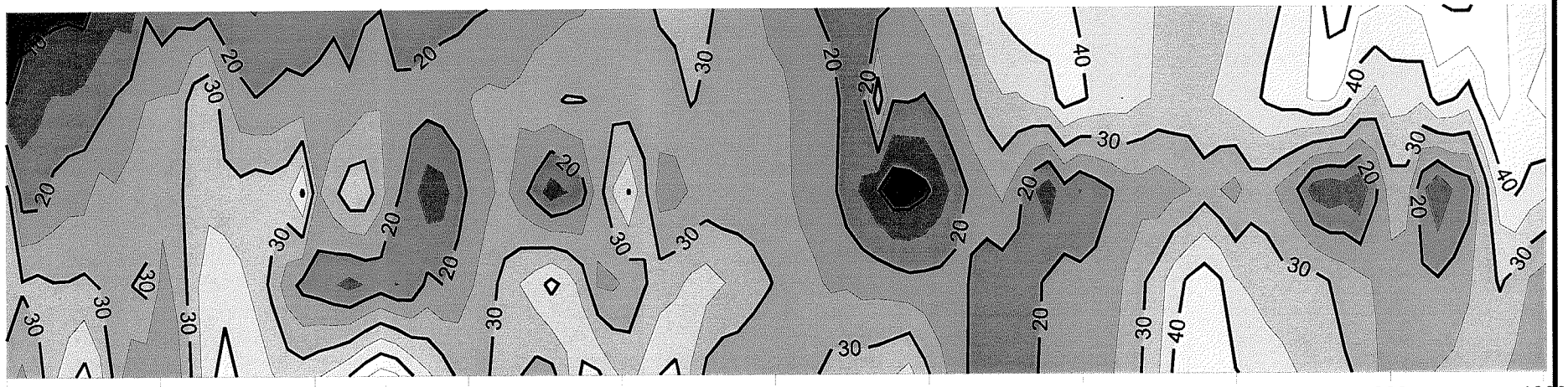
FIGURE 5

FILE NO. 02.1003

DATE April 2002

Bottom Thickness Map



0 100 200

Approximate Horizontal Scale 1" = 100'

0 20 40

Approximate Scale 1" = 20'

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Lutak Dock Rehabilitation
Haines, Alaska

FIGURE 6

FILE NO. 02.1003

DATE April 2002

APPENDIX C

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

Attachment to 32-1-01482
Dated: April 2002
To: Reid Middleton
Re: Lutak Dock Improvements

Important Information About Your Geotechnical/Environmental Report

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

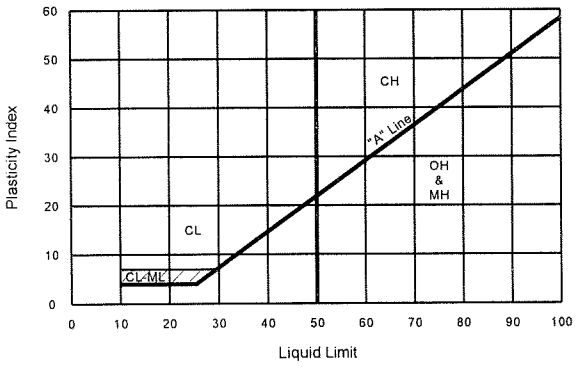
Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

Unified Soil Classification System

GROUP NAME Criteria for Assigning Group Names and Group Symbols			Soil Classification Group Symbol with Generalized Group Descriptions	
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVELS 50% or more of coarse fraction retained on No. 4 sieve	Clean GRAVELS Less than 5% fines	GW	Well-graded Gravels
			GP	Poorly-graded Gravels
		GRAVELS with fines More than 12% fines	GM	Gravel & Silt Mixtures
			GC	Gravel & Clay Mixtures
	SANDS More than 50% of coarse fraction passes No. 4 sieve	Clean SANDS Less than 5% fines	SW	Well-graded Sands
			SP	Poorly-graded Sands
		SANDS with fines More than 12% fines	SM	Sand & Silt Mixtures
			SC	Sand & Clay Mixtures
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid limit 50% or less	INORGANIC	ML	Non-plastic & Low-plasticity Silts
			CL	Low-plasticity Clays
		ORGANIC	OL	Non-plastic and Low-plasticity Organic Clays Non-plastic and Low-plasticity Organic Silts
	SILTS AND CLAYS Liquid limit greater than 50%	INORGANIC	CH	High-plasticity Clays
			MH	High-plasticity Silts
		ORGANIC	OH	High-plasticity Organic Clays High-plasticity Organic Silts
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat

PLASTICITY CHART



Descriptive Terminology Denoting Component Proportions

Description	Range of Proportion
Add the adjective "slightly"	5 - 12%
Add soil adjective[(a)]	12 - 50%
Major proportion in upper case, (e.g., SAND)	>50%

(a) Use gravelly, sandy, or silty as appropriate

NOTE: The soil descriptions used in the boring logs lists constituents from smallest percentage to largest percentage.

Lutak Dock Improvements
Haines, Alaska

UNIFIED SOIL CLASSIFICATION SYSTEM

April 2002 32-1-01482

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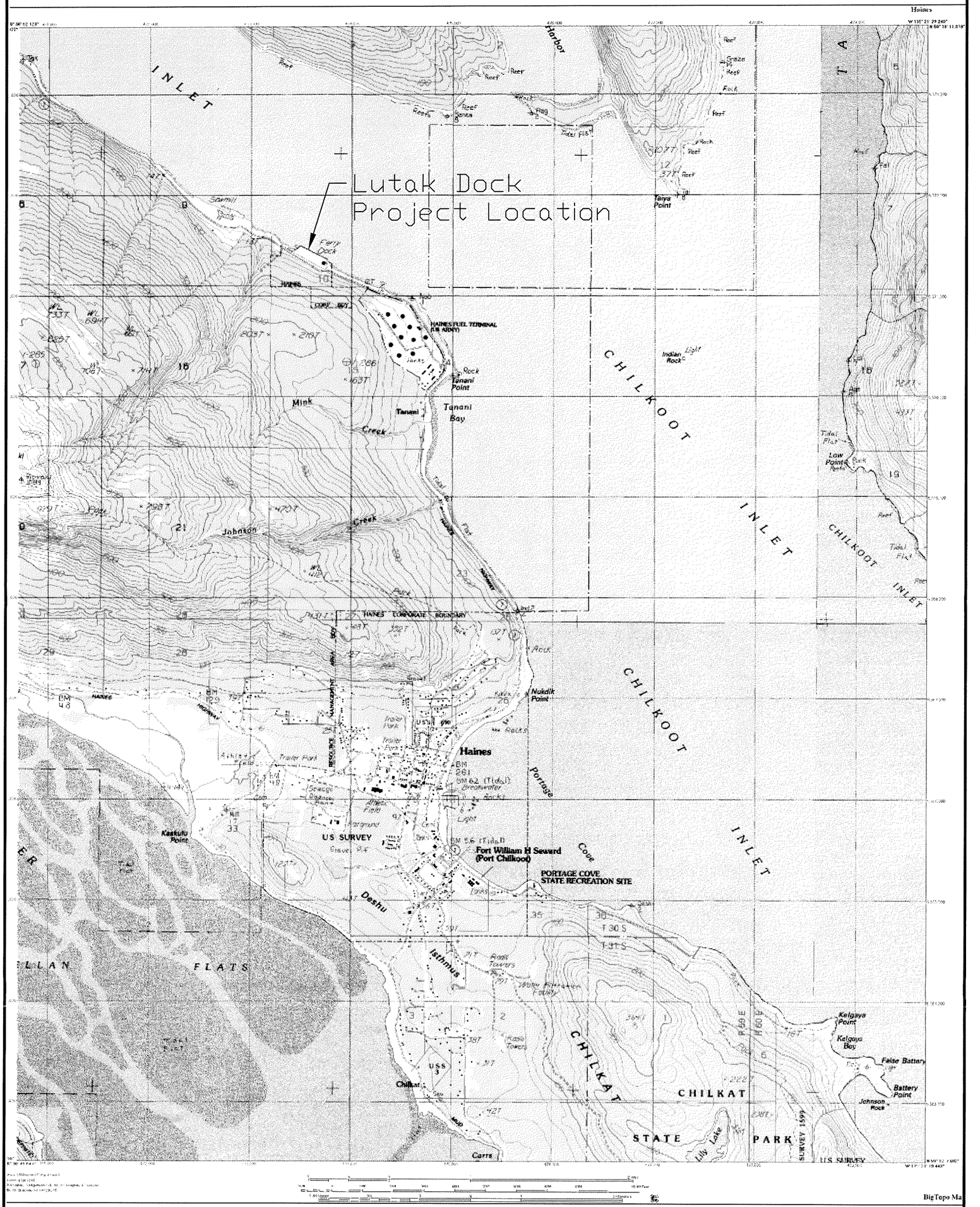
Fig. 3



Lutak Dock Improvements
Haines, Alaska

VICINITY MAP

April 2002

32-1-01482

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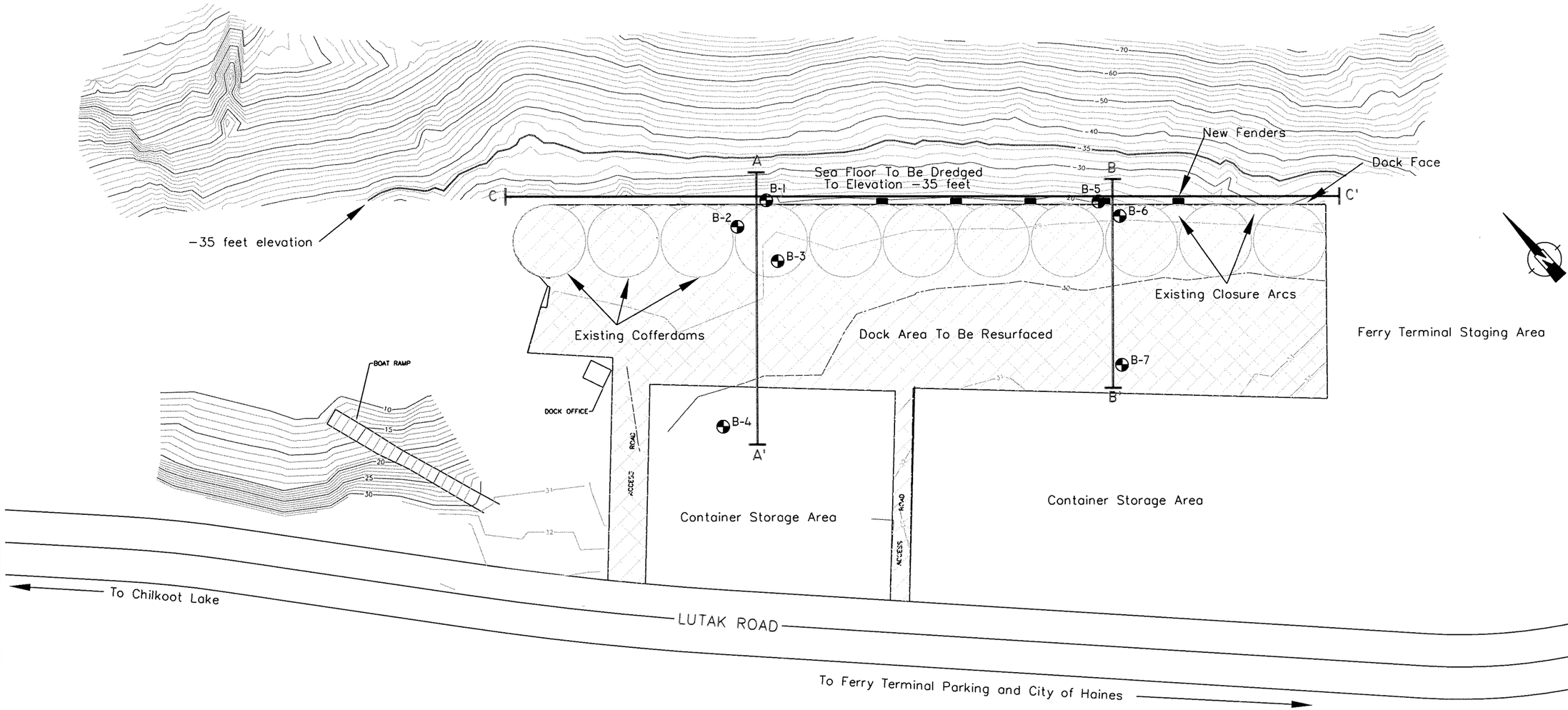
FIG. 1

LUTAK INLET



LEGEND

- B-1 — Approximate location of Boring B-1, advanced by Shannon & Wilson, February 2002
- A' — A — Approximate location Profile A-A', see Figure 6
  Profile B-B' is located on Figure 7
  Profile C-C' is located on Figure 10
- Approximate elevation in feet
  Elevation contour interval equal to 1 vertical foot
- Existing Lutak Dock surface

NOTES

1. Drawing adapted from preliminary plans provided by Reid Middleton.
2. Elevations shown on map provided by USKH site survey. Vertical datum established at Mean Lower Low Water (MLLW) level.
3. Additional proposed improvements are shown on Figures 6 and 7.

0 50 100 200
APPROXIMATE SCALE IN FEET

Lutak Dock Improvements
Haines, Alaska

SITE PLAN

April 2002 32-1-01482

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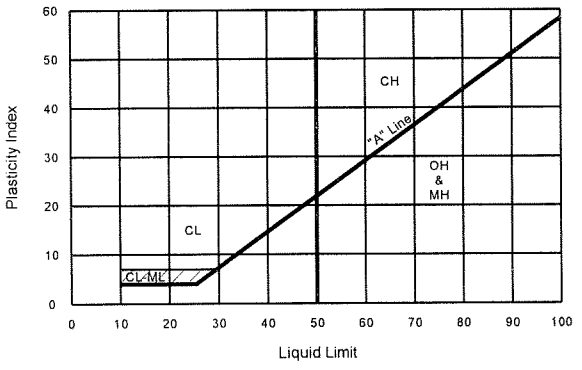
Geotechnical & Environmental Consultants

FIG. 2

Unified Soil Classification System

GROUP NAME Criteria for Assigning Group Names and Group Symbols			Soil Classification Group Symbol with Generalized Group Descriptions	
COARSE-GRAINED SOILS more than 50% retained on No. 200 sieve	GRAVELS 50% or more of coarse fraction retained on No. 4 sieve	Clean GRAVELS Less than 5% fines	GW	Well-graded Gravels
			GP	Poorly-graded Gravels
		GRAVELS with fines More than 12% fines	GM	Gravel & Silt Mixtures
			GC	Gravel & Clay Mixtures
	SANDS More than 50% of coarse fraction passes No. 4 sieve	Clean SANDS Less than 5% fines	SW	Well-graded Sands
			SP	Poorly-graded Sands
		SANDS with fines More than 12% fines	SM	Sand & Silt Mixtures
			SC	Sand & Clay Mixtures
FINE-GRAINED SOILS 50% or more passes the No. 200 sieve	SILTS AND CLAYS Liquid limit 50% or less	INORGANIC	ML	Non-plastic & Low-plasticity Silts
			CL	Low-plasticity Clays
		ORGANIC	OL	Non-plastic and Low-plasticity Organic Clays Non-plastic and Low-plasticity Organic Silts
	SILTS AND CLAYS Liquid limit greater than 50%	INORGANIC	CH	High-plasticity Clays
			MH	High-plasticity Silts
		ORGANIC	OH	High-plasticity Organic Clays High-plasticity Organic Silts
HIGHLY ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat

PLASTICITY CHART



Descriptive Terminology Denoting Component Proportions

Description	Range of Proportion
Add the adjective "slightly"	5 - 12%
Add soil adjective[(a)]	12 - 50%
Major proportion in upper case, (e.g., SAND)	>50%

(a) Use gravelly, sandy, or silty as appropriate

NOTE: The soil descriptions used in the boring logs lists constituents from smallest percentage to largest percentage.

Lutak Dock Improvements
Haines, Alaska

UNIFIED SOIL CLASSIFICATION SYSTEM

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Fig. 3

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FROST CLASSIFICATION

(after Municipality of Anchorage)

GROUP		P-200	USC SYSTEM
NFS	Sandy Soils	0 to 3	SW, SP
	Gravelly Soils	0 to 6	GW, GP, GW-GM, GP-GM
F1	Sandy Soils	3 to 6	SW, SP, SW-SM, SP-SM
	Gravelly Soils	6 to 13	GM, GW-GM, GP-GM
F2	Sandy Soils	6 to 19	SP-SM, SW-SM, SM
	Gravelly Soils	13 to 25	GM
F3	Sands, except very fine silty sands*	Over 19	SM, SC
	Gravelly Soils	Over 25	GM, GC
	Clays, PI>12		CL, CH
F4	All Silts		ML, MH
	Very fine silty sands*	Over 19	SM, SC
	Clays, PI<12		CL, CL-ML
	Varved clays and other fined grained, banded sediments		CL and ML CL, ML, and SM; SL, SH, and ML; CL, CH, ML, and SM

P-200 = Percent passing the number 200 sieve

* Very fine sand : greater than 50% of sand fraction passing the number 100 sieve

Lutak Dock Improvements
Anchorage, Alaska

FROST CLASSIFICATION

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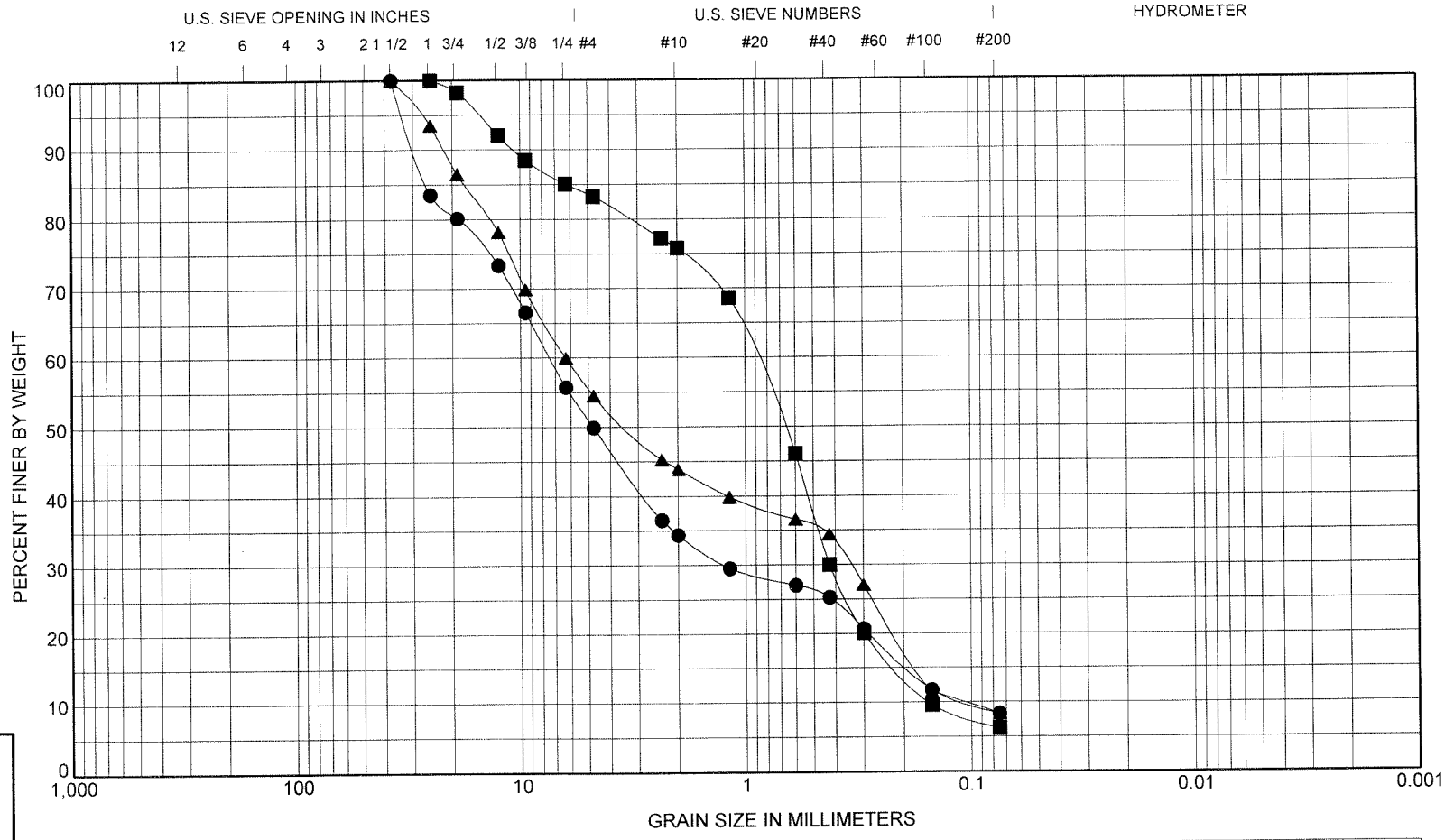
Fig. 4



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample		Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	B-1 S2	19.0 - 20.5	Slightly silty, sandy GRAVEL (GP-GM)				2.0	70.9
■	B-2 S5	25.0 - 26.5	Slightly silty, gravelly SAND (SP-SM)				1.3	5.9
▲	B-2 S12	60.0 - 61.5	Slightly silty, gravelly SAND (SP-SM)				0.2	59.1

Sample		Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-1 S2	19.0 - 20.5	37.5	7.4	1.24	0.1	50	42	8	
■	B-2 S5	25.0 - 26.5	25	0.91	0.42	0.15	17	77	6	
▲	B-2 S12	60.0 - 61.5	37.5	6.31	0.34	0.11	45	46	8	

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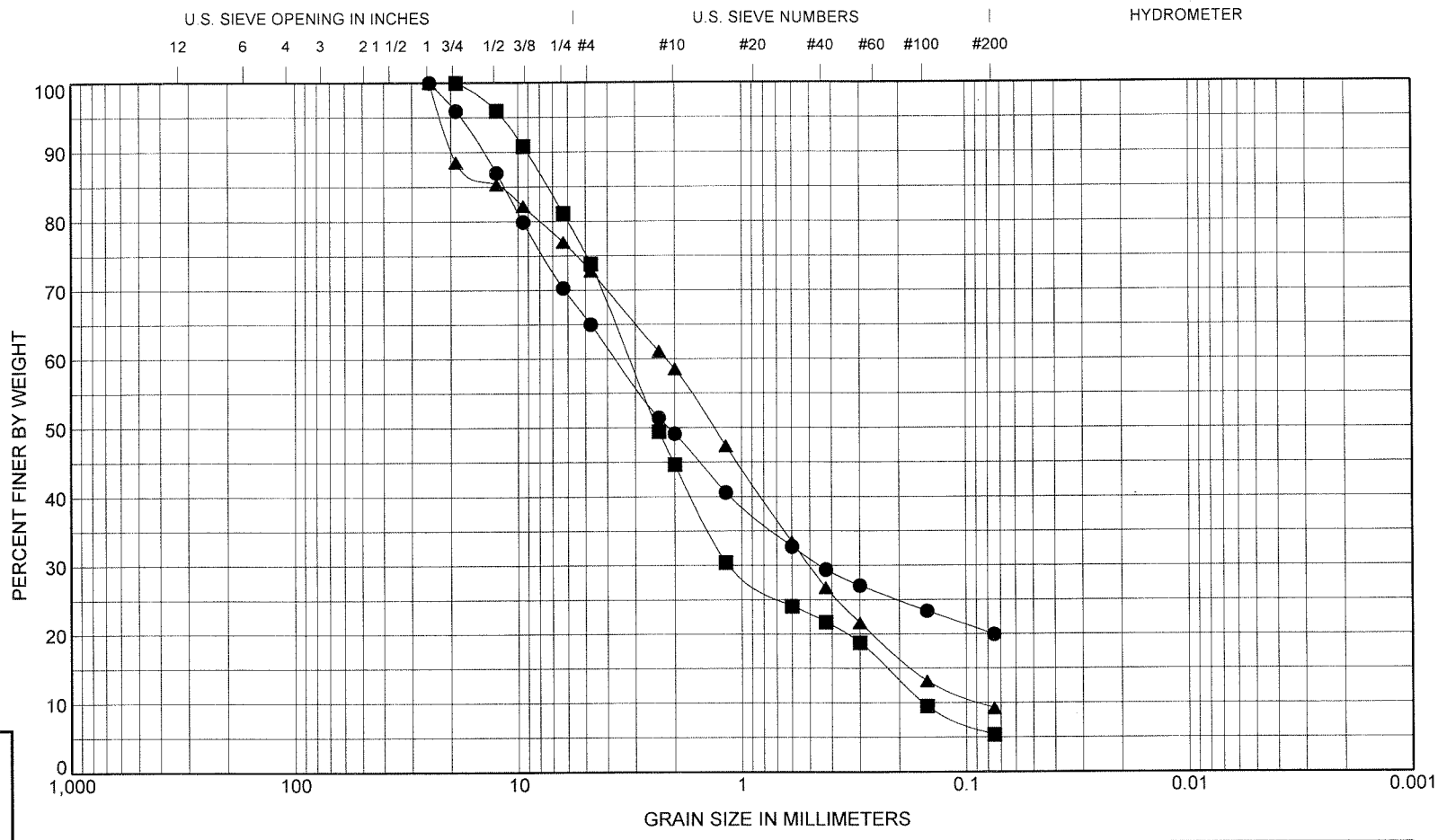
GRAIN SIZE CLASSIFICATION

April 2002

32-1-01482

Lutak Dock Improvements
Haines, Alaska

Fig. 5
Sheet 1 of 3



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample		Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	B-3 S3	15.0 - 16.5	Silty, gravelly SAND (SM)					
■	B-5 S3	13.0 - 14.5	Slightly silty, gravelly SAND (SP-SM)				2.5	20.6
▲	B-6 S3	20.0 - 21.5	Slightly silty, gravelly SAND (SP-SM)				1.3	25.8

Sample		Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-3 S3	15.0 - 16.5	25	3.67	0.45		35	45	20	
■	B-5 S3	13.0 - 14.5	19	3.19	1.12	0.16	26	68	5	
▲	B-6 S3	20.0 - 21.5	25	2.2	0.5	0.09	27	64	9	

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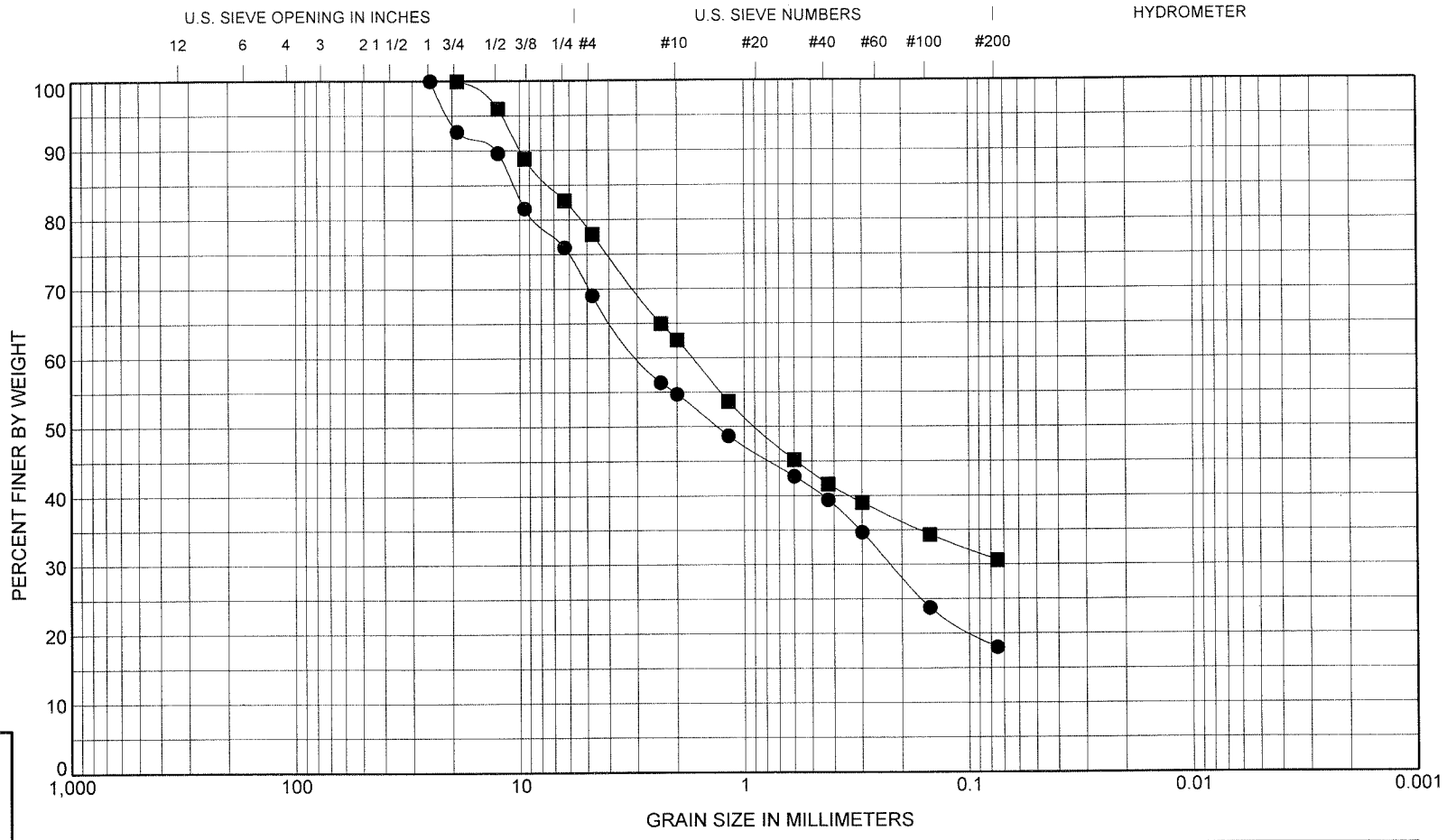
GRAIN SIZE CLASSIFICATION

April 2002

32-1-01482

Lutak Dock Improvements
Haines, Alaska

Fig. 5
Sheet 2 of 3



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

Sample		Depth, Ft	Classification	LL	PL	PI	Cc	Cu
●	B-6 S9	50.0 - 51.5	Silty, gravelly SAND (SM)					
■	B-7 S3	15.0 - 16.5	Silty, gravelly SAND (SM)					

Sample		Depth, Ft	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	B-6 S9	50.0 - 51.5	25	2.88	0.22		31	51	18	
■	B-7 S3	15.0 - 16.5	19	1.72			22	47	31	

Lutak Dock Improvements
Haines, Alaska

GRAIN SIZE CLASSIFICATION

April 2002

32-1-01482

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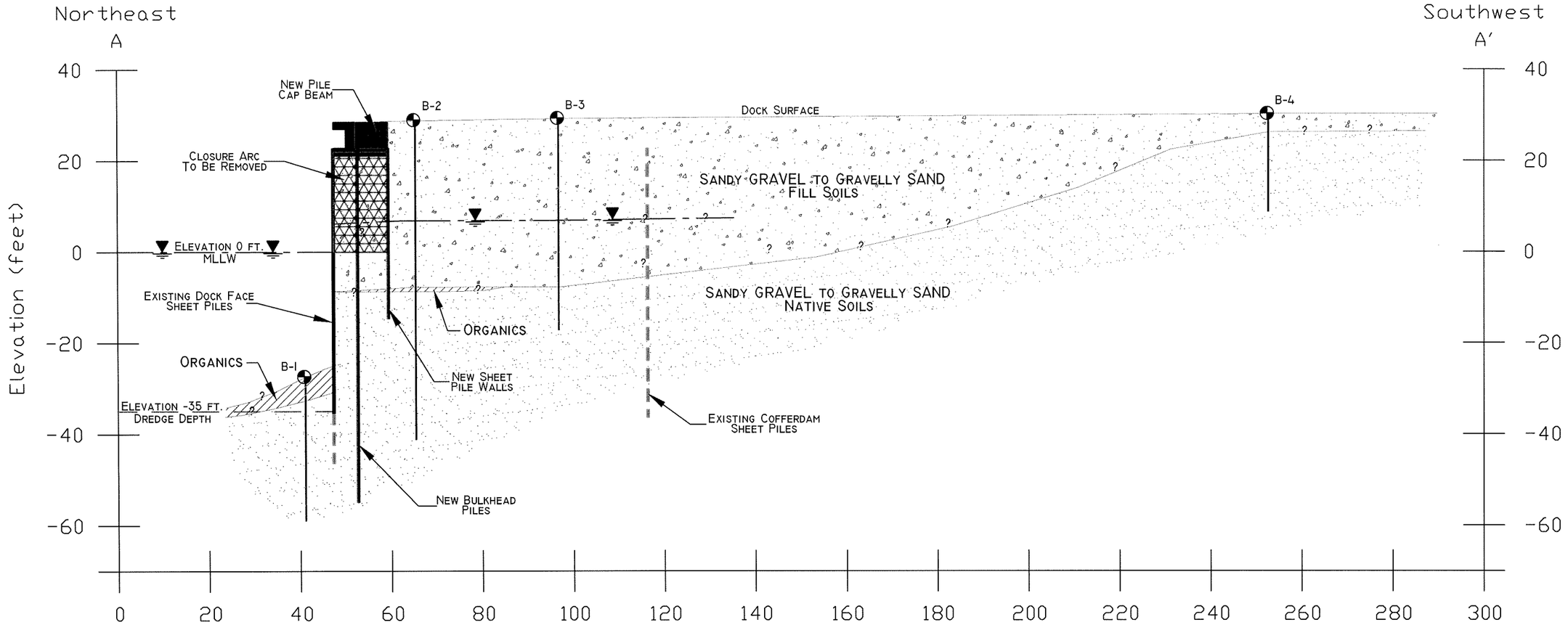
Fig. 5
Sheet 3 of 3



NOTES

1. Soil boundaries are approximate and interpolated from boring data. Actual soil contacts may vary between borings.

2. Existing sheet pile embedments shown in black are interpreted from geophysical analysis performed by Apollo Geophysics. Dashed green lines show sheet pile embedment according to design drawings for the existing dock structure.

3. Red features above indicate proposed improvements to the existing dock structure. Not shown above are the suggested resurfacing structural section and the new fender systems.

4. Elevations for borings and ground surface were interpreted from a site survey conducted by USKH. Elevation contours from this survey are presented on the Site Plan in Figure 2.

5. Elevations shown above are relative to Mean Lower Low Water (MLLW).

LEGEND

B-1 — Approximate location of Boring B-1, advanced by Shannon & Wilson, February 2002

Approximate elevation of groundwater surface inferred from boring data

Approximate embedment of existing cofferdam and closure arc sheet piles according to design plans for the existing dock structure

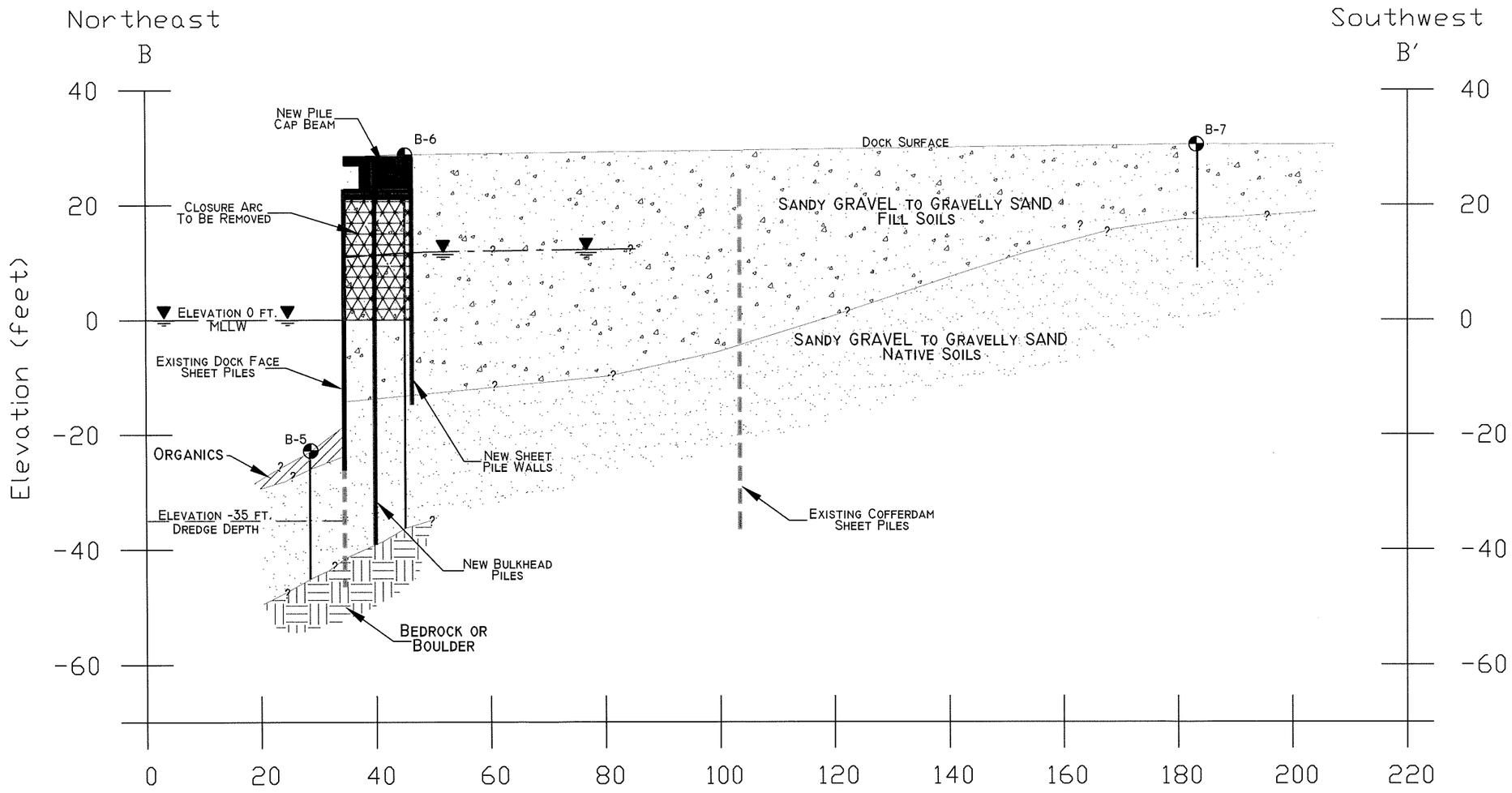
Lutak Dock Improvements
Haines, Alaska

PROFILE A-A'

April 2002 32-1-01482

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

FIG. 6



NOTES

1. Soil boundaries are approximate and interpolated from boring data. Actual soil contacts may vary between borings.

2. Existing sheet pile embedments shown in black are interpreted from geophysical analysis performed by Apollo Geophysics. Dashed green lines show sheet pile embedment according to design drawings for the existing dock structure.

3. Red features above indicate proposed improvements to the existing dock structure. Not shown above are the suggested resurfacing structural section and the new fender systems.

4. Elevations for borings and ground surface were interpreted from a site survey conducted by USKH. Elevation contours from this survey are presented on the Site Plan in Figure 2.

5. Elevations shown above are relative to Mean Lower Low Water (MLLW).

LEGEND

B-5	Approximate location of Boring B-5, advanced by Shannon & Wilson, February 2002
▼	Approximate elevation of groundwater surface inferred from boring data
- - -	Approximate embedment of existing cofferdam and closure arc sheet piles according to design plans for the existing dock structure

Lutak Dock Improvements
Haines, Alaska

PROFILE B-B'

April 2002 32-1-01482

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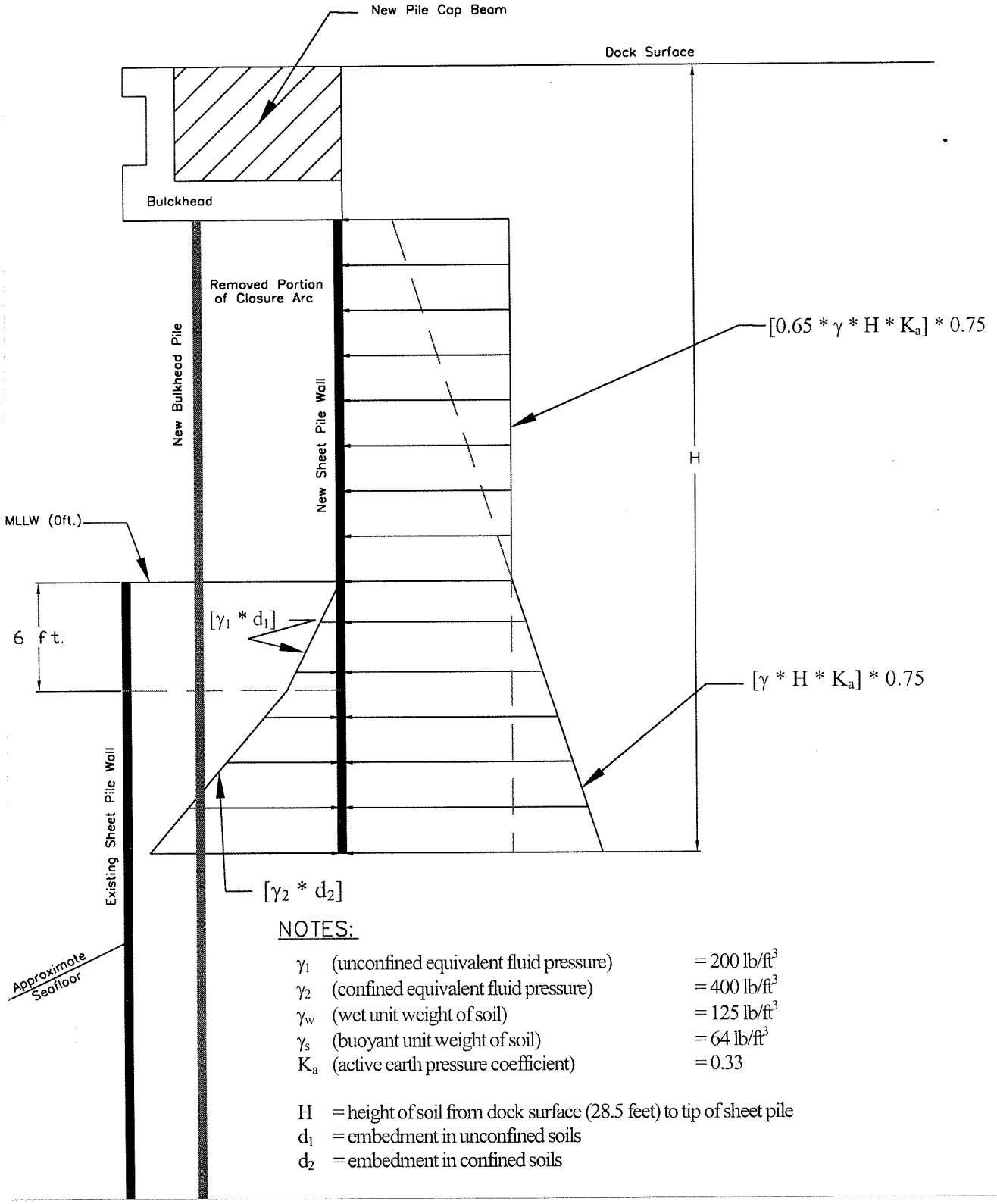
FIG. 7



NOTES:

γ_1	(unconfined equivalent fluid pressure)	$= 200$ lb/ft^3
γ_2	(confined equivalent fluid pressure)	$= 400$ lb/ft^3
γ_w	(wet unit weight of soil)	$= 125$ lb/ft^3
γ_s	(buoyant unit weight of soil)	$= 64$ lb/ft^3
K_a	(active earth pressure coefficient)	$= 0.33$

H = height of soil from dock surface (28.5 feet) to tip of sheet pile
d_1 = embedment in unconfined soils
d_2 = embedment in confined soils

0 4 8 16
APPROXIMATE SCALE IN FEET

Lutak Dock Improvements
Haines, Alaska

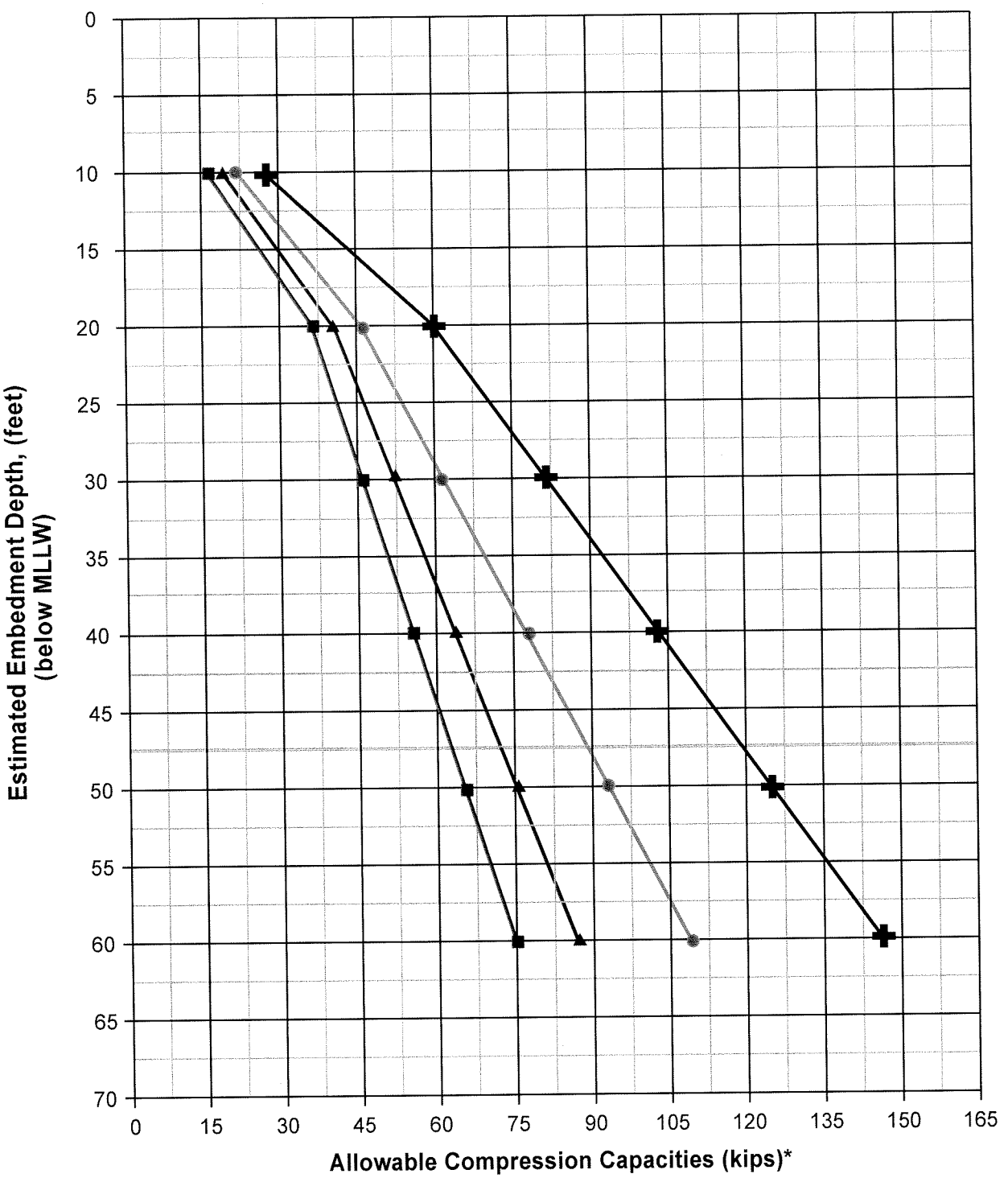
SHEET PILE WALL EARTH PRESSURE DETAIL

April 2002 32-1-01482

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

FIG. 8

EMBEDMENT VS. ALLOWABLE PILE CAPACITITES



LEGEND

- ■ 14-inch H pile
- ▲ 18-inch H pile
- ● 24-inch H pile
- ✚ 22-inch diameter closed ended pipe pile

* Factor of Safety of 2 on ultimate capacities

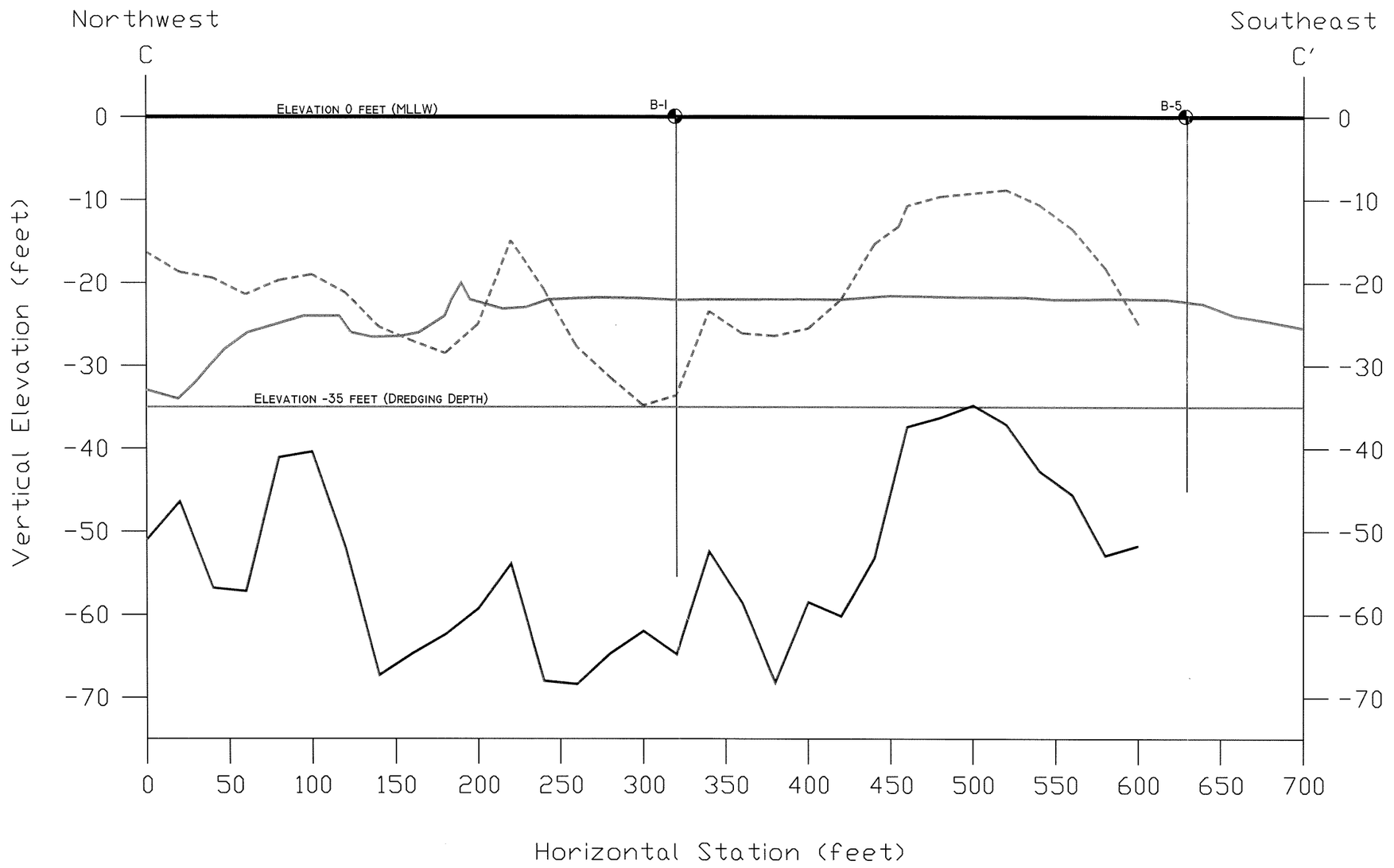
Lutak Dock Improvements
Anchorage, Alaska

ALLOWABLE PILE CAPACITIES

April 2002 32-1-01482

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

Fig. 9



NOTES

1. Elevations shown above are relative to Mean Lower Low Water (MLLW).

2. Horizontal stationing established by Apollo's over water survey.

3. Profile represented above is approximately 10 horizontal feet seaward of the dock face. Actual conditions at the dock face could be significantly different from those depicted above.

4. Subbottom elevation is an interpretation of seismic reflection data and should be verified by boring probes.

5. Vertical scale exaggeration = 5x

LEGEND

B-1 Approximate location of Boring B-1, advanced by Shannon & Wilson, February 2002.

Sea floor profile from USKH bathymetric survey.

Sea floor profile from Apollo survey

Subbottom (possible bedrock) profile from Apollo survey

Lutak Dock Improvements
Haines, Alaska

PROFILE C-C'

April 2002 32-1-01482

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

FIG. 10

REF #2

Geotechnical Report for Haines Ferry Terminal, Haines, AK – 1983. Prepared by Alaska Department of Transportation and Public Facilities.

GEOTECHNICAL REPORT

FOR

HAINES FERRY TERMINAL

HAINES, ALASKA

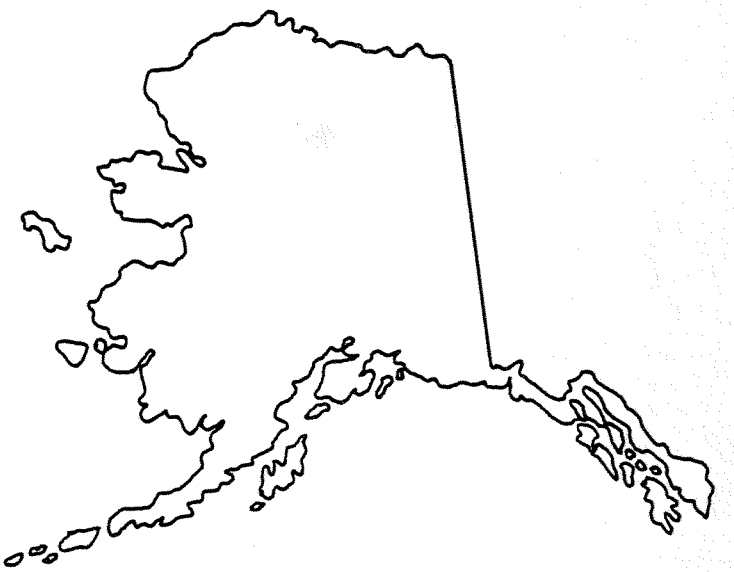
Project No. F-095-5(5)

Code No. A38512

Bridge No. 804

Southeast Region

January 27, 1983



PREPARED BY

ALASKA DEPARTMENT OF TRANSPORTATION & PUBLIC FACILITIES

TECHNICAL SERVICES

MATERIALS SECTION

ANCHORAGE, ALASKA

GEOTECHNICAL REPORT

FOR

HAINES FERRY TERMINAL

HAINES, ALASKA

Project No. F-095-5(5)

Code No. A38512

Bridge No. 804

Southeast Region

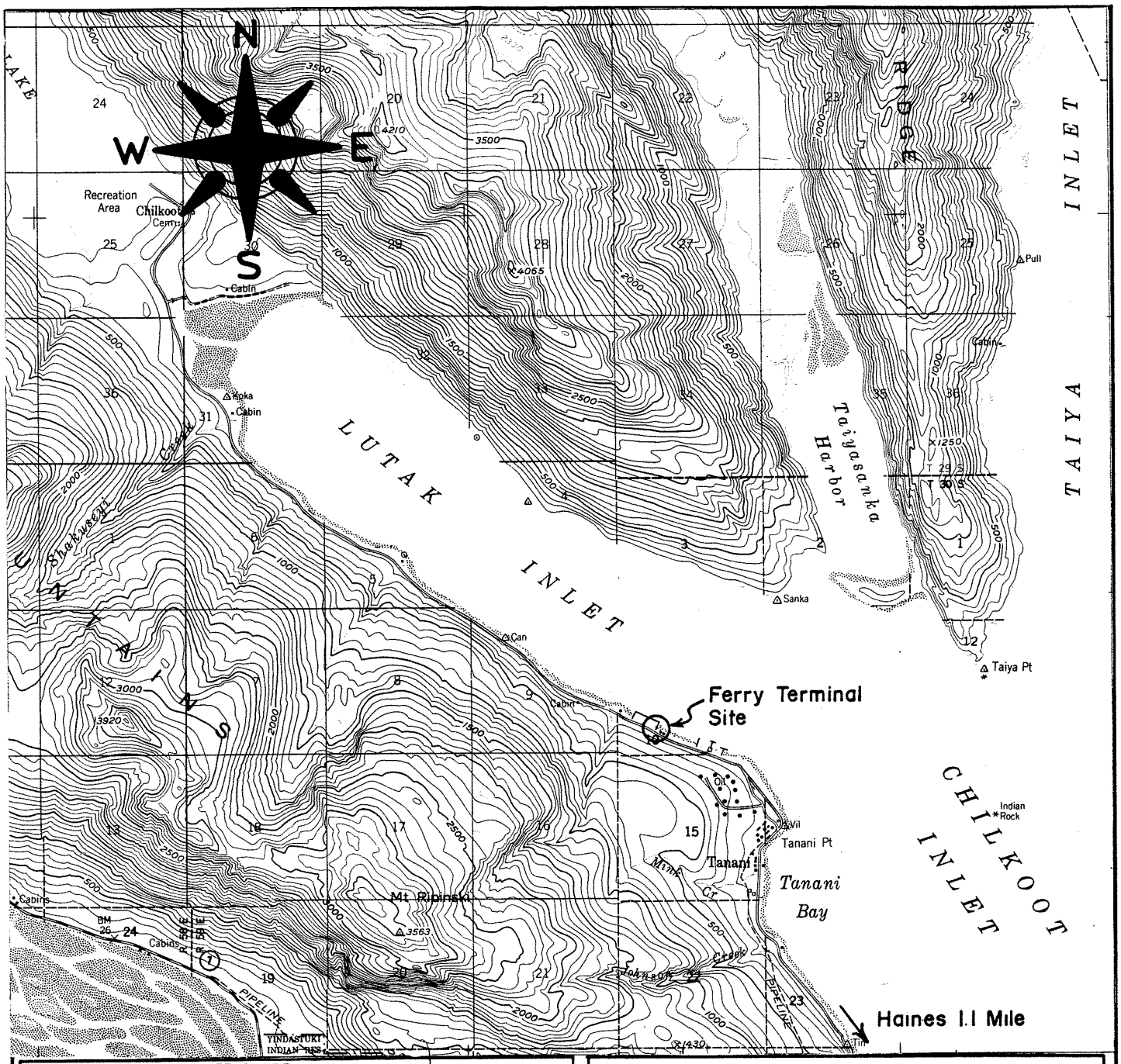
January 27, 1983

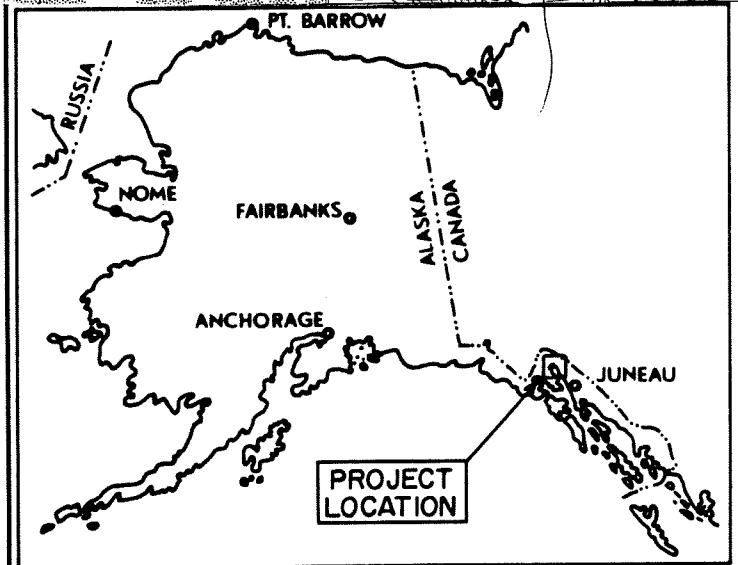
Approved by W. H. Slater

W. H. Slater
Chief of Geotechnical Services

Prepared by

Alaska Department of Transportation and Public Facilities
Standards and Technical Services
Materials Section
Anchorage, Alaska





HAINES FERRY TERMINAL

PROJECT No. F-095-5(5)

Code A38512

1/27/83

LOCATION MAP

Scale 1" = 1 Mile Data U.S.G.S.

INTRODUCTION

The Department of Transportation and Public Facilities proposes the construction of a new ferry terminal to replace the existing structure at Haines, MP 0.0 on the Haines Highway. At the request of the Harbors & Marine Facilities, the DOT Engineering Geology Branch conducted a foundation investigation at the proposed site in January 1983 under the supervision of Wilson Brown, Engineering Geologist. The purpose of the report is to describe the methods of exploration, the sampling and testing procedures and the site conditions at the time of the exploration. Preliminary information available at the time of the field study indicates that the proposed structure will consist of a 140 foot long transfer bridge, two lift structures, and three new dolphins and catwalk. The proposed structure is located at the site of the present ferry terminal.

Climate

The climate is predominantly marine and is characterized by mild winters and cool summers. The maximum temperature recorded was 90°F; the minimum was -16°F. Mean temperature is approximately 40°F. Average precipitation is approximately 60 inches and average snowfall is 133 inches. Winds, chiefly southeast and west, have been reported at 54 mph with gusts estimated to 65 mph (Alaska State Housing Authority, 1964).

Field Studies

The field exploration involved a surface geology examination and a subsurface soil study consisting of one wash boring and 4 friction penetrometer borings. The wash boring was drilled from the existing transfer bridge using BX casing, AW drill rod, and water as the circulating medium. The casing was advanced with a 140 pound trip hammer with a 30 inch free fall.

All of the borings were drilled with a track-mounted CME 45B rotary drill equipped with a CME automatic trip hammer. In boring 1 standard penetration tests were taken with the trip hammer approximately every 5 feet and consisted of driving a standard split barrel sampler (1.4"I.D., 2.0"O.D.) with a 140 pound weight having a free fall of 30 inches. The friction penetrometer test borings consisted of driving a 1.75"O.D., flush coupled, blunt-tipped steel AW drill rod with a 140 pound trip hammer having a free fall of 30 inches. Pullout (uplift resistance) pressure was determined by means of the hydraulic rams of the drill pulling the casing or penetrometer rod immediately after driving.

Soil samples recovered during the standard penetration test were examined, visually field classified and logged in the field by the geologist. All samples were sealed and transported to the Southeast Region Materials Laboratory located in Juneau, Alaska.

Graphic logs of the borings and penetrometer test are shown on the "Log of Test Holes" sheets included in this report.

Laboratory Testing

Soil samples obtained in the exploration were tested in the Southeast Materials Laboratory in Juneau for determination of the following properties.

1. Particle size distribution
2. Atterberg Limits
3. Moisture Content
4. Soil Classification
5. Specific Gravity

All samples tested were assigned the appropriate AASHTO Soil Classification designation, Alaska DOT Engineering Soil Classification, and the Frost Susceptibility Value. All test procedures were performed in accordance with methods approved by the Alaska Department of Transportation and Public Facilities, AASHTO and/or ASTM. The results of the laboratory testing are listed on the "Summary of Test Data Foundation Soils" sheets included in this report.

General Site Conditions

Regional

Recently published data by the USGS disclosed that the Haines area has risen 101 cm in the period from 1939 to 1979-80. The center of the uplift

area is near Glacier Bay and is apparently due to melting of glaciers and rebound of the land mass. It is assumed that it will continue to take place in the future.

Surface

The Haines Ferry Terminal is located in southeast Alaska on the southwest shore of Lutak Inlet approximately 4 miles north of Haines. The site is characterized by a typical "U" shaped glacial valley which is now a drowned fiord in which glacial, colluvial, marine, and modern beach deposits have been deposited over meta-basalts, phyllites, and diorites. Offshore the water depths reach over 300 feet and the mountain above the site rises to 3563 feet.

Subsurface

The subsurface materials commonly consist of slightly silty sandy gravel, sand, gravelly sand, and silt, all containing possible cobbles and boulders, which at Boring 1 may be generally divided into the following principal stratigraphic units:

Layer No.	General Description	Approx.Thickness
1	Very loose - loose sandy gravel, silty gravelly sand, and silty sand containing possible cobbles and boulders (Recent beach deposits).	22 feet
2	Firm to compact silty sandy gravel, sandy gravel and silty gravelly sand containing cobbles and boulders. (Marine deposits or glacial till.)	33 feet
3	Dense to very dense sandy and gravelly silt containing possible cobbles and boulders. (Glacial till.)	16 feet
4	Dark green meta-basalt, highly fractured, hard, slightly weathered to fresh	unknown

Free water should be anticipated near the ground surface or at the tidal elevation, whichever, occurs first.

The penetrometer data should be used with caution when making conclusions regarding the configuration of the bedrock surface. The data does indicate the presence of a very hard stratum, but this stratum may or may not be the bedrock surface.

Respectfully submitted,

Monte Weaver
Foundation Geologist

STATE OF ALASKA

DEPARTMENT OF TRANSPORTATION & PUBLIC FACILITIES

MATERIALS SECTION

Project No. F-095-5(5) Project Name Haines Ferry Terminal sheet 1 of 1 sheets

Boring and Sample No.	Depth in Feet	Laboratory Number 83C	Grading Analysis — % Passing: Gravel 1"	Gravel 1/2"	Gravel #4	Sand #10	Sand #40	Silt #200	Clay .02	Clay .005	Atterberg Limits: Liquid Limit	Atterberg Limits: Plastic Index	Nat. Dry Density P.C.F.	Nat. Moist %	Specific Gravity	F.S.V. (Corps of Eng)	AASHO Group Classif.			
1 043	43-45	20	88	78	65	52	33	18	-	-	NV	NP	-	11	2.87	F3	A-1-b	Silty Gravel		
1 053	53-55	21	100	71	48	31	18	6	-	-	NV	NP	-	9	2.90	NFS	A-1-a	Sandy Gravel		
1 058	58-60	22	100	68	54	47	30	13	-	-	NV	NP		7	2.91	F1	A-1-b	Silty Sandy Gravel		
1 063	63-65	23	COMBINED WITH SAMPLE 1-053																	
1 068	68-70	24	98	90	68	45	23	7	-		NV	NP	-	8	2.92	NFS	A-1-a	Sl. Silty Sandy Gravel		
1 073	73-75	25	100	88	67	47	24	9	-		NV	NP	-	9	2.91	F1	A-1-a	Sl. Silty Sandy Gravel		
1 078	78-80	26	100	97	76	57	17	6			NV	NP	-	10	2.89	NSF	A-1-b	Gravelly Sand		
1 083	83-85	27	100	91	79	69	47	36			17	NP	-	13	2.84	F3	A-4-0	Gravelly Sandy Silt		
1 088B	88-90	28	100	81	78	65	46	36			19	NP	-	14	2.81	F4	A-4-0	Sandy Gravelly Silt		
1 088A	88-90	29	-	100	99	98	90	80			28	9	-	34	2.63	F4	A-4-6	Silt		
1 093	93-95	30	100	100	93	86	69	46			20	NP	-	9	2.72	F4	A-4-0	Sandy Silt		
1 097	97-99	31	96	94	90	83	76	48			19	NP	-	10	2.73	F4	A-4-0	Sandy Silt		

Remarks: Note: Soil particles larger than 1.4" diameter were not recovered in the sampler as the inside diameter of the sampler was less than 1.4 inches.

Rev.

Jan. 13, 1981

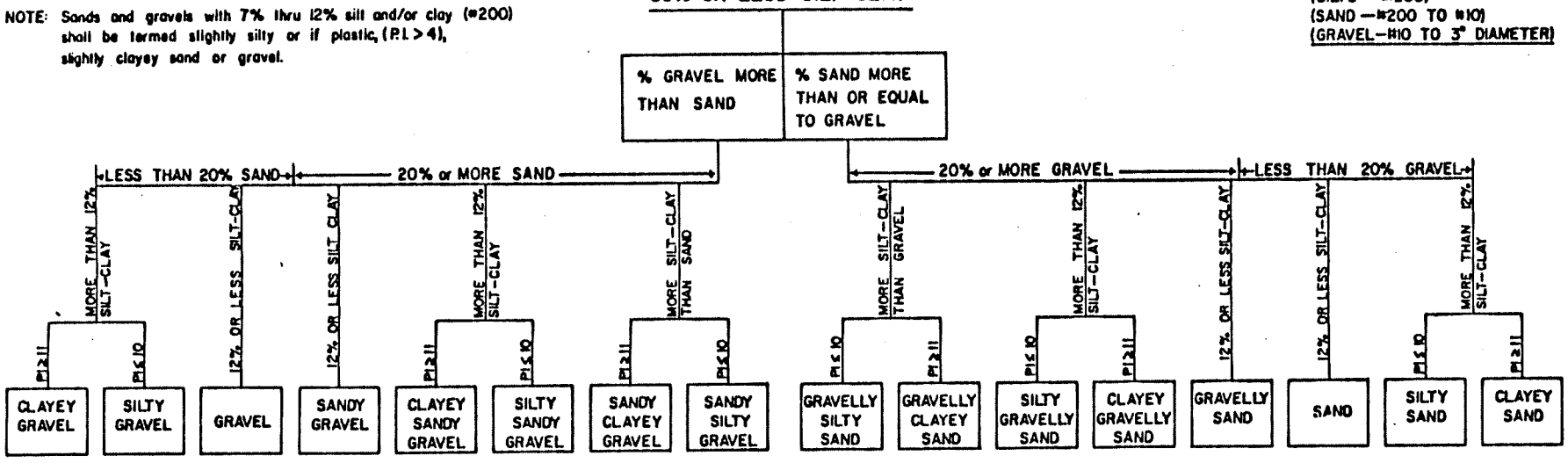
ALASKA DEPARTMENT OF TRANSPORTATION ENGINEERING SOIL CLASSIFICATIONS

NOTE: All silts with a plastic index > 4 shall be termed "clayey".

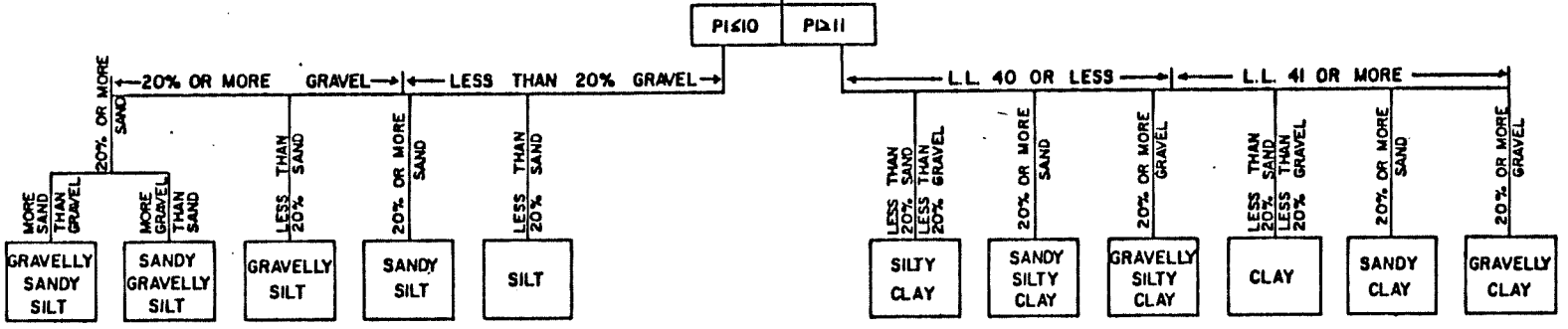
NOTE: Sands and gravels with 7% thru 12% silt and/or clay (#200) shall be termed slightly silty or if plastic, (P.I. > 4), slightly clayey sand or gravel.

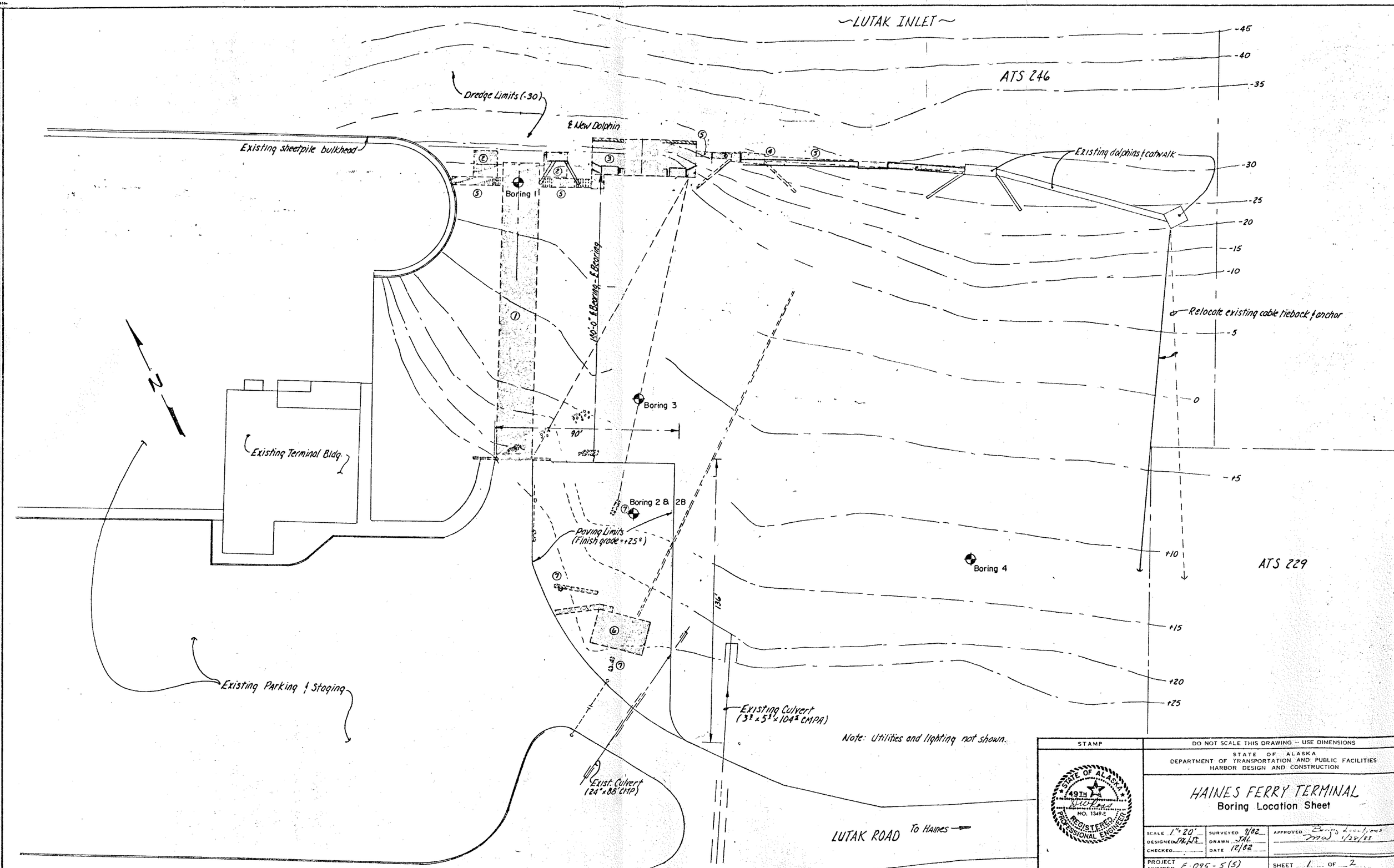
(CLAYS - <#200)
(SILTS — <#200)
(SAND — #200 TO #10)
(GRAVEL — #10 TO 3" DIAMETER)

COARSE-GRAINED SOILS
35% OR LESS SILT-CLAY



FINE-GRAINED SOILS
36% OR MORE SILT-CLAY



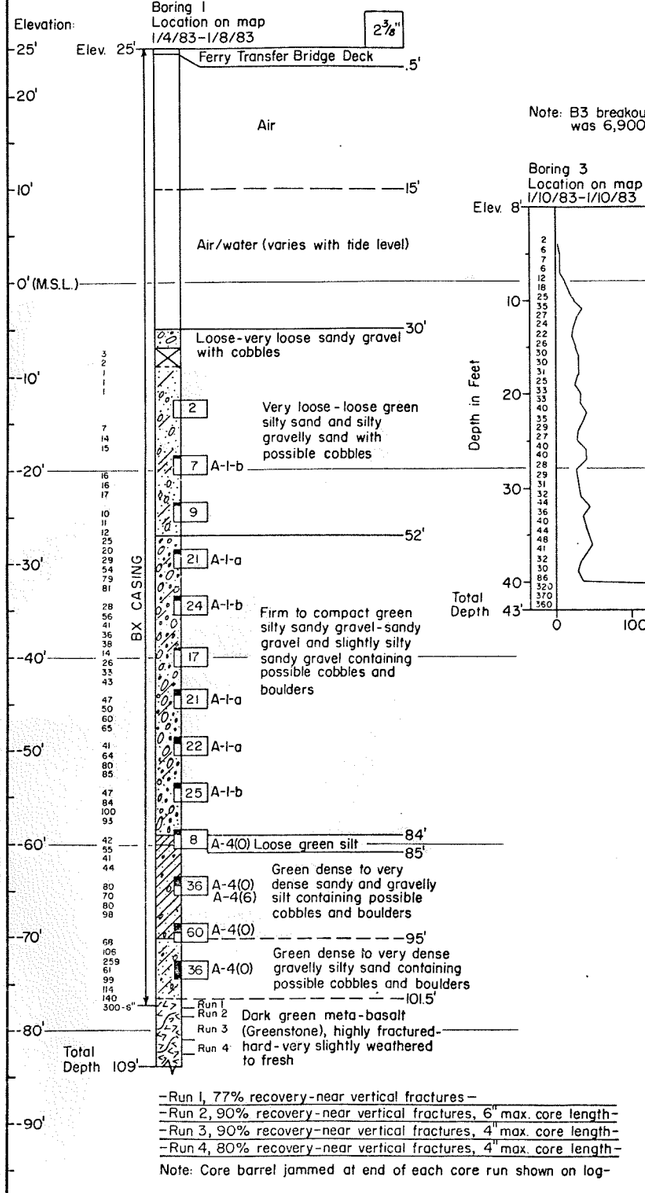


~LUTAK INLET~
ATS 246
ATS 229
Dredge Limits (-30)
℄ New Dolphin
Existing sheetpile bulkhead
Existing dolphins & catwalk
Boring 1
Boring 2 & 2B
Boring 3
Boring 4
140'-0" ℄ Bearing - ℄ Bearing
90'
136'
Existing Terminal Bldg.
Paving Limits (Finish grade = +25°)
Existing Parking & Staging
Existing Culvert (3'3" x 5'3" x 104'5" CMPA)
Exist. Culvert (24" x 88' CMP)
Relocate existing cable tieback & anchor
-45
-40
-35
-30
-25
-20
-15
-10
-5
0
+5
+10
+15
+20
+25
N
LUTAK ROAD
To Haines
Note: Utilities and lighting not shown.
STAMP
STATE OF ALASKA
49TH
NO. 1349-E
REGISTERED PROFESSIONAL ENGINEER
DO NOT SCALE THIS DRAWING – USE DIMENSIONS
STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION AND PUBLIC FACILITIES
HARBOR DESIGN AND CONSTRUCTION
HAINES FERRY TERMINAL
Boring Location Sheet
SCALE 1" = 20'
DESIGNED JAL/JS
CHECKED
SURVEYED 9/82
DRAWN JAL
DATE 12/82
APPROVED Boring Locations 1/28/83
PROJECT NUMBER F-095-5(5)
SHEET 1 OF 2

NORTHERN TRADING ASSOC. 135-3 5-71 45798

STATE	PROJECT DESIGNATION	YEAR	SHEET NO.	TOTAL SHEETS
ALASKA	A38512	1983	2	2

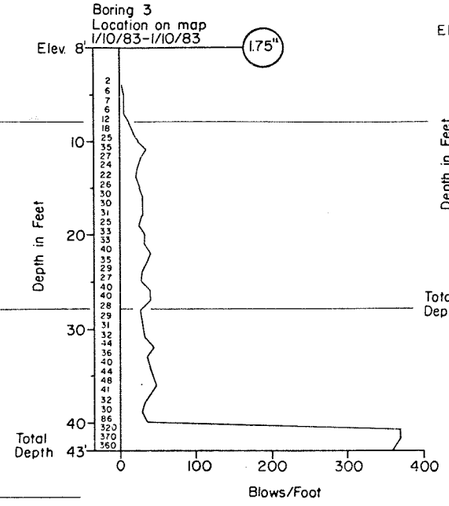
Note: B-1 breakout (pulling) pressure was 17,200 lb. on BX casing (2.875" O.D.)



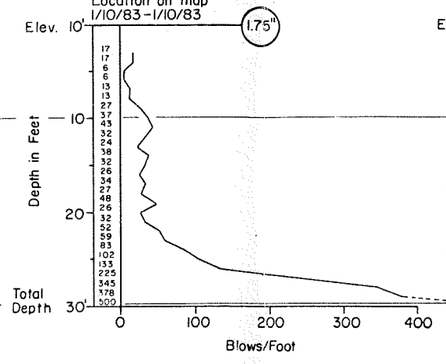
-Run 1, 77% recovery-near vertical fractures-
-Run 2, 90% recovery-near vertical fractures, 6" max. core length-
-Run 3, 90% recovery-near vertical fractures, 4" max. core length-
-Run 4, 80% recovery-near vertical fractures, 4" max. core length-

Note: Core barrel jammed at end of each core run shown on log-

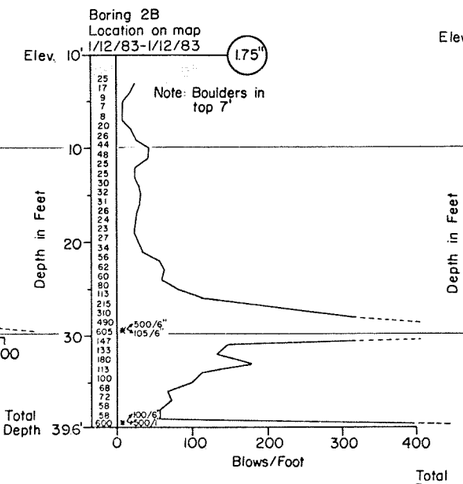
Note: B3 breakout (pulling) pressure was 6,9000 lbs. on AW drill rod.



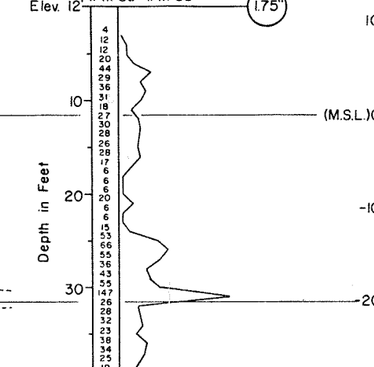
Note: B2 breakout (pulling) pressure was 11,800 lbs. on AW drill rod.

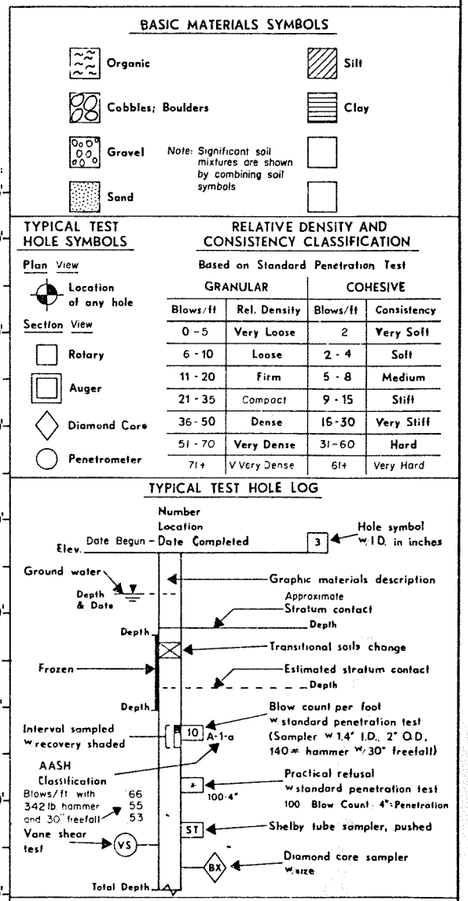


Note: B2B breakout (pulling) pressure was 10,800 lbs. with 4,900 lbs. sustained pull on AW drill rod.



Note: B4 breakout (pulling) pressure was 6,400 lbs. on AW drill rod.





LOG OF TEST HOLES

HAINES FERRY TERMINAL

SOIL GRAIN SIZE DEFINITIONS

Boulder (rounded)	>12" Diameter
Cobble (rounded)	3" Diameter - 12" Diameter
Broken rock (angular)	>3" Diameter
Gravel (rounded); Stone (angular)	#10 Sieve - 3" Diameter
Sand	#200 Sieve - #10
Silt/clay	<#200 Sieve

Note: Soil classifications are visual only unless AASHTO soil class is shown on the log.

State of Alaska
DEPARTMENT OF TRANSPORTATION & PUBLIC FACILITIES
Juneau, Alaska

Date JAN. 1983
Approved

BRIDGE NO. 804
DWHG. NO.

REF #3

Geology Data Report, Haines Terminal Improvements. Dated 2005. Prepared by Alaska Department of Transportation.

ALASKA

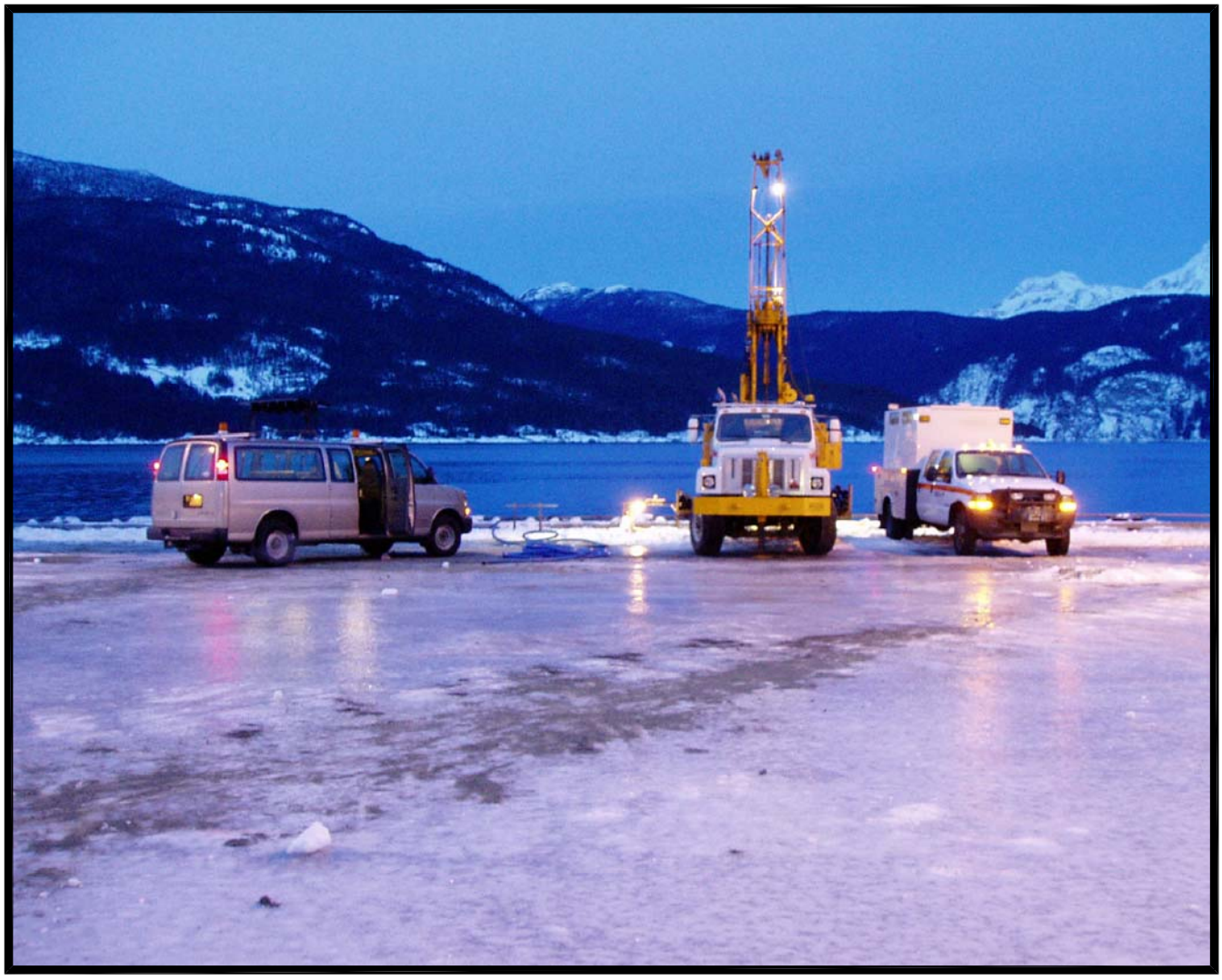
DEPARTMENT OF TRANSPORTATION
AND PUBLIC FACILITIES

GEOLOGY DATA REPORT

HAINES: TERMINAL IMPROVEMENTS

FED PROJECT: NH-095-5(14) AKSAS: 68433

APRIL 2005



GEOLOGY DATA REPORT

HAINES FERRY TERMINAL IMPROVEMENTS

Project No. 68433

Prepared by:

Alaska Department of Transportation and Public Facilities
Southeast Region Geotechnical Section
Juneau, Alaska

April 2005

Prepared: _____
Ralph Swedell, CPG
Regional Geologist
Southeast Region

Reviewed: _____
for Bruce Brunette, P.E.
Regional Material Engineer
Southeast Region

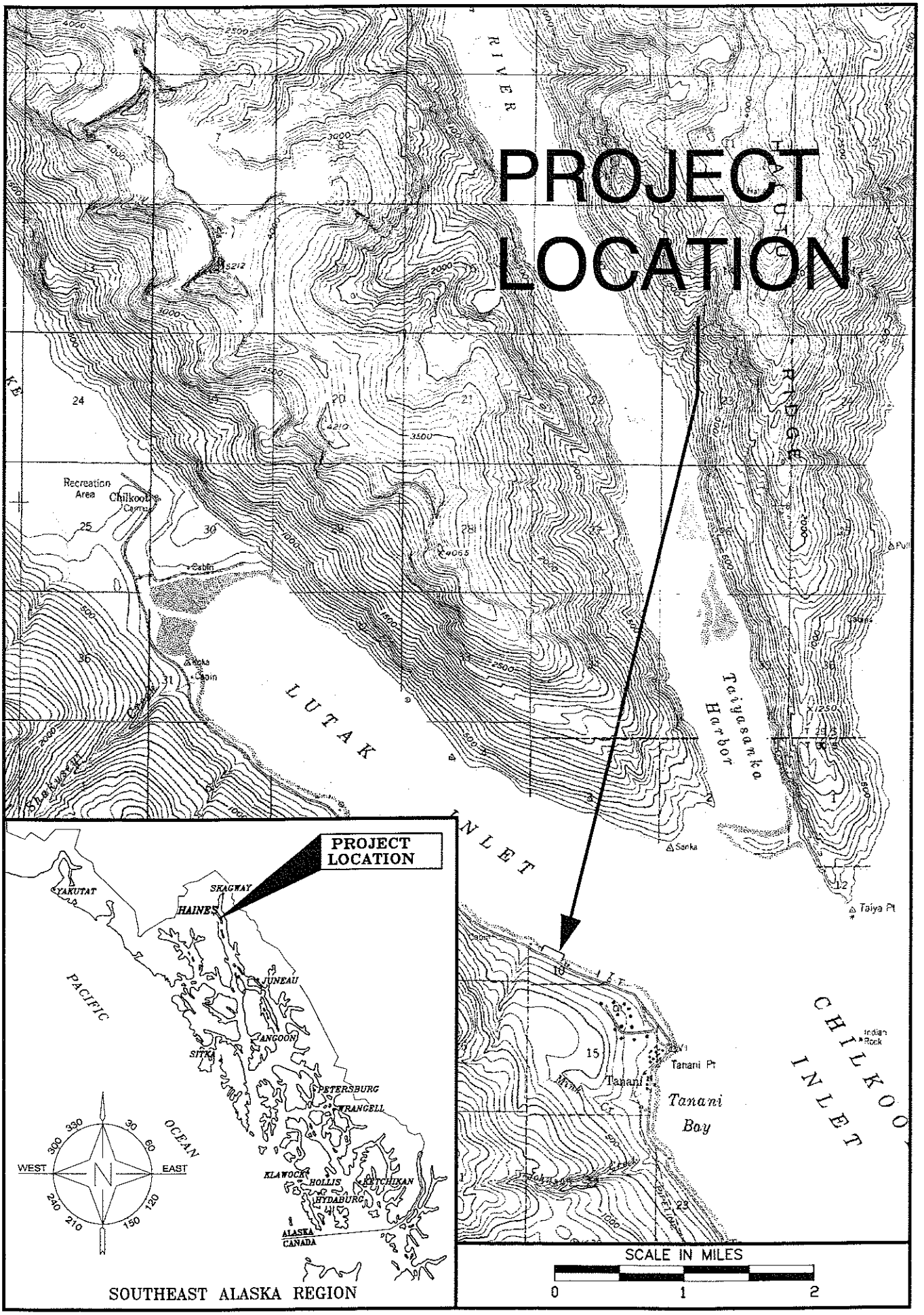
TABLE OF CONTENTS

LOCATION MAP	1
SUMMARY	2
INTRODUCTION	2
PREVIOUS INVESTIGATIONS	2
SITE DESCRIPTION	2
AREA GEOLOGY	4
FIELD INVESTIGATION	5
LABORATORY TESTING	6
SOIL UNITS DESCRIPTION	6
REFERENCES	6

APPENDIX A: Test Hole Location Map and Previous Studies Map

APPENDIX B: AKDOT&PF Test Holes and Penetrometer Logs

APPENDIX C: AKDOT&PF Laboratory Test Data


PROJECT LOCATION
RIVER
LUTAK INLET
Taiyasanka Harbor
Recreation Area
Chilkoot
Cabin
Sanka
Taiya Pt
Tanani Pt
Tanani
Tanani Bay
CHILKOOT INLET
Indian Rock
PROJECT LOCATION
YAKUTAT
SKAGWAY
HAINES
JUNEAU
ANGOON
SITKA
PETERSBURG
WRANGELL
KLAWOCK
HOLLIS
KETCHIKAN
HYDABURG
ALASKA
CANADA
PACIFIC
OCEAN
WEST
EAST
SOUTHEAST ALASKA REGION
SCALE IN MILES
0
1
2

GEOLOGY DATA REPORT

SUMMARY

The general soil conditions at the AMHS Haines Ferry Terminal (Lutak Dock sheet-pile bulkhead cells 1-4) consist of a layers of clean granular fill overlying a system of sheet pile cells. The cells form a 60-foot high retaining bulkhead for the staging area and terminal building. The sheet pile cells were driven into granular deposits and back filled with material ranging in size from silt to boulders. The depth of penetration of the sheet piles was not determined in this investigation. Test borings, penetrometers and piling logs indicate an irregular bedrock surface below the sheet piles.

INTRODUCTION

DOT&PF is planning to upgrade the AMHS Ferry Terminal facilities in Haines. The existing sheet-pile cellular bulkheads have been in service for over 50 years and have suffered corrosion and steel sectional loss. At present, plans have not been finalized, but the range of upgrades will consider cellular bulkhead reconstruction, or bulkhead removal and replacement with armored rock slope and the installation of stand-alone mooring piling. Marine Engineering Manager Bern Savikko, P.E. requested this geotechnical investigation. The purpose of this report is to present site geology information, test drilling data and laboratory testing results. This report does not attempt to describe or delineate the extensive concrete bull rail constructed on the outer edge of the bulkhead.

PREVIOUS INVESTIGATIONS

We have records from three previous geotechnical investigations in the project area. No geotechnical report has been located covering the original Lutak Dock construction, circa 1953. These geotechnical reports are available at the SE Region Geotechnical Section offices:

- *Haines Ferry Terminal, Project Number F-095-5 (5), Bridge No. 804*, AK DOT&PF, January 27, 1983. The investigation consisted of one test hole and three penetrometers near the AMHS transfer-bridge, east of the Lutak Dock. We also have copies of the pile driving logs from the 1983 Haines Ferry Terminal Reconstruction.
- *Geotechnical Report Lutak Dock Improvements*, Shannon & Wilson, Inc., April 2002. They drilled seven test borings and conduced a marine geophysical exploration for that portion of the Lutak Dock owned by the City of Haines (west of the AMHS facilities).
- *Geotechnical Report to USKH for Haines Highway Reconstruction, Alaska Marine Highway Ferry Terminal to the Junction with Mud Bay Road*, Golder Associates, October 2003. This investigation was for the Haines Highway reconstruction; one test boring and one test pit were located near the ferry terminal.

SITE DESCRIPTION

Topography

The Haines AMHS Ferry Terminal is located on the southern shoreline of Lutak Inlet. The ridge behind (south of) the terminal facility raises from sea level to 3563' (Mt. Ripinski) in a little over two miles. The terminal building and the western portion of the mooring facility are on a level embankment retained by a sheet-pile cellular bulkhead. The transfer-bridge and mooring dolphins east of the bulkhead are constructed over a gently sloping beach.

Drainage

During periods of torrential rainfall, the runoff down the slopes of the Mt. Ripinski ridge can result in very high stream flows. The stream west of the terminal sometimes washes sand, gravel and large boulders onto the road. The engineered drainage from the Lutak Dock adequately accommodates heavy runoff.

Man-made features

Lutak Road follows the Lutak Inlet shoreline, passing between the AMHS Ferry Terminal/ Lutak Dock facilities and the Mt. Ripinski ridge. The terminal building and staging area are located on the eastern portion of the Lutak Dock facility.

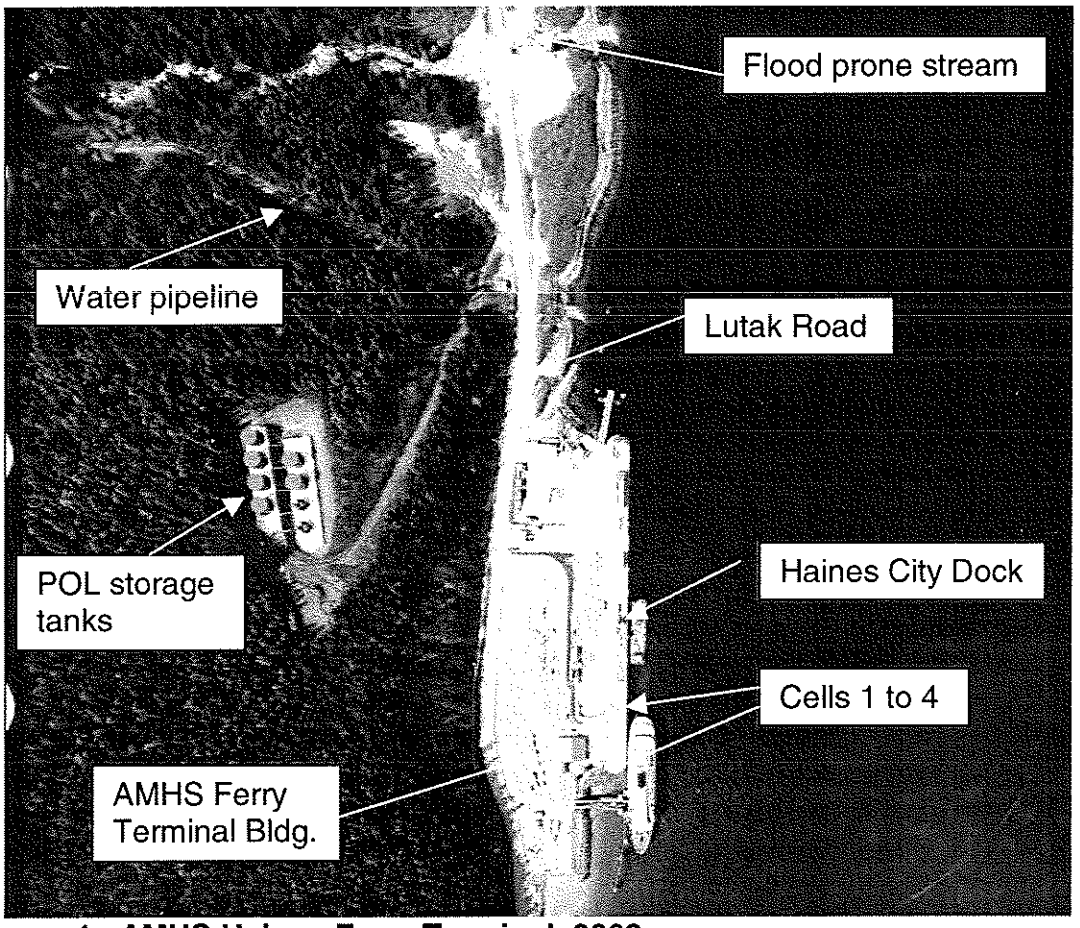


Figure 1. AMHS Haines Ferry Terminal, 2003.

AREA GEOLOGY

Glaciation

Lutak Inlet is a glacial scoured fiord. Ice to depths of 5000' occupied this area as recently as 10,000 years ago (Coulter, 1961). The glaciers gouged out U-shaped valleys and deep fiords; and in many locations scraped the ridges down to bare rock. After the melting and retreat of the glaciers, seawater flooded the valleys, rivers began depositing thick sequences of sediments on the valley floors and marine sediments settled into the fiords. The flank of the bedrock ridge bordering the southwest shore of Lutak Inlet is covered with a thick sequence of colluvium.

Flooding. The stream drainages that flow down the slopes of the south ridge can carry heavy bedloads that include a large number of cobbles and boulders. During periods of torrential rainfall material from these streams have covered Lutak Road (Figures 1 & 2). It is likely that flooding has deposited substantial amounts of cobbles and boulders into Lutak Inlet.

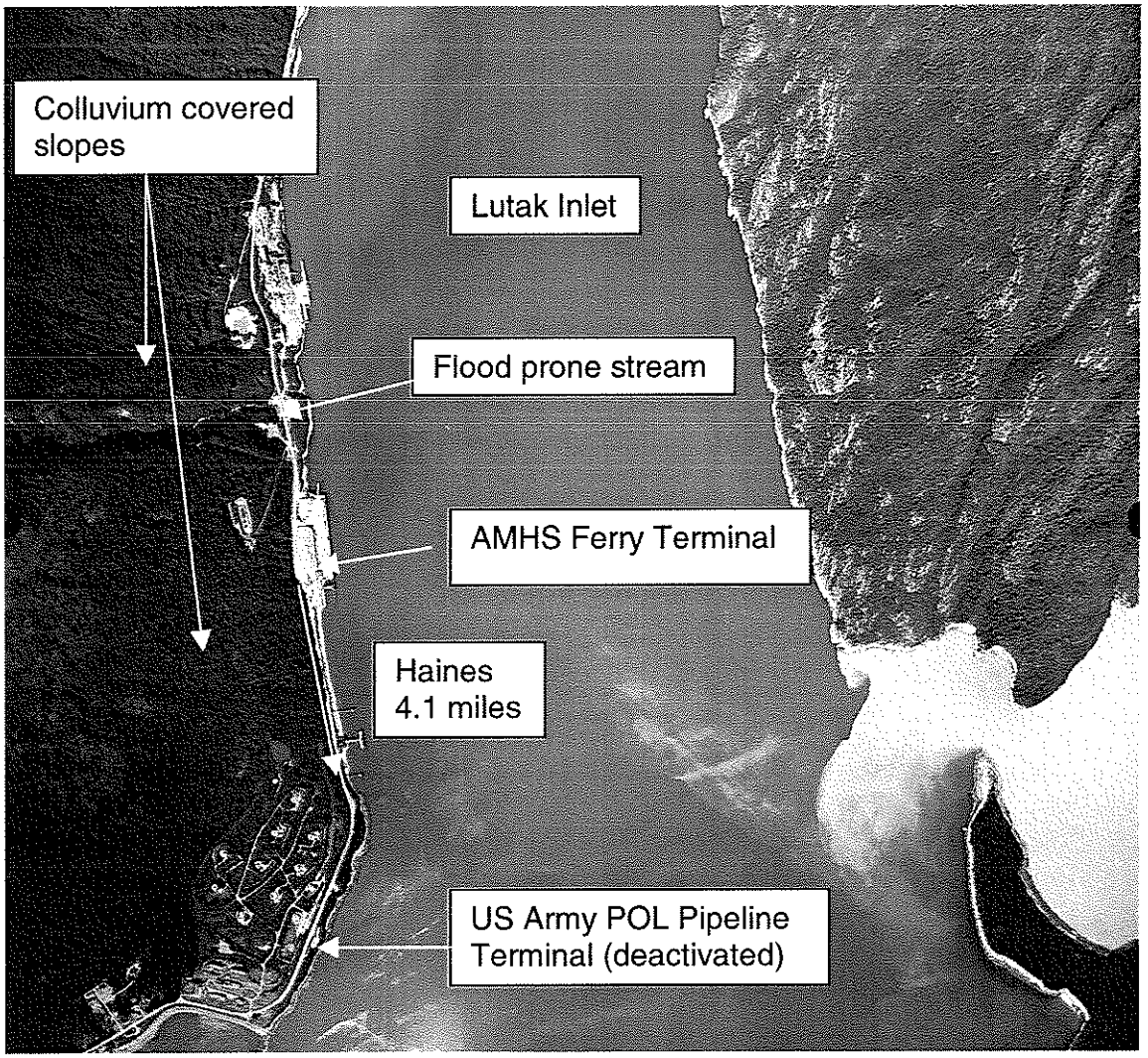


Figure 2. AMHS Haines Ferry Terminal, Lutak Inlet, August 2003.

Seismicity
Haines is in a seismic area with infrequent earthquakes. Within a radius of 35 miles of Haines, the Alaska Earthquake Information Center lists only 11events of magnitude 4 or greater since 1898. The strongest event was 6.9 magnitude, with its epicenter 24 miles southwest of Haines.

Two major known fault systems are capable of producing earthquakes that would effect the Haines area. The Chatham-Denali Fault crossing just west of Haines, in the Chilkat River & Inlet, and the Queen Charlotte fault located 95 miles to the southwest of Haines, offshore of the SE Alaska coastline.

An earthquake on the Chatham-Denali Fault could have a strong impact on the project area. A strong earthquake could cause considerable damage from avalanches, rockslides, liquefaction induced foundation failures and strong shaking of structures.

The effects of an earthquake along the Queen Charlotte Fault would be attenuated by the fault's distance from the Haines area. A number of magnitude 7 and greater earthquakes have occurred along this fault zone within the last two hundred years. Strong earthquakes along the Alaska coast have produced large waves that caused major damage to coastal towns. To our knowledge, Haines has not experienced any damage due to seismically induced waves.

Following is a list of some commonly used seismic design parameters.

- PGA 0.2 for an event with 475 year return period, USGS National Seismic Hazard Mapping Project, ftp://ghtftp.cr.usgs.gov/pub/hazmaps/deagg
- AASHTO Soil Profile Type I, Standard Specifications for Highway Bridges, 2002, pg. 449
- Site Class D, International Building Code, 2000, Table 1615.1.1, pg. 350

FIELD INVESTIGATION
January 8 to 16, 2005 we conducted geotechnical drilling operations at the Haines AMHS Ferry Terminal. The purpose of the investigation was to determine the soil conditions within and underlying the existing sheet pile cellular bulkheads. Geologist Mitch McDonald planned and supervised the investigation. Statewide Drilling Services provided the drill crew and equipment.

We drilled the test holes using rotary wash boring technique and 3.5" ID drill casing. We took drive samples with either a standard 1.4" OD x 24" long sampler driven with a 140 lb. auto hammer, in accordance with AASHTO T 206-87 (SPT), or with a 2" ID x 24" long sampler driven with a 340 lb. auto hammer (MPT). A double tube BX core barrel was used to confirm the presence of bedrock.

We drove continuous soundings with a 2.5" OD blunt-tippe rod using a 340 pound hammer falling 30 inches. Test hole and penetrometer logs are included in Appendix B.

LABORATORY TESTING

Samples were tested in the Southeast Region Materials Laboratory to determine one or more of the following properties: sieve analysis (AASHTO T 27 & T 11), Atterberg limits (AASHTO T 89 & T 90), moisture content (AASHTO T 265), Organic Content (AASHTO T 267), Specific Gravity (AASHTO T 133) and Unified Soil Classification (ASTM D2487). The laboratory test results are included in Appendix C.

SOIL UNITS DESCRIPTION

We used the test hole logs, penetrometer logs and sample test results to interpret the soil conditions encountered during this investigation. For the purposes of description and analysis, we divided the material into five units: surface fill, cell back-fill, beach deposits, diamicton and bedrock. The surface of the staging area is paved.

- Surface fill, sand with gravel (SW and SP), 9.5' to 10' thick at TH 1 and 2.
- Cell back-fill (TH1 and TH 2), silty sand with gravel (SM) and sand with silt and gravel (SP-SM), 33.0' to 42.5' thick.
- Beach deposits, sand (SP) and sand with gravel (SP), 21.5' to 31.5' thick.
- Diamicton (probably glacial-marine sediments), silt (ML), silt with sand (ML) and sandy silt (ML), 19' to 29.5' thick.
- Bedrock was encountered below the existing surface at 103.5' in TH 1 and at 93' in TH 2.

The percentage and size of cobbles and boulders cannot be accurately determined by the drilling process. Gravel is the largest particle that can be recovered by the drive samplers. However, interpretation of the drilling response of the soil indicated the presence of cobbles and boulders. In addition, very large boulders were encountered during the maintenance excavation of cell 4.

The water level in the test holes shortly after drilling varied from 13.5' to 17.9' below the surface. The water level near the sheet-pile bulkheads is effected by the stage of the tide, but will also vary depending on recent rainfall events and the season of the year.

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APPENDIX A
TEST HOLE LOCATION MAP

NOTE: AERIAL PHOTO IS DISTORTED AND SHOWS A BEND IN THE DOCK FACE THAT DOES NOT EXIST

EDGE OF CONCRETE

DOCK FACE

P-2 (AKDoT 2005)

P-1 (AKDoT 2005)

109 28.49' PLASCAP

110 29.09' ALCAP

108 28.67' MAG_MP4

107 28.40' MAG_RP4

106 28.17' MAG_RP3

101 28.59' MAG_MP3

104 28.58' MAG_MP2

105 28.89' MAG_RP2

103 28.41' MAG_MP1

102 28.11' MAG_RP1

FENCE LINE

CELL 4

CELL 3

CELL 2

CELL 1

TH-2 (AKDoT 2005)

TH-1 (AKDoT 2005)

TH-3 (AKDoT 2005)

FENCE LINE

FENCE LINE

112 31.27 ALCAP

EXISTING ACCESS GANGWAY

EXISTING MOORING STRUCTURES

EXISTING TIMBER MOORING STRUCTURES

EXISTING ACCESS GANGWAY & PLATFORM

REF POINT FOR LOCATING THE 1982 BORINGS

REF POINT FOR LOCATING THE 1982 BORINGS

EXISTING TRANSFER BRIDGE

TOE OF RIP RAP

UTILITY CLUSTER

TERMINAL BUILDING

EXISTING ELECTRICAL VAULTS

EDGE OF PAVEMENT

CONCRETE ABUTMENT

728 27.45' CTRL_PK

730 27.50' CTRL_PK

EDGE OF PAVEMENT

REF POINT FOR LOCATING THE 1982 BORINGS

STORAGE BUILDING

GENERATOR BUILDING

POWER POLE

SIGN

SIGN

EDGE O

19 33.99 HNS-5

SIGN

GUARDRAIL

POWER POLE

SIGN

POWER POLE

SIGN

DITCH

0 100 200

LUMIN

SIGN

POWER POLE

GUARDRAIL

SIGN

SIGN

SIGN

TOP OF ROCK

TEST HOLE LOCATION MAP

HNS: FERRY TERMINAL IMPROVEMENTS
PROJECT NO. 68433

SCALE 1:50

PRINTED ON: Thu, 10/Mar/05 12:13PM

PRINTED BY: mmcdonald

PATH: Q:\Vine\68433\GE\AutoCad Project\FERRY01\dwg\68433_Test Hole Location Map.dwg
TAB: Test Hole Location Map_Figure

STATE OF ALASKA
DEPARTMENT OF TRANSPORTATION & PUBLIC FACILITIES
STATEWIDE DESIGN & ENGINEERING SERVICES DIVISION

HNS: FERRY TERMINAL IMPROVEMENTS
PROJECT NO.68433

PROJECT DESIGNATION NUMBER	
NH-095-5 (14)	
STATE	YEAR
ALASKA	2005
APPENDIX	FIGURE
A	1

HNS: FERRY TERMINAL IMPROVEMENTS
PROJECT NO. 68433

PREVIOUS STUDIES

SCALE 1:100

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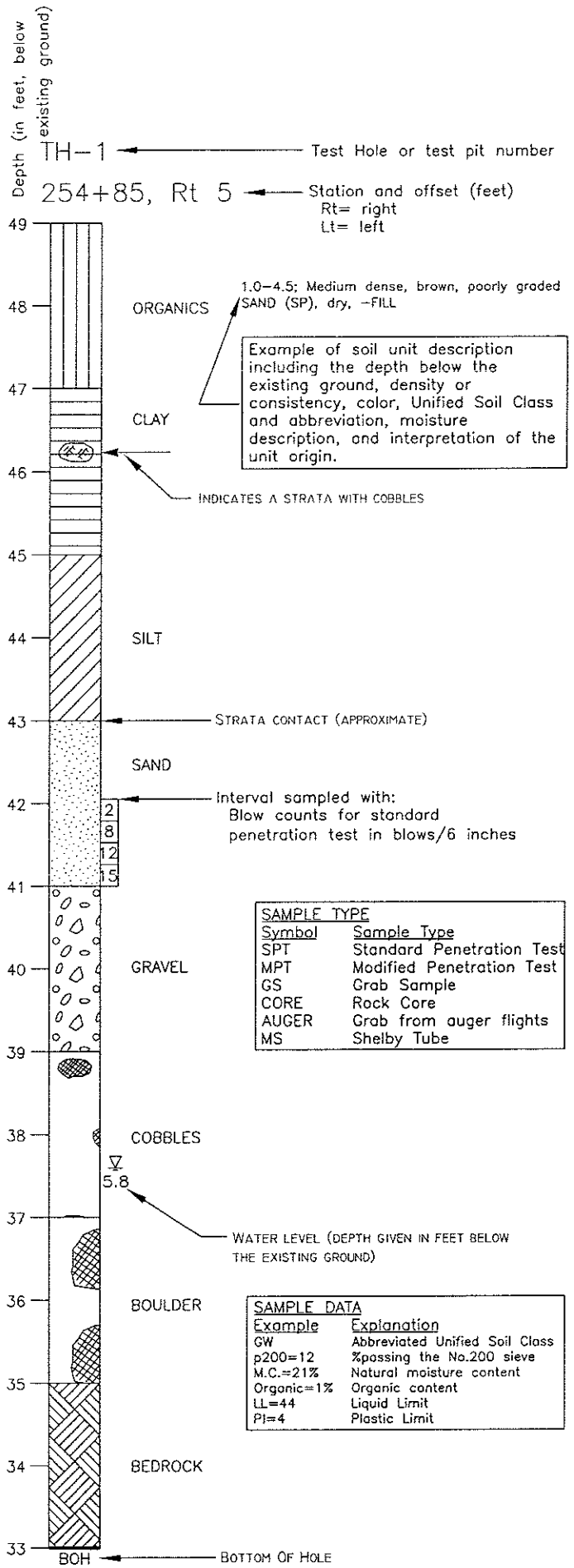
PRINTED ON: Thu, 10/Mar/05 12:22PM
PRINTED BY: mmcdonald

APPENDIX B
AKDOT & PF TEST HOLE AND PENETROMETER LOGS

TEST HOLE LOG EXPLANATION



SAMPLE TYPE	
Symbol	Sample Type
SPT	Standard Penetration Test
MPT	Modified Penetration Test
GS	Grab Sample
CORE	Rock Core
AUGER	Grab from auger flights
MS	Shelby Tube

SAMPLE DATA	
Example	Explanation
GW	Abbreviated Unified Soil Class
p200=12	%passing the No.200 sieve
M.C.=21%	Natural moisture content
Organic=1%	Organic content
LL=44	Liquid Limit
PI=4	Plastic Limit

EXPLANATION OF DESCRIPTIVE TERMS

GENERALIZED UNIFIED SOIL CLASSIFICATION SYSTEM

Coarse Grained Soils >50% retained on #200 seive	Gravels >50% of coarse fraction retained on #4 seive	Clean Gravels <5% fines	GW	Well-graded gravel
			GP	Poorly graded gravel
		Gravels >12% fines	GM	Silty gravel
			GC	Clayey gravel
	Sands 50% or more of coarse fraction passes #4 seive	Clean Sands <5% fines	SW	Well-graded sand
			SP	Poorly graded sand
		Sands >12% fines	SM	Silty sand
			SC	Clayey sand
Fine Grained Soils 50% or more passes the #200 seive	Silts and Clays LL < 50	PI<4 inorganic	ML	Silt
		PI>7 inorganic	CL	Lean clay
		Organic	OL	Organic clay / Organic silt
	Silts and Clays LL 50 or more	PI below A-line inorganic	MH	Elastic silt
		PI on/above A-line inorganic	CH	Fat clay
		Organic	OH	Organic clay / Organic silt
Highly Organic Soils			PT	Peat

GRAIN SIZE DISTRIBUTION (ASTM D2487)

Description	Sieve (in)	Sieve (mm)
Boulders	>12in	>300mm
Cobbles	3-12in	75mm-300mm
Gravel	No.4-3in	2.00mm-75mm
Sand	No.200-No.4	.075mm-2.00mm
Silt	<No.200, PI<4	<.075mm, PI<4
Clay	<No.200, PI>/=4	<.075mm, PI>/=4

SILTS / CLAYS

PLASTICITY (AASHTO)

Description	Blows/Ft.
none	0-5
low	5-10
moderate	10-20
plastic	20-40
highly plastic	>40

DRY STRENGTH (ASTM D2488)
- None: crumbles to powder with handling
- Low: crumbles to powder with finger pressure
- Med: breaks into pieces with high finger pressure
- High: won't crumble, break btw thumb and hard surface
- Very High: can't break btw thumb and hard surface

PLASTICITY (ASTM D2488)
- Nonplastic: can't be rolled
- Low: thread can barely be rolled
- Medium: thread easy and fast to roll
- High: thread takes long time to roll

DILATANCY (ASTM D2488)
- None: no visible change
- Slow: water appears slowly and disappears slowly when squeezed
- Rapid: water appears quickly and disappears quickly when squeezed

TOUGHNESS (ASTM D2488)
- Low: thread & lump is weak and soft
- Med: thread & lump has med stiffness
- High: thread & lump has high stiffness

CONSISTENCY (AASHTO)

Description	Blows/Ft.
very soft	0-1
soft	2-4
med stiff	5-8
stiff	9-15
very stiff	16-30
hard	31-60

CONSISTENCY (ASTM D2488)

Description	Criteria
very soft	thumb penetrates > 1 inch
soft	thumb penetrates 1 inch
firm	thumb indents ¼ inch
hard	thumb won't indent, thumbnail will
very hard	thumbnail won't indent

SANDS / GRAVELS

ANGULARITY (ASTM D2488)
- Angular: sharp edges, plane sides, unpolished
- Subangular: rounded edges
- Subrounded: nearly plane sides, well rounded corners and edges
- Rounded: smoothly curved sides, no edges

SHAPE (ASTM D2488)
- Flat: width/thickness >3
- Elongated: length/width >3
- Flat and Elongated: both

RELATIVE DENSITY(AASHTO)

Description	Blows/Ft.
very loose	0-4
loose	5-10
med dense	11-24
dense	25-50
very dense	> 50

SUPPLEMENTARY DESCRIPTIVE TERMS

MOISTURE (ASTM D2488)
- Dry: dusty, dry to touch
- Moist: damp, but no visible water
- Wet: visible free water, usually below water table

HCl REACTION (ASTM D2488)
- None: no visible reaction
- Weak: some reaction, bubbles forming slowly
- Strong: violent reaction, bubbles immediately

CEMENTATION AT FIELD MOISTURE (ASTM D2488)
- Weak: Crumbles or breaks with handling or little finger pressure
- Moderate: Crumbles or breaks with considerable finger pressure
- Strong: Will not crumble or break with finger pressure

PERCENTAGES (USCS D2488)

Trace	<5%
Few	5-10%
Little	15-25%
Some	30-45%
Mostly	50-100%

TEST HOLE LOG

Hole No. TH-1

STATE OF ALASKA DOT/PF
SE Region Materials
Geology Section

Project	Haines : Ferry Terminal Improvements
Project #	68433
Location	Cell 1 / center
Lat/Long	59°16'54.6" N / 135°27'44.6" W (Swing ties +/- 2 feet. GPS +/- 20 feet)
Elevation	28.5 feet Depth 113 feet
Horizontal Datum	WGS 84

Geologist	M. McDonald, Jr.
Field Crew	AK DOT: Ed Carman/Jason Love
Date Begin	1-8-05
Date End	1-10-05
Weather	15-35° F, cloudy
Equipment	CME 75
Hammer	CME Auto Hammer (140 and 340 lbs)

Drilling Method: Water Rotary- NW casing and tri-cone

Drilling Notes:
Hole located in the approximate center of Cell 1. Corresponding tide level at 7:56 am is 23.5'.

Depth in (Feet)	Casing Blows / ft.	Sample Method	Sample Number	SPT Blow Count	Recovery (feet)	N-Value	Sample Data	Elevation in (Feet)
1		MPT	TH-1-1	18, 22, 12, 9	2.0		M.C.=4.7% p200=10.9%	28
2								27
3		MPT	TH-1-2	6, 6, 6, 5	1.3		M.C.=5.4% p200=8.5%	26
4								25
5								24
6	11							23
7	22							22
8	12							21
9		SPT	TH-1-3a	11, 8, 17	1.0	25	M.C.=12.5% p200=14.7%	20
10		SPT	TH-1-3b	42	0.2			19
11							M.C.=7.3% p200=24.2%	18
12	32							17
13	20							16
14								15
15		SPT	TH-1-4	12, 11, 7, 10	1.0	18	M.C.=9.5% p200=35.2%	14
16								13
17								12
18	23							11
19		SPT	TH-1-5	11, 10, 9, 9	1.5	19	M.C.=8.7% p200=29.3%	10
20								9
21								8
22	22							7
23	17							6
24		SPT	TH-1-6	8, 8, 7, 11	0.5	15	M.C.=7.9% p200=22.4%	5
25								4
26	19							3
27	50							2
28								1
29		SPT	TH-1-7	13, 5, 10, 6	0.8	15	M.C.=7.4% p200=20.2%	0
30								-1
31								-2
32	16							-3
33	15							-4
34		SPT	TH-1-8	7, 6, 5, 6	0.8	11	M.C.=7.4% p200=19.8%	-5
35								-6
36								-7
37	17							-8
38	16							-9
39		GS	TH-1-9		1.0		M.C.=12.5% p200=16.9%	-10
40								-11

SUBSURFACE MATERIAL

0-0.15': ASPHALT

0.15-1.5': Dense, light greenish gray, well-graded **SAND with Gravel (SW)**, dry, ; (FILL)
gravel: subangular, fine-coarse grained
sand: medium grained
silt: trace

1.5-10': Medium dense, brown, medium grained, well-graded **SAND with Gravel(SW)**, moist, some cobbles throughout, boulders expected but not encountered; (FILL)
gravel: mostly fine grained with lesser coarse grained particles, subangular to subrounded
sand: subrounded to rounded, medium sphericity but some flat particles
silt: trace

10-52.5': Medium dense, dark greenish gray **SILTY SAND with Gravel (SM)**, moist to wet, ; (FILL)
gravel: angular to subrounded, fragments of diorite, flat and elongated schist and greenstone
sand: very fine - coarse grained, subrounded to subangular, mostly flat and elongated
silt: nonplastic to low plasticity

17' Observed wood debris in the drill cuttings

17.9'

Tide level was at 23.5' below the ground surface at the time the water table measurement was taken.

10-52.5' The silt content of this unit varies between 5% and 20% and soils range between Silty Sand with gravel (SM) and poorly graded Sand with Silt and Gravel (SP_SM)

38.5-39.5' Boulder

Sample Definition: **SPT** - 2' long, 2" O.D. split spoon driven with a 140 lb. hammer dropped 30 inches. **MPT** - 2' long, 2.5" O.D. split spoon driven with a 340 lbs hammer dropped 30 inches.

LOG OF TEST HOLE SE 3 68433_HNS FERRY TERMINAL IMPROVEMENTS.GPJ SE_AKDOT_MOD_0510.GDT 3/11/05

TEST HOLE LOG

STATE OF ALASKA DOT/PF

SE Region Materials

Geology Section

Hole No. TH-1

Project	Haines : Ferry Terminal Improvements
Project #	68433
Location	Cell 1 / center
Lat/Long	59°16'54.6" N / 135°27'44.6" W (Swing ties +/- 2 feet. GPS +/- 20 feet)
Elevation	28.5 feet Depth 113 feet
Horizontal Datum	WGS 84

Geologist	M. McDonald, Jr.
Field Crew	AK DOT: Ed Carman/Jason Love
Date Begin	1-8-05
Date End	1-10-05
Weather	15-35° F, cloudy
Equipment	CME 75
Hammer	CME Auto Hammer (140 and 340 lbs)

Drilling Notes:
Hole located in the approximate center of Cell 1. Corresponding tide level at 7:56 am is 23.5'.

Drilling Method	Depth in (Feet)	Casing Blows / ft.	Sample Data: Sample Method	Sample Number	SPT Blow Count	Recovery (feet)	N-Value	Sample Data	Soil Graph	Elevation in (Feet)
Water Rotary- NW casing and tri-cone	41	18								-12
	42	15								-13
	43	13								-14
	44		SPT	TH-1-10	9, 6, 6, 9	1.0	12	M.C.=7.7% p200=23.1%		-15
	45									-16
	46	19								-17
	47	39								-18
	48									-19
	49		SPT	TH-1-11	11, 8, 6, 6	0.1	14			-20
	50									-21
	51	21								-22
	52	35								-23
	53									-24
	54		SPT	TH-1-12	15, 16, 11, 11	1.0	27	M.C.=7.5% p200=6.5%		-25
	55									-26
	56									-27
	57	47								-28
	58	39								-29
	59		SPT	TH-1-13	29, 23, 17, 15	0.0	40			-30
	60									-31
	61									-32
	62	23								-33
	63	33								-34
	64		SPT	TH-1-14	10, 7, 6, 1	0.7	13	M.C.=6.6% p200=4.8%		-35
	65									-36
	66									-37
	67	20								-38
	68	24								-39
	69		SPT	TH-1-15	7, 12, 13, 16	0.7	25	M.C.=8.7% p200=7.4%		-40
	70									-41
	71									-42
	72	11								-43
	73	11								-44
	74	10								-45
	75	12								-46
	76	20								-47
	77	19								-48
	78	17								-49
	79		SPT	TH-1-16	14, 10, 12	0.0	22			-50
	80									-51

SUBSURFACE MATERIAL

52.5-74': Medium dense, light gray to brownish gray, fine to coarse grained, poorly graded **SAND (SP)**, wet, ; (BEACH and INTERTIDAL DEPOSITS)
gravel: fine grained, subrounded
silt: content varies

64.5-66' Soil is very loose

74-81.5': Soft to very stiff, gray, elastic **SILT (MH)**, very moist, consistency increases with depth, some clay content expected; (DEEP MARINE / GLACIOMARINE DEPOSITS)
sand: trace
silt: non-dilatant, medium plasticity, low-medium toughness

Sample Definition: **SPT** - 2' long, 2" O.D. split spoon driven with a 140 lb. hammer dropped 30 inches. **MPT** - 2' long, 2.5" O.D. split spoon driven with a 340 lbs hammer dropped 30 inches.

LOG OF TEST HOLE SE 3 68433_HNS FERRY TERMINAL IMPROVEMENTS.GPJ SE_AKDOT_MOD_0510.GDT 3/11/05

TEST HOLE LOG

Hole No. TH-1

STATE OF ALASKA DOT/PF
SE Region Materials
Geology Section

Project	Haines : Ferry Terminal Improvements
Project #	68433
Location	Cell 1 / center
Lat/Long	59°16'54.6" N / 135°27'44.6" W (Swing ties +/- 2 feet. GPS +/- 20 feet)
Elevation	28.5 feet
Depth	113 feet
Horizontal Datum	WGS 84

Geologist	M. McDonald, Jr.
Field Crew	AK DOT: Ed Carman/Jason Love
Date Begin	1-8-05
Date End	1-10-05
Weather	15-35° F, cloudy
Equipment	CME 75
Hammer	CME Auto Hammer (140 and 340 lbs)

Drilling Notes:
Hole located in the approximate center of Cell 1. Corresponding tide level at 7:56 am is 23.5'.

Drilling Method	Depth in (Feet)	Casing Blows / ft.	Sample Data: Sample Method	Sample Number	SPT Blow Count	Recovery (feet)	N-Value	Sample Data	Soil Graph
Water Rotary- NW casing and tri-cone	81		SPT	TH-1-17	13 3 0 30/refusal	1.0		M.C.=26.1% p200=81.7%	
	88		SPT	TH-1-18	9 10 13 28	2.0	23	M.C.=11.7% p200=48.7%	
	99		SPT	TH-1-19	12 3 3 11	2.0	6	M.C.=13.2% p200=54.3%	
Core- BX double tube	104		CORE	TH-1-CR1		4.5		Start=103.5' Rec=90% RQD=55.5%	
	109		CORE	TH-1-CR2		4.4		Start=108.5' Rec=98% RQD=45%	
	113							BOH	

SUBSURFACE MATERIAL

81.5-83.5': Hard, gray, elastic **SILT (MH)**, moist, similar to the previous unit but very hard; casing refusal in this interval; (DEEP MARINE / GLACIOMARINE DEPOSITS)

83.5-103.5': Stiff (consistency varies from medium stiff to very stiff in discreet layers), gray, elastic **SILT with Sand(MH)**, moist to very moist, some clay content expected; (DEEP MARINE / GLACIOMARINE DEPOSIT)
gravel: trace, fine grained
sand: very fine-coarse grained
silt: low-medium plasticity, non-dilatant, low toughness

86' Rock of unknown size

101' Cobble

103.5-113':BEDROCK

Bottom Of Hole 113'

Elevation in (Feet): -52 -53 -54 -55 -56 -57 -58 -59 -60 -61 -62 -63 -64 -65 -66 -67 -68 -69 -70 -71 -72 -73 -74 -75 -76 -77 -78 -79 -80 -81 -82 -83 -84 -85 -86 -87 -88 -89 -90 -91

Sample Definition: **SPT** - 2' long, 2" O.D. split spoon driven with a 140 lb. hammer dropped 30 inches. **MPT** - 2' long, 2.5" O.D. split spoon driven with a 340 lbs hammer dropped 30 inches.

TEST HOLE LOG

Hole No. TH-2

STATE OF ALASKA DOT/PF
SE Region Materials
Geology Section

Project	Haines : Ferry Terminal Improvements
Project #	68433
Location	Cell 2 / center
Lat/Long	59°16'55.0" N / 135°27'46.3" W (Swing ties +/- 2 feet. GPS +/- 20 feet)
Elevation	28.5 feet
Depth	97.6 feet
Horizontal Datum	WGS 84

Geologist	M. McDonald, Jr.
Field Crew	AK DOT: Ed Carman/Jason Love
Date Begin	1-10-05
Date End	1-11-05
Weather	5-15° F, cloudy, windy
Equipment	CME 75
Hammer	CME Auto Hammer (140 and 340 lbs)

Drilling Notes:
Hole located in the approximate center of Cell 2.

Drilling Method	Depth in (Feet)	Casing Blows / ft.	Sample Data: Sample Method	Sample Number	SPT Blow Count	Recovery (feet)	N-Value	Sample Data	Soil Graph
Water Rotary- NW casing and tri-cone	1		MPT	TH-2-1	40 / 19 / 16 / 8	2.0		M.C.=5.4% p200=17.1%	
	2								
	3		MPT	TH-2-2	6 / 8 / 7 / 6	1.3		M.C.=4.2% p200=6.9%	
	4								
	5	27							
	6	34							
	7	37							
	8								
	9		SPT	TH-2-3	9 / 22 / 30 / 40	1.4	52	M.C.=7.1% p200=35.8%	
	10								
	11	26							
	12	35							
	13								
	14		SPT	TH-2-4	8 / 11 / 10 / 12	2.0	21	M.C.=10.0% p200=34.2%	
	15								▼ 15.5'
	16	18							
	17	15							
	18								
	19		SPT	TH-2-5	9 / 15 / 12 / 10	0.8	27	M.C.=9.7% p200=30.7%	
	20								
	21	22							
	22	15							
	23								
	24		SPT	TH-2-6	7 / 11 / 9 / 12	0.8	20	M.C.=8.6% p200=23.9%	
	25								
	26	27							
	27	30							
	28								
	29		SPT	TH-2-7	10 / 7 / 16 / 8	0.9	23	M.C.=9.4% p200=27.3%	
	30								
	31	15							
	32	14							
	33								
	34		SPT	TH-2-8	4 / 5 / 6 / 9	0.6	11	M.C.=8.8% p200=18.2%	
	35								
	36	12							
	37	10							
	38								
	39		SPT	TH-2-9	6 / 6 / 7	0.8	13	M.C.=8.4% p200=18.4%	
	40								

SUBSURFACE MATERIAL

0-0.15': ASPHALT

0.15-0.5': Dense, brown, well-graded **SAND with Gravel(SW)**, dry, ; (FILL)
gravel: fine to coarse grained, rounded-subrounded, medium-high sphericity
sand: fine-medium grained, subrounded, high sphericity
silt: trace

0.5-3.5': Medium dense to dense, light greenish gray, well-graded **SAND with Gravel(SW)**, dry, ; (FILL)
gravel: fine-coarse grained, subangular
sand: medium grained
silt: trace

3.5-9.5': Medium dense, brown, poorly graded **SAND with Gravel(SP)**, moist to wet, some cobbles and trace boulders; (FILL)
gravel: fine-coarse grained, rounded-subrounded, medium- high sphericity
sand: fine-medium grained with lesser amounts of coarse grained particles, subrounded, high sphericity
silt: trace

9.5-36.5': Medium dense, dark greenish gray, poorly graded **SAND with Silt and Gravel(SP-SM)**, moist to wet, ; (FILL)
gravel: fine grained, subangular-subrounded, medium sphericity
sand: very fine-medium grained
silt: green

36.5-43.5': Loose to medium dense, dark gray, poorly graded **SAND with Silt and Gravel(SP-SM)**, wet, some cobbles and possibly boulders; (FILL)
gravel: fine grained, subangular, low sphericity with some flat particles
sand: fine-medium grained

Elevation in (Feet): 28 27 26 25 24 23 22 21 20 19 18 17 16 15 14 13 12 11 10 9 8 7 6 5 4 3 2 1 0 -1 -2 -3 -4 -5 -6 -7 -8 -9 -10 -11

Sample Definition: SPT - 2' long, 2" O.D. split spoon driven with a 140 lb. hammer dropped 30 inches. MPT - 2' long, 2.5" O.D. split spoon driven with a 340 lbs hammer dropped 30 inches.

LOG OF TEST HOLE SE 3 68433_HNS FERRY TERMINAL IMPROVEMENTS.GPJ SE_AKDOT_MOD_0510.GDT 3/11/05

TEST HOLE LOG

STATE OF ALASKA DOT/PF
SE Region Materials
Geology Section

Hole No. TH-2

Project	Haines : Ferry Terminal Improvements
Project #	68433
Location	Cell 2 / center
Lat/Long	59°16'55.0" N / 135°27'46.3" W (Swing ties +/- 2 feet. GPS +/- 20 feet)
Elevation	28.5 feet Depth 97.6 feet
Horizontal Datum	WGS 84

Geologist	M. McDonald, Jr.
Field Crew	AK DOT: Ed Carman/Jason Love
Date Begin	1-10-05
Date End	1-11-05
Weather	5-15° F, cloudy, windy
Equipment	CME 75
Hammer	CME Auto Hammer (140 and 340 lbs)

Drilling Notes:
Hole located in the approximate center of Cell 2.

Drilling Method	Depth in (Feet)	Casing Blows / ft.	Sample Data: Sample Method	Sample Number	SPT Blow Count	Recovery (feet)	N-Value	Sample Data	Soil Graph	SUBSURFACE MATERIAL	Elevation in (Feet)
Water Rotary- NW casing and tri-cone	41	14			8					silt: green	-12
	42	50								42.5' Cobble	-13
	43									**43.5-48.5':** Medium dense, light gray, poorly graded **SAND with Gravel (SP)**, wet, traces of wood debris, some cobbles, this unit appears to be bedded with layer thicknesses approximately 1' thick; (BEACH DEPOSIT)	-14
	44		SPT	TH-2-10	11	0.6	14	M.C.=6.4% p200=9.7%		gravel: fine grained, angular-subangular	-15
	45				9 5 6					sand: medium-coarse grained, subangular	-16
	46	32								silt: trace	-17
	47	12								46.5' Cobble	-18
	48			TH-2TH-2-11a						**48.5-50':** Stiff, light gray, elastic **SILT with Sand(MH)**, wet, distinctly spongy feel to this material; (INTERTIDAL / SHALLOW MARINE DEPOSIT)	-19
	49		SPT		3 8	0.8		M.C.=9.2%		sand: very fine-fine grained, as much as 15% sand sized shell fragments	-20
	50		SPT		11 12	0.7		p200=6.5%		silt: medium plasticity, non dilatant, low toughness	-21
	51	24						M.C.=27.0% p200=37.9%		**50-74':** Loose to medium dense, light gray to gray, poorly graded **SAND (SP)**, wet, shell fragments noted in the fine grained layers; (BEACH DEPOSITS)	-22
	52	34								gravel: noted only as angular fragments in the drill cuttings and trace amounts in the samples	-23
	53									sand: layers of very fine-fine grained mixed with layers of medium to coarse grained, subangular	-24
	54		SPT	TH-2-12	14 19 29 14	1.5	48	M.C.=12.8% p200=12.9%		silt: trace amounts from 50-65' becoming less below 65'	-25
	55										-26
	56	27									-27
	57	25									-28
	58										-29
	59		SPT	TH-2-13	11 14 10 9	0.7	24	M.C.=9.8% p200=7.4%			-30
	60										-31
	61	28									-32
	62	29									-33
	63	24									-34
	64	21									-35
	65	20									-36
	66	19									-37
	67	17									-38
	68										-39
	69		SPT	TH-2-14	4 6 6 7	0.7	12	M.C.=5.5% p200=2.3%			-40
	70										-41
	71	15									-42
	72	14									-43
	73	12									-44
	74										-45
	75	7	GS	TH-2-Cuttins				M.C.=20.6% p200=39.0%		**74-78.5';** gray, elastic **SILT with Sand(MH)**, wet, ; (SHALLOW MARINE DEPOSITS)	-46
	76	14								sand: fine-medium grained	-47
	77	15								silt: medium plasticity	-48
	78	57									-49
	79		SPT	TH-2-15	50/2"	0.0				**78.5-87.5':** Medium dense, light gray, poorly graded **SAND with Gravel (SP)**, wet, with cobbles and boulders between 78' and 83'; (BEACH	-50
	80										-51

Sample Definition: **SPT** - 2' long, 2" O.D. split spoon driven with a 140 lb. hammer dropped 30 inches. **MPT** - 2' long, 2.5" O.D. split spoon driven with a 340 lbs hammer dropped 30 inches.

LOG OF TEST HOLE SE 3 68433_HNS FERRY TERMINAL IMPROVEMENTS.GPJ SE_AKDOT_MOD_0510.GDT 3/11/05

TEST HOLE LOG

Hole No. TH-2

STATE OF ALASKA DOT/PF
SE Region Materials
Geology Section

Project	Haines : Ferry Terminal Improvements
Project #	68433
Location	Cell 2 / center
Lat/Long	59°16'55.0" N / 135°27'46.3" W (Swing ties +/- 2 feet. GPS +/- 20 feet)
Elevation	28.5 feet Depth 97.6 feet
Horizontal Datum	WGS 84

Geologist	M. McDonald, Jr.
Field Crew	AK DOT: Ed Carman/Jason Love
Date Begin	1-10-05
Date End	1-11-05
Weather	5-15° F, cloudy, windy
Equipment	CME 75
Hammer	CME Auto Hammer (140 and 340 lbs)

Drilling Notes:
Hole located in the approximate center of Cell 2.

Drilling Method	Depth in (Feet)	Casing Blows / ft.	Sample Data: Sample Method	Sample Number	SPT Blow Count	Recovery (feet)	N-Value	Sample Data	Soil Graph	Elevation in (Feet)
Core-BX double tube core barrel Water Rotary- open hole with tri-cone	81									-52
	82									-53
	83									-54
	84									-55
	85									-56
	86									-57
	87									-58
	88									-59
	89		SPT	TH-2-16	23 9 5 6	2.0	14	M.C.=9.1% p200=49.1%		-60
	90									-61
	91									-62
	92									-63
	93									-64
	94		SPT	TH-2-CR1		1.9		Start=93.7' Rec=49% RQD=0%		-65
	95									-66
	96									-67
	97									-68
	98								BOH	-69
	99									-70
	100									-71
	101									-72
	102									-73
	103									-74
	104									-75
	105									-76
	106									-77
	107									-78
	108									-79
	109									-80
	110									-81
	111									-82
	112									-83
	113									-84
	114									-85
	115									-86
	116									-87
	117									-88
	118									-89
	119									-90
	120									-91

SUBSURFACE MATERIAL

DEPOSITS)
81' Boulder

82.5' Cobble

87.5-93': Medium stiff to stiff, light gray **SANDY SILT(ML)**, wet, ; (DIAMICTON / GLACIOMARINE DEPOSITS)
gravel: trace, angular-subangular, low sphericity
sand: very fine-fine grained
silt: non plastic, slow-rapid dilatancy, low toughness, none-low dry strength
91.5-93' becomes extremely hard, some cobbles

93-97.6': BEDROCK similar rock type to TH-1 with same structural orientation

Bottom Of Hole 97.6'

Sample Definition: **SPT** - 2' long, 2" O.D. split spoon driven with a 140 lb. hammer dropped 30 inches. **MPT** - 2' long, 2.5" O.D. split spoon driven with a 340 lbs hammer dropped 30 inches.

TEST HOLE LOG

Hole No. TH-3

STATE OF ALASKA DOT/PF
SE Region Materials
Geology Section

Project	Haines : Ferry Terminal Improvements
Project #	68433
Location	Cell 4 / behind the sheet pile structure
Lat/Long	59°16'55.1" N / 135°27'48.8" W (Swing ties +/- 2 feet. GPS +/- 20 feet)
Elevation	28.5 feet Depth 73.5 feet
Horizontal Datum	WGS 84

Geologist	M. McDonald, Jr.
Field Crew	AK DOT: Ed Carman/Jason Love
Date Begin	1-15-05
Date End	1-15-05
Weather	15°F, cloudy
Equipment	CME 75
Hammer	CME Auto Hammer (140 and 340 lbs)

Drilling Method: Water Rotary- NW casing and tri-cone

Drilling Notes:
Hole located at the limits of the temporary excavation made to repair Cell 4.

Depth in (Feet)	Casing Blows / ft.	Sample Method	Sample Number	SPT Blow Count	Recovery (feet)	N-Value	Sample Data	Soil Graph	SUBSURFACE MATERIAL	Elevation in (Feet)
1		MPT	TH-3-1	100/3"	0.0				**0-0.15': ASPHALT**	28
2	192	SPT	TH-3-2	50/4"	0.2				**0.15-1.5':** Frozen, light brown, poorly graded **SAND with Gravel(SP)**, dry, trace silt; (FILL)	27
3	58								**1.5-29':** Medium dense, dark greenish gray, poorly graded **SAND with Silt (SP-SM)**, moist to wet, with cobbles and boulders; (FILL)	26
4	33								gravel: fine-coarse grained, angular-subangular, elongated-low sphericity particles;	25
5		SPT	TH-3-3	14, 13, 11, 9	0.9	24	M.C.=8.6% p200=21.9%		sand: medium grained;	24
6									silt: green;	23
7	32								2.5-3.5' Boulder	22
8	24								3.7' Cobble	21
9	25									20
10		SPT	TH-3-4	12, 8, 5, 23	0.9	13	M.C.=7.8% p200=21.4%			19
11										18
12	33									17
13	31								13' Cobble	16
14	35								▼ 13.4'	15
15		SPT	TH-3-5	20, 12, 12, 15	1.1	24	M.C.=7.8% p200=18.8%			14
16										13
17	38								16.5' Cobble	12
18	31								17' Cobble	11
19	30									10
20		SPT	TH-3-6	11, 16, 13, 28	0.9	29	M.C.=9.0% p200=25.4%			9
21										8
22	21									7
23	19									6
24	17									5
25		SPT	TH-3-7	5, 4, 7, 9	0.4	11	M.C.=5.7% p200=11.6%		24' Decrease in the relative density and increased gravel content	4
26										3
27	13									2
28	8									1
29	9									0
30		SPT	TH-3-8	6, 5, 8, 9	1.1	13	M.C.=15.1% p200=22.8%		**29-33':** Medium dense, dark gray, slightly organic **SILTY SAND(SM)**, wet, traces of shell fragments and fibrous organics; (BEACH / SHALLOW MARINE DEPOSIT)	-1
31	20								gravel: trace amounts, blocky, angular particles;	-2
32	20									-3
33	36									-4
34		SPT	TH-3-9	17, 14, 14, 11	1.0	28	M.C.=6.8% p200=0.0%		**33-42':** Medium dense, light gray to gray, poorly graded **SAND with Gravel (SP)**, wet, (BEACH DEPOSIT)	-5
35									gravel: fine grained, rounded-subrounded;	-6
36	25								sand: coarse grained, subrounded-subangular;	-7
37	21								silt: trace amounts but generally less below 36';	-8
38	25									-9
39				7, 9			M.C.=8.8%			-10
40										-11

Sample Definition: **SPT** - 2' long, 2" O.D. split spoon driven with a 140 lb. hammer dropped 30 inches. **MPT** - 2' long, 2.5" O.D. split spoon driven with a 340 lbs hammer dropped 30 inches.

TEST HOLE LOG

Hole No. TH-3

STATE OF ALASKA DOT/PF
SE Region Materials
Geology Section

Project	Haines : Ferry Terminal Improvements
Project #	68433
Location	Cell 4 / behind the sheet pile structure
Lat/Long	59°16'55.1" N / 135°27'48.8" W (Swing ties +/- 2 feet. GPS +/- 20 feet)
Elevation	28.5 feet Depth 73.5 feet
Horizontal Datum	WGS 84

Geologist	M. McDonald, Jr.
Field Crew	AK DOT: Ed Carman/Jason Love
Date Begin	1-15-05
Date End	1-15-05
Weather	15°F, cloudy
Equipment	CME 75
Hammer	CME Auto Hammer (140 and 340 lbs)

Drilling Notes:
Hole located at the limits of the temporary excavation made to repair Cell 4.

Drilling Method	Depth in (Feet)	Casing Blows / ft.	Sample Data: Sample Method	Sample Number	SPT Blow Count	Recovery (feet)	N-Value	Sample Data	Soil Graph	Elevation in (Feet)
Water Rotary- open hole with tri-cone	41	43	SPT	TH-3-10	9 11	0.9	18	p200=3.2%		-12
	42									-13
	43		CORE	TH-3-CR1		0.5		Start=42.5' RQD=0%		-14
	44									-15
	45									-16
	46									-17
	47									-18
	48		SPT	TH-3-11	50/2"	0.0				-19
	49									-20
	50									-21
	51									-22
	52									-23
	53									-24
	54									-25
	55									-26
	56									-27
	57									-28
	58									-29
	59									-30
	60									-31
	61									-32
	62									-33
	63									-34
	64									-35
	65									-36
	66									-37
	67									-38
	68									-39
	69									-40
	70									-41
	71									-42
	72									-43
	73									-44
	74								BOH	-45
	75									-46
	76									-47
	77									-48
	78									-49
	79									-50
	80									-51

SUBSURFACE MATERIAL

42-44': BOULDER

44-73.5': Very stiff to hard, greenish gray **SANDY SILT to SILTY SAND (ML / SM)** wet, possible traces of fine grained shell fragments; this unit is very hard and required 1200 to 1500 lbs of down pressure to drill with a tri-cone bit, no samples were recovered in this interval and it was logged from drill cuttings only; (ORIGIN UNKNOWN)
sand: very fine-fine grained, approximately 40-50%;
silt: green;

Bottom Of Hole 73.5'

Sample Definition: **SPT** - 2' long, 2" O.D. split spoon driven with a 140 lb. hammer dropped 30 inches. **MPT** - 2' long, 2.5" O.D. split spoon driven with a 340 lbs hammer dropped 30 inches.

LOG OF PENETROMETER

Hole No. **P-1**

STATE OF ALASKA DOT/PF
SE Region Materials
Geology Section

Project	Haines : Ferry Terminal Improvements
Project #	68433
Location	Closure arc / between cell 1 & 2
Lat/Long	59°16'55.1" N / 135°27'45.4" W
Elevation	28 feet
Depth	107.2 feet
Geologist	M. McDonald, Jr.
Field Crew	AK DOT: Ed Carman/Jason Love
Date Begin	1-13-05
Date End	1-15-05
Weather	0° F, cloudy
Equipment	CME 75 / CME Auto Hammer (340 lbs)

Drilling Notes:
Hole located at proposed mooring structure.

Penetrometer- 2.5" O.D.

GRAPH OF PENETROMETER IN BLOWS/FOOT

Depth in (Feet)	Penetrometer Blows / ft.
1	
2	
3	16
4	7
5	8
6	38
7	40
8	
9	
10	4
11	7
12	7
13	8
14	8
15	6
16	8
17	5
18	8
19	16
20	7
21	6
22	7
23	7
24	14
25	10
26	10
27	11
28	7
29	9
30	17
31	17
32	14
33	8
34	7
35	9
36	8
37	8
38	12
39	12
40	15

Elevation in (Feet)	
28	0-0.15' Asphalt
27	0.15-1.5' Frozen soil
26	
25	
24	
23	
22	5.5' Rocks
21	
20	7-9' Concrete, cored with NQ wireline
19	
18	
17	
16	
15	
14	
13	
12	
11	
10	
9	
8	
7	
6	
5	23.2' Rock
4	
3	
2	
1	
0	
-1	
-2	
-3	
-4	
-5	
-6	
-7	
-8	
-9	
-10	
-11	
-12	

Note: Unless otherwise noted the penetrometer test is a 2.5 inch O.D., flat tipped, thick walled rod driven continuously with a 340 lbs. CME auto hammer.

FOUNDATION PENETROMETER SE 68433_HNS FERRY TERMINAL IMPROVEMENTS.GPJ SE_AKDOT_MOD_040504.GDT 3/10/05

LOG OF PENETROMETER

Hole No. P-1

STATE OF ALASKA DOT/PF
SE Region Materials
Geology Section

Project	Haines : Ferry Terminal Improvements
Project #	68433
Location	Closure arc / between cell 1 & 2
Lat/Long	59°16'55.1" N / 135°27'45.4" W
Elevation	28 feet Depth 107.2 feet

Geologist	M. McDonald, Jr.
Field Crew	AK DOT: Ed Carman/Jason Love
Date Begin	1-13-05
Date End	1-15-05
Weather	0° F, cloudy
Equipment	CME 75 / CME Auto Hammer (340 lbs)

Drilling Notes:
Hole located at proposed mooring structure.

GRAPH OF PENETROMETER IN BLOWS/FOOT

Drilling Method	Depth in (Feet)	Penetrometer Blows / ft.	Elevation in (Feet)
Penetrometer- 2.5" O.D.	41	13	-12
	42	14	-13
	43	11	-14
	44	17	-15
	45	12	-16
	46	11	-17
	47	11	-18
	48	12	-19
	49	8	-20
	50	12	-21
	51	19	-22
	52	27	-23
	53	14	-24
	54	6	-25
	55	7	-26
	56	7	-27
	57	9	-28
	58	15	-29
	59	13	-30
	60	9	-31
	61	10	-32
	62	15	-33
	63	18	-34
	64	24	-35
	65	26	-36
	66	15	-37
	67	17	-38
	68	20	-39
	69	42	-40
	70	116	-41
	71	128	-42
	72	97	-43
	73	77	-44
	74	80	-45
	75	105	-46
	76	88	-47
	77	63	-48
	78	43	-49
	79	39	-50
	80	44	-51
			-52

42' Strata change

77' Strata change

Note: Unless otherwise noted the penetrometer test is a 2.5 inch O.D., flat tipped, thick walled rod driven continuously with a 340 lbs. CME auto hammer.

LOG OF PENETROMETER

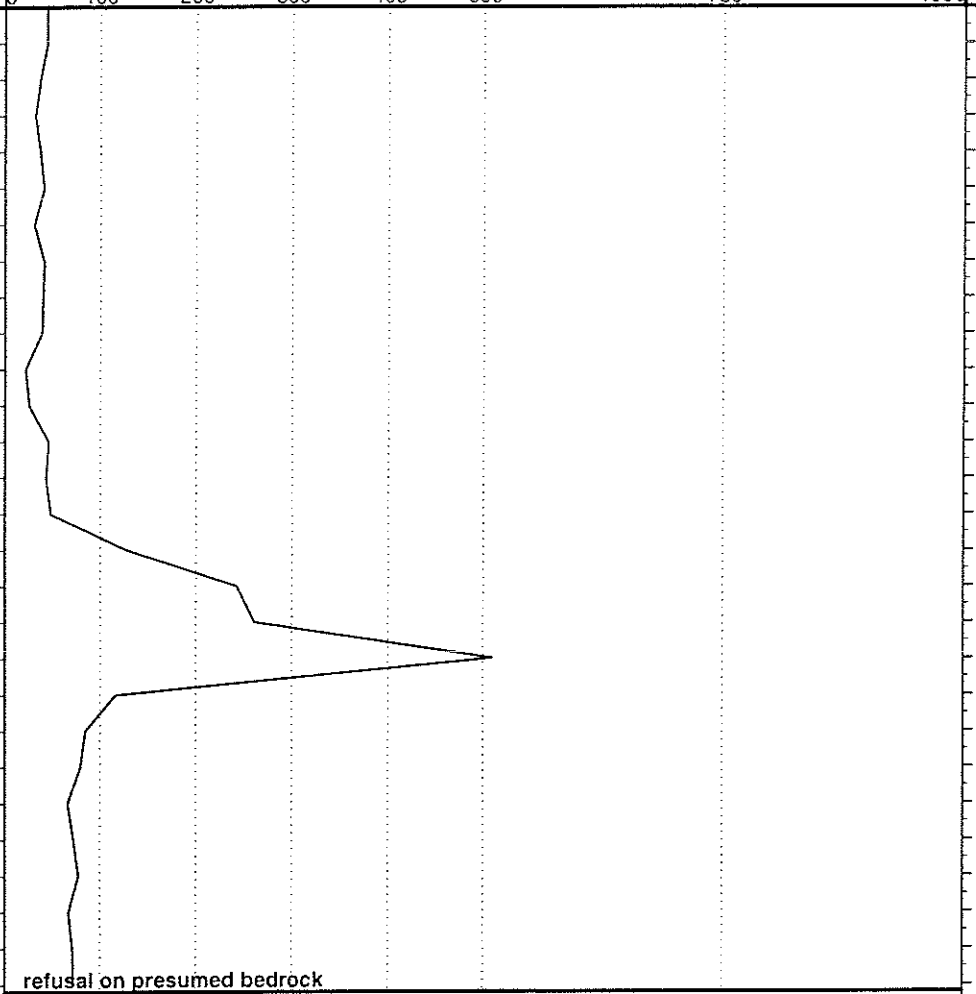
Hole No. P-1

STATE OF ALASKA DOT/PF
SE Region Materials
Geology Section

Project	Haines : Ferry Terminal Improvements
Project #	68433
Location	Closure arc / between cell 1 & 2
Lat/Long	59°16'55.1" N / 135°27'45.4" W
Elevation	28 feet Depth 107.2 feet
Geologist	M. McDonald, Jr.
Field Crew	AK DOT: Ed Carman/Jason Love
Date Begin	1-13-05
Date End	1-15-05
Weather	0° F, cloudy
Equipment	CME 75 / CME Auto Hammer (340 lbs)

Drilling Notes:
Hole located at proposed mooring structure.

GRAPH OF PENETROMETER IN BLOWS/FOOT



Drilling Method: Penetrometer- 2.5" O.D.

Depth in (Feet)	Penetrometer Blows / ft.	Elevation in (Feet)	
81	44	-52	
82	37	-53	
83	32	-54	
84	37	-55	
85	41	-56	
86	31	-57	
87	41	-58	
88	40	-59	
89	39	-60	
90	22	-61	
91	26	-62	
92	46	-63	91' Strata change
93	43	-64	
94	48	-65	
95	128	-66	
96	242	-67	94.8' Strata change
97	260	-68	
98	509	-69	
99	116	-70	
100	84	-71	98.2' Strata change
101	79	-72	
102	66	-73	
103	72	-74	
104	77	-75	
105	67	-76	
106	71	-77	
107	72	-78	
108	153/1	-79	1300 psi required to break the rods free
109		-80	
110		-81	
111		-82	
112		-83	
113		-84	
114		-85	
115		-86	
116		-87	
117		-88	
118		-89	
119		-90	
120		-91	
		-92	

refusal on presumed bedrock

BOTTOM OF HOLE at 107.2'

Note: Unless otherwise noted the penetrometer test is a 2.5 inch O.D., flat tipped, thick walled rod driven continuously with a 340 lbs. CME auto hammer.

FOUNDATION PENETROMETER SE 68433_HNS FERRY TERMINAL IMPROVEMENTS.GPJ SE_AKDOT_MOD_040504.GDT 3/10/05

LOG OF PENETROMETER

Hole No. P-2

STATE OF ALASKA DOT/PF
SE Region Materials
Geology Section

Project	Haines : Ferry Terminal Improvements
Project #	68433
Location	Closure arc / between cell 2 & 3
Lat/Long	59°16'55.4" N / 135°27'46.6" W
Elevation	28 feet
Depth	94 feet
Geologist	M. McDonald, Jr.
Field Crew	AK DOT: Ed Carman/Jason Love
Date Begin	1-13-05
Date End	1-13-05
Weather	-4° F, clear
Equipment	CME 75 / CME Auto Hammer (340 lbs)

Drilling Notes:
Hole located at proposed mooring structure.

GRAPH OF PENETROMETER IN BLOWS/FOOT

Depth in (Feet)	Penetrometer Blows / ft.	Elevation in (Feet)	
1		28	0-0.15' Asphalt
2	33	27	0.15-1.5' Frozen soil
3	24	26	
4	25	25	
5	43	24	
6	24	23	5' Rocks
7		22	
8		21	7-9' Concrete, cored with NQ wireline
9	12	20	
10	22	19	
11	16	18	
12	13	17	
13	13	16	
14	13	15	
15	14	14	
16	25	13	16-17' Rocks
17	29	12	
18	29	11	17.8-18.5' Rocks
19	14	10	
20	28	9	
21	19	8	20.2' Rock 20.8' Rock
22	18	7	
23	14	6	
24	13	5	
25	14	4	
26	19	3	
27	10	2	
28	6	1	27' Strata change
29	6	0	
30	9	-1	
31	9	-2	
32	13	-3	
33	18	-4	
34	12	-5	
35	11	-6	
36	7	-7	
37	8	-8	
38	9	-9	
39	11	-10	
40		-11	
		-12	

Drilling Method: Penetrometer- 2.5" O.D.

Note: Unless otherwise noted the penetrometer test is a 2.5 inch O.D., flat tipped, thick walled rod driven continuously with a 340 lbs. CME auto hammer.

FOUNDATION PENETROMETER SE 68433_HNS FERRY TERMINAL IMPROVEMENTS.GPJ SE_AKDOT_MOD_040504.GDT 3/10/05

LOG OF PENETROMETER

Hole No. P-2

STATE OF ALASKA DOT/PF
SE Region Materials
Geology Section

Project	Haines : Ferry Terminal Improvements
Project #	68433
Location	Closure arc / between cell 2 & 3
Lat/Long	59°16'55.4" N / 135°27'46.6" W
Elevation	28 feet
Depth	94 feet
Geologist	M. McDonald, Jr.
Field Crew	AK DOT: Ed Carman/Jason Love
Date Begin	1-13-05
Date End	1-13-05
Weather	-4° F, clear
Equipment	CME 75 / CME Auto Hammer (340 lbs)

Drilling Notes:
Hole located at proposed mooring structure.

Drilling Method: Penetrometer- 2.5" O.D.

GRAPH OF PENETROMETER IN BLOWS/FOOT

Depth in (Feet)	Penetrometer Blows / ft.	Elevation in (Feet)	
	0 100 200 300 400 500 750 1000	-12	
41	16	-13	
42	8	-14	
43	15	-15	
44	13	-16	
45	11	-17	
46	9	-18	
47	10	-19	
48	15	-20	
49	8	-21	
50	9	-22	
51	17	-23	
52	10	-24	
53	11	-25	
54	8	-26	
55	9	-27	
56	7	-28	
57	8	-29	
58	10	-30	
59	7	-31	
60	7	-32	
61	15	-33	60' Strata change
62	23	-34	
63	17	-35	
64	27	-36	63' Strata change
65	49	-37	
66	71	-38	
67	66	-39	
68	57	-40	
69	30	-41	
70	22	-42	69' Strata change
71	51	-43	70' Strata change
72	94	-44	
73	80	-45	
74	49	-46	
75	46	-47	
76	67	-48	75.4-75.9' Rock
77	40	-49	
78	35	-50	
79	41	-51	
80	40	-52	

Note: Unless otherwise noted the penetrometer test is a 2.5 inch O.D., flat tipped, thick walled rod driven continuously with a 340 lbs. CME auto hammer.

LOG OF PENETROMETER

Hole No. P-2

STATE OF ALASKA DOT/PF
SE Region Materials
Geology Section

Project	Haines : Ferry Terminal Improvements
Project #	68433
Location	Closure arc / between cell 2 & 3
Lat/Long	59°16'55.4" N / 135°27'46.6" W
Elevation	28 feet
Depth	94 feet
Geologist	M. McDonald, Jr.
Field Crew	AK DOT: Ed Carman/Jason Love
Date Begin	1-13-05
Date End	1-13-05
Weather	-4° F, clear
Equipment	CME 75 / CME Auto Hammer (340 lbs)

Drilling Notes:
Hole located at proposed mooring structure.

GRAPH OF PENETROMETER IN BLOWS/FOOT

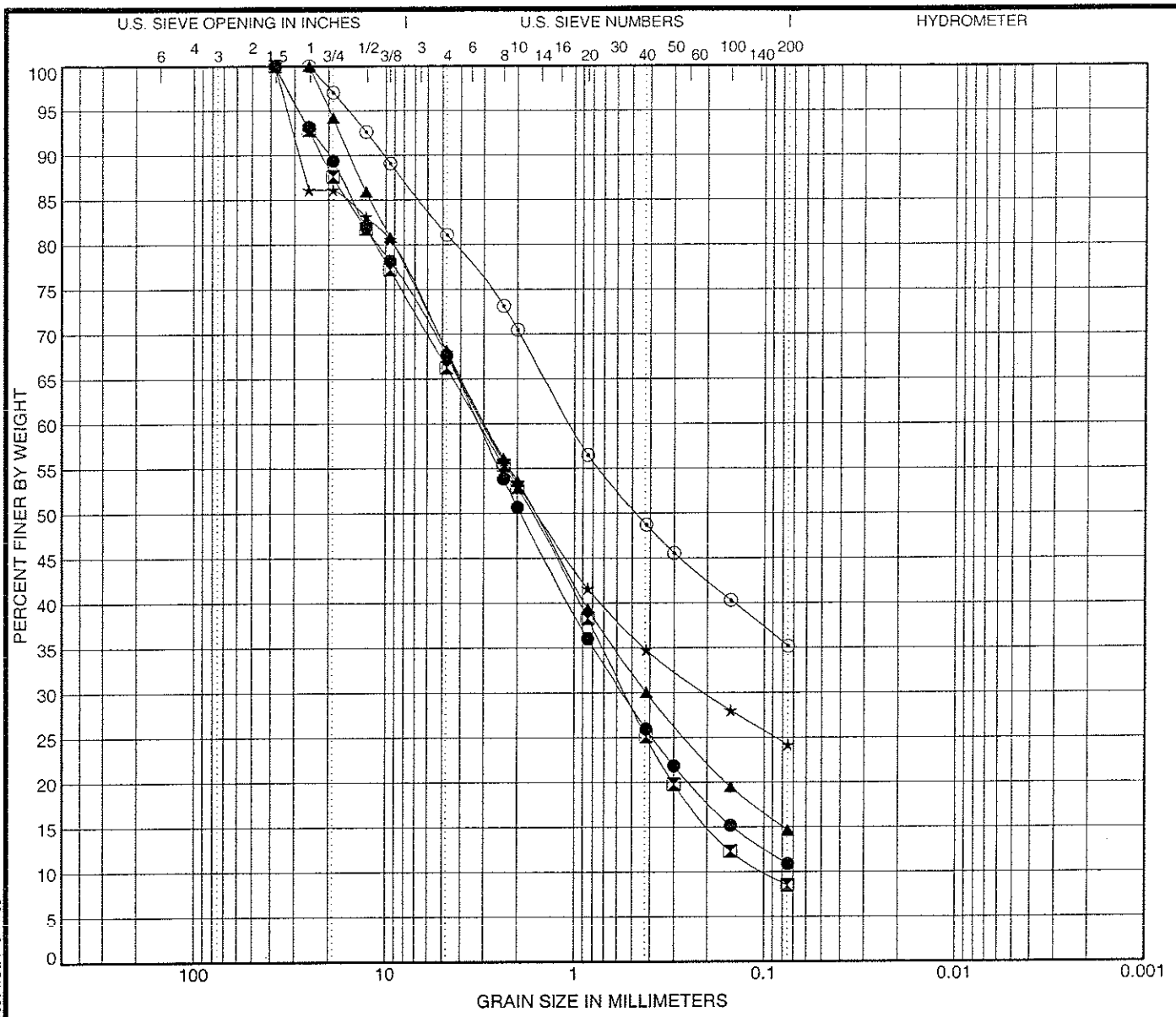
Drilling Method	Depth in (Feet)	Penetrometer Blows / ft.
Penetrometer- 2.5" O.D.	81	44
	82	37
	83	35
	84	34
	85	28
	86	19
	87	209
	88	309
	89	356
	90	119
	91	90
	92	70
	93	65
	94	89
	94	1000/2" refusal on presumed bedrock
	94	BOTTOM OF HOLE at 94'

Elevation in (Feet)	
-52	
-53	81
-54	82
-55	83
-56	84
-57	85 84' Strata change
-58	86
-59	87 86.2' Strata change
-60	88
-61	89
-62	90 89.5' Strata change
-63	91
-64	92
-65	93
-66	94
-67	95 1600 psi required to
-68	96 break the rods
-69	97 free
-70	98
-71	99
-72	100
-73	101
-74	102
-75	103
-76	104
-77	105
-78	106
-79	107
-80	108
-81	109
-82	110
-83	111
-84	112
-85	113
-86	114
-87	115
-88	116
-89	117
-90	118
-91	119
-92	120

Note: Unless otherwise noted the penetrometer test is a 2.5 inch O.D., flat tipped, thick walled rod driven continuously with a 340 lbs. CME auto hammer.

FOUNDATION PENETROMETER SE 68433_HNS FERRY TERMINAL IMPROVEMENTS.GPJ SE_AKDOT_MOD_040504.GDT 3/10/05

APPENDIX C
AKDOT & PF
LABORATORY TEST DATA



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	(frost susceptible soils % finer 0.02mm)

Sample # / Depth		Lab #	USCS Classification	%Org	% MC	SpG	LL	PL	PI
● TH-1-1	0.5	05C-21	WELL-GRADED SAND with SILT and GRAVEL(SW-SM)		4.7	2.7	NV	NP	NP
⊠ TH-1-2	2.5	05C-22	POORLY GRADED SAND with SILT and GRAVEL(SP-SM)		5.4	2.8	NV	NP	NP
▲ TH-1-3a	8.5	05C-23	SILTY SAND with GRAVEL(SM)		12.5		NV	NP	NP
★ TH-1-3b	10.0	05C-24	SILTY SAND with GRAVEL(SM)		7.3		NV	NP	NP
⊙ TH-1-4	14.5	05C-25	SILTY SAND with GRAVEL(SM)		9.5	2.9	NV	NP	NP

Sample # / Depth		Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay	Frost Susceptible Soils (% finer 0.02mm)
● TH-1-1	0.5	1.48	49.68	38	3.225	0.556		32.4	56.7	10.9		
⊠ TH-1-2	2.5	0.95	32.24	38	3.178	0.545	0.099	33.7	57.8	8.5		
▲ TH-1-3a	8.5			25.4	2.952	0.418		31.8	53.5	14.7		
★ TH-1-3b	10.0			38	3.002	0.203		32.0	43.8	24.2		
⊙ TH-1-4	14.5			25.4	1.053			18.9	45.9	35.2		

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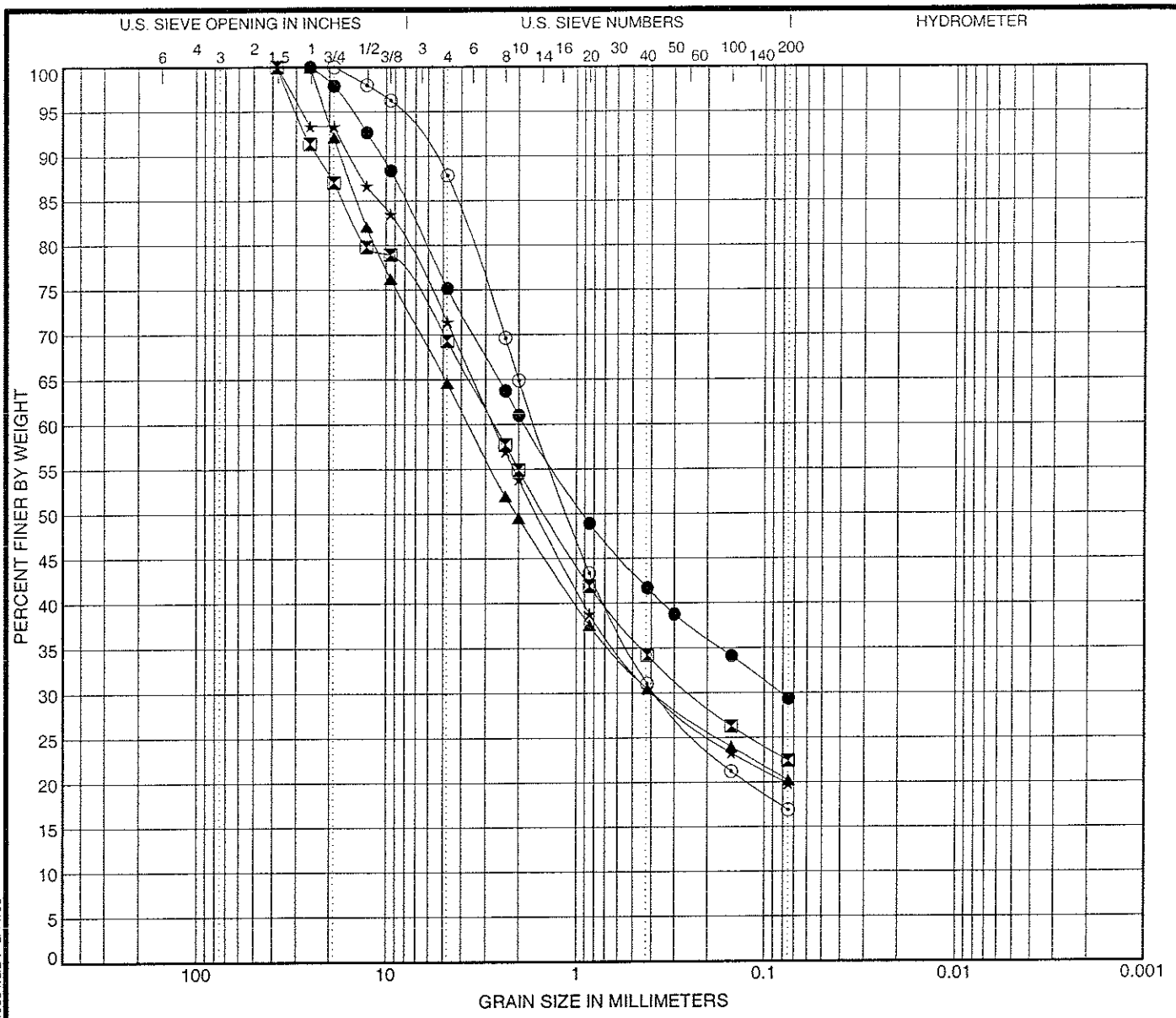
SUBSURFACE DATA: Results of Laboratory Testing Sheet 1 of 9

Location: Haines

Project: Ferry Terminal Improvements

Number: 68433

US_GRAIN_SIZE_MOD 2_68433_HNS FERRY TERMINAL IMPROVEMENTS.GPJ_SE_AKDOT_MOD_040504.GDT 3/11/05



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	(frost susceptible soils % finer 0.02mm)

	Sample # / Depth		Lab #	USCS Classification	%Org	% MC	SpG	LL	PL	PI
●	TH-1-5	18.5	05C-26	SILTY SAND with GRAVEL(SM)		8.7	2.9	NV	NP	NP
⊠	TH-1-6	23.5	05C-27	SILTY SAND with GRAVEL(SM)		7.9		NV	NP	NP
▲	TH-1-7	28.5	05C-28	SILTY SAND with GRAVEL(SM)		7.4		NV	NP	NP
★	TH-1-8	33.5	05C-28	SILTY SAND with GRAVEL(SM)		7.4		NV	NP	NP
⊙	TH-1-9	38.5	05C-29	SILTY SAND(SM)		12.5		NV	NP	NP

	Sample # / Depth		Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay	Frost Susceptible Soils (% finer 0.02mm)
●	TH-1-5	18.5			25.4	1.861	0.083		24.8	45.8	29.3		
⊠	TH-1-6	23.5			38	2.706	0.243		30.7	46.9	22.4		
▲	TH-1-7	28.5			25.4	3.679	0.389		35.4	44.4	20.2		
★	TH-1-8	33.5			38	2.731	0.395		28.6	51.6	19.8		
⊙	TH-1-9	38.5			19	1.646	0.376		12.1	70.9	16.9		

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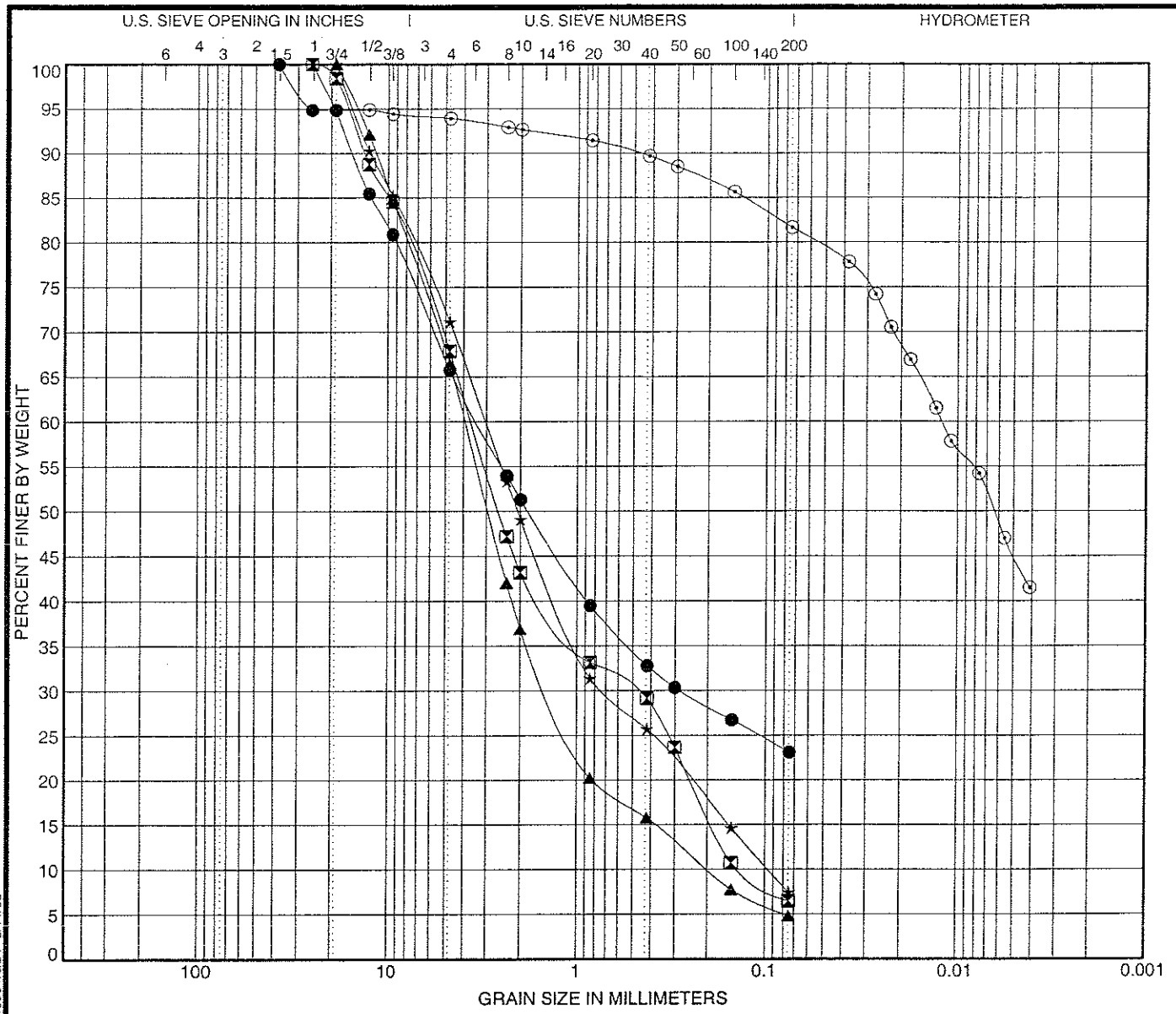
SUBSURFACE DATA: Results of Laboratory Testing Sheet 2 of 9

Location: Haines

Project: Ferry Terminal Improvements

Number: 68433

US_GRAIN_SIZE_MOD 2_68433_HNS FERRY TERMINAL IMPROVEMENTS.GPJ_SE_AKDOT_MOD_040504.GDT 3/11/05



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	(frost susceptible soils % finer 0.02mm)

	Sample # / Depth	Lab #	USCS Classification	%Org	% MC	SpG	LL	PL	PI
●	TH-1-10 43.5	05C-30	SILTY SAND with GRAVEL(SM)		7.7	3.0	NV	NP	NP
⧓	TH-1-12 53.5	05C-32	POORLY GRADED SAND with SILT and GRAVEL(SP-SM)		7.5	2.9	NV	NP	NP
▲	TH-1-14 63.5	05C-33	WELL-GRADED SAND with GRAVEL(SW)		6.6		NV	NP	NP
★	TH-1-15 68.5	05C-34	WELL-GRADED SAND with SILT and GRAVEL(SW-SM)		8.7		NV	NP	NP
⊙	TH-1-17 80.5	05C-35	SILTY CLAY with SAND(CL-ML)		26.1	2.8	28	21	7

	Sample # / Depth	Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay	Frost Susceptible Soils (% finer 0.02mm)
●	TH-1-10 43.5			38	3.377	0.281		34.3	42.6	23.1		
⧓	TH-1-12 53.5	0.49	27.23	25.4	3.64	0.487	0.134	32.1	61.4	6.5		
▲	TH-1-14 63.5	2.49	19.72	19	3.951	1.403	0.2	33.6	61.6	4.8		
★	TH-1-15 68.5	1.74	31.95	19	3.066	0.715	0.096	28.9	63.7	7.4		
⊙	TH-1-17 80.5			25.4	0.012			6.1	12.3	36.7	45.0	68.5

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SUBSURFACE DATA: Results of Laboratory Testing Sheet 3 of 9

Location: Haines

Project: Ferry Terminal Improvements

Number: 68433

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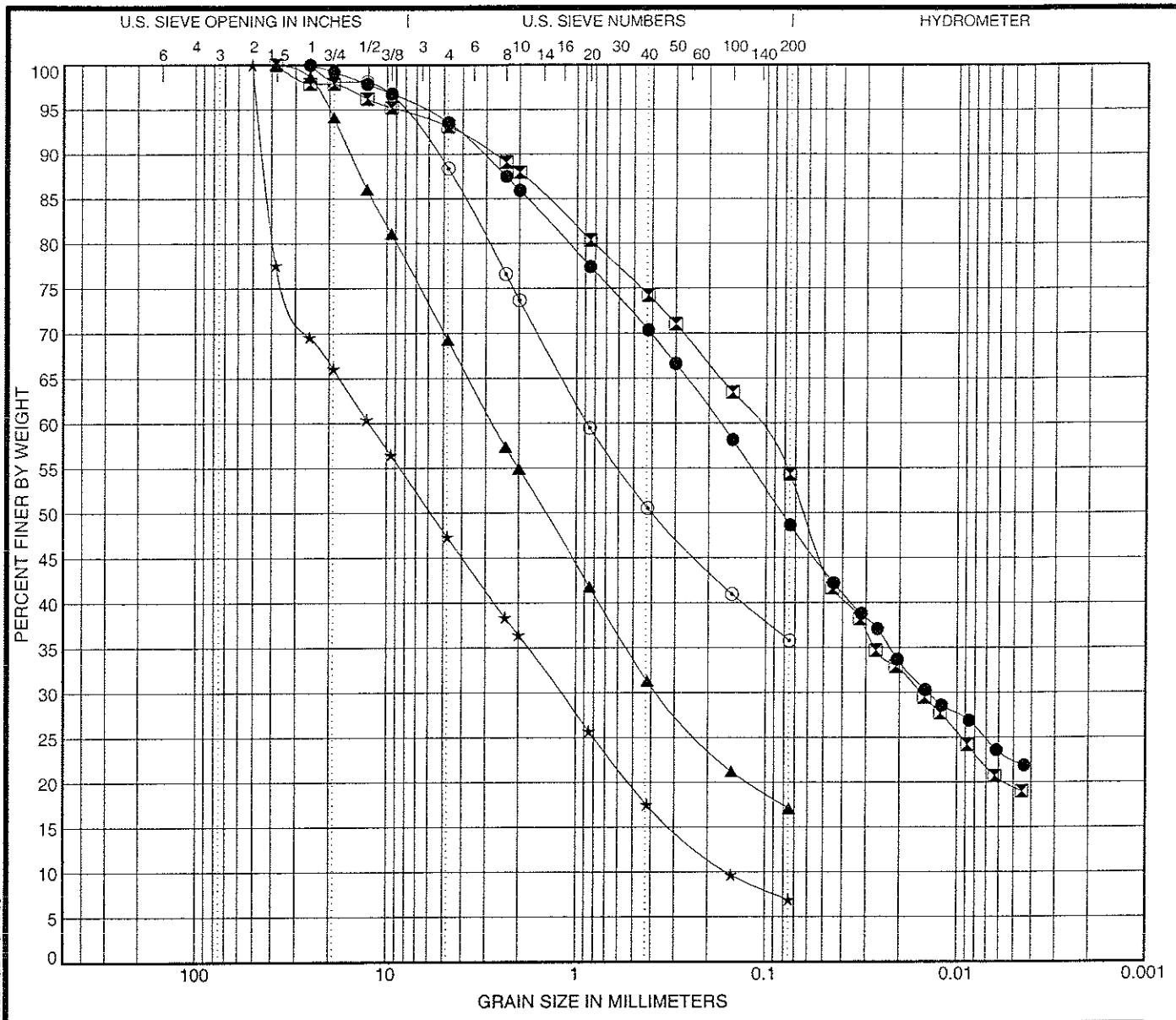

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	(frost susceptible soils % finer 0.02mm)

	Sample # / Depth		Lab #	USCS Classification	%Org	% MC	SpG	LL	PL	PI
●	TH-1-18	88.0	05C-36	SILTY, CLAYEY SAND(SC-SM)		11.7	2.7	20	13	7
⊠	TH-1-19	98.5	05C-37	SANDY SILT(ML)		13.2	2.7	17	NP	NP
▲	TH-2-1	0.5	05C-38	SILTY SAND with GRAVEL(SM)		5.4		NV	NP	NP
★	TH-2-2	2.5	05C-39	POORLY GRADED GRAVEL with SILT and SAND(GP-GM)		4.2		NV	NP	NP
⊙	TH-2-3	8.5	05C-40	SILTY SAND(SM)		7.1		NV	NP	NP

	Sample # / Depth		Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay	Frost Susceptible Soils (% finer 0.02mm)
●	TH-1-18	88.0			25.4	0.174	0.014		6.5	44.9	26.1	22.5	33.3
⊠	TH-1-19	98.5			38	0.115	0.016		6.9	38.8	34.8	19.5	32.4
▲	TH-2-1	0.5			38	2.753	0.367		30.7	52.2	17.1		
★	TH-2-2	2.5	0.74	78.68	50.8	12.308	1.196	0.156	52.7	40.4	6.9		
⊙	TH-2-3	8.5			25.4	0.876			11.6	52.6	35.8		

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SUBSURFACE DATA: Results of Laboratory Testing Sheet 4 of 9

Location: Haines

Project: Ferry Terminal Improvements

Number: 68433

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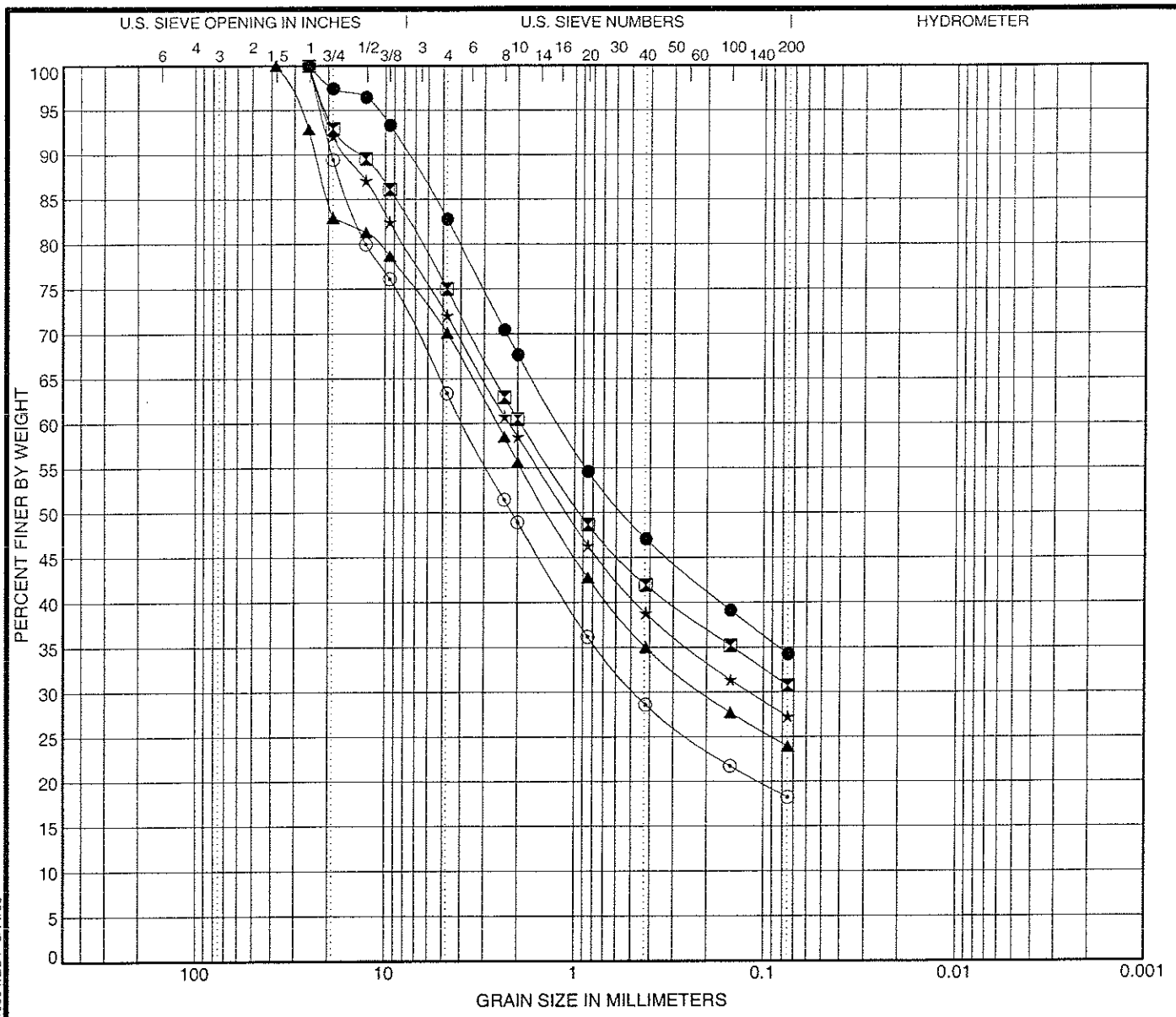

COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	(frost susceptible soils % finer 0.02mm)

	Sample # / Depth		Lab #	USCS Classification	%Org	% MC	SpG	LL	PL	PI
●	TH-2-4	13.5	05C-41	SILTY SAND with GRAVEL(SM)		10.0		NV	NP	NP
⊠	TH-2-5	18.5	05C-42	SILTY SAND with GRAVEL(SM)		9.7		NV	NP	NP
▲	TH-2-6	23.5	05C-43	SILTY SAND with GRAVEL(SM)		8.6		NV	NP	NP
★	TH-2-7	28.5	05C-44	SILTY SAND with GRAVEL(SM)		9.4		NV	NP	NP
⊙	TH-2-8	33.5	05C-45	SILTY SAND with GRAVEL(SM)		8.8		NV	NP	NP

	Sample # / Depth		Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay	Frost Susceptible Soils (% finer 0.02mm)
●	TH-2-4	13.5			25.4	1.209			17.2	48.5	34.2		
⊠	TH-2-5	18.5			25.4	1.931			25.0	44.3	30.7		
▲	TH-2-6	23.5			38	2.583	0.207		29.9	46.2	23.9		
★	TH-2-7	28.5			25.4	2.229	0.119		28.0	44.8	27.3		
⊙	TH-2-8	33.5			25.4	3.897	0.478		36.6	45.1	18.2		

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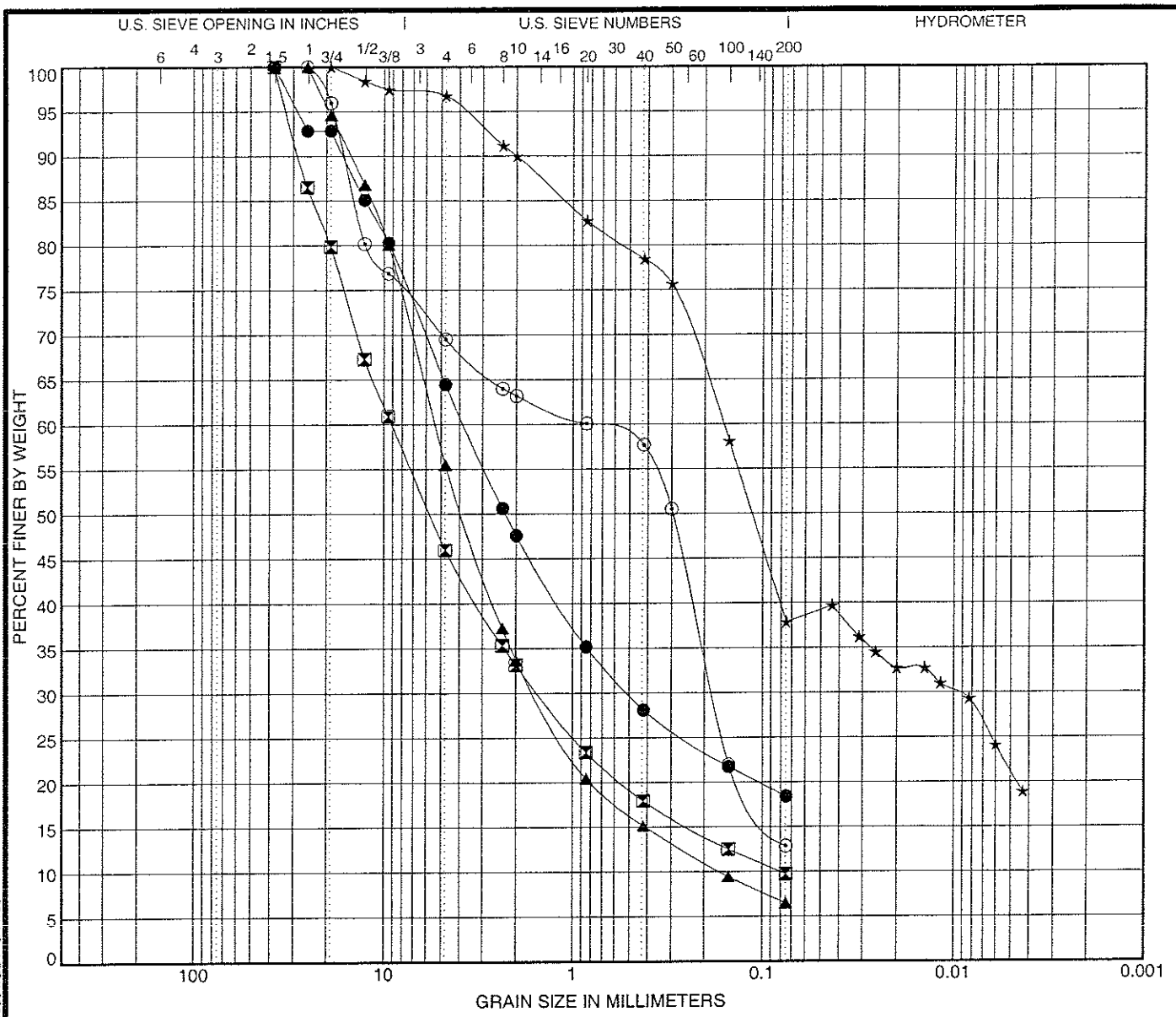
SUBSURFACE DATA: Results of Laboratory Testing Sheet 5 of 9

Location: Haines

Project: Ferry Terminal Improvements

Number: 68433

US_GRAIN_SIZE_MOD 2 68433_HNS FERRY TERMINAL IMPROVEMENTS.GPJ SE_AKDOT_MOD_040504.GDT 3/11/05



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	(frost susceptible soils % finer 0.02mm)

	Sample # / Depth	Lab #	USCS Classification	%Org	% MC	SpG	LL	PL	PI
●	TH-2-9 38.5	05C-46	SILTY SAND with GRAVEL(SM)		8.4		NV	NP	NP
⊠	TH-2-10 43.5	05C-47	POORLY GRADED GRAVEL with SILT and SAND(GP-GM)		6.4		NV	NP	NP
▲	TH-2-11a 48.5	05C-48	WELL-GRADED SAND with SILT and GRAVEL(SW-SM)		9.2		NV	NP	NP
★	TH-2-11b 49.5	05C-49	SILTY SAND(SM)		27.0	2.8	NV	NP	NP
⊙	TH-2-12 53.5	05C-50	SILTY SAND with GRAVEL(SM)		12.8		NV	NP	NP

	Sample # / Depth	Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay	Frost Susceptible Soils (% finer 0.02mm)
●	TH-2-9 38.5			38	3.788	0.507		35.5	46.1	18.4		
⊠	TH-2-10 43.5	3.16	113.66	38	9.105	1.517	0.08	54.0	36.3	9.7		
▲	TH-2-11a 48.5	2.77	32.50	25.4	5.395	1.575	0.166	44.5	49.0	6.5		
★	TH-2-11b 49.5			19	0.162	0.01		3.2	58.9	16.6	21.3	32.7
⊙	TH-2-12 53.5			25.4	0.827	0.182		30.5	56.7	12.9		

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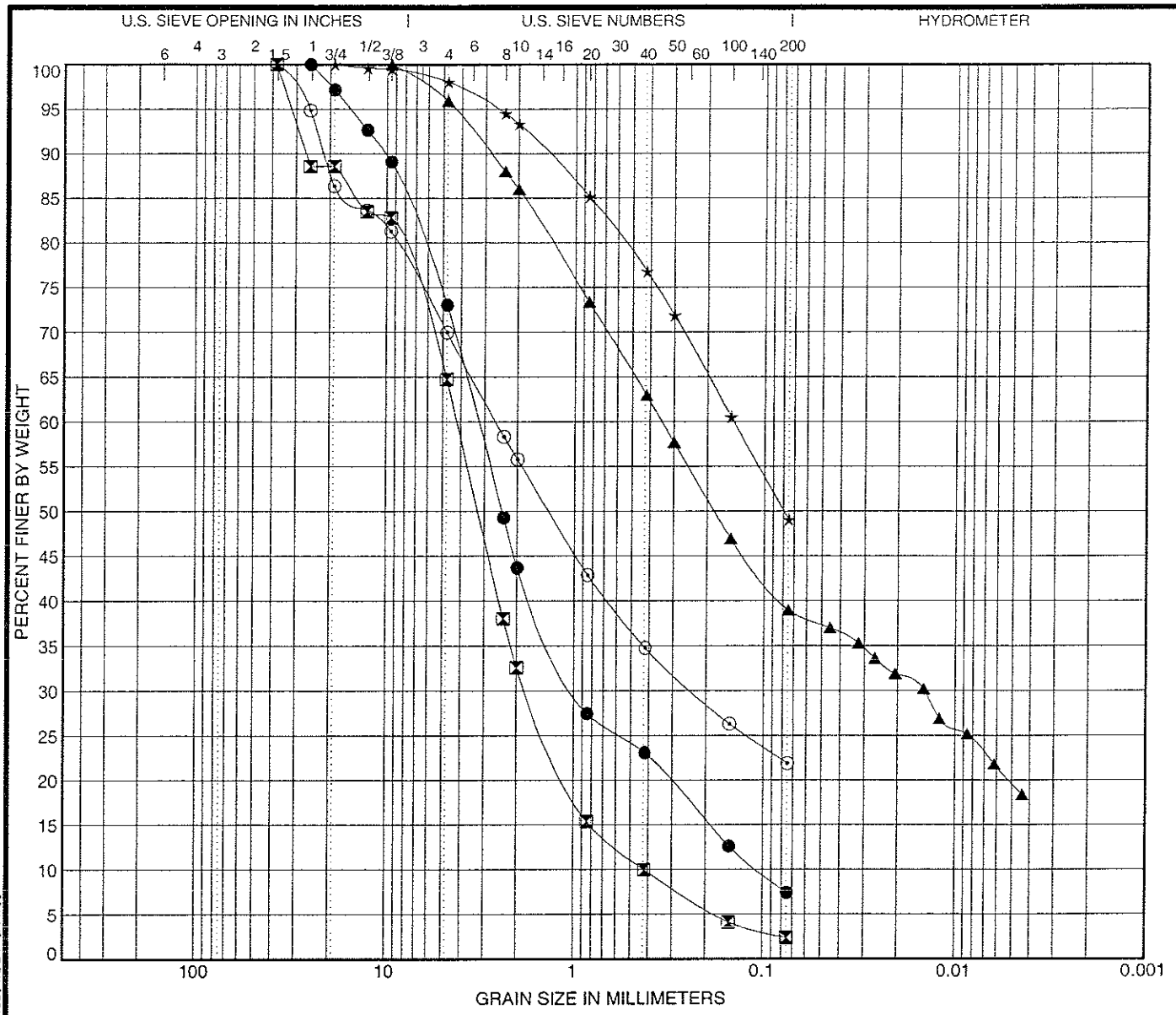
SUBSURFACE DATA: Results of Laboratory Testing Sheet 6 of 9

Location: Haines

Project: Ferry Terminal Improvements

Number: 68433

US_GRAIN_SIZE_MOD 2_68433_HNS FERRY TERMINAL IMPROVEMENTS.GPJ_SE_AKDOT_MOD_040504.GDT 3/11/05



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	(frost susceptible soils % finer 0.02mm)

	Sample # / Depth	Lab #	USCS Classification	%Org	% MC	SpG	LL	PL	PI
●	TH-2-13 58.5	05C-51	WELL-GRADED SAND with SILT and GRAVEL(SW-SM)		9.8		NV	NP	NP
⊠	TH-2-14 68.5	05C-52	WELL-GRADED SAND with GRAVEL(SW)		5.5		NV	NP	NP
▲	TH-2-Cutting 4.0	05C-53	SILTY SAND(SM)		20.6	2.7	17	15	2
★	TH-2-16 88.5	05C-54	SILTY SAND(SM)		9.1	2.7	14	NP	NP
⊙	TH-3-3 4.0	05C-55	SILTY SAND with GRAVEL(SM)		8.6		NV	NP	NP

	Sample # / Depth	Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay	Frost Susceptible Soils (% finer 0.02mm)
●	TH-2-13 58.5	2.75	30.54	25.4	3.235	0.971	0.106	27.0	65.6	7.4		
⊠	TH-2-14 68.5	1.75	9.95	38	4.196	1.759	0.422	35.3	62.4	2.3		
▲	TH-2-Cutting 4.0			9.5	0.348	0.015		4.1	56.9	19.3	19.7	31.7
★	TH-2-16 88.5			19	0.145			1.9	49.0	49.1		
⊙	TH-3-3 4.0			38	2.608	0.235		30.0	48.1	21.9		

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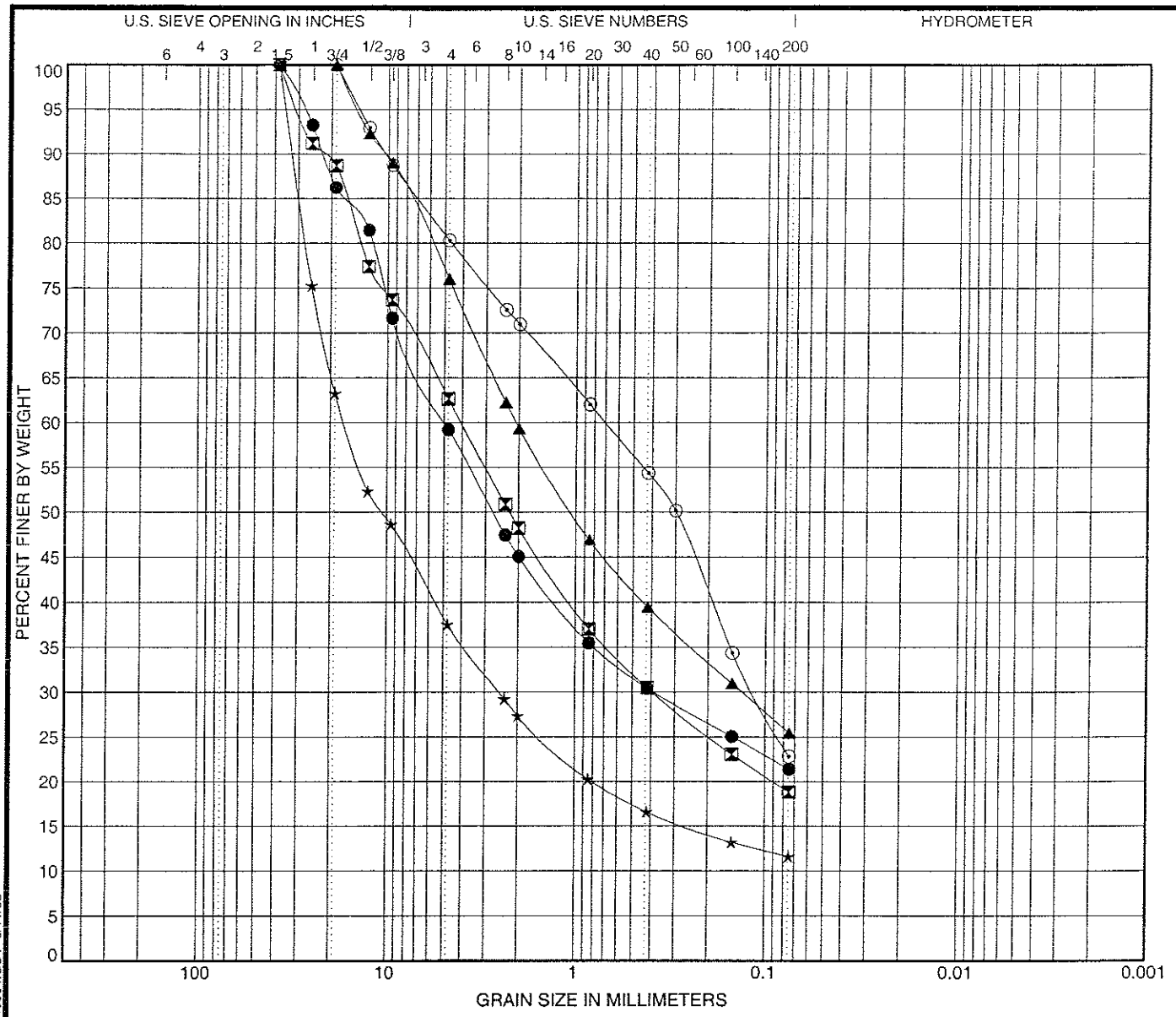
SUBSURFACE DATA: Results of Laboratory Testing Sheet 7 of 9

Location: Haines

Project: Ferry Terminal Improvements

Number: 68433

US_GRAIN_SIZE_MOD 2_68433_HNS FERRY TERMINAL IMPROVEMENTS.GPJ SE_AKDOT_MOD_040504.GDT 3/11/05



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	(frost susceptible soils % finer 0.02mm)

	Sample # / Depth		Lab #	USCS Classification	%Org	% MC	SpG	LL	PL	PI
●	TH-3-4	9.0	05C-56	SILTY GRAVEL with SAND(GM)		7.8		NV	NP	NP
⊠	TH-3-5	14.0	05C-57	SILTY SAND with GRAVEL(SM)		7.8		NV	NP	NP
▲	TH-3-6	19.0	05C-58	SILTY SAND with GRAVEL(SM)		9.0		NV	NP	NP
★	TH-3-7	24.0	05C-59	POORLY GRADED GRAVEL with SILT and SAND(GP-GM)		5.7		NV	NP	NP
⊙	TH-3-8	29.0	05C-60	SILTY SAND with GRAVEL(SM)		15.1		NV	NP	NP

	Sample # / Depth		Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay	Frost Susceptible Soils (% finer 0.02mm)
●	TH-3-4	9.0			38	4.962	0.385		40.8	37.8	21.4		
⊠	TH-3-5	14.0			38	4.056	0.393		37.3	43.8	18.8		
▲	TH-3-6	19.0			19	2.076	0.133		24.0	50.6	25.4		
★	TH-3-7	24.0	10.16	455.76	38	16.84	2.515		62.5	25.9	11.6		
⊙	TH-3-8	29.0			19	0.705	0.115		19.7	57.5	22.8		

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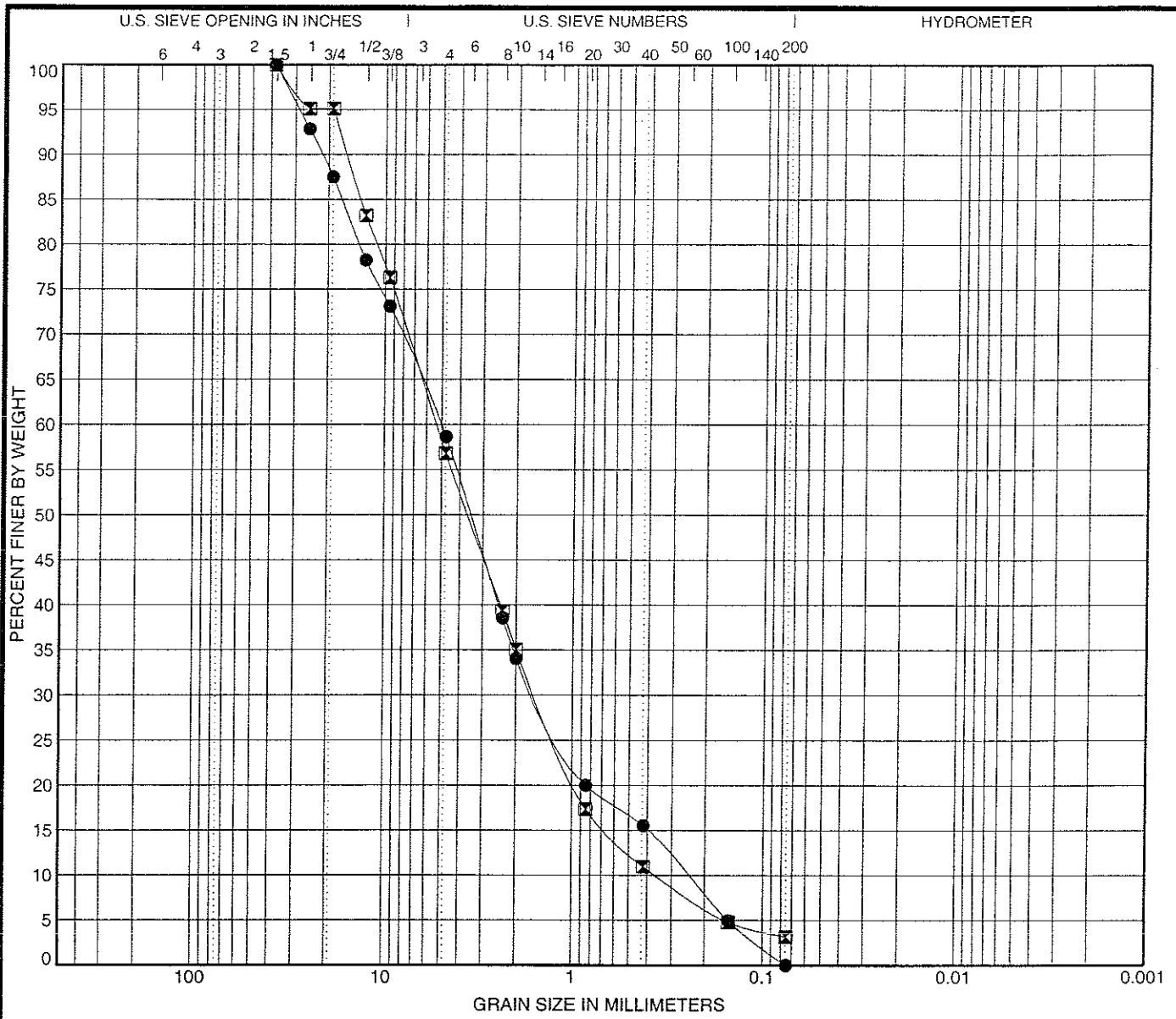
SUBSURFACE DATA: Results of Laboratory Testing Sheet 8 of 9

Location: Haines

Project: Ferry Terminal Improvements

Number: 68433

US_GRAIN_SIZE_MOD 2_68433_HNS_FERRY TERMINAL IMPROVEMENTS.GPJ_SE_AKDOT_MOD_040504.GDT_3/11/05



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	(frost susceptible soils % finer 0.02mm)

	Sample # / Depth		Lab #	USCS Classification	%Org	% MC	SpG	LL	PL	PI
●	TH-3-9	34.0	05C-61	WELL-GRADED SAND with GRAVEL(SW)		6.8		NV	NP	NP
⊠	TH-3-10	39.0	05C-62	WELL-GRADED SAND with GRAVEL(SW)		8.8		NV	NP	NP

	Sample # / Depth		Cc	Cu	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay	Frost Susceptible Soils (% finer 0.02mm)
●	TH-3-9	34.0	1.96	20.68	38	5.068	1.561	0.245	41.4	58.6	0.0		
⊠	TH-3-10	39.0	1.29	14.90	38	5.326	1.566	0.358	43.2	53.6	3.2		

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SUBSURFACE DATA: Results of Laboratory Testing Sheet 9 of 9

Location: Haines

Project: Ferry Terminal Improvements

Number: 68433

US_GRAIN_SIZE_MOD 2_68433_HNS FERRY TERMINAL IMPROVEMENTS.GPJ_SE_AKDOT_MOD_040504.GDT_3/11/05

REF #4

Echelon Engineering Report – Inspection and Assessment of Lutak Dock, Haines, AK. Prepared May 2014.

REF #5

Echelon Engineering Report – Underwater Inspection Lutak Dock, Haines, AK. Dated April 2002.

ECHELON ENGINEERING, INC.

Civil/Marine Consulting Engineers

INSPECTION REPORT

FOR

LUTAK DOCK

Haines, Alaska

Prepared For:

Reid Middleton, Inc.
4300 B Street, Suite 403
Anchorage, Alaska 99503

ATTN: Mr. Craig Freas, PE
Project Manager
Tel: (907) 562-3439

Prepared By:

Echelon Engineering, Inc.
3837 13th Avenue West, Suite 205
Seattle, Washington 98119

ATTN: Ms. Shelley Sommerfeld, P.E.
President
Tel: (206) 286-6699

April 2002
02-2163, Rev 1

ECHELON ENGINEERING, INC.
Civil/Marine Consulting Engineers

April 22, 2002

Reid Middleton, Inc.
4300 B Street, Suite 403
Anchorage, Alaska 99503

ATTN: Mr. Craig Freas
Project Manager

RE: FINAL REPORT – Underwater Inspection Lutak Dock, Haines, Alaska

Dear Mr. Freas:

This letter is submitted to document the findings of our recent inspection of the City owned portion of the Lutak Dock, located on Lutak Inlet, Haines Alaska. The project was carried out in support of your ongoing evaluation and rehabilitation project for the structure.

INSPECTION METHODOLOGY

The project included a sample inspection of the steel sheet pile cells that comprise the structure. Specifically, the scope of the investigation included a Level I visual swim-by inspection of the submerged portion of the 10 Main Cells located at the northwestern end of the wharf. Additionally detailed Level II cleaning and Level III ultrasonic thickness measurements were obtained at representative test sites on 3 of the 10 Main Cells. On each of these 3 Main Cells, two piling were cleaned and tested at representative intervals from the concrete cap to the mudline.

Due to the dense marine growth and the silty glacial sediments that were contained within the marine fouling, Level II cleaning of the 6 sample piling was carried out prior to the actual inspection. During the cleaning operations, the underwater visibility reduced to near 0 ft. as the glacial sediments contained within the animals and/or their attachment matrix was released into the surrounding water. As shown by the accompanying photographs, cleaning prior to inspection allowed for much greater visibility of the cleaned piling.

The Main Cells selected for detailed inspection and ultrasonic testing were Cells 3, 6 and 10. Two separate locations were selected on each of the these cells; one at or near the outer radius of the cell closest to the berth; and the second near the southeastern intersection with the adjacent closure arc. These intermediate sites are identified as locations 3.5, 6.5 and 10.5. Along each of the 6 test piles, specific sites were selected for detailed ultrasonic measurements. Due to the scale and pitting present at most of the test sites, further surface preparation was required in order to obtain surfaces smooth enough to facilitate ultrasonic readings. This preparation was accomplished using a pneumatic grinder which proved effective in removal of surface irregularities such that consistent and repeatable thickness measurements could be

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obtained both above and below water. Test sites were selected at various elevations ranging from the concrete cap to the mudline and dependant upon specific conditions and accessibility.

Thickness readings were obtained utilizing a Cygnus Ultrasonic Thickness gauge, which through it's multiple echo design will display only those measured thickness which have consistent repeatable readings (3 echoes), thus ensuring that only accurate actual measured values are recorded. In addition to the wall thickness measurements, localized pit depth measurements at several sites were also obtained utilizing a Thorpe Pit Depth Gauge.

QUALIFICATIONS OF INSPECTORS

The investigation was conducted by a crew composed of a combination of professional and technical personnel capable and experienced in both the underwater and topside inspection and assessment of structural members. The inspection crew utilized on this project included the following Echelon Engineering personnel:

S.D. Sommerfeld, P.E.	Project Manager/Engineer - Diver Licensed Professional Engineer, WA, Guam 17 Yrs Experience Specializing in Marine Structures Inspection & Design
E.B. Vegsund, B.Sc.	Marine Specialist/Biologist - Diver BS in Marine Biology - Emphasis on Marine Borer Studies 29 Yrs Experience Specializing in Marine Structures Inspection
J. Svornich	Inspection Technician/Commercial Diver 25 Yr Experience Specializing in Marine Structures Inspection
I. Thomas	Inspection Technician/Commercial Diver 1 Yr Experience Specializing in Marine Structures Inspection

OBSERVED CONDITIONS

The field investigation was carried out during the interval of April 3 – 7, 2002. Weather during this period was clear and cold with strong winds and seas ranging from calm to small craft conditions and wave heights of up to 2 ft. Tidal elevations during the daylight inspection periods were found to range from a low of + 1.9 ft. (MLLW) to a high of +13.5 ft. (MLLW). Underwater visibility ranged from near 0 ft. during the cleaning and site preparation work to approximately 10 ft. during inspection activities.

To facilitate the identification of the test site locations, the Main Cells have been numbered consecutively 1 – 10 from the northwest end. Test site locations have been referenced to the Cell number and to the site at which the readings were taken. Test sites were numbered sequentially from the top of the pile to the mudline.

Representative photographs depicting typical conditions encountered are presented in Appendix A. A drawing identifying the identification system used is presented in Appendix B. Pile thickness measurements as obtained using an Ultrasonic Thickness Gauge, are presented in Appendix C, Table 1. Table 2 of Appendix C presents the data by grouping it into ranges of elevation or exposure zones along the pile, (i.e. Splash Zone ~El. +19.25 ft. to +15.0 ft.; Intertidal Zone ~El. +15.0 ft. to + 0.0 ft.;

Submerged Zone ~El. +0.0 ft to mudline. Table 3 presents a percentile ranking of the 48 individual test site locations at which ultrasonic thickness readings were obtained.

The findings of the inspection are as follows:

Steel Sheet piling

1. The overall condition of the steel sheet piles within the Main Cells is fair. No evidence of perforation of the piling was noted and all of the closure knuckles connecting the individual sheets were observed to be intact with no evidence of separation.
2. A variable covering of marine fouling organisms which consisted primarily of blue muscles and sea urchins was found to be typical on the exposed face of the piling. In some areas the fouling was so dense as to preclude visual assessment of the wall integrity. In other areas the amount of marine fouling was minimal exposing the steel piling and allowing for visual inspection of the member.
3. Throughout the inspected length of the wharf from Cell 1 – 10 generalized surface corrosion was found from the intertidal zone to the mudline.
4. One pile was noted to have a number of drilled holes ~3-4 inches in diameter and spaced 2-3 ft. apart. These holes were found to extend along the submerged length of the pile.
5. A sampling of six (6) piles were subjected to Level III ultrasonic thickness testing. All of the selected test sites were found to be in fair to good condition based on visual inspection. The individual piles were cleaned for the majority of their length from the intertidal zone to the mudline. Cleaning was conducted across the width of the pile including both closure knuckles. Further scraping, wire brushing and grinding was conducted at the individual test sites to facilitate the ultrasonic thickness readings. Throughout the inspected cells, no perforations of the piles was observed.
6. Thickness readings obtained on the sample piling ranged from a maximum thickness of 0.453 in. to a minimum of 0.147 inches. Analysis shows the mean of the readings to be 0.355 in. with a standard deviation of 0.073 inches.
7. Heavy pitting was found at virtually all test sites. Pit depth measurements were found to range from 0.04 in. to 0.11 inches.

General Observations

1. Although not included in the scope of this investigation, the overall condition of the closure arc piling was found to be poor. Extensive corrosion was observed within the intertidal zone resulting in numerous perforation of the individual piles. Cursory inspection of the submerged and mudline zones of these piling showed them to be in better condition than that found in the intertidal zone. Below the waterline, the piles were found to be in similar condition to that encountered on the Main Cell piling with evidence of surface corrosion. Additionally, the connection knuckles were noted to be undamaged.
2. The bottom along the length of Main Cells 1 – 10 was found to slope gradually downward from ~El. –13 ft. to ~El. –22 ft. Glacial sediments were noted predominantly from Cell 1 to Cell 9, where the composition then changed to primarily small rock (~2-4 inch diameter).

3. Cursory observation of the steel H-piling located along the berth noted areas of severe corrosion and deterioration from the lower intertidal zone to the mudline. A number of these piling were also noted to be loose at their top.

EVALUATION

In summary, this sample inspection found general surface corrosion of the Main Cell sheet piles, as well as thinning of the pile sections. Although no perforation of the sheet piles was noted, heavy marine fouling may have provided cover preventing detection of damaged areas. Based on the thickness readings obtained, review of the nonparametric percentiles and the normal probability curve (Figure 1 and 2) indicate that 85% of the readings are above ¼ inch thickness (0.250 in.) and that 50% of the readings are above $^3/_8$ inch thickness (0.375 in.). Conversely, 15% of the observations show a remaining thickness below ¼ inch (0.250 in.) and 50% show a minimum thickness below $^3/_8$ inch (0.375 in.).

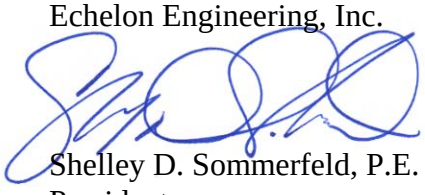
Typically, there can be an elevation effect on corrosion with the higher corrosion rates evident in the splash zone just above MHW and in the submerged zone just below MLW. The inspection data do not demonstrate pronounced accelerated corrosion at any particular elevation (Figure 3). Based on the data, the mean thickness within the range of El. 0 to El. –5 ft was found to be 0.295 inches (Figure 4). This is approximately 20% below the median remaining thickness of 0.375 inches. However, the statistic is derived from a limited number of measurements with a large variation.

Corrosion of the steel sheetpile cells appears to be fairly uniform with a median remaining wall thickness of $^3/_8$ inch and 85% of the observations above ¼ inch thickness. No perforations or severe localized corrosion of the steel piles was noted during the investigation.

In contrast to the main cells, the condition of the steel sheet piles in the closure arcs appear subject to accelerated non-uniform corrosion typical of corrosion caused by differential conditions or differential metallurgy. Alternatively, assuming that the sheet piling in the Closure Arcs was originally thinner than that of the piling in the Main Cells, the greater deterioration of these piling may be the apparent failure at the end of the corrosion allowance. Given the corrosion loss of the main Cell piling to date, this would demonstrate the need for corrosion control to protect the remaining section.

We sincerely appreciate the opportunity to assist you with this project. Should you have any questions regarding this report, or if we can be of further assistance, please do not hesitate to contact our office.

Yours Truly,
Echelon Engineering, Inc.



Shelley D. Sommerfeld, P.E.
President

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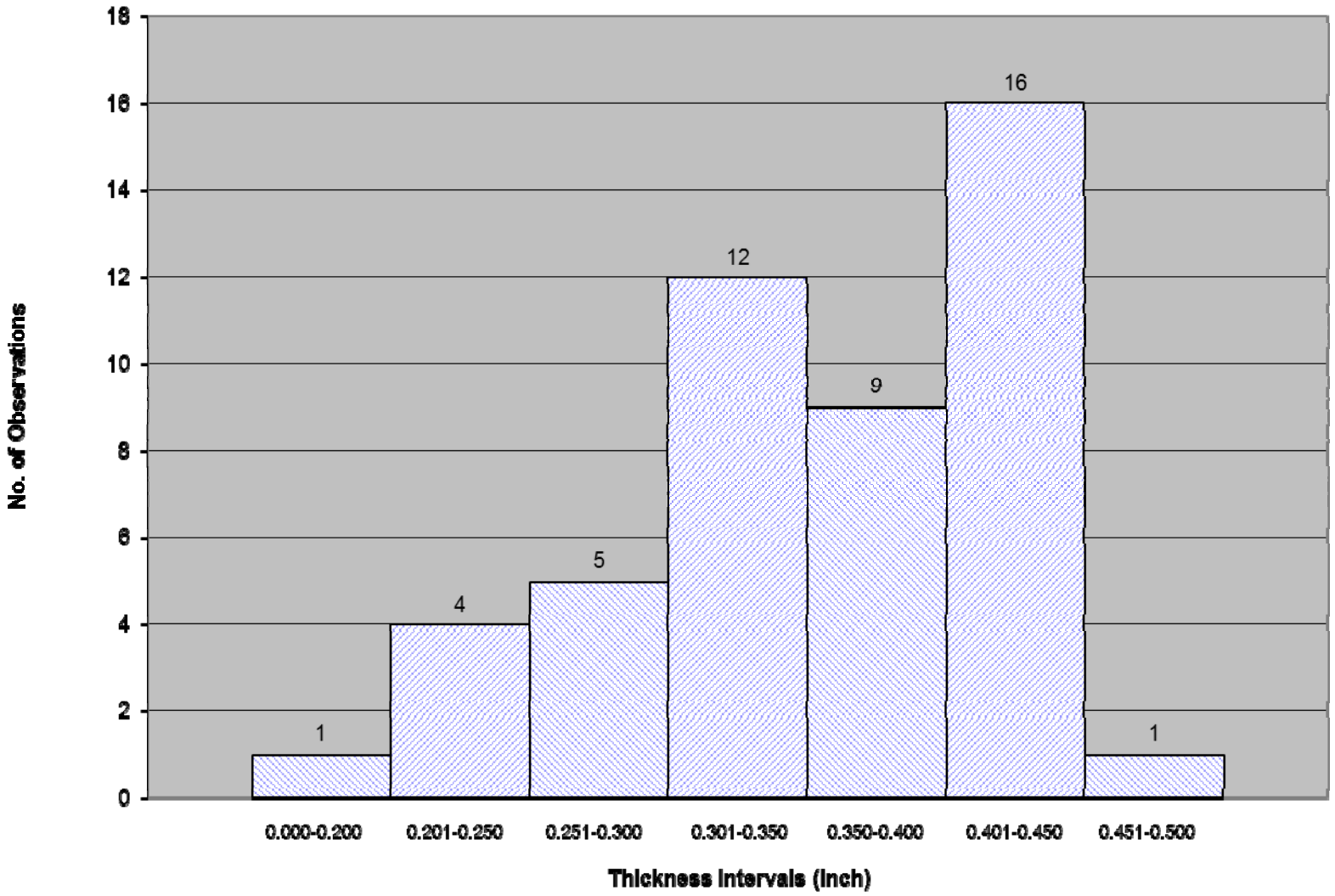
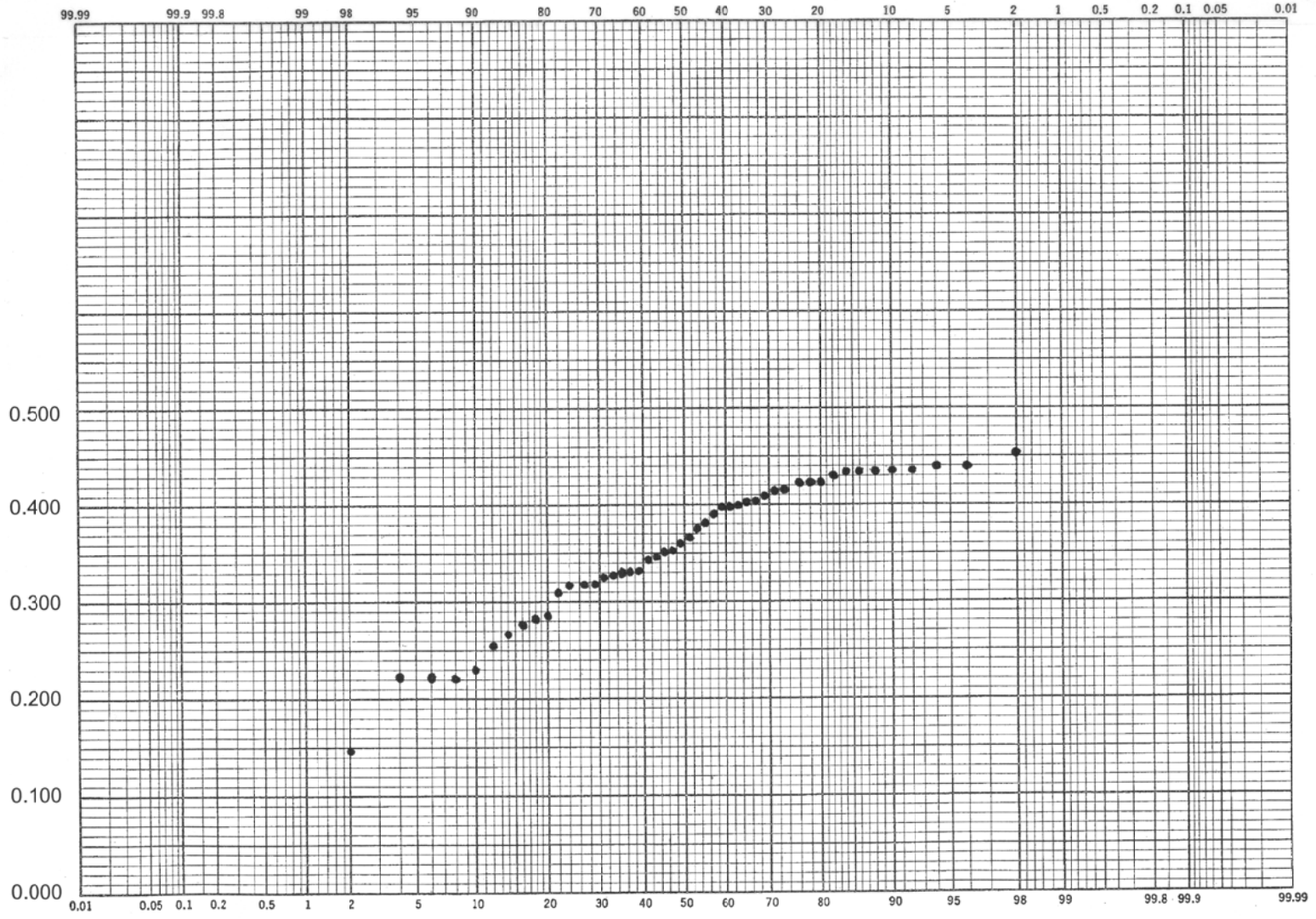


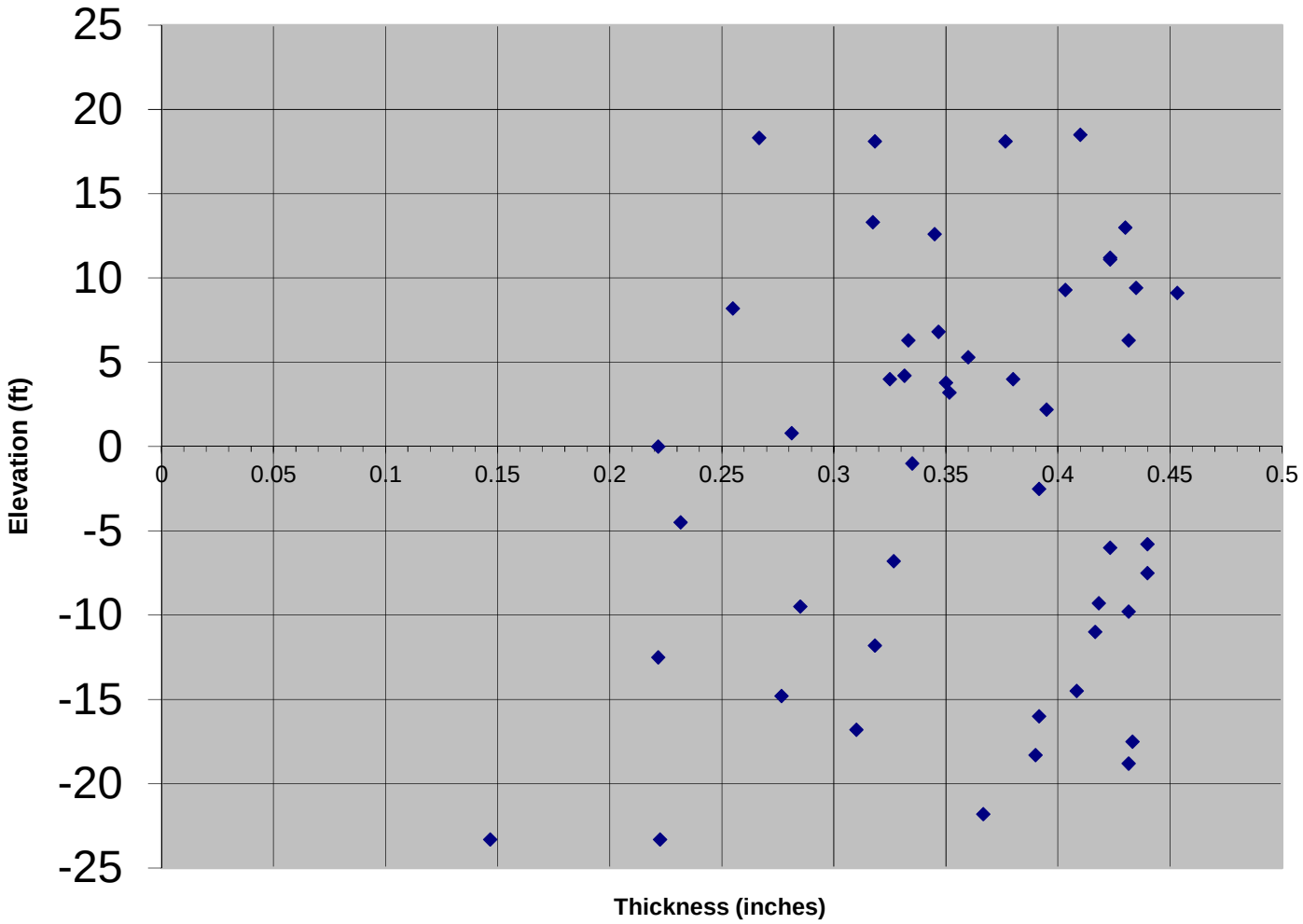
WALL THICKNESS FREQUENCY HISTOGRAM

FIGURE 1



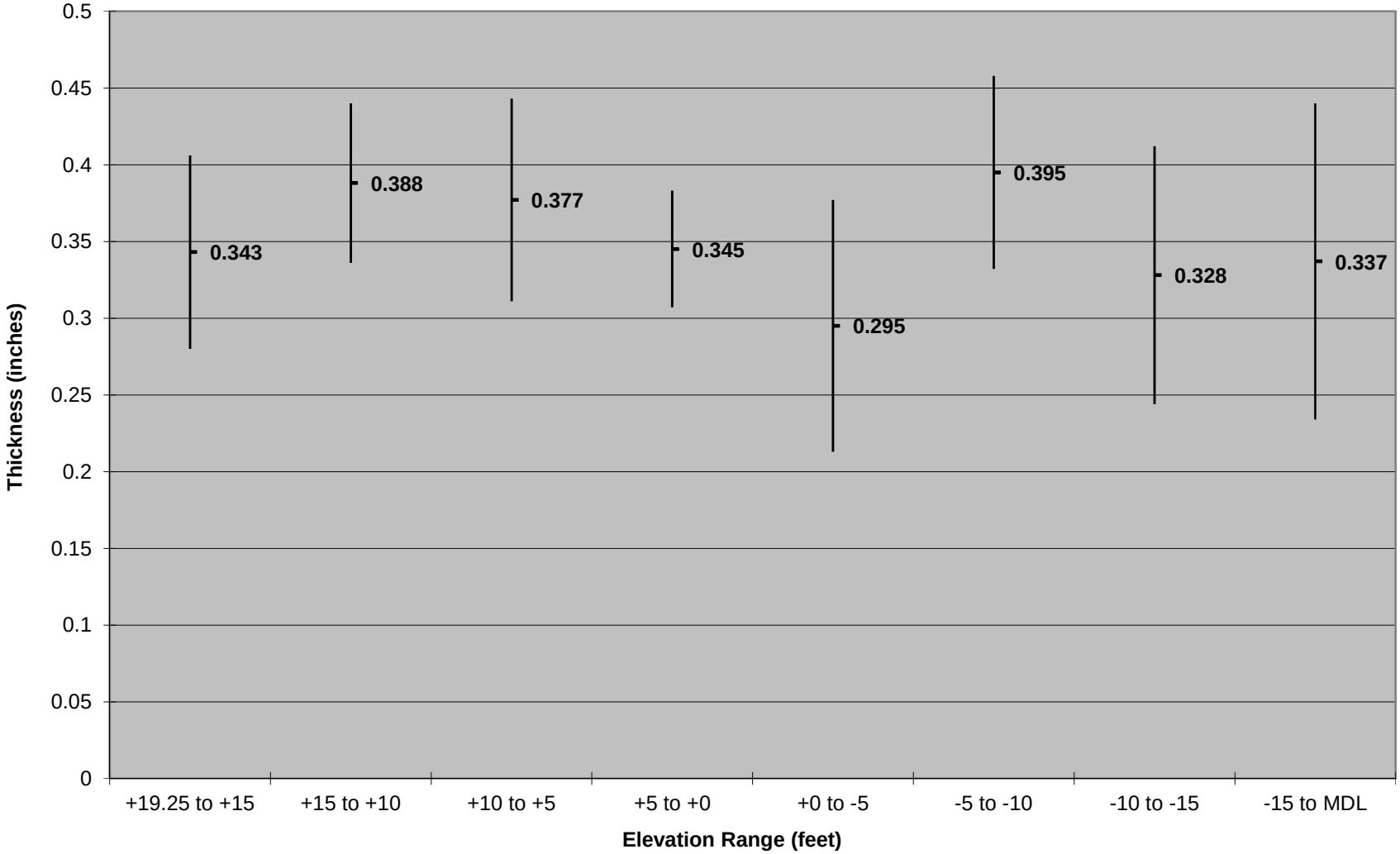
NORMAL PROBABILITY PLOT

FIGURE 2



WALL THICKNESS vs. ELEVATION

FIGURE 3



MEAN THICKNESS +/- 1 STANDARD DEVIATION vs ELEVATION

FIGURE 4

PHOTO No. 1: Lutak Dock, Cells No. 1-10, Looking NW - Note Cell No. 1 on the far right and Cell No. 10 on the left. Also note the intermittent fender system along the face of the wharf.

PHOTO No. 2: Cell No. 3, Looking NW - Note the surface corrosion and scale in the splash zone and upper intertidal zone of the piling. This was found to be typical of the conditions encountered. Also note the good condition of the upper portion of the 2.5 Closure Arc in the background.

PHOTO No. 3: Cell No. 3, Test Site 3.5-1, El. +13.3 ft. - Note the technician using a pneumatic grinding disc to prepare the test site for Level III ultrasonic thickness testing.

PHOTO No. 4: Cell No. 3, Test Site 3.5-5, El. +2.2 ft. - Note the Level II cleaning of the sheet pile and knuckles and good condition of both the pile and the closure joint knuckles. Also note the approximately 2 in. diameter localized Level III test site near the water surface.

PHOTO No. 5: Cell No. 10, Test Site 10-0, El. +18.5 ft. - Note the Level III cleaned site approximately 1 ft. below the bottom of the concrete cap. Also note the heavy scale, which was typically found in the splash zone of the inspected piles.

PHOTO No. 6: Cell No. 3, Test Site 3-1, El. +13.0 ft. - Note the Level II cleaning of the pile and the connection knuckles. Also note the test site prepared for Level III thickness measurements (bottom center).

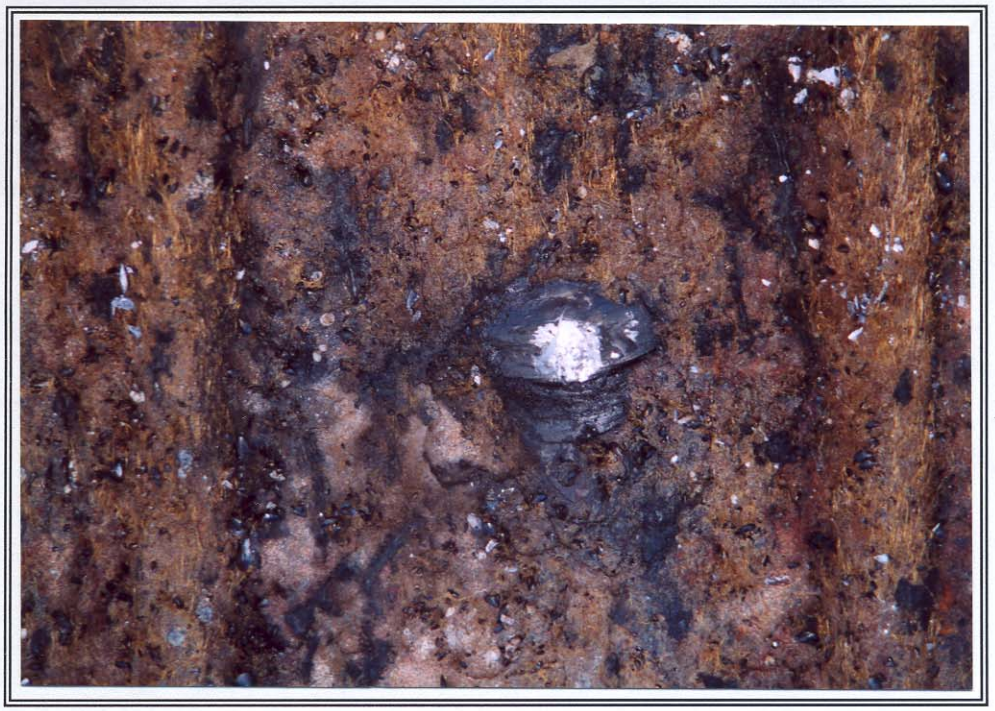


PHOTO No. 7: Cell No. 6, Test Site 6-4, El. +3.2 ft. - Note the thick layer of black iron oxide surrounding the test site. This layer was removed to facilitate ultrasonic readings.

PHOTO No. 8: Cell No. 10, Test Site 10.5-4, El. -6.8 ft. - Note the yellow crayon mark indicating the test site (right center) located on Cell No. 10. Also note the uniform surface corrosion and minimal marine fouling evident on the Closure Arc pile to the left.

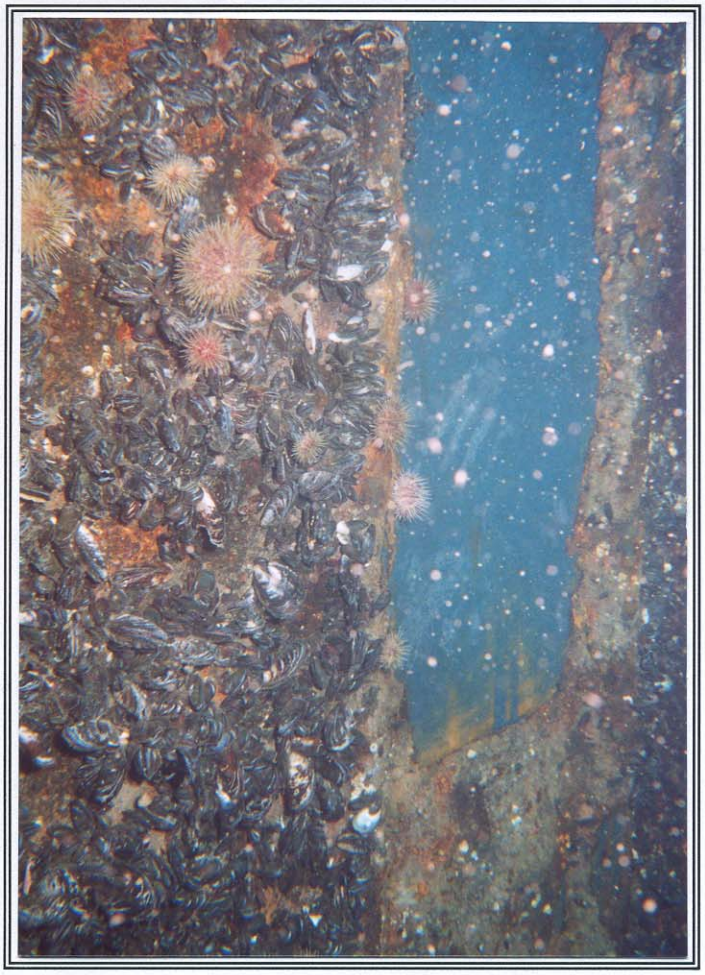


PHOTO No. 9: Cell No. 10, Test Site 10.5-5, El. -11.8 ft. - This location was found to have uniform surface corrosion. Note the Level II cleaned site prepared for ultrasonic thickness measurements. Also note the typical marine fouling of blue mussels and sea urchins surrounding the cleaned area.

PHOTO No. 10: Cell No. 10, Test Site 10.5-6, El. -16.8 ft. - Note the uniform surface corrosion on the pile and the fill material visible through a 3 in. diameter hole immediately below the test site. Also note the yellow crayon marking the location where ultrasonic thickness readings were taken at El. -16.8 ft.

PHOTO No. 11: Cell No. 10, Test Site 10-6, El. -23.3 ft. - Note the surface corrosion on the pile and the heavy pitting evident in the cleaned test site. Thickness measurements at this location ranged from 0.147 - 0.225 inches, the minimum encountered.

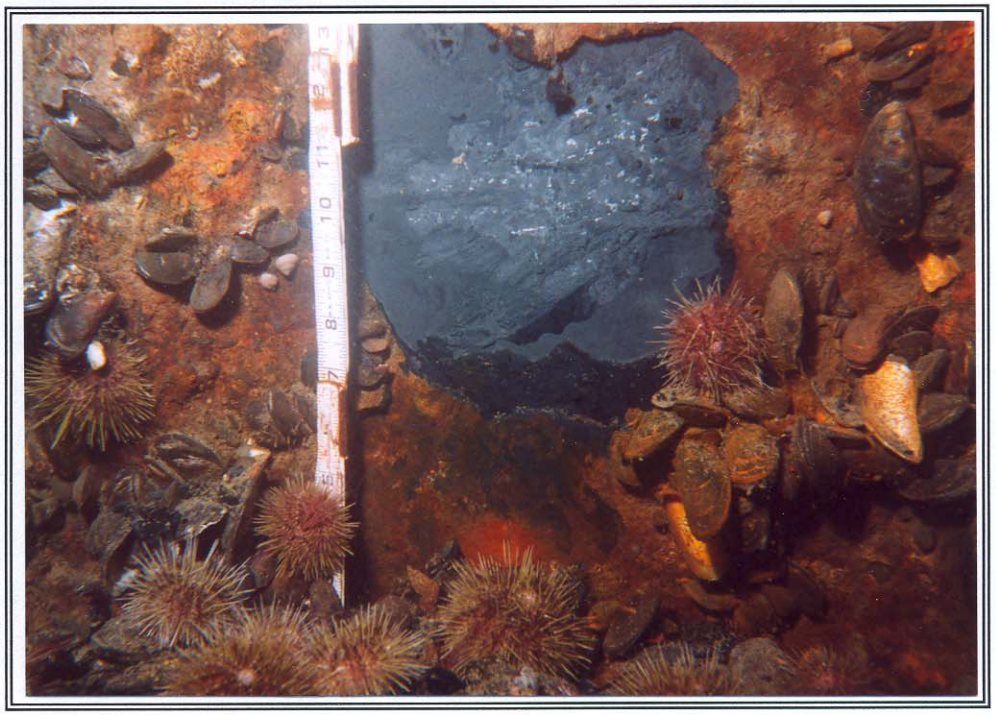


PHOTO No. 12: Cell 6, Test Site 6-9, El. -18.8 ft. - Note the surface corrosion around the Level III cleaned site. Thickness readings at this location were found to average 0.277 in.

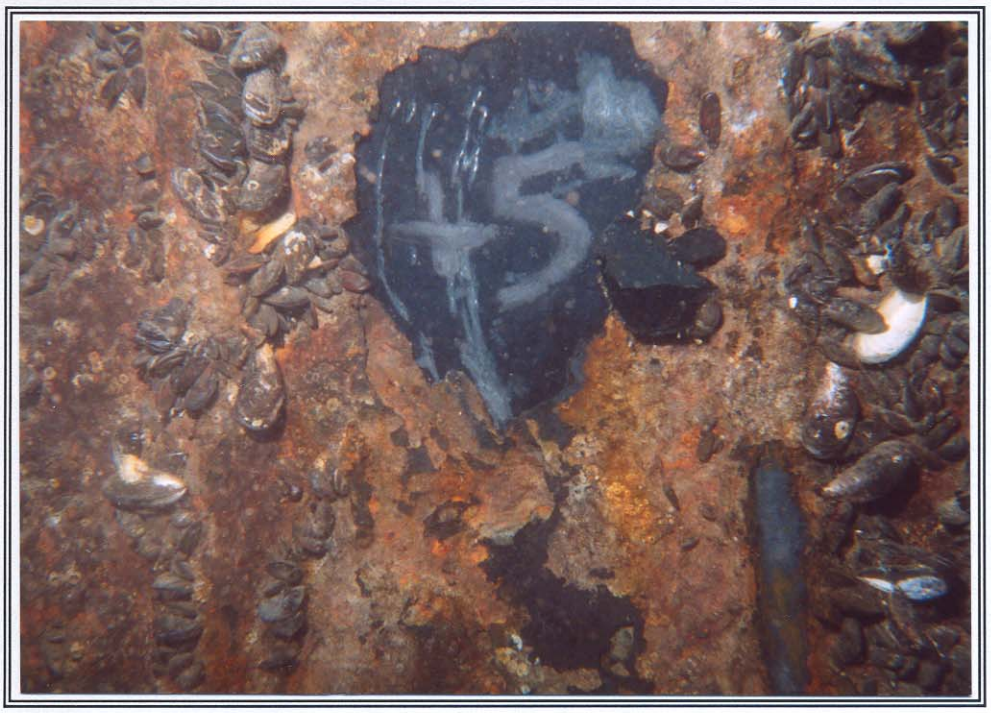


PHOTO No. 13: Cell No. 3, Test Site 6.5-7, El. -11.0 ft. - Note the smooth surface of the cleaned steel at the Level III test site. The ultrasonic thickness readings at this site averaged 0.417 in. Also note the minimal marine growth at this location.

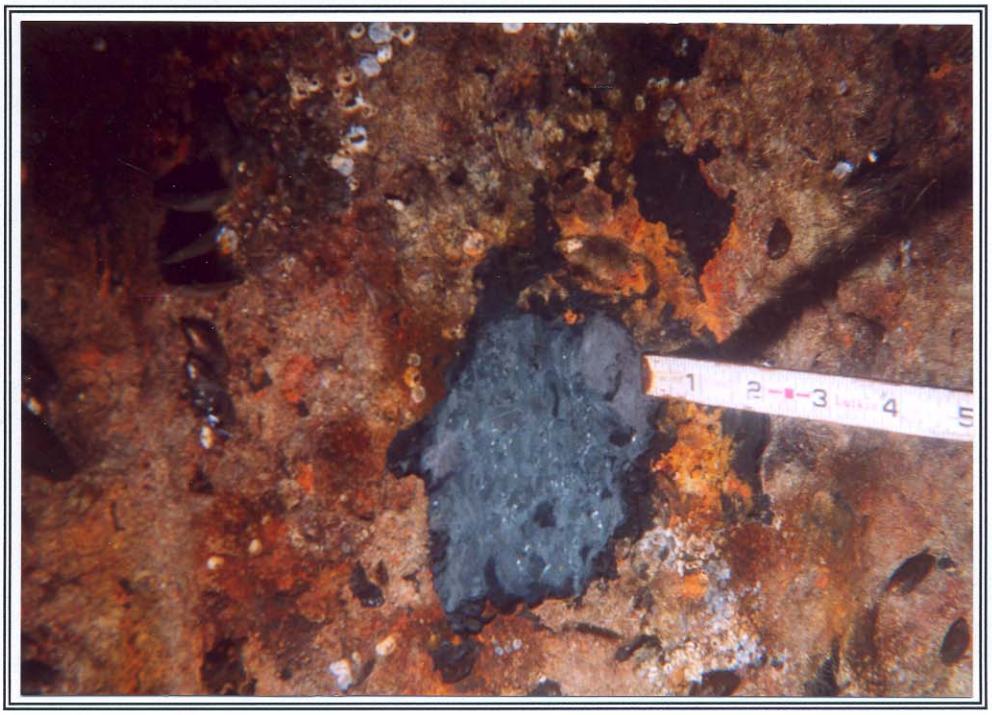


PHOTO No. 14: Cell No. 6, Test Site 6-8, El. -14.8 ft. - Note the approximate 1/4 in. build up of corrosion products at this test site. The average ultrasonic reading at this location was found to be 0.277 in.

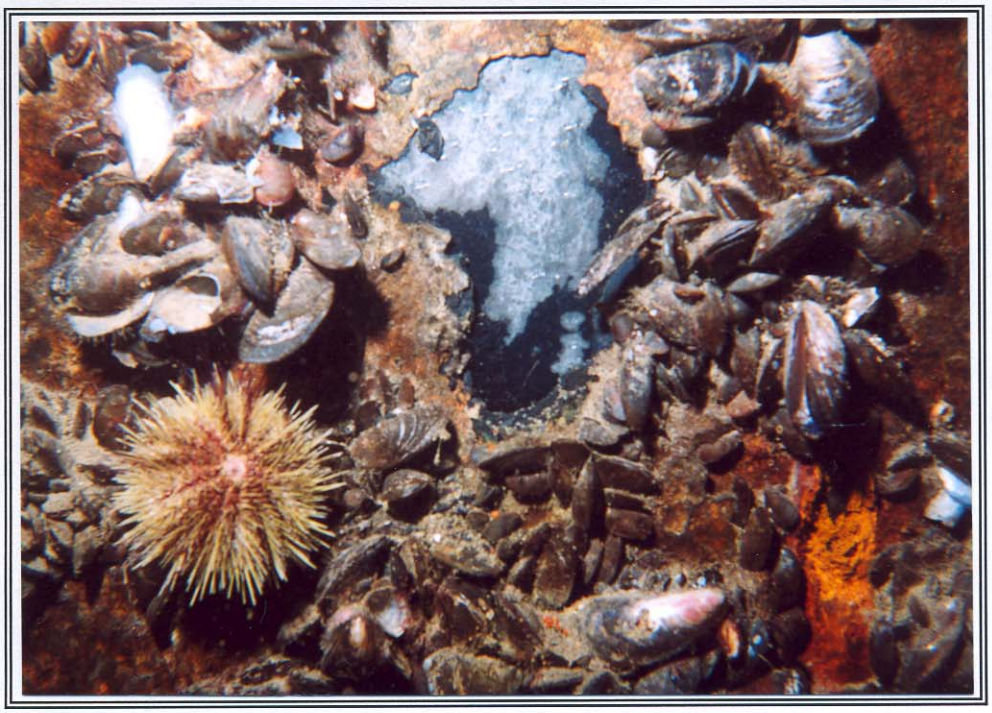
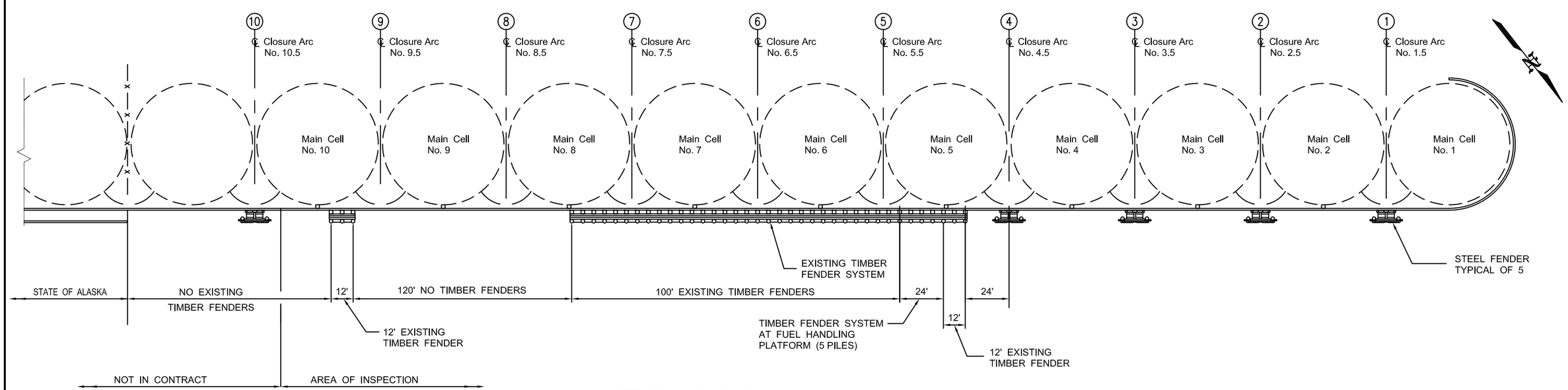


PHOTO No. 15: Cell No. 3, Test Site 3-6, El. -7.5 ft. - Note the smooth steel surface revealed by Level II cleaning. Ultrasonic thickness readings indicated the average thickness at this location of 0.440 in. Also note the more dense covering of marine fouling at this elevation.

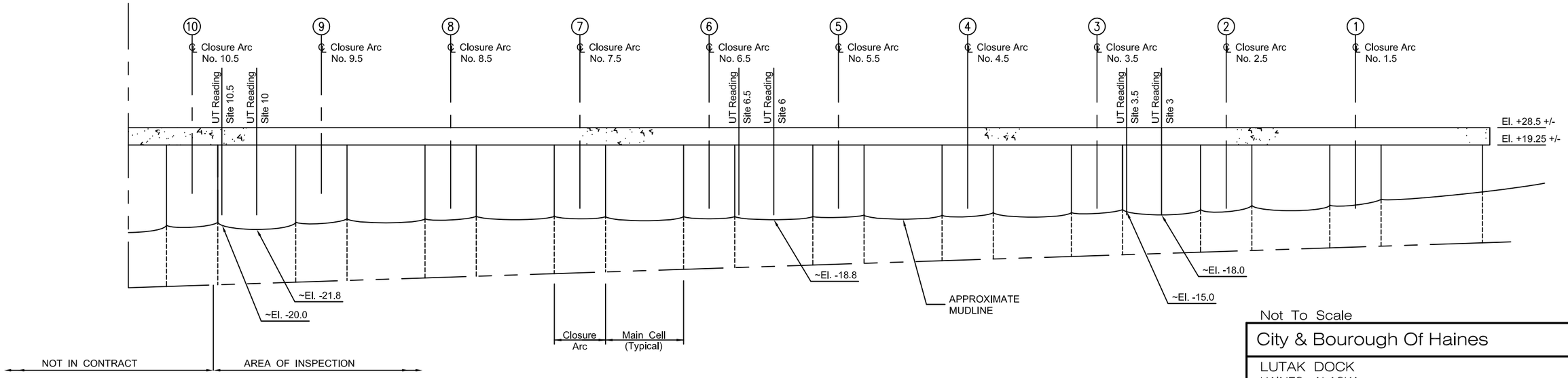
PHOTO No. 16: Cell No. 3, Test Site 3-7, El. -12.5 ft. - The average ultrasonic reading at this site was 0.222 in. This photo was taken 1 day after cleaning was conducted. Note the amount of surface corrosion that has occurred during this short period of exposure. Also note the dense covering of marine fouling.

PHOTO No. 17: Cell No.5 & Closure Arc, Mudline - Cell No. 5 is located to the right with the closure arc on the left. Note the upward slope of the accumulated silty sediment against the face of the cell. This type of accumulation was typical throughout Cells No. 1 - 9.

PHOTO No. 18: Cell No. 10, Mudline - Note the absence of silty sediment at this location. It appears that the sediments have been displaced at this location, likely by prop/thruster wash. Also note the small rocks which have become bonded to the bottom 2 - 3 ft. on the sheet piles.



PLAN



ELEVATION

Not To Scale

City & Bourough Of Haines

LUTAK DOCK
HAINES, ALASKA

UNDERWATER INSPECTION
PLAN & ELEVATION

DATE:	April 2002
PROJECT:	02-2163
SHEET:	1 of 1
DRAWN:	JDS/SDS

ECHELON ENGINEERING, INC.
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Shelley D. Sommerfeld, Echelon Engineering, Inc. 4/22/2002 4:05 PM 2163-SHT1.dwg

TABLE 1
STEEL THICKNESS READINGS

LOCATION	Elevation	THICKNESS READINGS (inches)					CONDITION / DAMAGE
Cell - Site No.	(Chart Datum)	Reading 1	Reading 2	Reading 3	Reading 4	Average	
MAIN CELL No. 3							
3 - 0	18.3	0.265	0.265	0.270		0.267	
3 - 1	13.0	0.425	0.430	0.435		0.430	
3 - 2	9.1	0.450	0.455	0.455		0.453	Max. Pit Depth = 0.045 in.
3 - 3	6.3	0.430	0.435	0.430		0.432	
3 - 4	3.8	0.345	0.345	0.350	0.360	0.350	
3 - 5	-2.5	0.390	0.395	0.390		0.392	
3 - 6	-7.5	0.440	0.435	0.445		0.440	
3 - 7	-12.5	0.215	0.225	0.225		0.222	
3 - 8	-17.5	0.435	0.430	0.435		0.433	Mudline
MAIN CELL No. 3 @ East Closure Arch							
3.5 - 0	18.1	0.320	0.315	0.320		0.318	
3.5 - 1	13.3	0.315	0.320			0.318	
3.5 - 2	9.3	0.405	0.400	0.405		0.403	
3.5 - 3	6.8	0.355	0.345	0.340		0.347	
3.5 - 4	4.2	0.330	0.335	0.330		0.332	
3.5 - 5	2.2	0.390	0.400	0.395		0.395	
3.5 - 6	0.0	0.215	0.225	0.225		0.222	
3.5 - 7	-4.5	0.225	0.230	0.240		0.232	
3.5 - 8	-9.5	0.225	0.315	0.325	0.275	0.285	
3.5 - 9	-14.5	0.405	0.410	0.410		0.408	Mudline
MAIN CELL No. 6							
6 - 0							Not Accessible
6 - 1	12.6	0.320	0.355	0.360		0.345	Max. Pit Depth = 0.040 in.
6 - 2	9.4	0.430	0.435	0.440		0.435	
6 - 3							Not Accessible
6 - 4	3.2	0.245	0.415	0.395		0.352	
6 - 5	0.8	0.215	0.290	0.325	0.295	0.281	
6 - 6	-5.8	0.435	0.435	0.450		0.440	
6 - 7	-9.8	0.430	0.435	0.430		0.432	
6 - 8	-14.8	0.270	0.275	0.285		0.277	
6 - 9	-18.8	0.435	0.430	0.430		0.432	Mudline

TABLE 1
STEEL THICKNESS READINGS

LOCATION	Elevation	THICKNESS READINGS (inches)					CONDITION / DAMAGE
Cell - Site No.	(Chart Datum)	Reading 1	Reading 2	Reading 3	Reading 4	Average	
MAIN CELL No. 6 @ East Closure Arch							
6.5 - 0							Not Accessible
6.5 - 1							Not Accessible
6.5 - 2							Not Accessible
6.5 - 3							Not Accessible
6.5 - 4	4.0	0.380	0.380	0.380		0.380	
6.5 - 5	-1.0	0.320	0.350			0.335	
6.5 - 6	-6.0	0.420	0.415	0.435		0.423	
6.5 - 7	-11.0	0.420	0.415	0.415		0.417	
6.5 - 8	-16.0	0.390	0.395	0.390		0.392	
MAIN CELL No. 10							
10 - 0	18.5	0.440	0.390	0.400		0.410	
10 - 1	11.2	0.415	0.425	0.430		0.423	Max. Pit Depth = 0.11
10 - 2	8.2	0.250	0.255	0.260	0.255	0.255	
10 - 3	5.3	0.355	0.360	0.365		0.360	
10 - 4	-9.3	0.420	0.410	0.425		0.418	
10 - 5	-18.3	0.390				0.390	
10 - 6, 1st Set	-23.3	0.145	0.145	0.150		0.147	Mudline
10 - 6, 2nd Set	-23.3	0.220	0.220	0.225	0.225	0.223	Mudline
MAIN CELL No. 10 @ East Closure Arch							
10.5 - 0	18.1	0.370	0.380	0.380		0.377	
10.5 - 1	11.1	0.415	0.425	0.430		0.423	
10.5 - 2	6.3	0.320	0.340	0.340		0.333	
10.5 - 3	4.0	0.315	0.325	0.335		0.325	Max. Pit Depth = 0.08 in.
10.5 - 4	-6.8	0.305	0.340	0.335		0.327	
10.5 - 5	-11.8	0.335	0.305	0.315		0.318	
10.5 - 6	-16.8	0.310	0.315	0.305		0.310	
10.5 - 7	-21.8	0.370	0.365	0.365		0.367	Mudline

TABLE 2
STEEL THICKNESS READINGS - BY ELEVATION RANGE

ELEVATION RANGE	Elevation (Chart Datum)	AVERAGE THICKNESS READINGS (inches)						Mean	Standard Deviation
		Cell 3	Cell 3.5	Cell 6	Cell 6.5	Cell 10	Cell 10.5		
Elevation +19.25 to +15	18.5 18.3 18.1	0.267	 0.318			0.410	 0.377	0.343	0.063
Elevation +15 to +10	13.3 13.0 12.6 11.2 11.1	0.430	0.318	0.345		0.423	0.423	0.388	0.052
Elevation +10 to +5	9.4 9.3 9.1 8.2 6.8 6.3 5.3	0.453 0.432	0.403 0.347	0.435		0.255 0.360	0.333	0.377	0.066
Elevation +5 to +0	4.2 4.0 3.8 3.2 2.2 0.8	0.350	0.332 0.395	0.352 0.281	0.380		0.325	0.345	0.038
Elevation +0 to -5	0.0 -1.0 -2.5 -4.5	0.392	0.222 0.232		0.335			0.295	0.082
Elevation -5 to -10	-5.8 -6.0 -6.8 -7.5 -9.3 -9.5 -9.8	0.440	 0.285	0.440 0.432	0.423	 0.418	0.327	0.395	0.063

TABLE 2
STEEL THICKNESS READINGS - BY ELEVATION RANGE

ELEVATION RANGE	Elevation (Chart Datum)	AVERAGE THICKNESS READINGS (inches)						Mean	Standard Deviation
		Cell 3	Cell 3.5	Cell 6	Cell 6.5	Cell 10	Cell 10.5		
Elevation -10 to -15	-11.0 -11.8 -12.5 -14.5 -14.8	0.222	0.408	0.277	0.417		0.318	0.328	0.084
Elevation -15 to MDL	-16.0 -16.8 -17.5 -18.3 -18.8 -21.8 -23.3 -23.3	0.433		0.432	0.392	0.390 0.147 0.223	0.310 0.367	0.337	0.103

Mean of all readings = 0.355
Standard Deviation of all readings = 0.073

TABLE 3
NONPARAMETRIC PERCENTILES

Ranking	Average Thickness (in)	Percentile	LOCATION Cell - Site No.	Elevation (Chart Datum)
1	0.147	2	10 - 6, 1st Set	-23.3
2	0.222	4	3-7	-12.5
3	0.222	6	3.5 - 6	0.0
4	0.223	8	10 - 6, 2nd Set	-23.3
5	0.232	10	3.5 - 7	-4.5
6	0.255	12	10 - 2	8.2
7	0.267	14	3-0	18.3
8	0.277	16	6 - 8	-14.8
9	0.281	18	6 - 5	0.8
10	0.285	20	3.5 - 8	-9.5
11	0.310	22	10.5 - 6	-16.8
12	0.318	24	3.5 - 0	18.1
13	0.318	27	3.5 - 1	13.3
14	0.318	29	10.5 - 5	-11.8
15	0.325	31	10.5 - 3	4.0
16	0.327	33	10.5 - 4	-6.8
17	0.332	35	3.5 - 4	4.2
18	0.333	37	10.5 - 2	6.3
19	0.335	39	6.5 - 5	-1.0
20	0.345	41	6 - 1	12.6
21	0.347	43	3.5 - 3	6.8
22	0.350	45	3-4	3.8
23	0.352	47	6 - 4	3.2
24	0.360	49	10 - 3	5.3
25	0.367	51	10.5 - 7	-21.8
26	0.377	53	10.5 - 0	18.1
27	0.380	55	6.5 - 4	4.0
28	0.390	57	10 - 5	-18.3
29	0.392	59	3-5	-2.5
30	0.392	61	6.5 - 8	-16.0
31	0.395	63	3.5 - 5	2.2
32	0.403	65	3.5 - 2	9.3
33	0.408	67	3.5 - 9	-14.5
34	0.410	69	10 - 0	18.5
35	0.417	71	6.5 - 7	-11.0
36	0.418	73	10 - 4	-9.3
37	0.423	76	6.5 - 6	-6.0
38	0.423	78	10 - 1	11.2
39	0.423	80	10.5 - 1	11.1
40	0.430	82	3-1	13.0
41	0.432	84	3-3	6.3
42	0.432	86	6 - 7	-9.8
43	0.432	88	6 - 9	-18.8
44	0.433	90	3-8	-17.5
45	0.435	92	6 - 2	9.4
46	0.440	94	3-6	-7.5
47	0.440	96	6 - 6	-5.8
48	0.453	98	3-2	9.1

REF #6

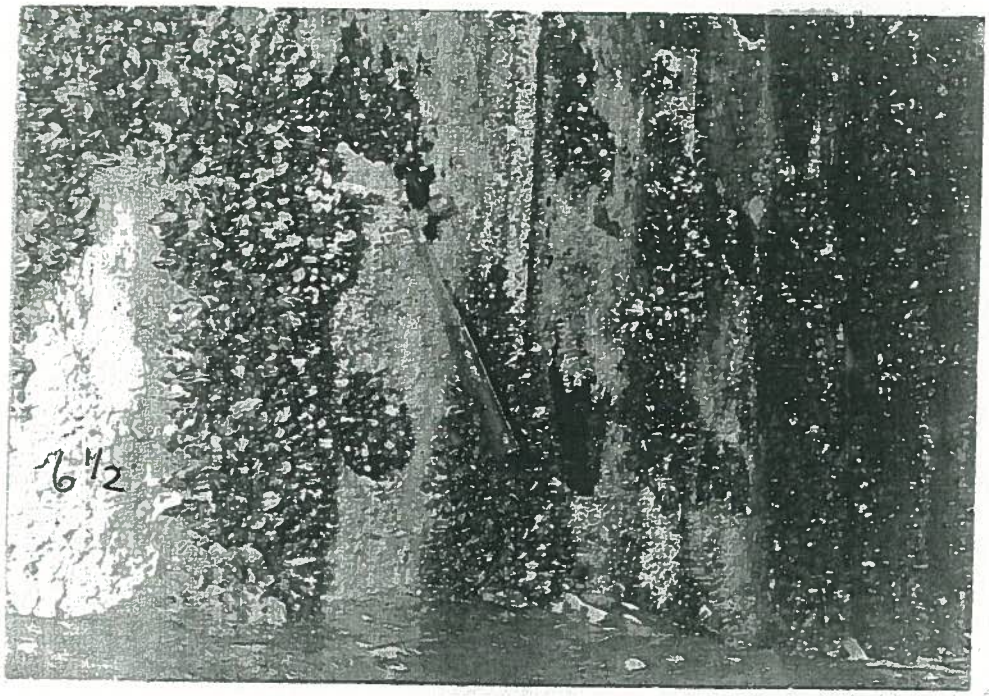
Condition Survey Lutak Dock – Prepared by PND Engineers. Date October 1988.

8847 ~~8825~~

98262
Background

CONDITION SURVEY

LUTAK DOCK



PREPARED FOR

CITY OF HAINES

Dispos

Peratrovich, Nottingham & Drage, Inc.
Engineering Consultants
2205 North Jordan Avenue, JUNEAU, AK 99801
(907) 789-5006

10/26/88

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INTRODUCTION

Recognizing that the Lutak Dock is an important asset to the local economy, and that the dock has deteriorated considerably since its construction in 1953; the City of Haines has retained Peratrovich, Nottingham and Drage to inspect the dock and make recommendations for repairs to the facility.

Field inspections were conducted August 29 through 31 and again on October 5, 1988. The following discussion describes current conditions and makes recommendations for repairs.

HISTORY

The dock was originally constructed in 1953 by the U.S. Army Corps of Engineers. An impressed current cathodic protection system was included in the initial construction. It is uncertain when the cathodic protection system was deactivated, but the dock has not been protected from corrosion for at least two decades.

Prior to this survey the facility was inspected on 3 earlier occasions. In 1975, Steen and Matlock, Consulting Engineers, conducted a condition survey of the dock. They did not discover any damage to the concrete cap or the sheet pile cells, although they made recommendations for replacement of the fender system.

In 1976 Dennis Nottingham, working for R & M Consultants, Inc. conducted a second condition survey of the dock for the Alaska Division of Waters and Harbors. That report indicated that significant metal loss had occurred in main cells and connector arcs. However, the report indicated that the dock was structurally sound at that time.

Peratrovich, Nottingham and Drage, Inc. was retained by the Alaska Department of Transportation and Public Facilities in 1982 to conduct research into corrosion in the Alaskan marine environment. As part of that effort, the Lutak Dock was again inspected. This report stated that "... corrosion damage to this structure deserves serious consideration and remedial action."

GENERAL DESCRIPTION OF FACILITY

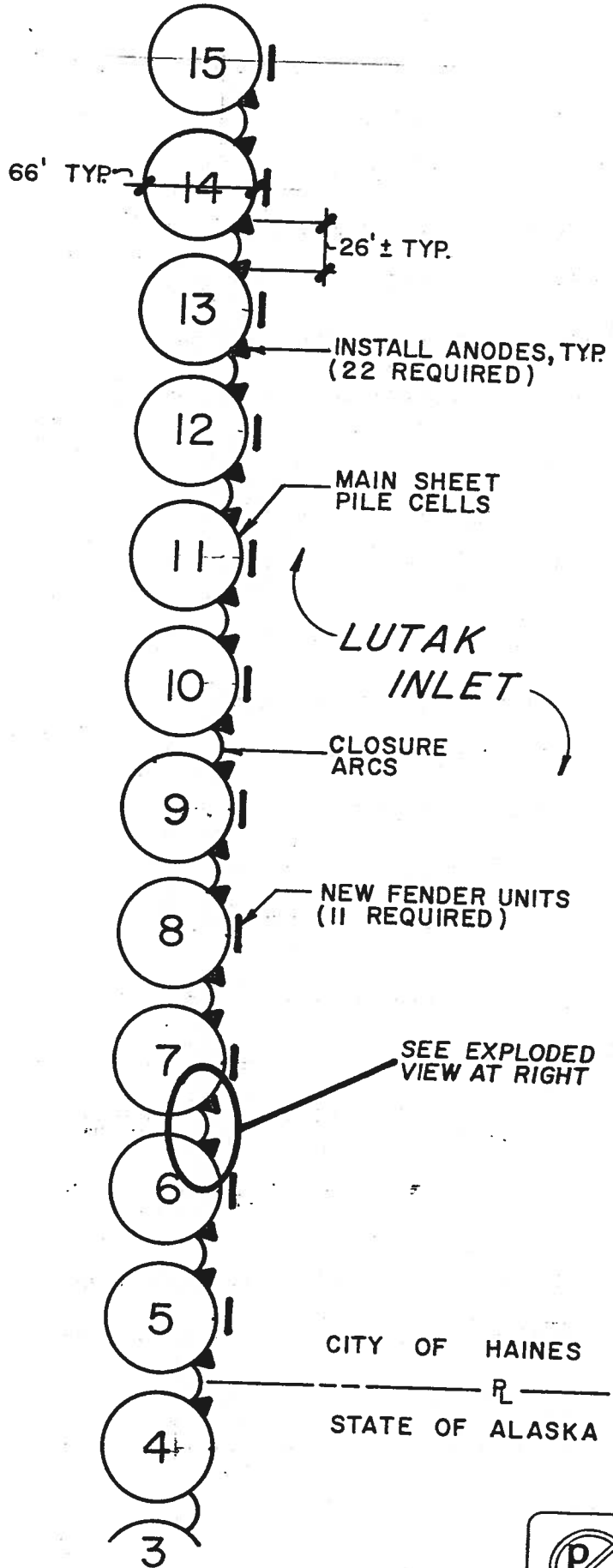
The Lutak Dock consists of a series of circular 1/2" thick steel sheet pile cells which are connected by closure arcs constructed of steel sheet piles 3/8" thick. (See Figure 1.) The closure arcs have been penetrated by numerous weep holes to allow the movement of groundwater through the wall. The sheet pile structure is topped by a concrete curb and pile cap system.

Ownership of the dock is shared between the City of Haines and the State of Alaska. The northern 11 of the 15 total sheet pile cells are on City of Haines property. Approximately 3/4 of the total dock length (960 feet) belongs to the City of Haines.

The original fendering system has been damaged and replaced by a system and which is now ineffective or has been completely damaged and removed.

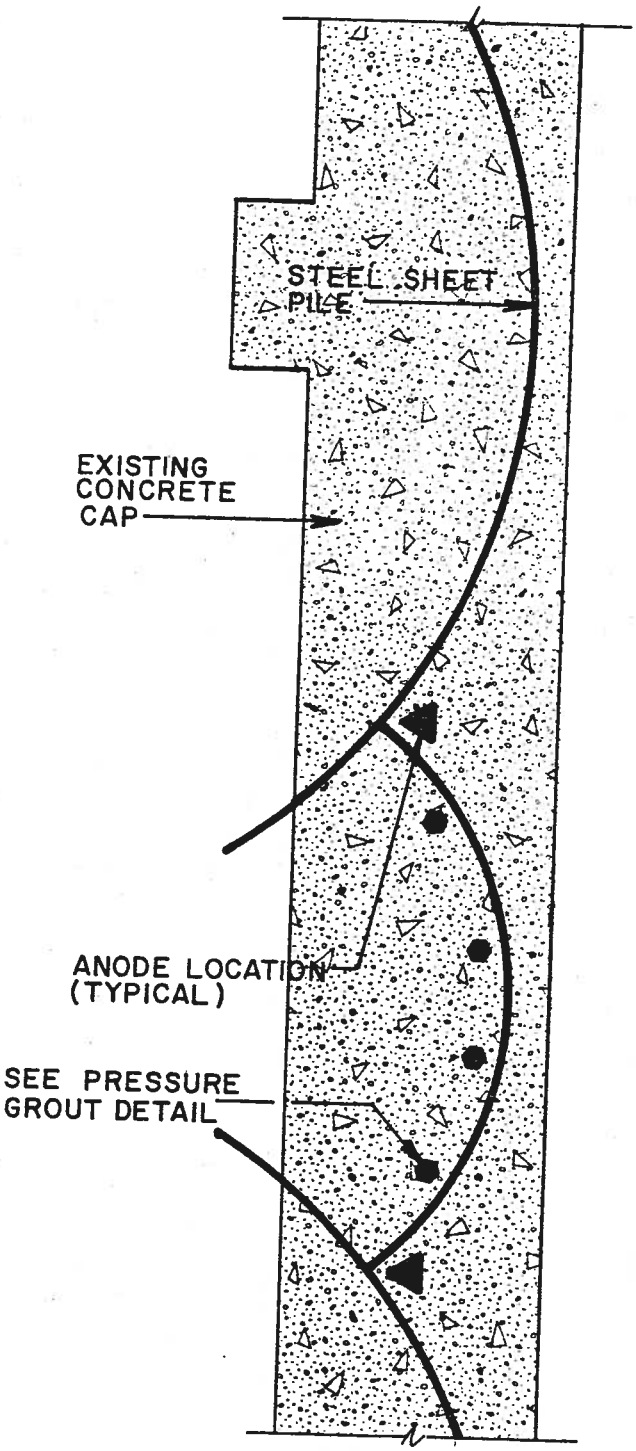
CORROSION DAMAGE

Our inspection focused primarily on the conditions of the steel sheet piles. The structural soundness is based on the remaining steel; that material not lost to corrosion. We found corrosion to be greatest in a band between -2 ft. to +8 ft. M.L.L.W.



PLAN VIEW

SCALE: 1" = 100'



SCALE: 1" = 10'

FIGURE 1

LUTAK DOCK - HAINES, ALASKA

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(907) 789-5006

Corrosion was most severe in the closure arcs where adjacent gaping holes up to 5 feet long were observed. Virtually all closure arcs had locations of full corrosion penetration. The closed sheet pile cells while having suffered severe metal loss to corrosion, were not completely penetrated anywhere on the dock. The purpose of the closure arcs is to tie the main cells together; therefore, their condition is critical to the structure. There are two reasons closure arcs exhibited much greater damage than the main closed cells:

1. The closure arcs were constructed of thinner material; i.e. 1/2" thickness for full cells verses 3/8" thickness of closure arcs.

2. A large number of weep holes were cut into the closure arcs to provide an outlet for groundwater flowing between the closed cells. This allows highly oxygenated water to pass along both faces of the sheet piles during each tide cycle. Consequently active corrosion has taken place from both faces simultaneously. The closed cells corroded from the outer face only.

Steel "H" piles that were originally designed to support lateral loads from the fender system are severely corroded at the low tide zone and will not perform their intended function.

FENDER SYSTEM

The fender system from cell 11 1/2 to the north has been removed with only broken remnants remaining. The fender system from cell 11 1/2 south to the City's property boundary is in poor condition. This system depends on the vertical steel "H" piles to provide lateral resistance to loads. As mentioned earlier these "H" piles have been severely corroded at the low tide line and have only a fraction of their original strength remaining.

CONCRETE PILE CAP

Although some cracking of the concrete pile cap has occurred, functionally it remains intact.

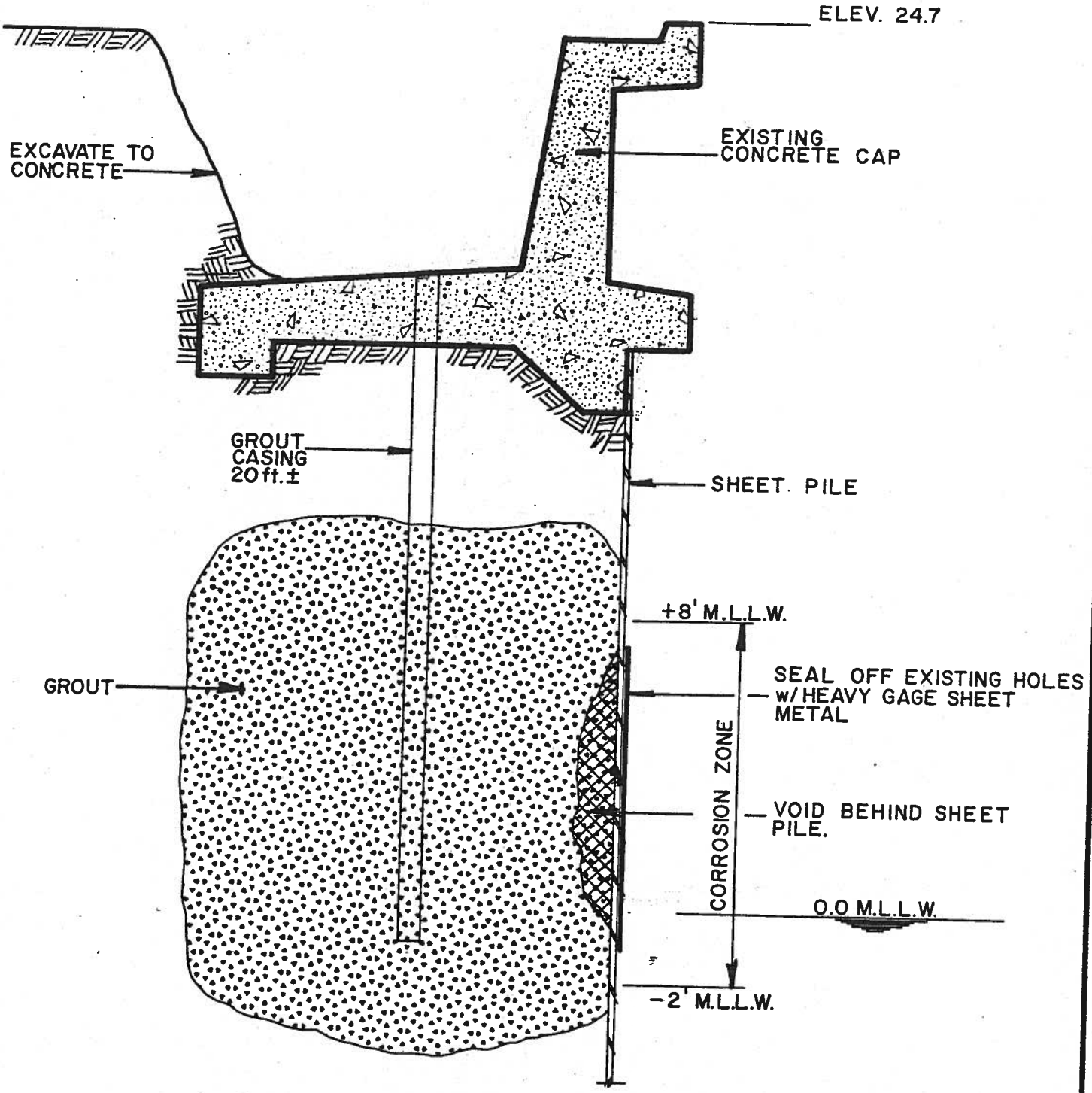
SAFETY LADDERS

Safety ladders on the existing dock are unusable or totally broken. Anyone falling into the water would have an extremely difficult time climbing out.

RECOMMENDATIONS

Closure arcs must be repaired where corroded. This can be accomplished by pressure grouting inside the corroded zones to stabilize the material inside the damaged cells. (Figure 2) The area above each damaged cell would be excavated to the concrete pile cap. A line of casings would be drilled through the concrete cap to the zone behind the corroded cells. Cement grout under pressure would be injected into the soil creating a solid mass. This would seal off and stabilize the damaged areas.

Once corrosion damage is repaired a cathodic protection system must be installed to prevent further corrosion damage. The recommended system is to attach sacrificial zinc or aluminum anodes to the sheet pile docks. The recommended system would suspend, the anodes from above by a galvanized steel cable at 0 ft. M.L.L.W. elevation. (Figure 3) A heavy copper cable would allow the anodes to be observed at minus tides without the necessity to have a diver. It would also make replacing anodes a relatively simple operation.



PRESSURE GROUTING

NO SCALE

FIGURE 2

LUTAK DOCK – HAINES, ALASKA

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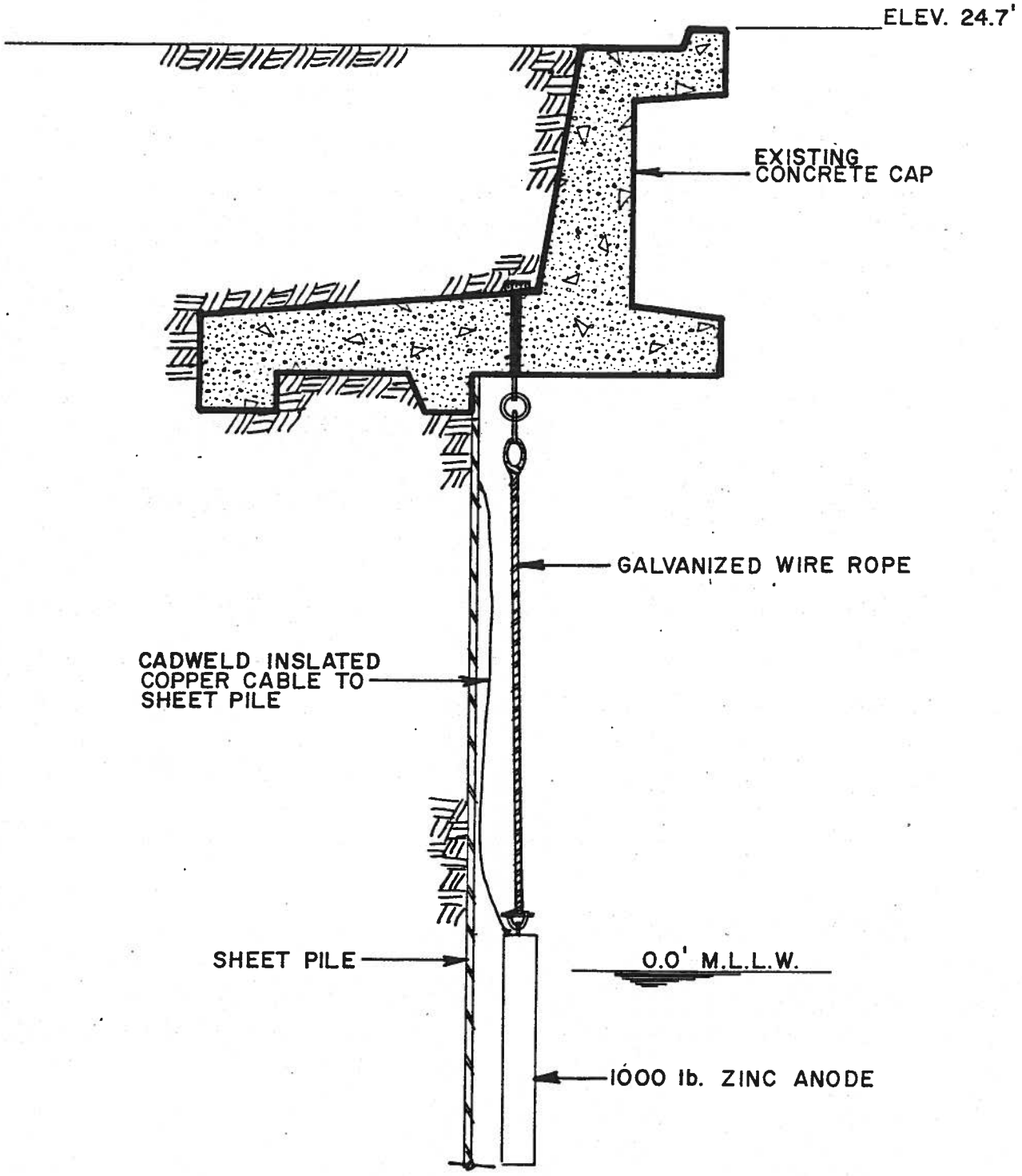

CATHODIC PROTECTION-ANODE INSTALLATION

NO SCALE

LUTAK DOCK-HAINES, ALASKA

FIGURE 3

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Engineering Consultants
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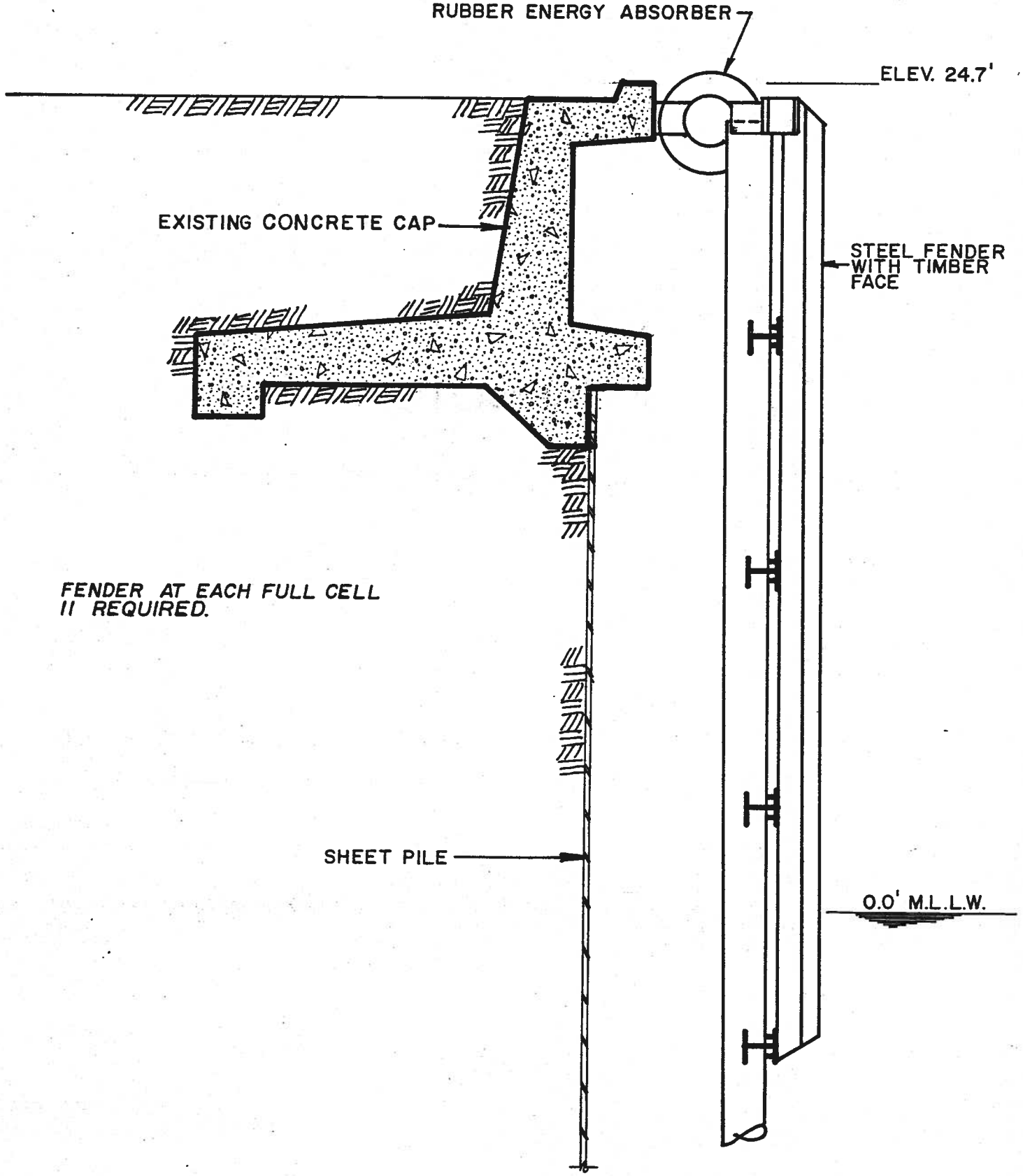
Finally a modern energy absorbing fender system should be installed along the dock face. Steel pin piles supporting a timber faced, steel fender backed up by rubber doughnut energy absorbers at the center of each full cell would provide a maintenance free system for the remaining useable life of the dock. (Figure 4) Safety ladders at the center of each fender would be included.

The existing fender system should be removed including steel "H" piles.

CONSTRUCTION COST ESTIMATE

REMOVE EXISTING FENDER SYSTEM	Lump Sum	$ 20,000
PRESSURE GROUTING	11 Cells @ $15,000	165,000
CATHODIC PROTECTION	22 Anodes @ $5,000	110,000
FENDER SYSTEM	11 Fenders @ $40,000	440,000
MOBILIZATION	(10% other items)	74,000
	Subtotal	$809,000
	15% Contingencies	121,000
	Total	$930,000



FENDERS

NO SCALE

FIGURE 4

LUTAK DOCK - HAINES, ALASKA

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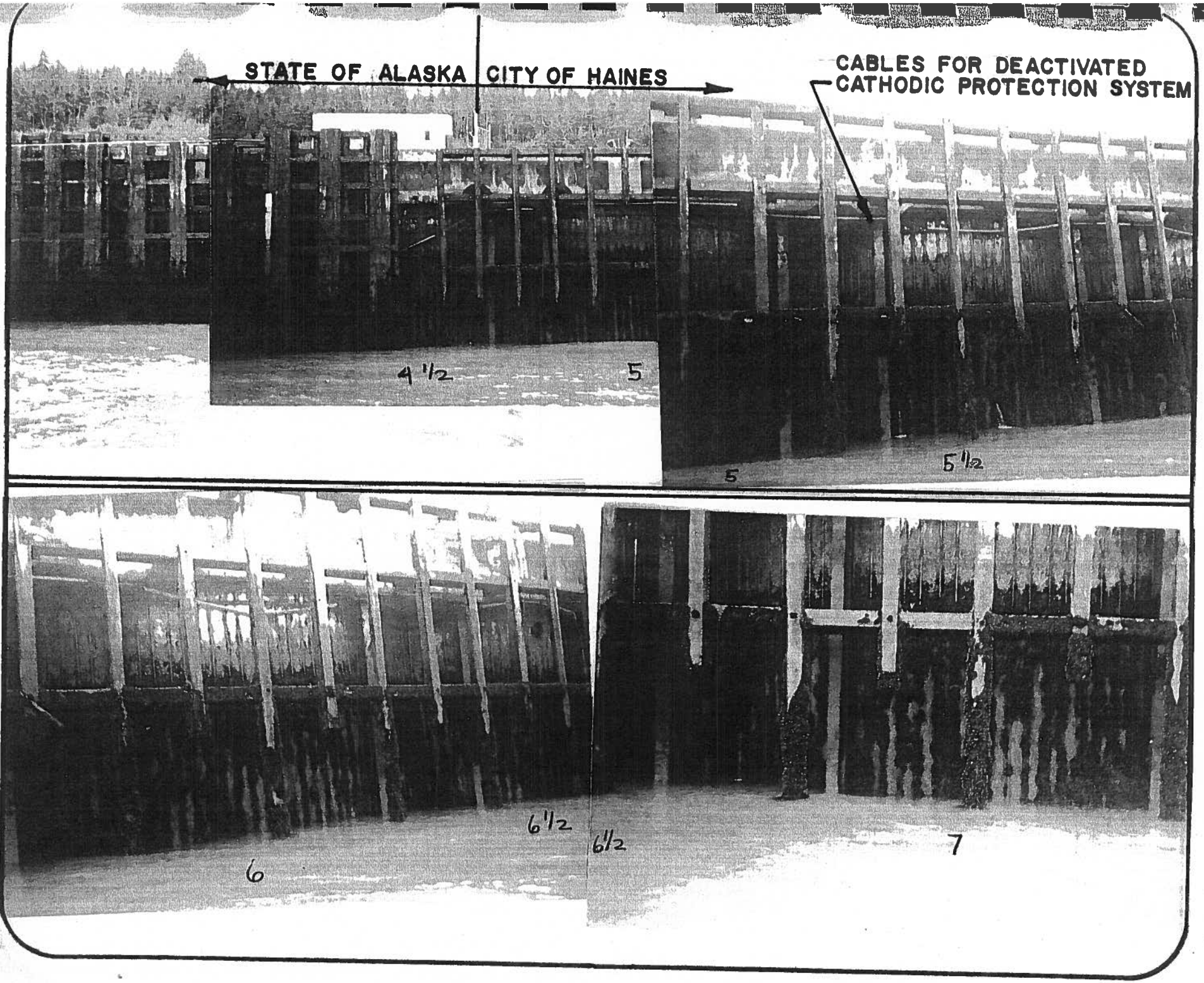
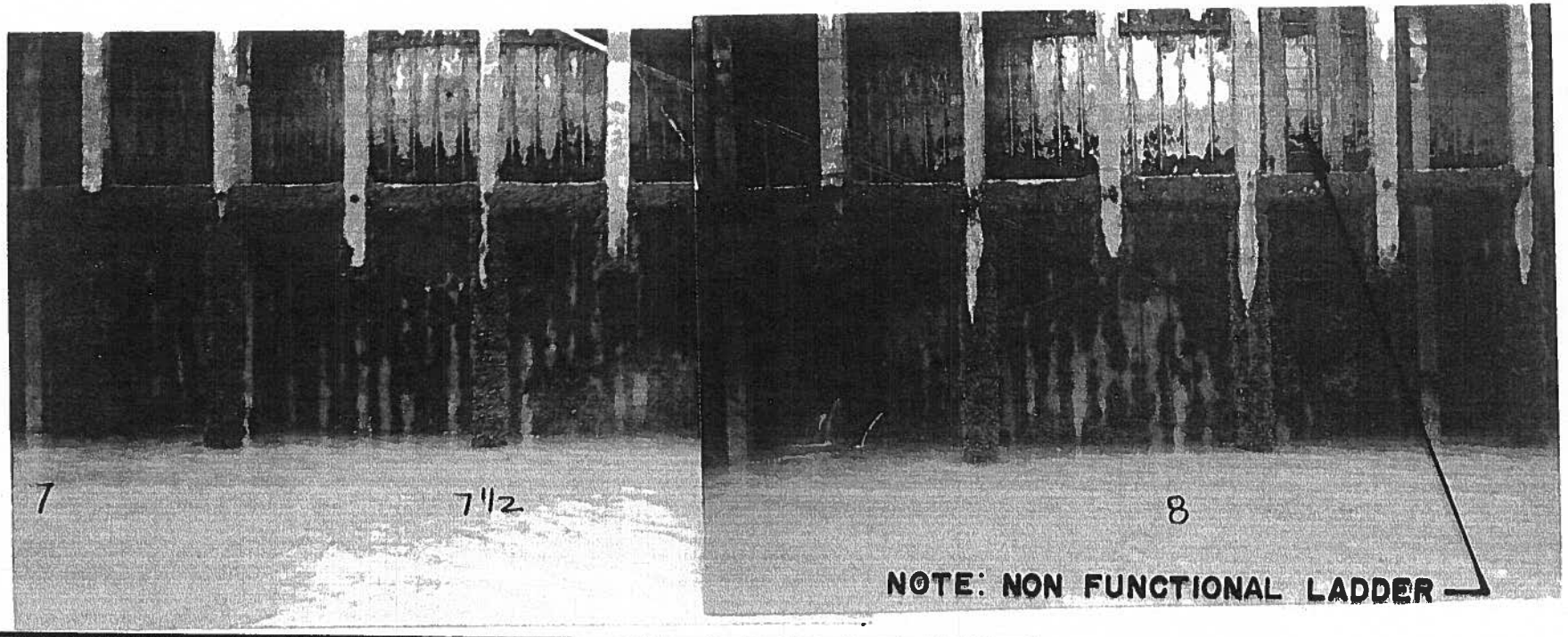
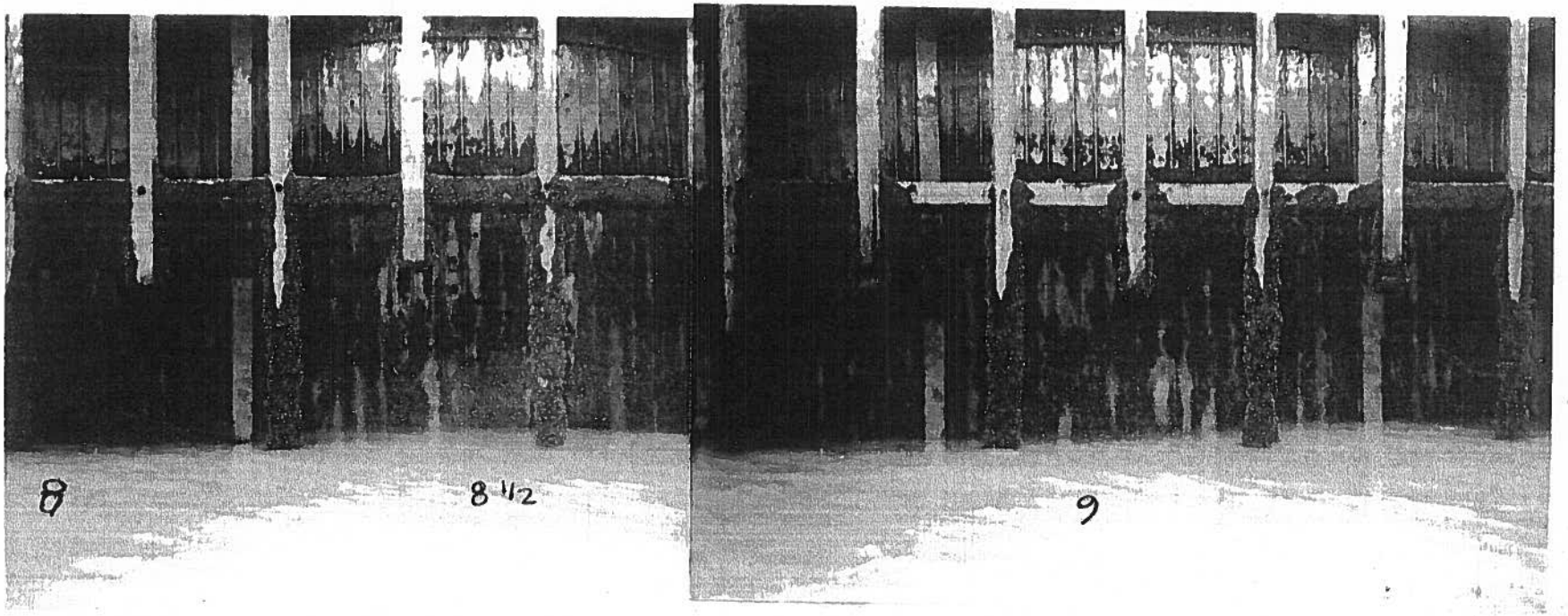
PHOTOGRAPHS


STATE OF ALASKA
CITY OF HAINES
CABLES FOR DEACTIVATED
CATHODIC PROTECTION SYSTEM
4 1/2
5
5
5 1/2
6
6 1/2
6 1/2
7


7
7 1/2
8
NOTE: NON FUNCTIONAL LADDER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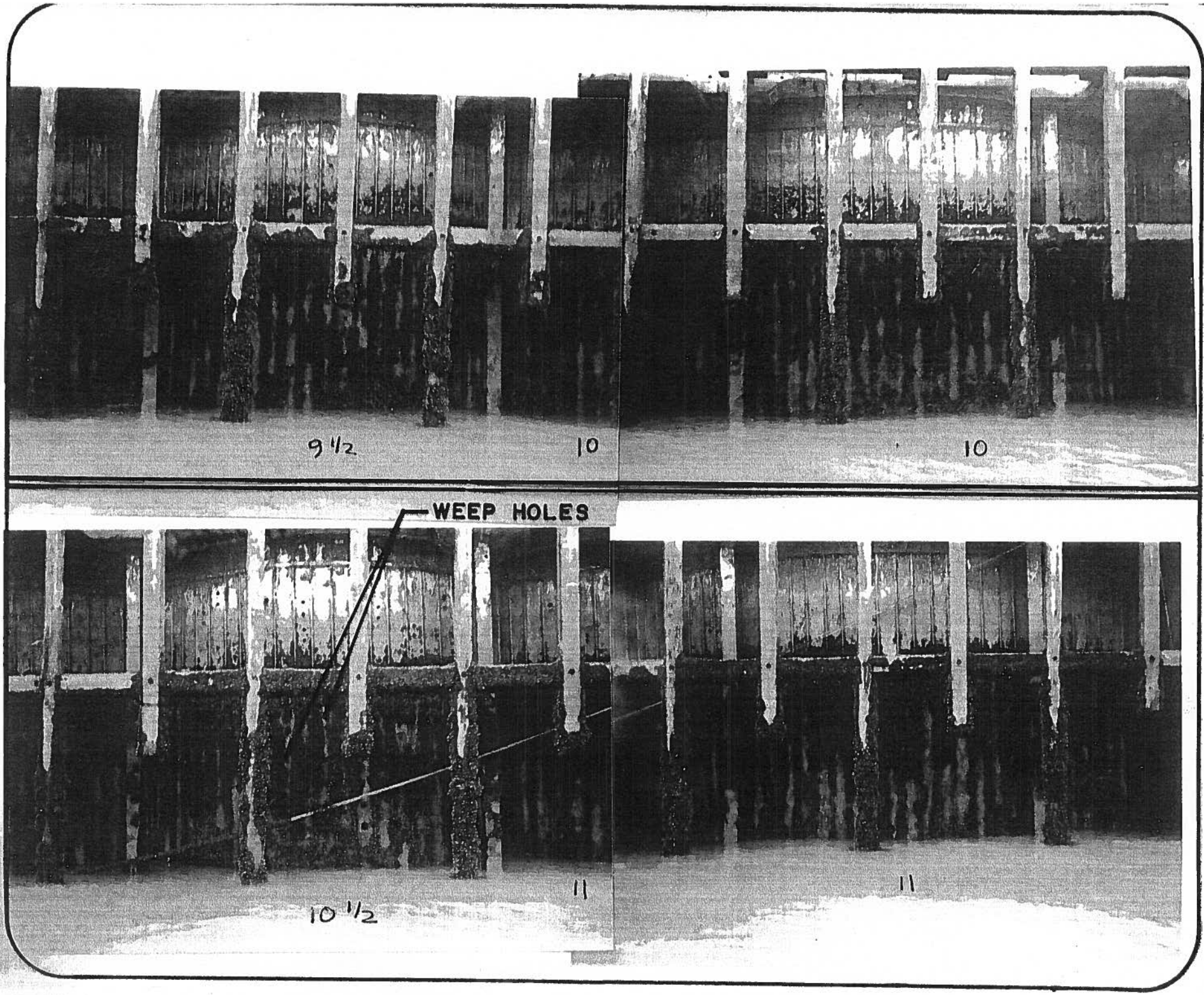
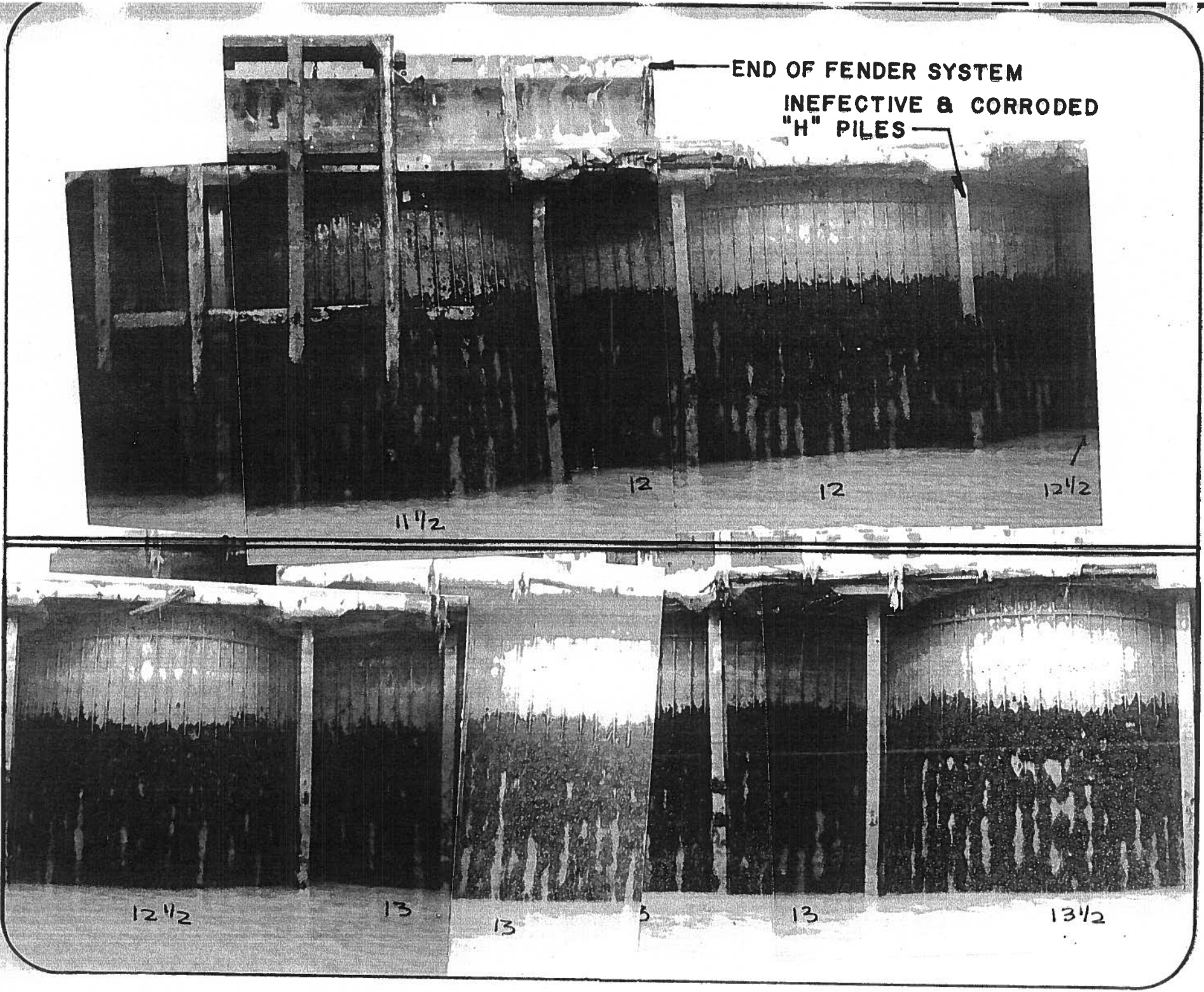
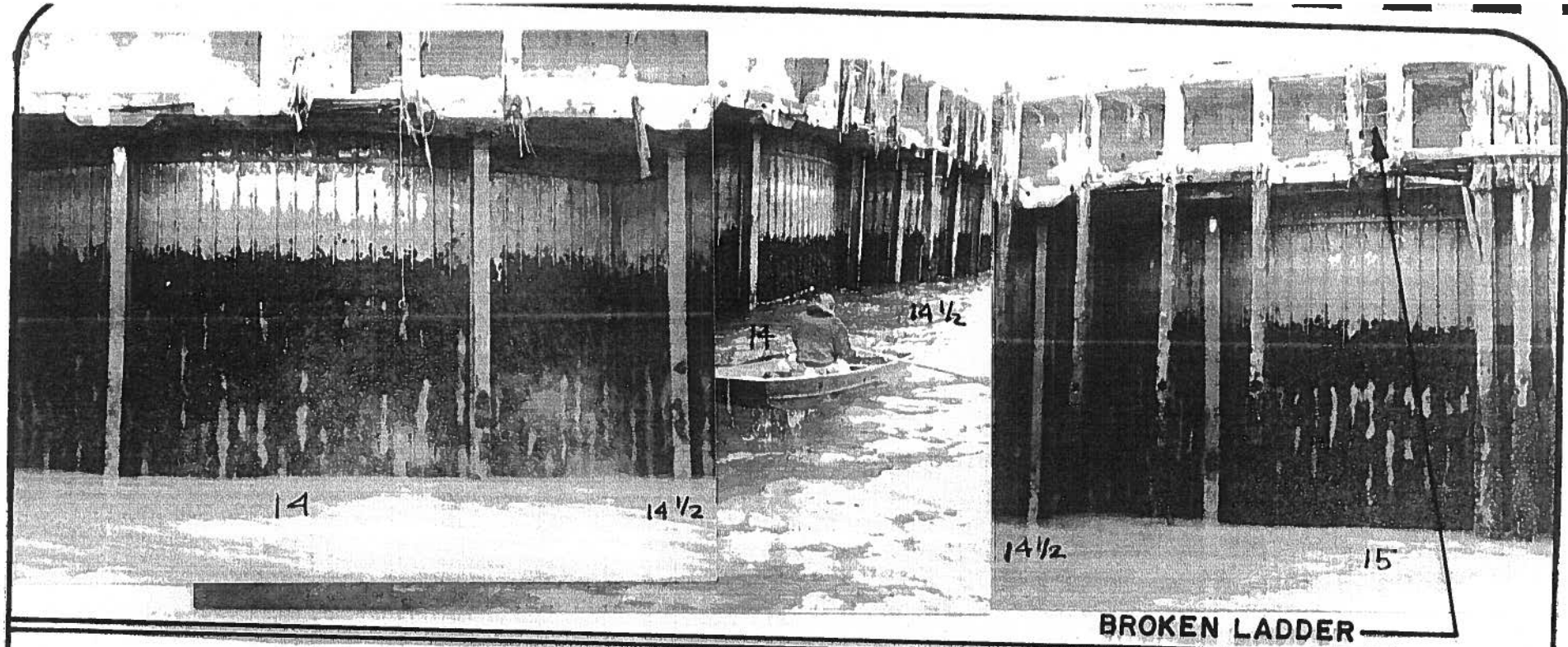
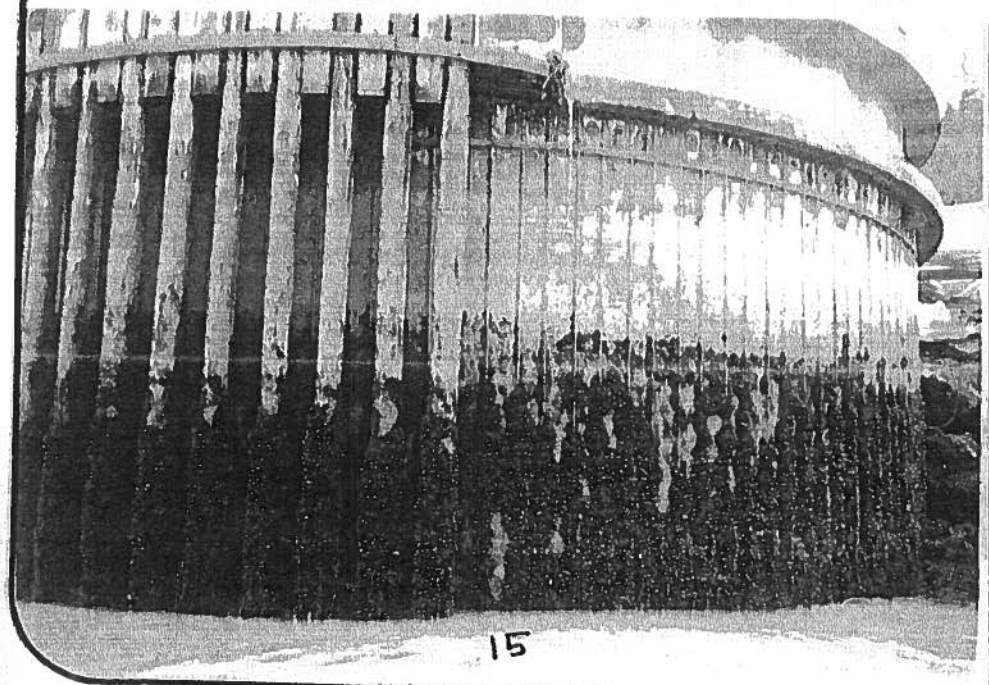
8 1/2
9


9 1/2
10
10
WEEP HOLES
10 1/2
11
11


END OF FENDER SYSTEM
INEFECTIVE & CORRODED
"H" PILES
11½
12
12
12½
12½
13
13
13
13½


14
14 1/2
14 1/2
15
BROKEN LADDER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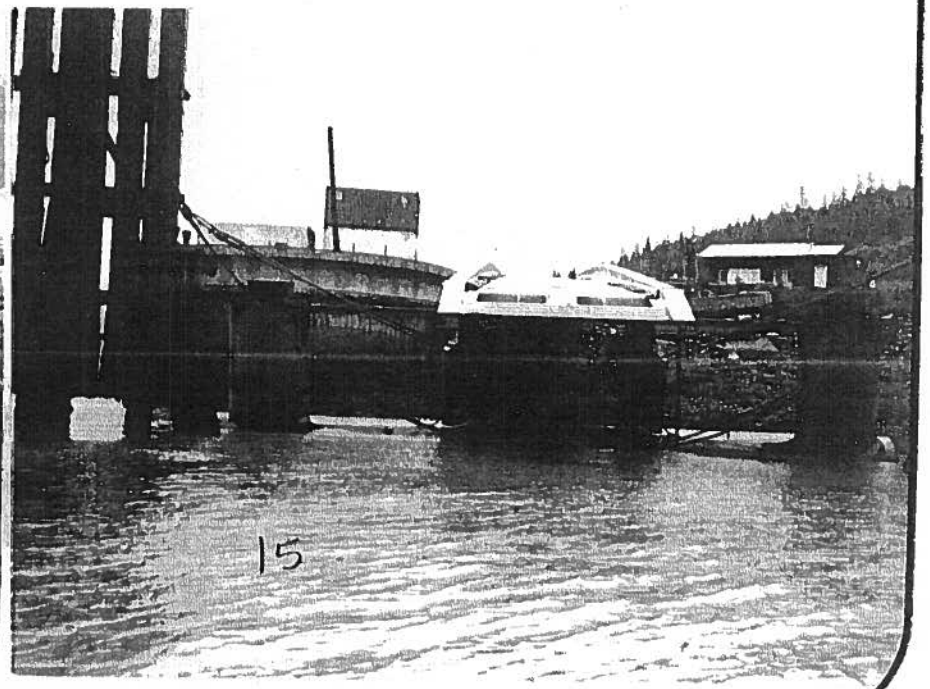
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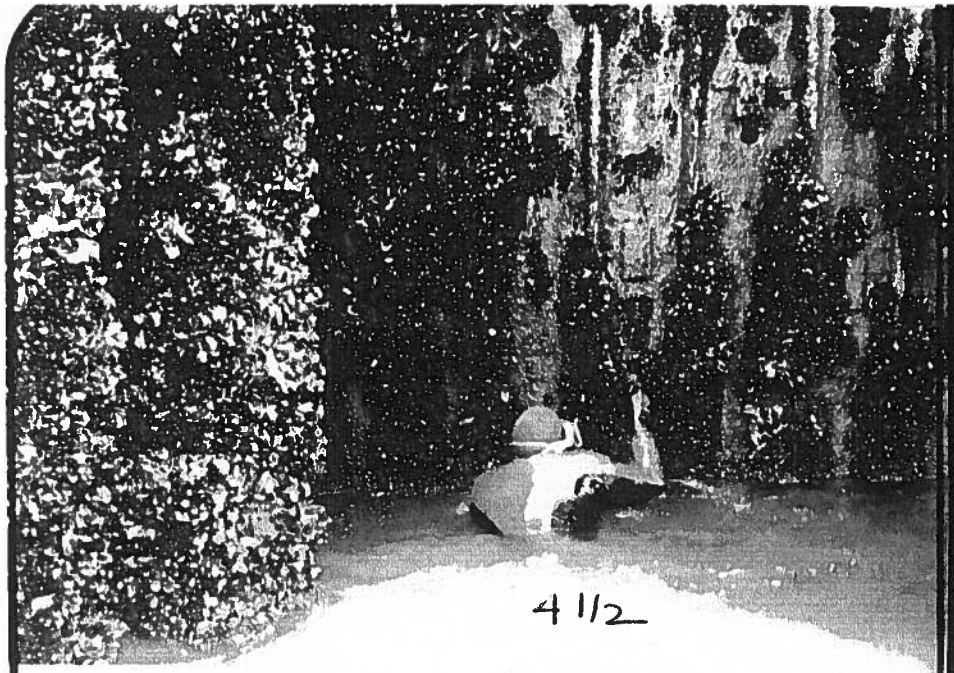
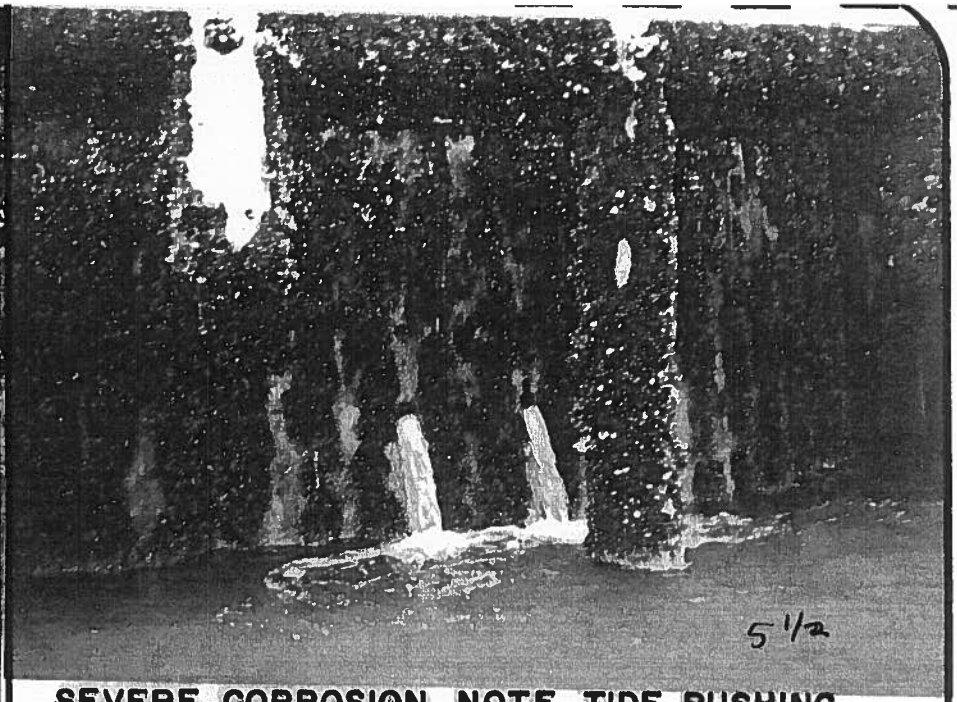
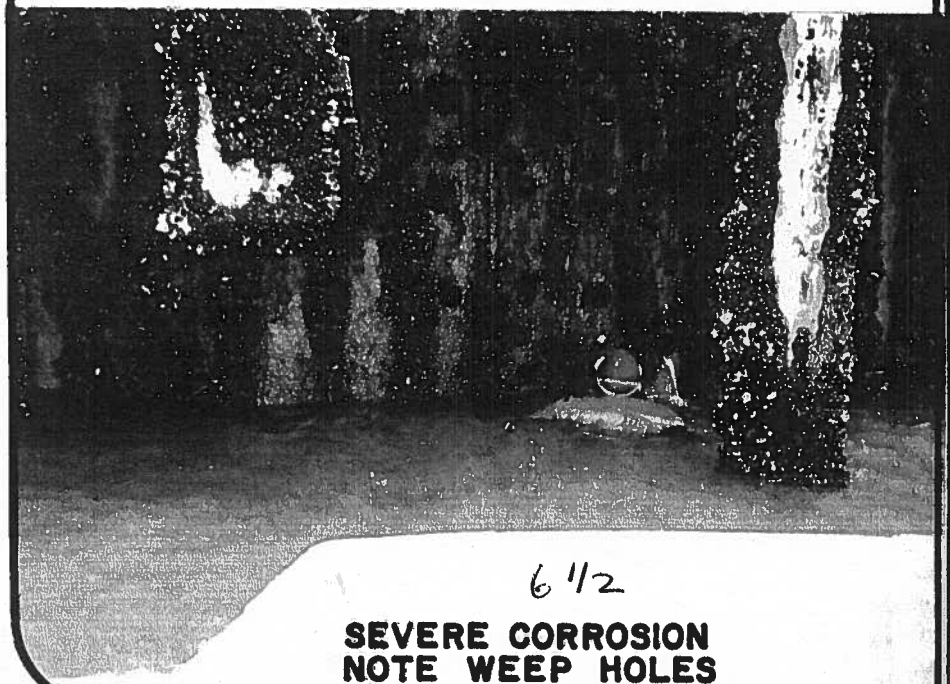


SEVERE CORROSION

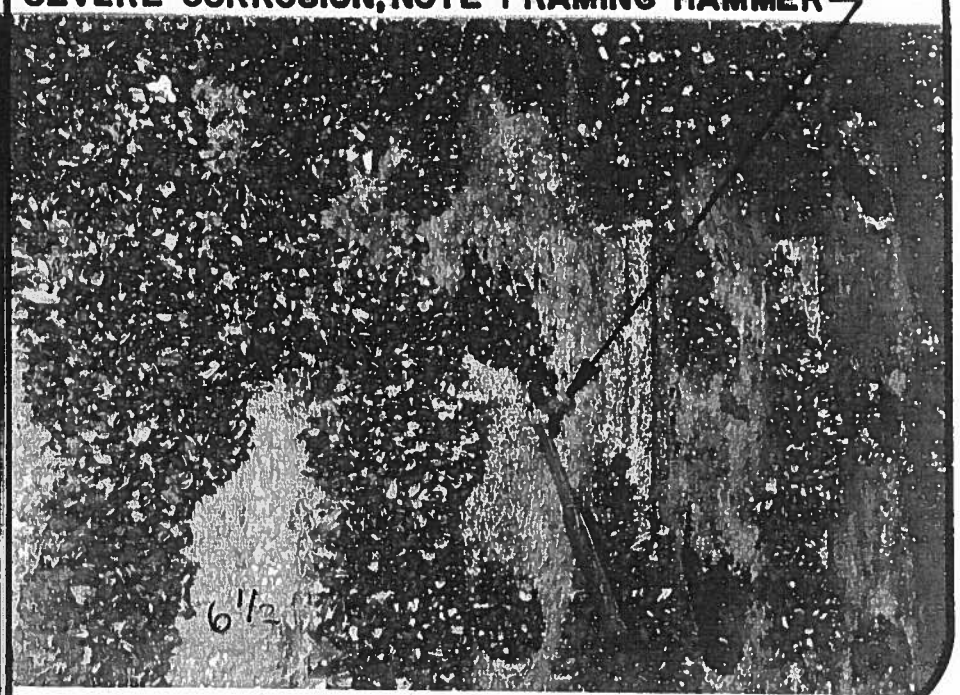


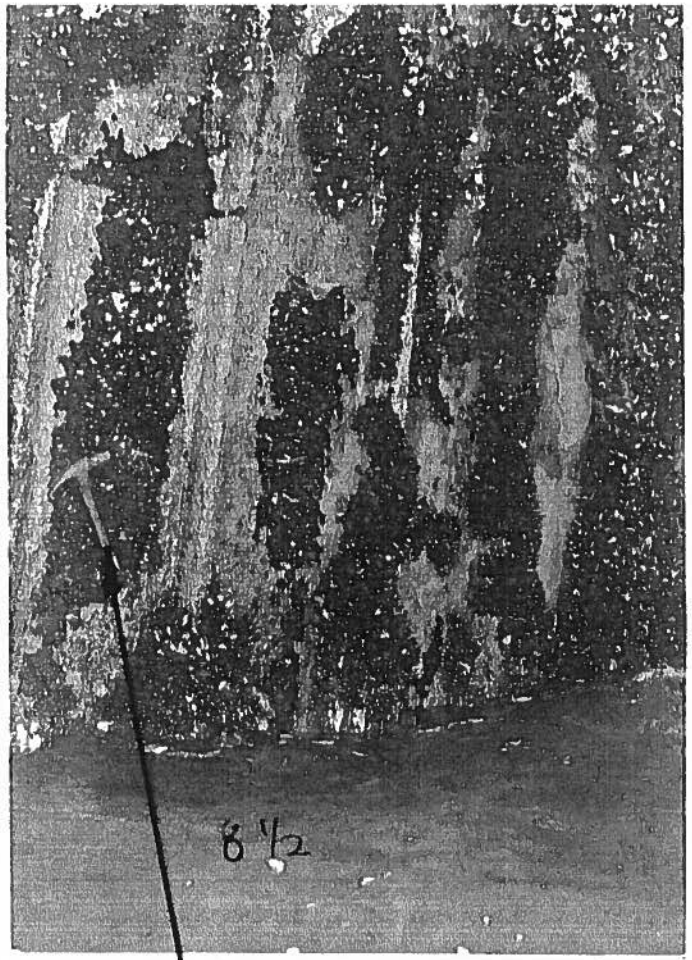
SEVERE CORROSION. NOTE TIDE RUSHING FROM WEEP HOLES



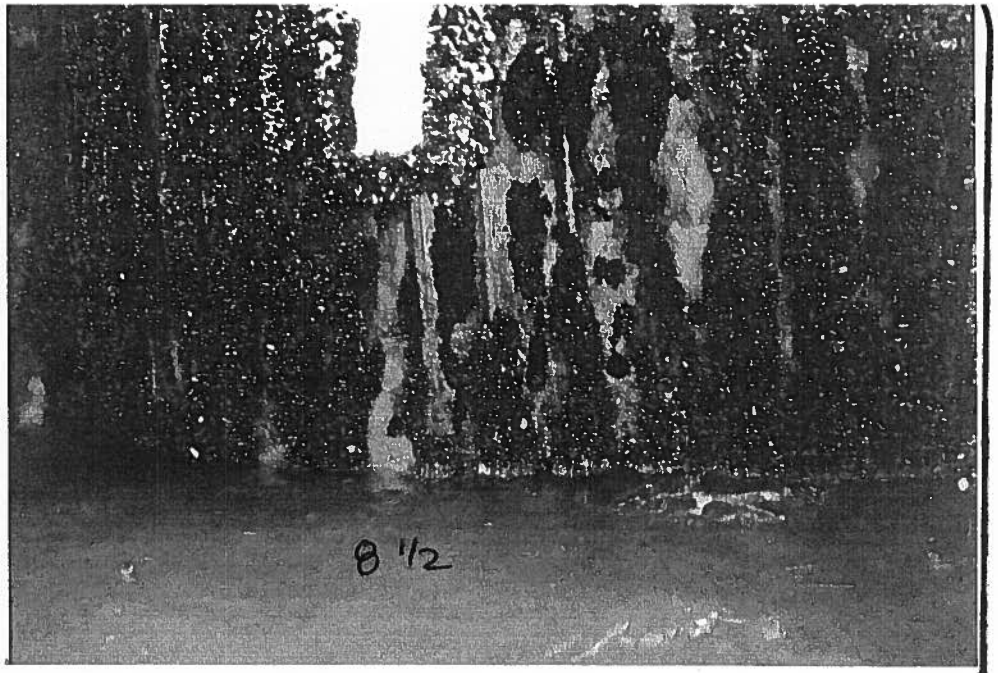
SEVERE CORROSION
NOTE WEEP HOLES

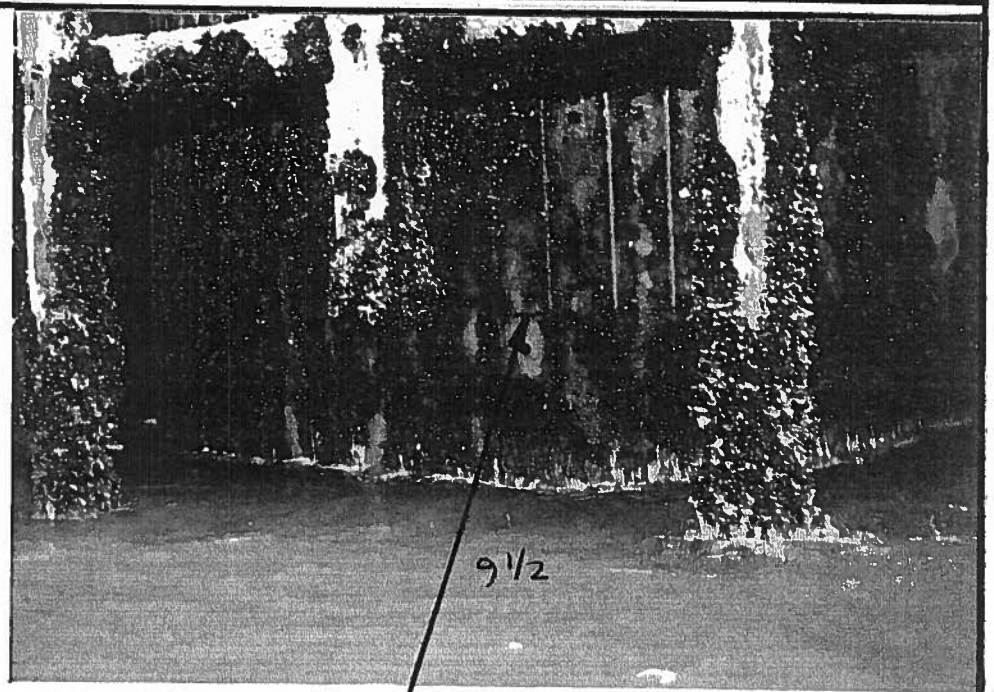
SEVERE CORROSION, NOTE FRAMING HAMMER



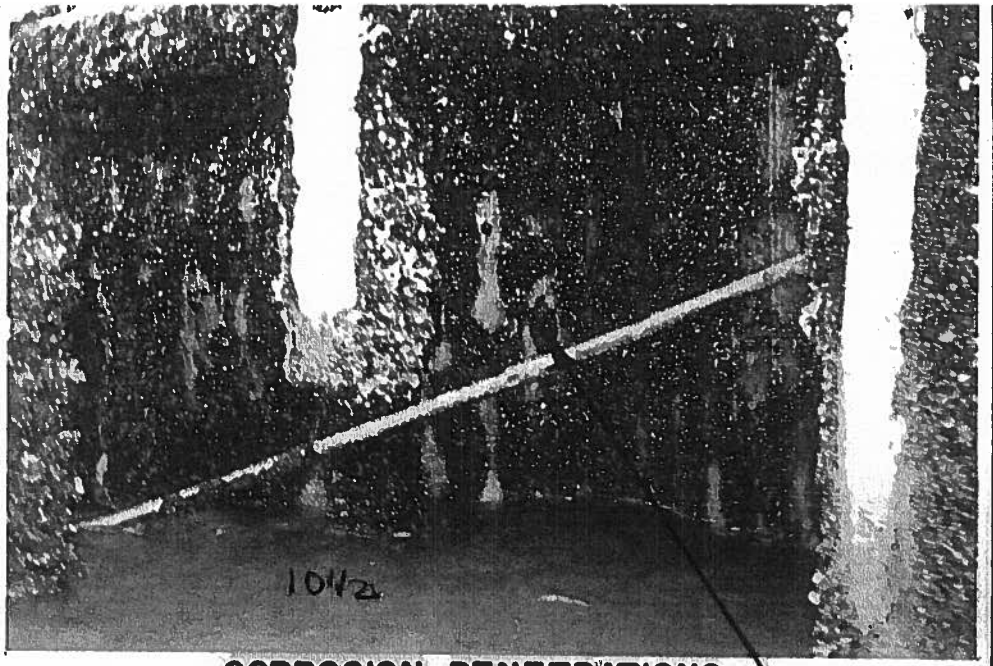


SEVERE CORROSION, NOTE FRAMING HAMMER





HEAVY CORROSION — SHEET PILE SPLICE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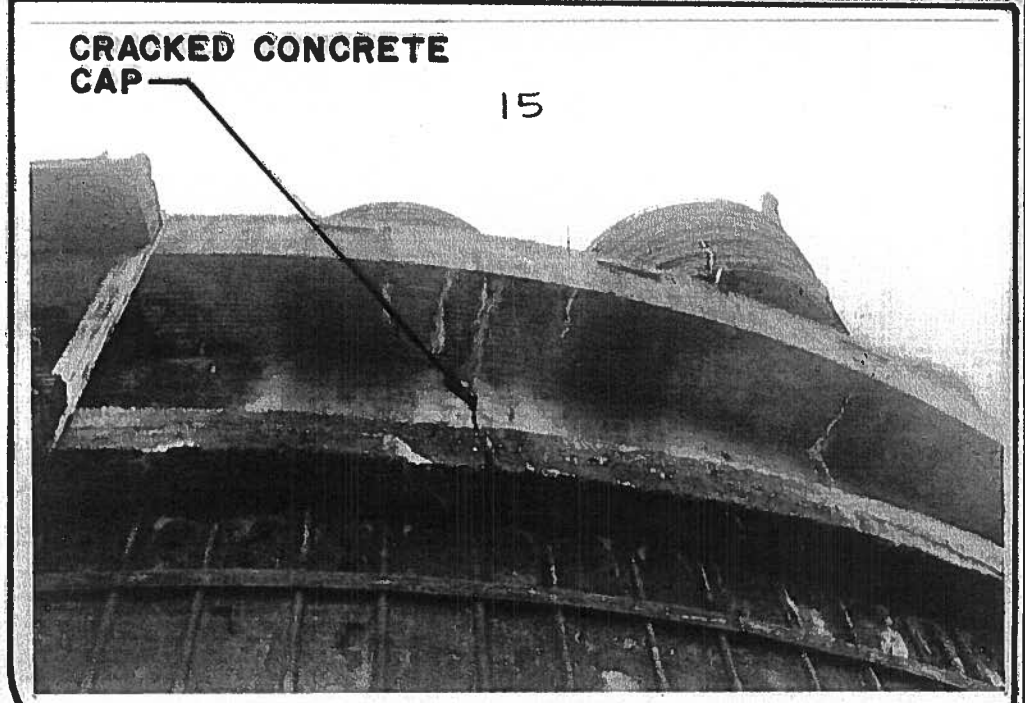
CORROSION PENETRATIONS


CRACKED CONCRETE CAP
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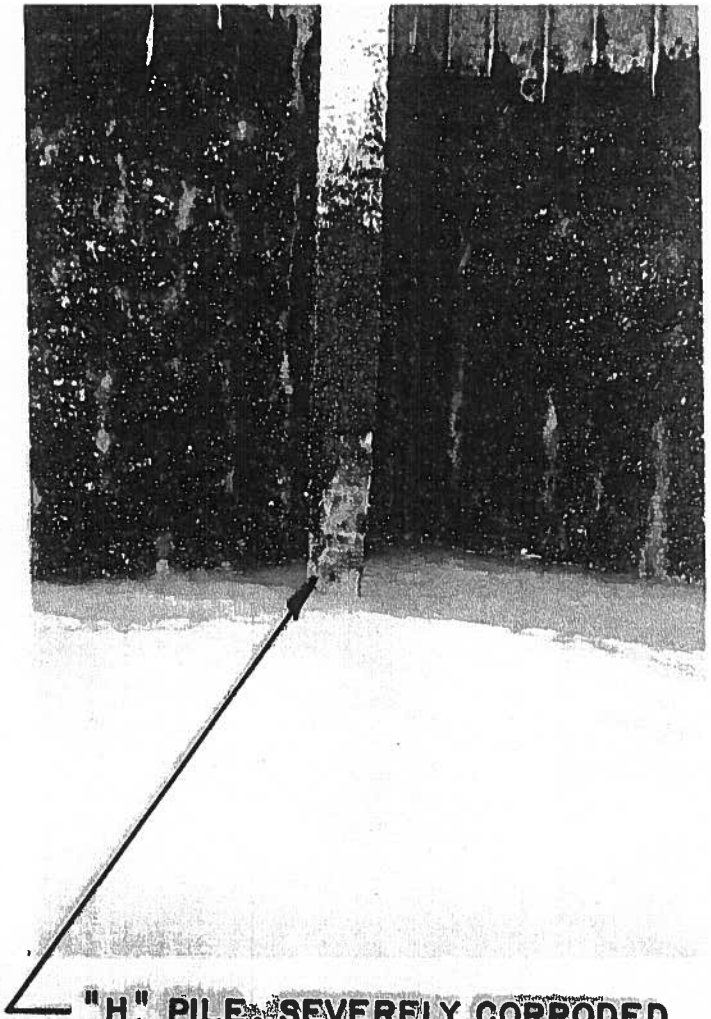
"H" PILE, SEVERELY CORRODED AT LOW TIDE.

REF #7

Haines-Lutak Inlet Port Facility Engineering Condition Report – Dated April 1976. Prepared by R&M Consultants.

R & M CONSULTANTS, INC.

ANCHORAGE
FAIRBANKS
JUNEAU

THREE TREES PROF. BLDG. • 6-1/2 MILE GLACIER HWY. • P. O. BOX 1786 • JUNEAU, ALASKA 99801 • TELEPHONE 907-789-0880 • TELEX 090-35419

98262
Background

April 5, 1976

State of Alaska
Department of Public Works
Division of Water and Harbors
130 Seward Street
Juneau, Alaska 99801

Attn: Mr. Don Statter
Director

Re: Haines-Lutak Inlet Port Facility
Engineering Condition Report
R & M Project No. 633110

Gentlemen:

We have completed work as per our agreement of March 24, 1976 regarding an evaluation of the Haines-Lutak Port Facility and are transmitting two copies of a report containing our findings.

Samples of steel sheet piling taken from the dock are also being transmitted.

Diving and survey work was hampered somewhat by the presence of ship log loading operations near the northwest end, however, we tried to work around this and get the best results possible.

We hope this report will give an accurate picture of the condition and remaining life of the basic structure and will be sufficient for your needs.

Should there be questions or if we may be of further service in any manner, please do not hesitate to contact us at your convenience.

Sincerely,

R & M CONSULTANTS, INC.

Dennis Nottingham, P.E.
Associate
DN:fej

for James A. Pung, P.E.
Civil Engineer

Attachment

ENGINEERING CONDITION REPORT

HAINES-LUTAK INLET PORT FACILITIES

(HAINES DRY CARGO DOCK)

R&M CONSULTANTS, INC.

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Appendix:

Table 110-A
Drawing No. 110-A
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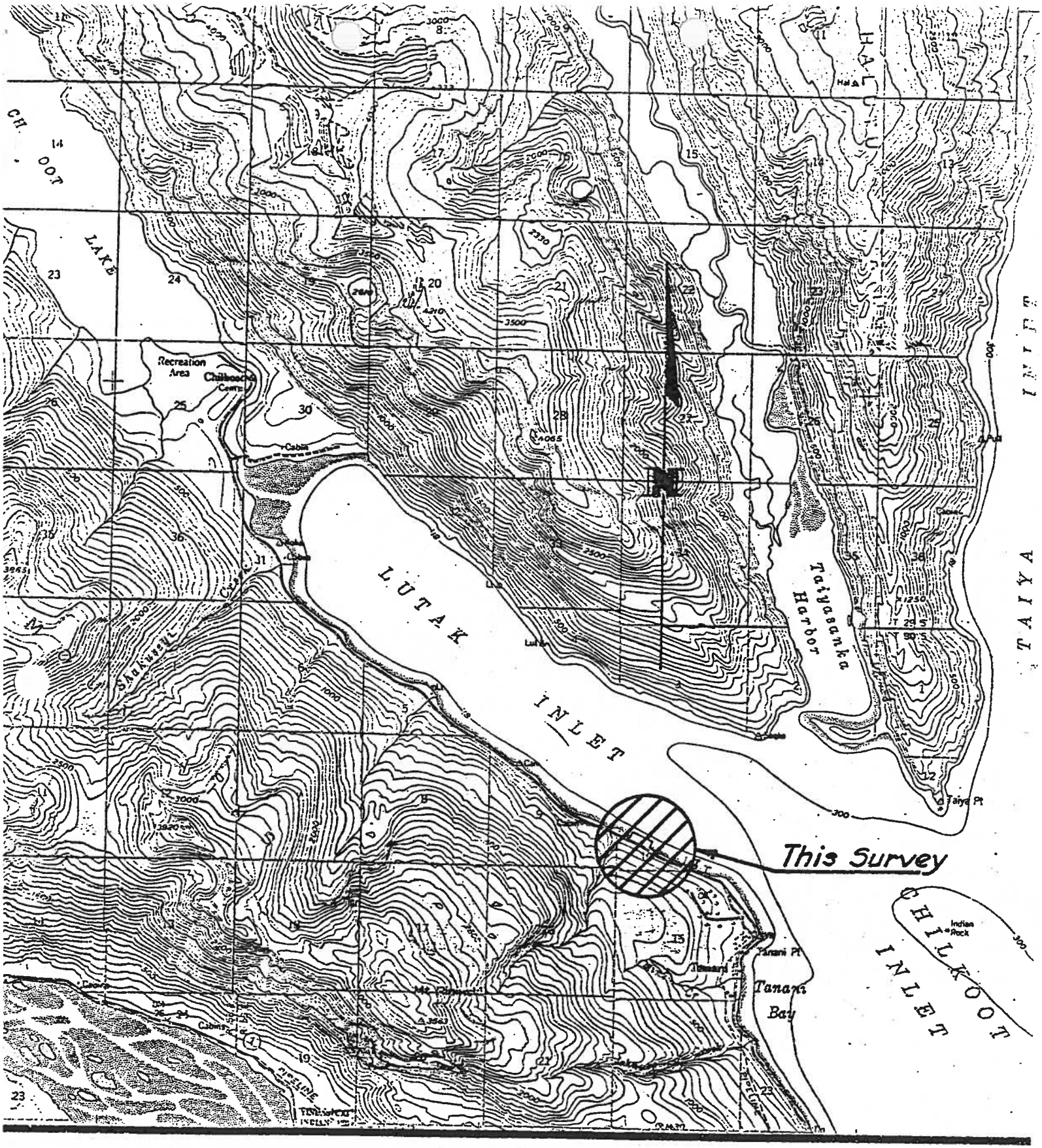


VICINITY MAP

Source : U.S.G.S. Quad. Skagway B-2

Scale = 1:63,360

Date: 1954 Rev.: 1963

ENGINEERING CONDITION REPORT

HAINES-LUTAK INLET PORT FACILITIES

(HAINES DRY CARGO DOCK)

INTRODUCTION

Civil engineering "condition" reviews requested of this firm by the State of Alaska, Department of Public Works, Division of Water and Harbors have been completed. The request for a condition engineering review of the Haines - Lutak Inlet Port Facility, owned by the General Services Administration, was requested by Mr. Don Statter, Director, Division of Water and Harbors on March 19, 1976 with a formal agreement being reached and initiated on March 24, 1976. Research of design and "as-built" records of this facility began immediately. Field reviews began on March 29, 1976 and were completed on April 4, 1976.

The following is our report of findings for this port facility condition study.

SCOPE OF WORK

By the contract for engineering services, the character and extent of said services is such that the engineer was to conduct such correspondence, research, site investigations, surveys and analysis as may seem necessary to complete a facilities condition report which specifically shall include, but not necessarily be limited to, the following:

1). Run vertical control surveys along the dock face to determine the amount of settlement of the dock area, if any.

2). Conduct sufficient horizontal location surveys to determine the amount of horizontal displacement within the dock, if any.

3). Obtain samples of the circular sheet piles by methods deemed feasible by the Engineer, ascertaining the condition of the circular sheet pile structure at four locations along the dock face at three elevations;

a). splash zone
b). approximate Elevation -15 MLLW
c). near bottom

These samples or measurements are to determine any sheet pile deterioration due to corrosion, electrolysis and/or other causes.

4). Perform visual inspection by a diver or divers of the entire dock face with particular attention focused on all sheet pile interlocks and welded joints.

5). Perform any other tests determined by the Engineer to be necessary to establish the existing structural integrity of the dock structure and probable remaining structure life.

As in many instances within Alaska, historic records for many facilities are limited to non-existent. We have obtained various "as-built" and design documentation from the U.S. Army, Corps of Engineers concerning the structural composition of this port facility. No records appear to exist relative to driving characteristics of the sheet piles; construction field survey notes; and/or ease/difficulty of construction for this facility. The following is our report of findings.

ENGINEERING SURVEYS (MOVEMENT)

Drawing dated July 30, 1953 of the "as-built" port facility by Schuman Johnson Manson-Osberg of Seattle, Washington was prepared by

W. Muncaster, L.S. of Haines, Alaska. This drawing has been used extensively to determine vertical/horizontal movement of the existing facilities.

A review of the drawing denotes that the elevation (vertical) control for construction was by U.S.C. & G.S. (United States Coast and Geodetic Survey) triangulation station "Pile" located within Lutak Inlet at approximate Latitude 69°17'N and Longitude 135°28'W. In employing this data, the field surveyor (Muncaster) had Mean Lower Low Water (MLLW) of 0.00' equal to Elevation 100. Field reviews were able to recover this triangulation station enabling necessary vertical control surveys for this project on the same (original) datum.

Vertical Control

Specifications employed on this project for vertical control surveys were those published by the Department of Commerce, National Ocean and Atmospheric Administration, National Ocean Survey in February 1974. The title of this specification is "Classification, Standards of Accuracy, and General Specifications of Geodetic Control Surveys". Table 3 - "Classification Standards of Accuracy and General Specifications for Vertical Control" was employed throughout this survey. In particular, Third Order, Class II measurements were made. Vertical control surveys were conducted utilizing as a basic survey instrument, a Zeiss Ni2 self-indexing level standardized for the procedures required. A Chicago, calibrated leveling rod with level bubble, was used in the rodding purposes. In all cases, three wire precision levels were conducted exceeding the particular order and class outlined for Third Order leveling

within the above referenced specifications. In doing this, programmed adjustments of loops were performed with balanced errors not exceeding .005 of a foot.

Horizontal Control

In addition to the vertical control surveys, a horizontal control base line was established utilizing as basic instrumentation a Wild T2E (one-second) Theodolite as well as a Hewlett-Packard Model 3800 short range electronic distance measuring device. Conducting this horizontal control survey, Table 2 - "Classifications, Standards of Accuracy and General Specifications for Horizontal Control (Traverse)" was followed throughout. In this instance, Second Order, Class I procedures were followed exclusively to determine movement, if any, of the wall face and the curb stop.

Surveys (General)

The vertical control survey was conducted such that fifteen to twenty-five foot lineal base "spot" measurements were conducted along the top of the concrete curb, integrally tied to the circular sheet pile structure. In addition, vertical lean of all main cells and spot checks on lean of connector arcs was done utilizing a four foot level with all measurements being taken at outer seaward face of sheet pile cells.

Results of these surveys are shown on Drawing No. 110-1.

The survey was conducted under the general instructions of M. A. Menzies, P.E., L.S. and executed by Mr. James Pung, P.E. and Mr. Robert Hungerford, Surveyor.

UNDERWATER SURVEYS

Underwater surveys were conducted on April 2, 3 and 4, 1976 by Messrs. Dennis Nottingham, P.E. and William Conyers, P.E. In addition to a general visual review of the main circular sheet pile cells and connector arcs, samples were extracted from the splash zone and the submerged zone which represent the condition of these circular sheet pile cells. By employing a K & E pneumatic drill underwater borings at specific areas on the circular cells were taken and splash zone and below mud line samples were taken using a cutting torch at low tide. The results of these sample thickness measurements are shown by Table 110-A.

Underwater work was conducted utilizing dry diving suits and Scuba (self contained underwater breathing apparatus) equipment. Two diver/engineers were employed for safety reasons while face position measurements of the diving team's location was conducted by the "land team".

DRY CARGO BULKHEAD AND EMBANKMENT REVIEW

Presently the Haines, G.S.A. Port Facility has duel utilization by individual corporations of the Alaska Timber Industry as well as State of Alaska, Department of Public Works, Division of Marine Transportation.

The westerly portion of this facility is used for loading/off-loading of various cargo facilities as well as cant and round logs by the Haines Sawmill to exporting ships. In addition, the easterly limit of the facility is used by the Division of Marine Transportation for loading/off-loading of passenger and vehicle traffic of Alaska's Marine Ferry System. Existing on the bulkhead embankment is the Marine Transportation ferry terminal which is a one-story, wood frame constructed building.

In 1973, R & M conducted an exploratory subsurface investigation of the bulkhead embankment to determine the feasibility of foundation conditions for the Marine Highway System ferry terminal building. For informational purposes a copy of the three test borings conducted, utilizing a CME 45C drill and rotary auger methods of subsurface exploration, are shown on Drawing No. 110-2.

The findings of this work conducted in 1973 denote that there are three soil types identified by two subsurface test borings. The uppermost soil was a fill material, usually constituting a sand of granitic origin. This material is pervious and has been compacted to a relatively high degree, undoubtedly by construction equipment and vehicle traffic over the parking area. The second soil type can be classified as a sandy gravel. The material source was apparently a sidehill cut of a nearby bluff. The gravel or rock portion of the material found within is a weathered green schist and surpentine. The degree of compaction of this material decreases with depth. The third soil type extending below Elevation 1'(MLLW) appears to be a natural beach deposit, in place prior

to embankment filling of the area. Density of this material can be classified as very good.

On the Log of Test Borings, both the material encountered at relative depths as well as the soil density, are shown by the split-spoon sampling "blow count". This blow count was achieved generally at 5' intervals or stratum changes of soil and was obtained by driving a 1.4" I.D. split-tube sampler in accordance with methods outlined by ASTM Test Designation 1586-67. This test utilizes a 140 pound drive hammer having a free-fall of 30". All samples were examined in the field by the geologist in charge of the drilling operation and logged in accordance with the Unified Soils Classification System.

A general, surficial review of the concrete curb and pile cap system denotes concrete of the highest quality was utilized for this construction. Very little pitting, erosion, popping or other detrimental conditions commonly found with pier and/or low air entrainment concrete are evident throughout the apron face.

The circular sheet pile cells with 90° tee connections, connector arcs and "H" piling, appear, at first glance, to be preserved quite well to the atmospheric and tidal action conditions that are existent within the harsh northern regions of Southeastern Alaska. Partial fender system replacement at the easterly end is currently underway as designed by the State of Alaska, Department of Public Works, Division of Water and Harbors.

The Haines - Lutak Inlet circular cofferdam structure is a self-supporting, gravity type system which is particularly applicable to large areas and deep water construction where both area and depth preclude the use of a single wall, internally braced structure. Cellular type sheet pile retaining walls are, as the name implies, complete closed circles of sheet piling. Each cell is an independent unit that is filled with a "gravity" mass of embankment material. The connecting arcs of circular sheet piles between cells are joined by a variety of methods. In the case of the Lutak Inlet port facility, the cells are interlocked and welded.

Driving dimensions rendered for various sheet piling profiles and plan dimension are nominal. Because of normal load tolerances and probable variations of on-sight conditions, sheet piles may be driven either short or long in a wall. Even when they are carefully lined-up and driven with a template variations in plan and vertical lean can be expected.

CORROSION

Steel corrodes because of its natural tendency to revert to a more stable electrical or chemical state and almost all metals exhibit this characteristic. While a corrosion process can be quite complex owing to the interaction of various natural and manmade factors, there is sufficient experience and data on hand to both understand and effectively control it. The type of environment that most piles may encounter include;

a). atmospheric (air)

b). soil (both disturbed and undisturbed)

c). water (fresh, polluted and salt)

Exposure of sheet pile structures in water can be described as occurring in one or more of five different exposure zones. These exposure zones are designated; atmospheric, splash, tidal, submerged and burial. These zones are denoted by Drawing No. 110-A. The overall performance of sheet piling can be judged by its weight loss and/or by its corresponding reduction of thickess in these various zones. While scattered pitting of various depths will occur, the overall significance of these pits is not usually critical in terms of piling performance. It is the load bearing ability or "beam" strength that is proportionate to the amount of metal remaining and availability for maintaining safety factors against structural yield forces.

Average thickness measurements or the average amount of metal remaining in critical stress areas and/or zones rather than appearance represents the criteria to be used in judging the corrosion effect the environment has on sheet piling and the potential service life of this structure.

Historically, steel piling have been continuously used with great success and with long life in a variety of applications. In areas of rural atmosphere, mild industrial conditions and relatively clean, fresh water, it is found that corrosion rates of sheet pile vary between two to about five mils per year for the first several years, after which,

the weight reduction or corrosion drops to a very low value. In retrospect, for local polluted conditions, heavy industrial atmospheres and heavier concentrations of sea (salt water), the life of sheet pile structures is reduced and may require some type of surface treatment to reduce exposure and increase the sheet pile life.

Various technical papers appearing in the American Society of Civil Engineer's magazine as well as publications by such corporations as United States Steel denote that in marine environment, examination of sheet piles over a multi-year period, the submerged zone has revealed that the corrosion rate averages about five mils per year for the first several years of exposure after which the rate tapers off to approximately two mils per year. It is apparent, therefore, that the underwater surface of plain carbon steel sheet pile structures have performed quite well in relatively clean salt water. The most critical zone for salt water service of sheet piles is the splash zone, that is the area from Mean Low Water (Tide) to the upper limits of wave action. This area experiences a corrosion rate several magnitudes in excess of either the atmospheric or submerged zones. Drawing No. 110-A denotes typical pile wall thickess loss at various zones for conventional A328 steel in the various stages of atmospheric/submerged and burial depths of the sheet pile. The life/loss charts are plotted for both five and nine years.

The Haines Lutak dock was reportedly, for a short time, provided protection by a proven method for that portion of the sheet pile which is

continually emersed in salt water. The protection method utilized was cathodic protection which utilizes the current flow obtained from an on-shore source or from submerged sacrificial metal. In either case, the corrosion of the submerged portion of the structure is usually arrested as long as the protective system is well maintained and operational. However, cathodic systems are only slightly effective in tidal zones and ineffective in the splash and atmospheric zones.

The "as-built" plans of the structure denote a 1/2" thick seaward surface cellular pile while the connecting arcs are reported as 3/8" thick. Our measurements denote that the loss of wall thickness is significant in the splash zone (see aforementioned table), especially at the connector arc portions. These arcs have a large number of hole cuts which appear to increase corrosion at these points.

STRUCTURAL STABILITY

From the "as-built" and pre-construction surveys, it can be shown that the existing ocean floor slope of Lutak Inlet near the dock has an average seaward trend of up to 40% (2.5:1). Subsurface soils investigation taken to an Elevation of approximately -8' Mean Lower Low Water, denotes a relatively dense in-place gravelly SAND throughout. It is felt that this gravelly SAND is relatively compressed by the glacial ice age last occurring in relatively recent geological times (10,000 years past). Comparison of soundings, underwater inspection and design drawings indicate the stability of this material is significant to withstand

erosion from tidal current from Lynn Canal and Lutak Inlet and ship induced turbulence as well as imposed dock loads.

Results of vertical and horizontal surveys shown on Drawing No. 110-1 indicate:

1). Little relative elevation change along the central portion of the dock with deviations near each end of up to about four inches downward.

2). Utilizing "as-built" plan datum and elevations as a base all present elevations appear to be approximately one foot lower. This could be due to very uniform settlement or some deviation in the base datum utilized. No effort was made to verify base datum from tidal observations because of wind and seas.

3). Vertical plumb measurements indicate that main cells for the most part are near vertical, some having slight back slope and some with slight seaward lean. All appear to have been built near to these positions as plan deviation of the curb line from the average is three inches or less and a minimum of concrete cracking is noticeable. "As-built" plans also indicate fairly large dimension deviations.

4). Sheet pile connector arcs almost all lean seaward and appear to have been built in this position or attained this position during fill operations before top concrete was placed.

From the above discussion, it appears that stability of the cellular system as a whole is intact with some movement being in evidence. Some long term soil creep and movement should be expected when a soil mass such as placed in this type of cellular structure imposes large stress on underlying materials. Since records of movement and settlement with time are not available to absolutely guarantee that soil creep and movement is stabilizing, it may be desirable to periodically monitor this dock in the future.

Component-wise deterioration of sheet piling in splash zone above low tide is beginning to be a consideration in some areas. Connector arc

sheets have been reduced in thickness in some places to less than 2/3 of the original. Structurally speaking loss of these components would not seriously endanger the dock as a whole, but would add greatly to maintenance and repair expense.

Main cell exposed face sheets in general have not yet deteriorated to the 3/8" thickess used in fill areas, thus these must still be considered structurally sound at this time. Interlocks and welds inspected appear to be in good condition. All steel that is below scour effects of wave and debris action is coated with two protective layers derived from chemical action, the outermost being a reddish color and the inner layer a black color overlying fairly bright to grey uniform appearing steel sheet piles. Table 110-A indicates that steel under this coating has a lower thickness rate loss than steel exposed to elements that may damage formation of this material.

"H" piling supporting portions of concrete top are in good structural condition with the exception of a few that appear to have been bent near points where fender systems had been attached.

Concrete is in adequate structural shape with a minimum of cracks and other deterioration.

At this time there is every indication that this dock, excluding the fender system, is in safe structural condition.

STRUCTURAL LIFE

Ultimate structural life of this sheet pile cellular structure will

probably depend on the integrity of seaward main cell piling in the splash zone. High repair costs will probably occur when connector arc sheet piling on the seaward face become severely corroded in the splash zone above or near low tide. High repair costs will also be associated with the present deteriorated fender system although this study is not a part of this report.

Based on limited thickness measurements and rates of thickness loss over 23 years at this location approximate estimates of remaining life can be made.

Using a soil weight of 100 psf, live load surcharge of 600 psf and an active soil pressure coefficient of 0.3 a design interlock pressure of about 3 kips/inch is calculated at MLLW in main cells. This should be considered an absolute minimum design load since tidal fluxuation could create water pressure within the cell which could produce pressure up to twice the minimum value. Stress in 3/8" main cell sheets at MLLW could then range from 8 to 16 ksi.

Safe allowable sheet stresses should probably not exceed 25 ksi to 30 ksi at the end of its structure life. Thus on the conservative side main cell sheets should not be allowed to deteriorate much more than to about 1/4" in thickness. At a conservative rate of 7 mils per year and with a present thickness of 0.40 inches this could take 20 more years.

Lateral pressure in connector arcs is about 1/3 that in main cells thus indicating that only about 1/8" or less of sheet thickness is required.

At such thin thicknesses and with the presence of numerous perforations in the piling, vertical load support would be impaired and corrosion could also advance at higher rates. Average thicknesses less than 3/16" at the end of its structure life should not be considered desirable. Using the same corrosion rates as the preceeding and starting with a present effective thickness of 0.25 inches potential maintenance problems could develop in 8 to 10 years.

REPLACEMENT COSTS

Sheet pile cell type docks are expensive to build but do offer an excellent solution capable of handling large storage and transfer loads.

Excluding fender systems but including sheet piling, concrete and fill a basic dock and fill of the type at Lutak Inlet could be expected to cost five to six million dollars to replace new and in kind. Very approximate quantities and unit costs are listed:

Item	Units	Unit Cost	Cost
Steel Piling	6,000,000 lb.	$0.50	$3,000,000
Concrete Cap (slab not included)	2,000 yd.	$400	$ 800,000
Fill	300,000 cu.yd.	$5	$1,500,000
	Total		$5,300,000

CONCLUSIONS

Based on results of land and underwater surveys and engineering computations the following points appear valid:

1). There has been some vertical and horizontal movement occuring in the sheet pile dock structure or underlying soil support. The magnitude of this movement is not large (a matter of a few inches) and at this time does not appear to be a problem. Since no time-movement records were examined it is recommended that after a period of time another survey be done to establish rates of movement and to further verify the system stability.

2). Examination of "as-built" plans and field observations indicate that sheet pile cells were constructed with some deviation from vertical and design plan dimensions.

3). Most severe corrosion of seaward sheet piling occurs near the lower tide levels and is considerably less below this elevation. Apparent maintenance efforts in the past including painting in the splash zone have helped reduce corrosion as some paint is still evident at higher levels. A cathodic protection system installed in the past is not operable.

4). Maximum remaining structural life of the main cell system is estimated to be about 20 years.

5). Remaining life of sheet pile connector arcs is estimated to be less than 10 years. Maintenance efforts may be required in these areas before the 20 year remaining structural life of the basic main cells can be realized.

6). Concrete portions of this dock should be capable of lasting up to 20 more years with some maintenance.

7). Existing fender systems over the majority of the dock are deteriorated and inadequate and require extensive repair.

APPENDIX

TABLE 110-A

I. Samples from 3/8"(0.38) Connector Arcs

Elevation	Sample No.	Thickness	Thickness Loss	23 Year Average mil/year loss
*El. +4±	1	0.29	0.10	4
*El. +2±	2	0.22	0.16 (Near Hole)	7
**El. -15±	3	0.31	0.07	3
**El. -30±	4	0.34	0.04 (Near Mud Line)	2

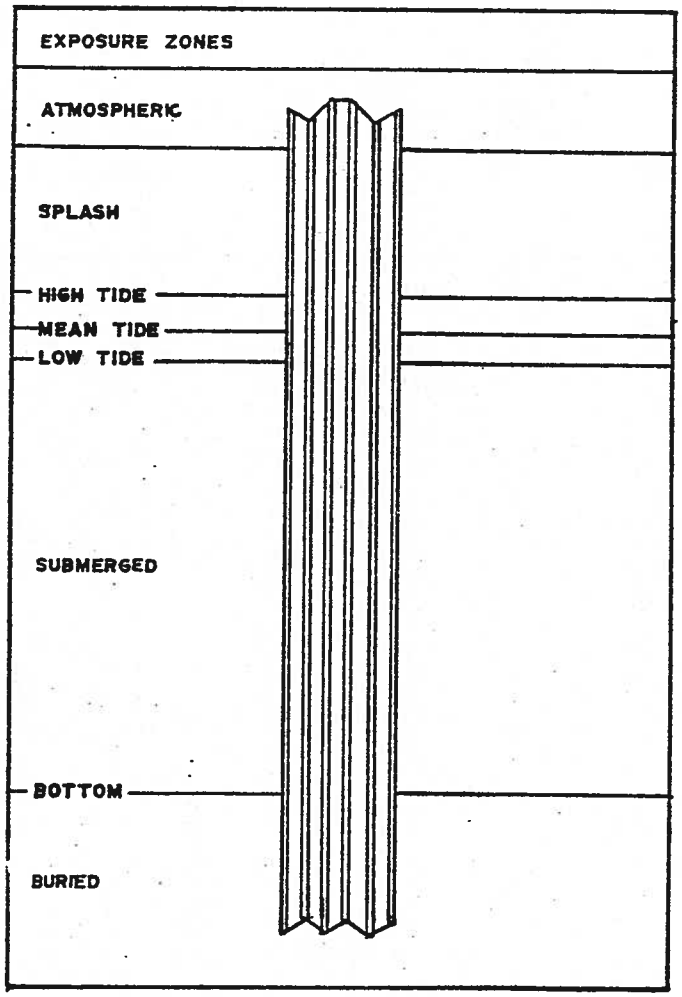
II. Samples from 1/2" (0.50) Main Cells

Elevation	Sample No.	Thickness	Thickness Loss	23 Year Average mil/year loss
*El. +4±	5	0.44	0.06	3
*El. +3±	6	0.36	0.14	6
*El. +3±	7	0.41	0.09	4
*El. +2±	8	0.41	0.09 (Below Mud Line)	4
*El. +2±	9	0.41	0.09 (Below Mud Line)	4
**El. -5±	10	0.44	0.06	3
**El. -15±	11	0.47	0.03	1
**El. -30±	12	0.47	0.03 (Near Mud Line)	1

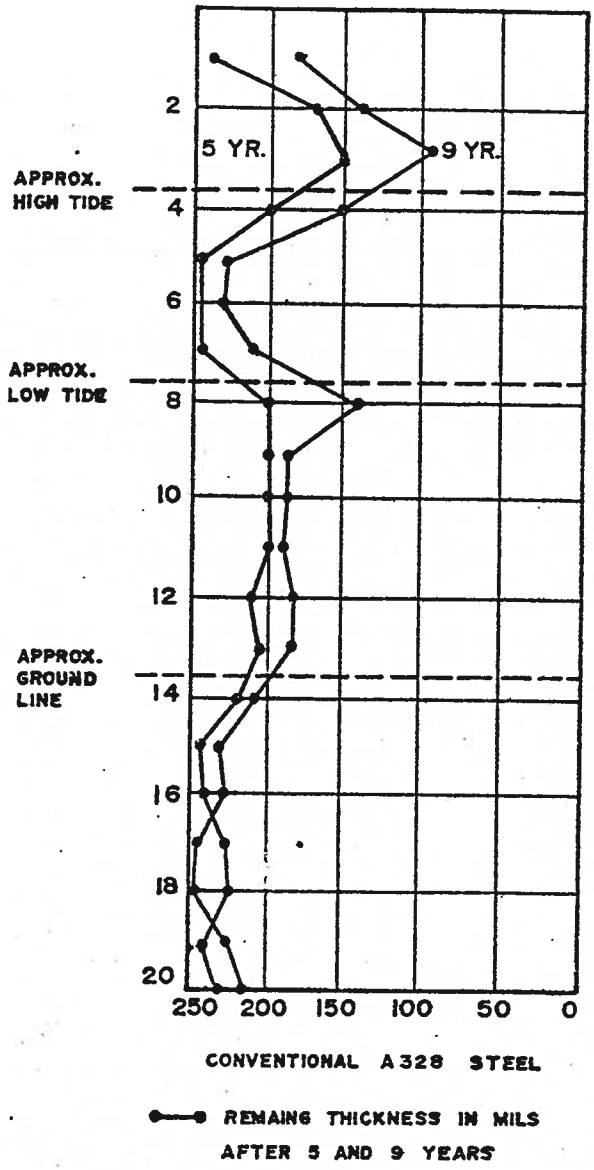
*Samples taken by cutting torch at low tide.

**Samples taken by underwater drilling and thickness measurement.

FIVE EXPOSURE ZONES IN SEA WATER ENVIRONMENTS



STEEL CORROSION RATE IN MARINE ENVIRONMENTS



TYPICAL CORROSION RATE AT VARIOUS ENVIRONMENTAL ZONES

SOURCE - A.S.C.E. / U.S.S.

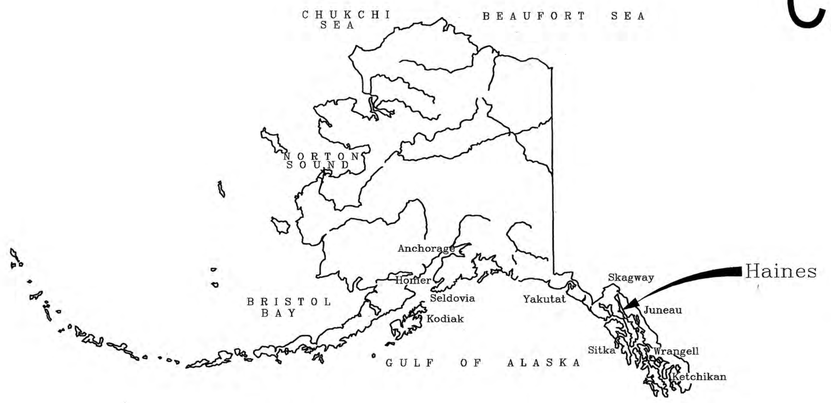
R&M CONSULTANTS, INC.

DATE 4-2-76	SCALE NONE	DWN BY D.B.	CHKD BY M.M.	PROJ. NO 633110	DWG NO. 110-A

REF #8

Lutak Dock Rehabilitation Project Drawings. Prepared by Reid Middleton. Dated November 2003.

LUTAK DOCK REHABILITATION PROJECT
CITY OF HAINES, ALASKA
EDA PROJECT NO. 07-79-4967



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LIST OF DRAWINGS

G1	COVER SHEET
G2	GENERAL NOTES
G3	CONTRACTOR WORK AND STAGING AREAS
C1	EXISTING SITE PLAN
C2	SITE GRADING PLAN
C3	CIVIL DETAILS
D1	DEMOLITION PLAN
D2	DREDGING PLAN
D3	DREDGE DISPOSAL SITE PLAN AND SECTIONS
S1	FENDER LAYOUT PLAN AND DETAILS
S2	FENDER DETAILS
S3	CLOSURE ARC MODIFICATIONS
S4	MISCELLANEOUS DETAILS
S5	RO-RO TRNSFER BRIDGE
CP1	CATHODE PROTECTION DETAILS
CP2	CATHODE PROTECTION NOTES AND DETAILS

STRUCTURAL ABBREVIATIONS

&	And	DB	Diaphragm Boundary	GA.	GA.	(N)	New	SOG	Slab-on-Grade
@	At	DBL	Double	GALV	Galvanized	NFS	Non-Frost Susceptible	SPCG	Spacing
℄	Center Line	DEMO	Demolition	GLB	Glue-Laminated Beam	NIC	Not in Contract	SPEC	Specification
		DIAG	Diagonal	GR	Grade	NO	Number	SQ	Square
AB	Anchor Bolt	DICA	Drilled-in-Concrete Anchor	GWB	Gypsum Wallboard	NOM	Nominal	SSH	Short Slotted Hole
ADJ	Adjustable, Adjacent	DIM	Dimension			NS	Near Side	STT	Stainless Steel
AGGR	Aggregate	DIMA	Drilled-in-Masonry Anchor	HD	Hold-down	NTS	Not to Scale	STD	Standard
ALT	Alternate	DL	Dead Load	HDG	Hot-Dip Galvanized			STFNR	Stiffener
ALUM	Aluminum	DN	Down	HDR	Header	OA	Overall	STL	Steel
ANCH	Anchor, Anchorage	DO	Ditto	HOR	Horizontal	OC	On Center	STOR	Storage
APPROX	Approximate	DTL	Detail	HT	Height	OD	Outside Diameter (Dim.)	STRUCT	Structural
ARCH	Architectural, Architect	DWG	Drawing	HSL	Horizontal Long Slotted Holes	OPNG	Opening	SYM	Symmetrical
ASTM	American Society for Testing and Materials					OPP	Opposite		
		(E)	Existing	ID	Inside Diameter	OSB	Oriented Strand Board	T	Top
AWW	All Weather Wood	EA	Each	IN OR "	Inch			TEMP	Temporary
		EC	Epoxy Coat	INCL	Include	P	Plate	T&B	Top & Bottom
BLDG	Building	EF	Each Face	INSUL	Insulation	PAR	Parallel	T&G	Tongue & Groove
BLK	Block	EJ	Expansion Joint	INT	Interior	PC	Pre-Cast	THK or t	Thick
BLKG	Blocking	ELEC	Electrical	INTMD	Intermediate.	PERIM	Perimeter	THRU	Through
BM	Beam	ELEV	Elevation			PLF	Pounds Per Linear Feet	TO	Top of
BOT	Bottom	ENG	Engineer	JST	Joist	PLYWD	Plywood	TOB	Top of Beam
BO	Bottom of	EQ	Equal, Earthquake	JT	Joint	PMJ	Pre-molded Joint	TOC	Top Of Concrete
BRG	Bearing	EQUIP	Equipment			PSF	Pounds Per Square Foot	TOS	Top of Steel
BSMT	Basement	EXP	Expansion	K	Long, Length, Angle	PSI	Pounds Per Square Inch	TOW	Top of Wall
BTWN	Between	EXST	Existing	KSI	Development Length	PT	Point	TRANS	Transverse
		EXT	Exterior			PT	Pressure – Treated	TS	Tube Steel
C	Channel Section	EW	Each Way	L	Long, Length, Angle			TYP	Typical
CIP	Cast-in-Place			LKD	Development Length	QTY	Quantity		
CJ	Control Joint	FB	Flat Bar	LLH	Long Leg Horizontal			UBC	Uniform Building Code
CLG	Ceiling	FD	Floor Drain	LLV	Long Leg Vertical	R	Radius, Reaction	UNO	Unless Noted Otherwise
CLR	Clear	FDN	Foundation	LOC	Location	REF	Reference, Refer		
CMU	Concrete Masonry Unit	FF	Finished Floor	LONG	Longitudinal	REINF	Reinforced	VER	Verify
COL	Column	FIN	Finish			REQ'D	Required	VERT	Vertical
CON JT	Construction Joint	FLR	Floor	MATL	Material	REV	Revision	VEST	Vestibule
CONC	Concrete	FO	Face of	MAX	Maximum	RMV	Remove	VSL	Vertical Slotted Hole
CONN	Connection	FOBM	Face of Beam	MC	Moment Connection/Misc. Channel	RM	Room		
CONSTR	Construction	FOC	Face of Concrete	MECH	Mechanical	RO	Rough Opening	WH	Weep Hole
CONT	Continuous	FOS	Face of Steel	MEMB	Membrane			WHS	Welded Headed Stud
CONTR	Contractor	FRMG	Framing	MFR	Manufacturer	SCHED	Schedule	W/	With
COORD	Coordinate	FS	Far Side	MID	Middle	SECT	Section	W/O	Without
CP	Complete Penetration	FT	Foot or Feet	MIN	Minimum	SH PL	SHEAR PLATE		Wide flange, Wide, Width
CTJ	Control Joint, Contraction Joint	FTG	Footing	MISC	Miscellaneous	SHT	Sheet	WD	Wood
CTR	Center	FV	Field Verify	MTD	Mounted	SIM	Similar	WP	Work Point, Water Proofing
CTSK	Countersunk			MTL	Metal	SL	Snow Load, Slope	WWF	Welded Wire Fabric

REVISION | BY | DATE | NO.

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LUTAK DOCK REHABILITATION PROJECT
CITY OF HAINES – EDA PROJECT NO. 07-79-04967

COVER SHEET

DWG

DES. CF/JJ | SHEET NO.
DR. CSB | G1
CH. CF/JJ | or 16 SHEETS
P.B.
DATE 05/02/02
JOB NO. 40-02-010

GENERAL NOTES

GENERAL

THE CONTRACTOR SHALL VERIFY AND COORDINATE ALL DIMENSIONS AMONG THE DRAWINGS BEFORE STARTING ANY WORK OR FABRICATION. ANY DISCREPANCIES FOUND AMONG THE DRAWINGS, SITE CONDITIONS, SPECIFICATIONS, AND THESE NOTES SHALL BE REPORTED TO THE OWNER'S REPRESENTATIVE.

ALL CONSTRUCTION SHALL COMPLY WITH THE STATE OF ALASKA DEPARTMENT OF TRANSPORTATION AND PUBLIC FACILITIES STANDARD SPECIFICATIONS.

SAFETY – THE CONTRACTOR IS RESPONSIBLE FOR MEETING ALL O.S.H.A. SAFETY STANDARDS. THE CONTRACTOR IS IN CHARGE OF ALL SAFETY MATTERS ON AND AROUND THE JOB SITE.

DEMOLITION

CONTRACTOR SHALL SUBMIT A DEMOLITION PLAN PRIOR TO BEGINNING WORK. PLAN SHALL CLEARLY DESCRIBE MEANS AND METHODS OF DEMOLITION AND IDENTIFY TO WHERE THE MATERIALS ARE TO BE TRANSPORTED AND BY WHAT MEANS.

THE EXISTING TIMBER FENDERS TO BE DEMOLISHED SHALL BE DISPOSED OF AT A LANDFILL AUTHORIZED TO RECEIVED CREOSOTE TREATED MATERIAL OR SHALL BE SALVAGED FOR REUSE BY THE CONTRACTOR. ALL OF THE TIMBER AND TIMBER PILES OF THE FENDERING SYSTEM ARE BELIEVED TO BE CREOSOTE TREATED.

THE NEW H–PILES, SHEET PILES AND CAP BEAM MODIFICATIONS SHALL BE COMPLETE BEFORE REMOVAL OF THE UPPER SECTION OF THE (E) CLOSURE ARCS AND THE FILL BEHIND THEM.

METHOD OF REMOVAL OF THE FILL BEHIND THE DEMOLISHED PORTION OF THE CLOSURE ARCS SHALL BE SUCH THAT THE CONDITIONS OF THE US ARMY CORPS OF ENGINEERS PERMIT FOR THIS PROJECT ARE NOT VIOLATED.

STRUCTURE DESIGN DATA

STRUCTURE MODIFICATIONS HAVE BEEN DESIGNED FOR THE FOLLOWING OPERATIONAL LOADS ON THE COMPLETED STRUCTURES. ADEQUACY OF EXISTING STRUCTURES TO SUPPORT THESE LOADS HAS NOT BEEN VERIFIED.

LIVE LOADS:

RO–RO BRIDGE MODIFICATIONS:	130 KIP AXLE WEIGHT
CLOSURE ARC MODIFICATIONS:	1000PSF UNIFORM LOAD OR CRANE TRACK PRESSURE OF 3500 PSF OVER AREA 4'X23'
FENDER SYSTEM:	VESSEL IMPACT ENERGY OF 230,000 FT – LBS @ 0 DEGREES

SPECIAL INSPECTION

THE FOLLOWING WORK SHALL BE INSPECTED BY A SPECIAL INSPECTOR:

1. CONCRETE. DURING THE TAKING OF TEST SPECIMENS AND PLACING OF REINFORCED CONCRETE.
2. BOLTS INSTALLED IN CONCRETE. PRIOR TO AND DURING THE PLACEMENT OF CONCRETE AROUND BOLTS.
3. REINFORCING STEEL. DURING PLACING OF REINFORCING STEEL. THE SPECIAL INSPECTOR NEED NOT BE PRESENT CONTINUOUSLY DURING PLACING OF REINFORCING STEEL PROVIDED THE SPECIAL INSPECTOR HAS INSPECTED FOR CONFORMANCE TO THE APPROVED PLANS PRIOR TO THE CLOSING OF FORMS OR THE DELIVERY OF CONCRETE TO THE JOBSITE.
4. STRUCTURAL WELDING. DURING THE WELDING OF ANY MEMBER, EXCEPT FOR WELDING DONE IN AN AISC APPROVED FABRICATOR'S SHOP. THE SPECIAL INSPECTOR NEED NOT BE CONTINUOUSLY PRESENT DURING WELDING OF SINGLE–PASS FILLET WELDS NOT EXCEEDING 5/16 INCH (7.9 MM) IN SIZE, PROVIDED THE MATERIALS, QUALIFICATIONS OF WELDING PROCEDURES AND WELDERS ARE VERIFIED PRIOR TO THE START OF WORK; PERIODIC INSPECTIONS ARE MADE OF WORK IN PROGRESS; AND A VISUAL INSPECTION OF ALL WELDS IS MADE PRIOR TO COMPLETION OR PRIOR TO SHIPMENT OF SHOP WELDING:
5. PILES. DURING DRIVING OF PILES

FOUNDATION NOTES

ALLOWABLE SOIL BEARING PRESSURE:	3000 PSF
CAP SUPPORT H–PILE CAPACITY REQ'D:	400 KIPS

REFER TO SOILS REPORT FOR PILE INSTALLATION, SUB–GRADE PREPARATION, AND LATERAL EARTH PRESSURES.

STRUCTURAL CONCRETE NOTES

ALL CONCRETE SHALL HAVE A WATER REDUCING ADMIXTURE MEETING ASTM C 494, TYPE A (TYPE E FOR COLD WEATHER CONCRETING) AND NOT MORE THAN 0.1 PERCENT CHLORIDE IONS. MAXIMUM WATER/CEMENT RATIO SHALL BE 0.40. MAXIMUM SLUMP BEFORE ADDING THE RANGE WATER REDUCING ADMIXTURE SHALL BE THREE INCHES. MAXIMUM AGGREGATE SIZE SHALL BE ¾". COLD WEATHER CONCRETING SHALL CONFORM TO ACI SPECIFICATION 306.1 AND ACI 306R. CALCIUM CHLORIDE SHALL NOT BE USED. MINIMUM CEMENT CONTENT SHALL BE 5–½ SACKS.

CAST–IN–PLACE CONCRETE:	f'c = 4,000 PSI.
AIR ENTRAINING ADMIXTURE:	ASTM C260
AGGREGATE:	ASTM C33
EPOXY ADHESIVE:	ASTM C881
REINFORCING BARS:	ASTM A 615, GRADE 60.
WATER:	ASTM C94, SECTION 4.1.3
CEMENT:	ASTM C150

THE FOLLOWING MINIMUM CONCRETE COVER SHALL BE PROVIDED FOR REINFORCEMENT FOR CAST–IN–PLACE CONCRETE:

A. CONCRETE CAST AGAINST AND PERMANENTLY EXPOSED TO EARTH	3 INCHES
B. ALL OTHER CONCRETE	2 INCHES

ALL WELDED WIRE FABRIC (WWF): ASTM A 185 OR ASTM A 497. CONTRACTOR SHALL TAKE SPECIAL CARE TO MAKE SURE WWF IN SLABS–ON–GRADE IS SUPPORTED IN ITS PROPER LOCATION.

WHERE REQUIRED, DOWELS SHALL MATCH SIZE AND QUANTITY OF MAIN REINFORCING.

NO WELDING OF REBAR IS ALLOWED WITHOUT ENGINEER'S APPROVAL.

DRILLED IN ADHESIVE ANCHORING

DRILLED IN ADHESIVE ANCHORING FOR BOTH ANCHOR BOLTS AND REBAR SHALL BE DONE WITH HILTI HVA 2421 EPOXY ADHESIVE OR AN APPROVED EQUAL. UNLESS OTHERWISE NOTED MINIMUM EMBEDMENT INTO CONCRETE SHALL BE 8 DIAMETERS. ALL INSTALLATION SHALL CONFORM TO THE MANUFACTURER'S INSTRUCTIONS.

STRUCTURAL STEEL NOTES

STRUCTURAL STEEL:	ASTM A 36, UNLESS OTHERWISE NOTED.
STRUCTURAL STEEL TUBES (HSS):	ASTM A 500, GRADE B.
STRUCTURAL STEEL PIPES:	ASTM A 53, GRADE B.
BOLTS:	ASTM A 325 EXCEPT WHERE NOTED A307
HARDENED WASHERS:	ASTM F 436
NUTS:	AS RECOMMENDED BY ASTM IN THE BOLT SPECIFICATION
WELDED HEADED STUDS:	ASTM A 108
THREADED RODS:	ASTM A 36 OR A307
NON–SHRINK GROUT:	ASTM C 1107
WELDING ELECTRODES :	E70, LOW–HYDROGEN

GALVANIZING: ALL STEEL FABRICATION AND PILING (EXCEPT SHEET PILES) SHALL BE GALVANIZED IN ACCORDANCE WITH ASTM A 123, UNLESS OTHERWISE NOTED.

STEEL SHEET PILES

SHEET PILES SHALL BE "Z" – SHAPED, SHALL HAVE A 50 KSI MINIMUM YIELD STRENGTH, AND SHALL, AS A MINIMUM, HAVE THE FOLLOWING SECTION PROPERTIES PER LINEAL FOOT OF WALL:

AREA IN^2	SECTION MODULUS IN^3	MOMENT OF INERTIA IN^4	WEIGHT PSF
11.2	46.8	281	38

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NO.	DATE	BY	REVISION

LUTAK DOCK REHABILITATION PROJECT
CITY OF HAINES – EDA PROJECT NO. 07–79–04967

GENERAL NOTES

DWG

DES. CF/JJ | SHEET NO. G2
DR. CSB
CH. CF/JJ
F.B.
of 16 SHEETS

DATE 05/02/02

JOB NO. 40–02–010

NOTES:

1. THE CONTRACTOR MUST COMPLETE REPAIR OF CLOSURE ARCS 6–10 AND INSTALL THE 5 NEW FENDER UNITS BEFORE PERFORMING REPAIRS ON CLOSURE ARCS 1–5.

2. WHILE WORK IS PROCEEDING ON REPAIR OF CLOSURE ARCS 1–5 BARGE CARGO ACTIVITIES WILL BE RESTRICTED TO THE EAST WORK AREA. WHILE WORK IS PROCEEDING ON REPAIR OF CLOSURE ARCS 6–10 BARGE CARGO ACTIVITIES WILL BE RESTRICTED TO THE THE WEST WORK AREA.

3. RESURFACING WORK CAN PROCEED CONCURRENTLY WITH THE DOCKSIDE REPAIRS BUT MUST FOLLOW THE SAME SEQUENCE AS THE CLOSURE ARCS, THAT IS DO THE WORK EAST OF THE FUEL HEADER FIRST AND WHEN THAT IS COMPLETE DO WORK WEST OF THE HEADER.

4. DREDGING AND FENDER DEMOLITION MAY PROCEED CONCURRENTLY WITH THE OTHER OPERATIONS. HOWEVER, THE WORK MUST ACCOMMODATE ARRIVAL AND OFF–LOADING OF SCHEDULED VESSEL TRAFFIC.

5. CONTAINERS AND OTHER EQUIPMENT RELATED TO CARGO ACTIVITIES MUST BE STORED OUTSIDE THE CONTRACTOR'S ACTIVE WORK AREAS.

6. THE CONTRACTOR SHALL PROVIDE THE CITY WITH A SCHEDULE SHOWING THE TIMES HE WILL BE OPERATING IN THE VARIOUS WORK AREAS AND NOTIFY THE CITY IMMEDIATELY OF ANY CHANGES IN THE SCHEDULE BUT NOT LESS THAN 72 HOURS FROM THE POSTED SCHEDULE.

7. THE CONTRACTOR STAGING AREA IS FOR THE EXCLUSIVE USE OF THE CONTRACTOR THROUGHOUT THE CONSTRUCTION PERIOD.

8. CONTRACTOR WILL ARRANGE AND PAY FOR TEMPORARY POWER AND PHONE THROUGH ALASKA POWER AND TELEPHONE.

9. NON–POTABLE WATER IS AVAILABLE FOR THE CONTRACTORS USE DURING CONSTRUCTION WHERE SHOWN.

LUTAK INLET

TUG

150'x60' FUEL BARGE

ELECTRICAL BOX AND WATER SPIGOT

EXISTING CONCRETE CAP ELEVATION 28.98'

BARGE RAMP

BOAT RAMP

CONTRACTOR SHALL FIELD LOCATE AND PROTECT EXISTING FACILITIES

WATER BOX

NON–POTABLE WATER FOR CONSTRUCTION

POWER POLE

DOCK OFFICE

TRACT A

FR. Lot 5 City of Haines

COMMON AREA

WEST WORK AREA

EAST WORK AREA

CONTRACTOR STAGING AREA

STAGING AREA

ACCESS ROAD

15 FT ACCESS FOR USE BY DELTA WESTERNFUEL TRUCKS DURING CONSTRUCTION

40' R

POINT 102

EXISTING 10' CHAIN LINK FENCE

ACCESS ROAD

20' R

EXISTING 6' CHAIN LINK FENCE

LUTAK ROAD

TRACT B

TRACT C

TRACT D

TRACT E

A T S 1 4 5 4

POINT 101

SW 1/4, NE 1/4, SW 1/4, SW 1/4
City of Haines

FR. Lot 6
City of Haines

FR. Lot 6
City of Haines

FR. Lot 7
State of Alaska

FR. Lot 8
State of Alaska

SECTION 10
T 30 S, R 59 E
C. R. M.

0 20 40 80

NORTH

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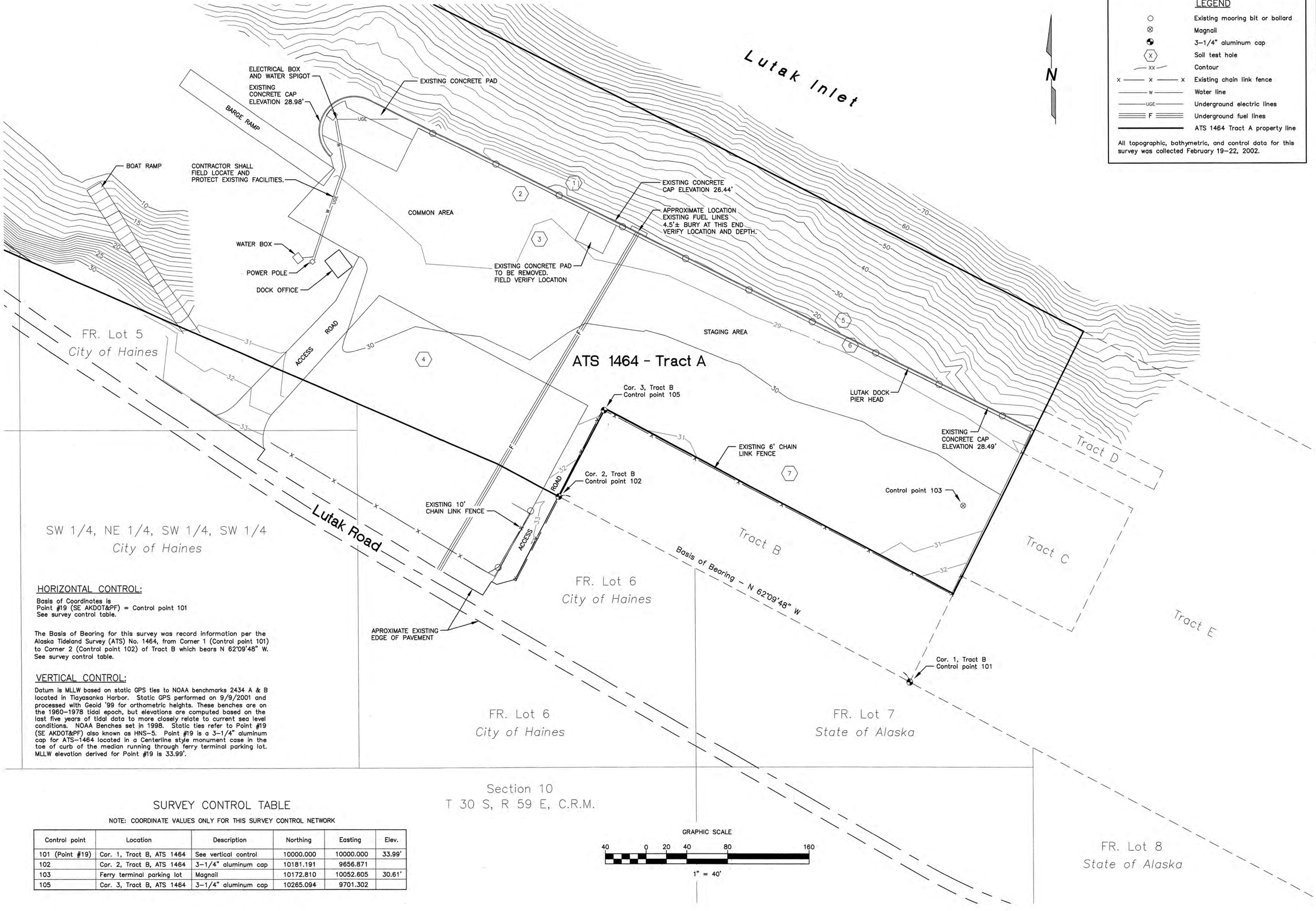
LUTAK DOCK REHABILITATION
CITY OF HAINES – EDA PROJECT NO. 07–79–04967

CONTRACTOR WORK AND STAGING AREAS

DWG		
DES. CF/JJ	SHEET NO.	
DR. CSB	G3	
CH. CF/JJ		
F.B.	OF 16 SHEETS	
DATE 05/02/02		
JOB NO. 695600		



LEGEND

- Existing mooring bit or bollard
- Magnail
- 3-1/4" aluminum cap
- Soil test hole
- Contour
- Existing chain link fence
- Water line
- Underground electric lines
- Underground fuel lines
- ATS 1464 Tract A property line

All topographic, bathymetric, and control data for this survey was collected February 19–22, 2002.

GRAPHIC SCALE

40 0 20 40 80 160

1" = 40'

HORIZONTAL CONTROL:

Basis of Coordinates is
Point #19 (SE AKDOT&PF) = Control point 101
See survey control table.

The Basis of Bearing for this survey was record information per the Alaska Tideland Survey (ATS) No. 1464, from Corner 1 (Control point 101) to Corner 2 (Control point 102) of Tract B which bears N 62°09'48" W. See survey control table.

VERTICAL CONTROL:

Datum is MLLW based on static GPS ties to NOAA benchmarks 2434 A & B located in Tlayasanka Harbor. Static GPS performed on 9/9/2001 and processed with Geoid '99 for orthometric heights. These benches are on the 1960–1978 tidal epoch, but elevations are computed based on the last five years of tidal data to more closely relate to current sea level conditions. NOAA Benches set in 1998. Static ties refer to Point #19 (SE AKDOT&PF) also known as HNS-5. Point #19 is a 3-1/4" aluminum cap for ATS-1464 located in a Centerline style monument case in the toe of curb of the median running through ferry terminal parking lot. MLLW elevation derived for Point #19 is 33.99'.

SURVEY CONTROL TABLE

NOTE: COORDINATE VALUES ONLY FOR THIS SURVEY CONTROL NETWORK

Control point	Location	Description	Northing	Easting	Elev.
101 (Point #19)	Cor. 1, Tract B, ATS 1464	See vertical control	10000.000	10000.000	33.99'
102	Cor. 2, Tract B, ATS 1464	3-1/4" aluminum cap	10181.191	9656.871	
103	Ferry terminal parking lot	Magnail	10172.810	10052.605	30.61'
105	Cor. 3, Tract B, ATS 1464	3-1/4" aluminum cap	10265.094	9701.302	

STATE OF ALASKA
49TH
Ron C. Heidemann
LS-10352
REGISTERED PROFESSIONAL LAND SURVEYOR

NO.	DATE	BY	REVISION

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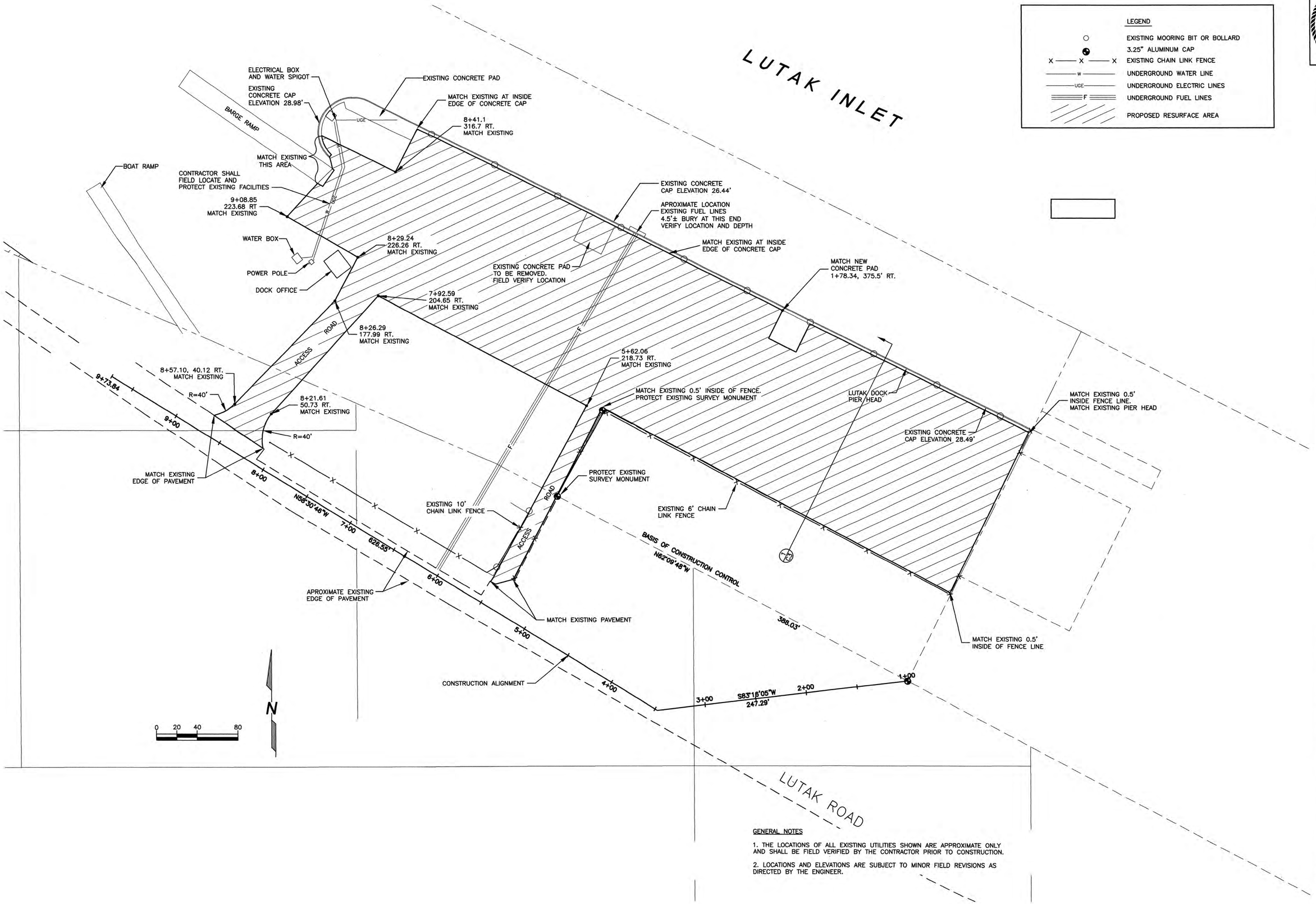
LUTAK DOCK REHABILITATION
CITY OF HAINES - EDA PROJECT NO. 07-79-04967

EXISTING SITE PLAN

DWG	695600MAST
DES.	MJ
DR.	CB
CH.	RH
F.B.	
SHEET NO.	C1
	of 15 SHEETS
DATE	MAY 2, 2002
JOB NO.	40-02-010



GENERAL NOTES

1. THE LOCATIONS OF ALL EXISTING UTILITIES SHOWN ARE APPROXIMATE ONLY AND SHALL BE FIELD VERIFIED BY THE CONTRACTOR PRIOR TO CONSTRUCTION.

2. LOCATIONS AND ELEVATIONS ARE SUBJECT TO MINOR FIELD REVISIONS AS DIRECTED BY THE ENGINEER.

STATE OF ALASKA 49TH D. Lance Meerig CE-7435 REGISTERED PROFESSIONAL ENGINEER

NO.	DATE	BY	REVISION

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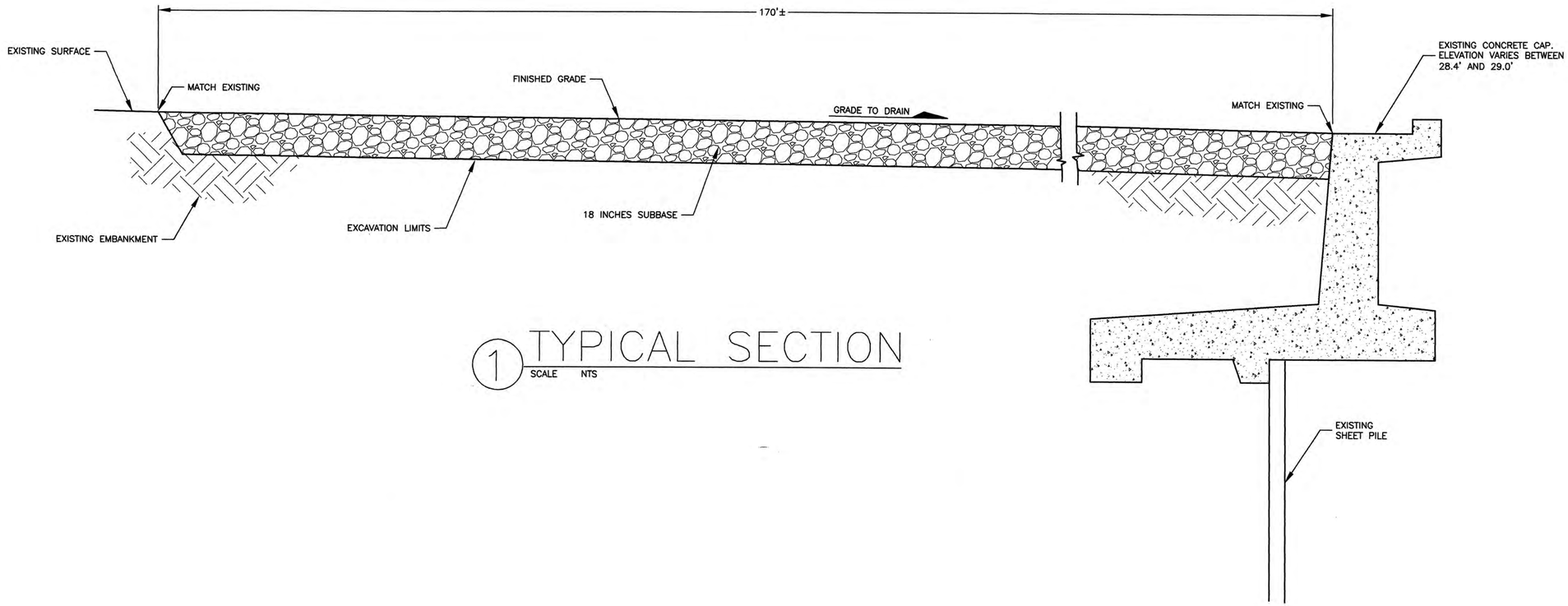
LUTAK DOCK REHABILITATION
CITY OF HAINES – EDA PROJECT NO. 07-79-04967

SITE GRADING PLAN

DWG	695600MAST
DES.	CB
DR.	CB
CH.	LM
F.B.	
SHEET NO.	C2 of 15 SHEETS
DATE	MAY 2, 2002
JOB NO.	40-01-010



1 TYPICAL SECTION
SCALE NTS

STATE OF ALASKA 49TH
D. Lance Meerig
CE-7435
REGISTERED PROFESSIONAL ENGINEER
24 Apr 02

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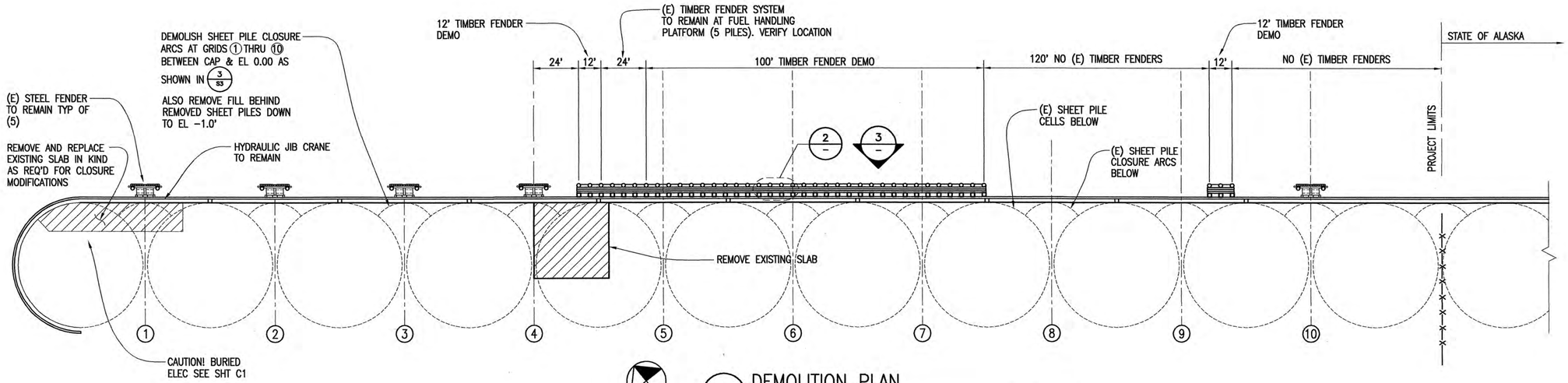
LUTAK DOCK REHABILITATION
CITY OF HAINES – EDA PROJECT NO. 07-79-04967

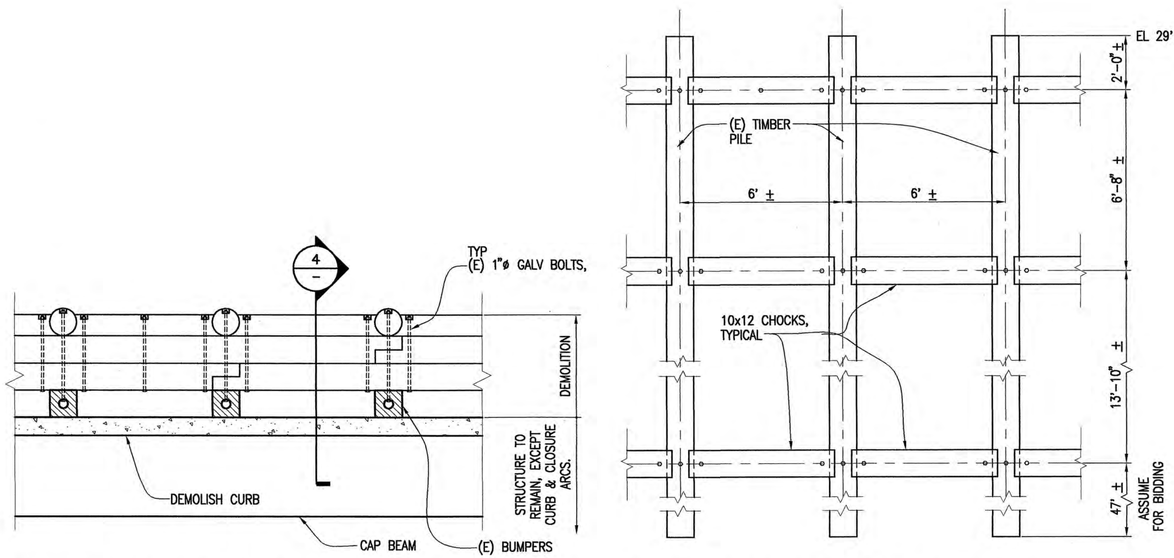
CIVIL DETAILS

DWG	695600MAST
DES.	CB
DR.	CB
CH.	LM
F.B.	
SHEET NO.	C3 of 15 SHEETS
DATE	MAY 2, 2002
JOB NO.	40-02-010



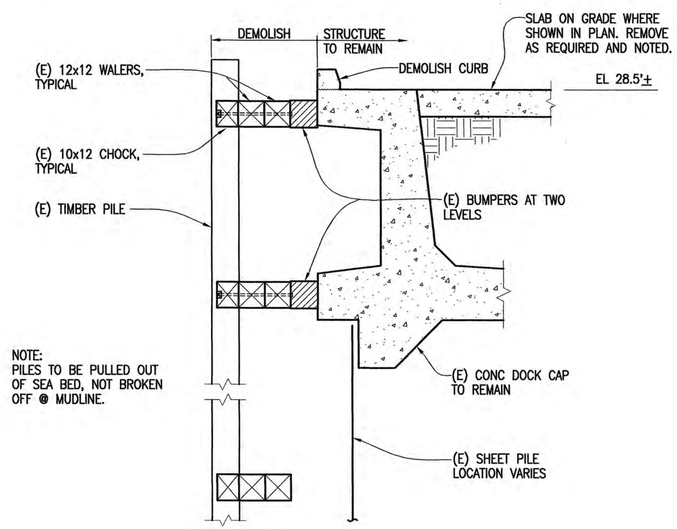
NORTH

1 DEMOLITION PLAN
SCALE 1" = 30 FEET



2 (E) TIMBER FENDER PARTIAL PLAN
SCALE 3/8" = 1'–0"

3 (E) TIMBER FENDER ELEVATION
SCALE 3/8" = 1'–0"



4 (E) TIMBER FENDER SECTION
SCALE 3/8" = 1'–0"

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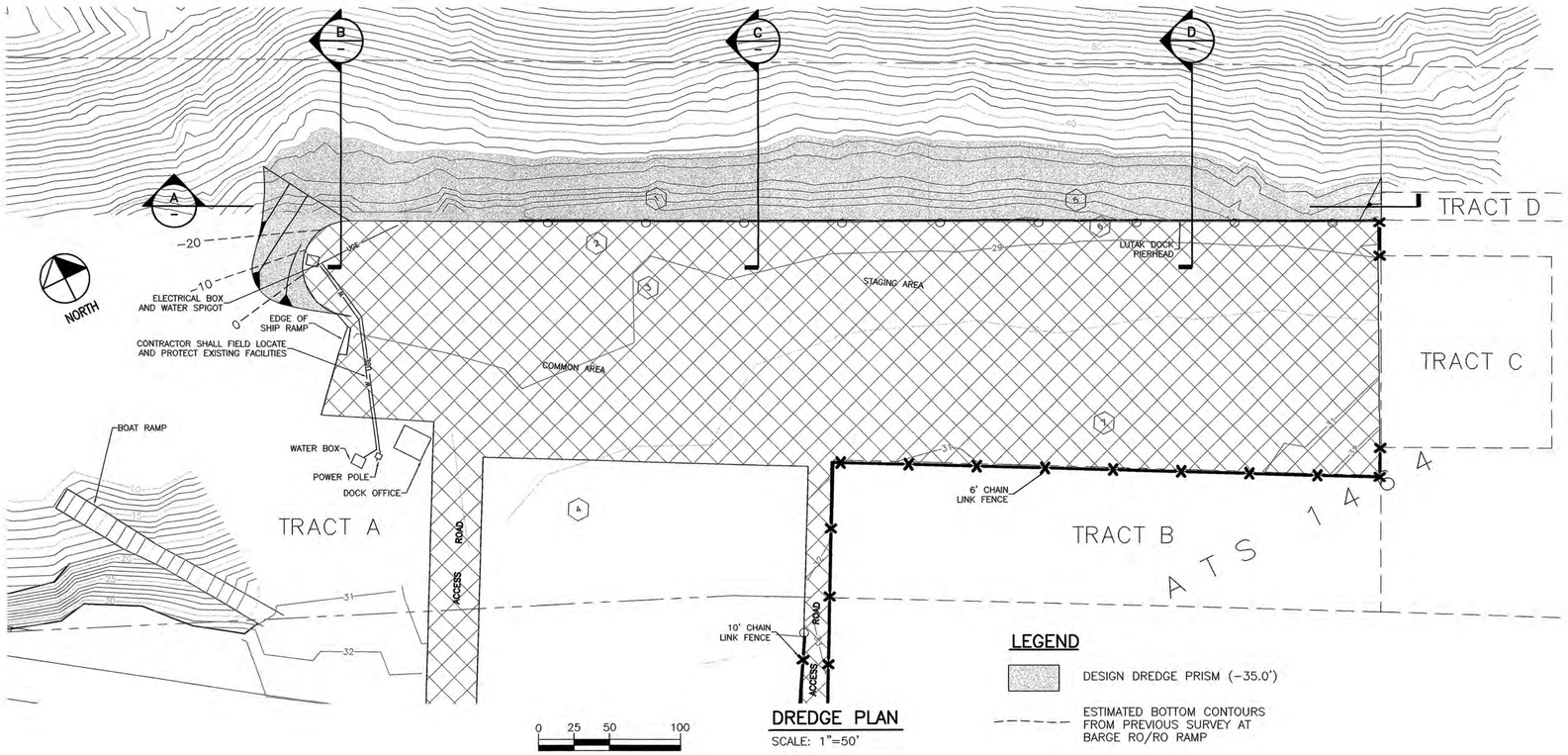
LUTAK DOCK REHABILITATION PROJECT
CITY OF HAINES – EDA PROJECT NO. 07–79–04967

DEMOLITION PLAN

NO.	DATE	BY	REVISION

DWG

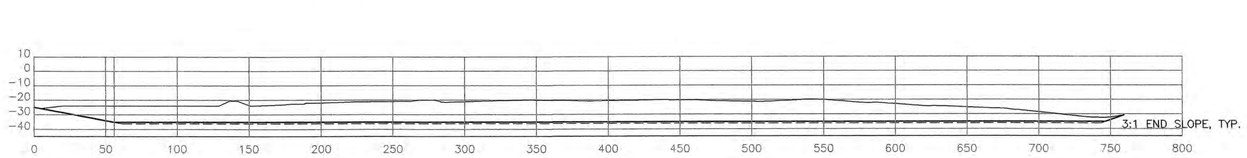
DES. CF/JJ	SHEET NO. D1
DR. CSB	
CH. CF/JJ	
F.B.	OF 16 SHEETS
DATE 05/02/02	
JOB NO. 40–02–010	



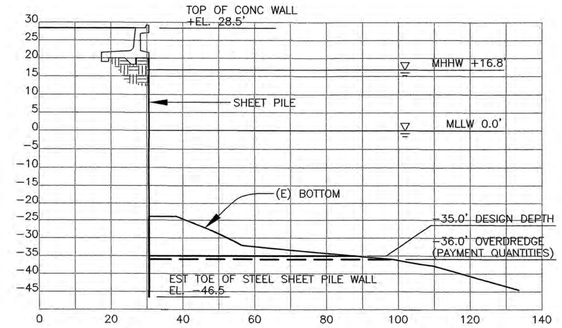
DREDGE PLAN
SCALE: 1"=50'

LEGEND

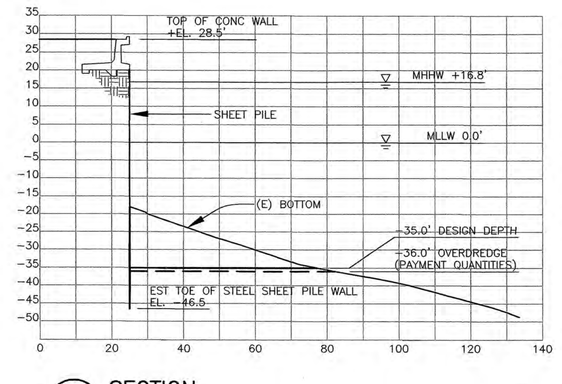
- DESIGN DREDGE PRISM (-35.0')
- ESTIMATED BOTTOM CONTOURS FROM PREVIOUS SURVEY AT BARGE RO/RO RAMP



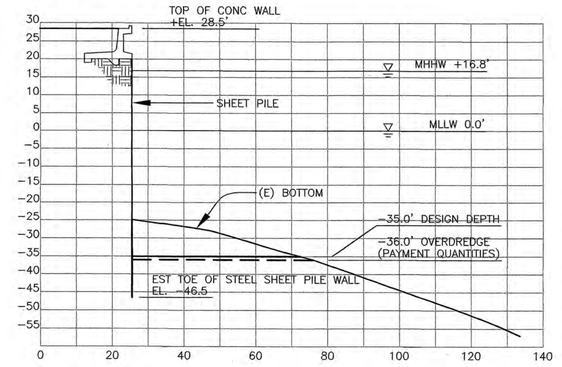
A SECTION
SCALE 1" = 50'



B SECTION
SCALE 1" = 20'



C SECTION
SCALE 1" = 20'



D SECTION
SCALE 1" = 20'

HORIZONTAL CONTROL:

BASIS OF COORDINATES IS
HNS-5 (SE AKDOT&PF) = USKH CONTROL PT 101
SEE SURVEY CONTROL TABLE

THE BASIS OF BEARING FOR THIS SURVEY WAS RECORD INFORMATION PER THE ALASKA TIDELAND SURVEY (ATS) No. 1464. FROM CORNER 1 (USKH 101) TO CORNER 2 (USKH 102) OF TRACT B, ATS 1464, WHICH BEARS N 62°09'48" W.
SEE SURVEY CONTROL TABLE

VERTICAL CONTROL:

SOURCE OF ELEVATION - SE AKDOT&PF

BENCH MARK (HNS-5)

ELEV. = 33.99 ft. (MLLW)

DESC. = ALUMINUM CAP IN A MONUMENT CASE ALONG THE OF A CURB OF THE MEDIAN RUNNING THROUGH FERRY TERMINAL PARKING LOT.

SURVEY CONTROL TABLE

NOTE: COORDINATE VALUES ONLY FOR THIS SURVEY CONTROL NETWORK

POINT NO.	LOCATION	DESCRIPTION	NORTHING	EASTING	ELEV.
101	FERRY TERMINAL PARKING LOT	3-1/4" ALCAP (HNS-5)	10000.00	10000.00	33.99'
102	FERRY TERMINAL PARKING LOT	3-1/4" ALCAP	10181.19	9656.87	

CONTROL NETWORK, TOPO AND BATHYMETRIC DATA WAS COLLECTED February 19 - 22, 2002.
VERTICAL CONTROL LOOPS; February 19, 2002

SURFACE = 6956A Existing ground

LEGEND

- BOLLARD
- TEST HOLE SHANNON & WILSON FEB 2002
- CONTOUR (-132.5-)
- CHAIN LINK FENCE
- ATS PROPERTY LINE
- ADJUSTED PROPERTY LINE
- RESURFACE AREA

TIDAL DATUM
HAINES, CHILKOOT INLET, LYNN CANAL LAT. 59°13.8' LONG. 135°26.0'
ESTIMATED EXTREME HIGH WATER (EHW) +22.5'
MEAN HIGHER HIGH WATER (MHHW) +16.8'
MEAN HIGH WATER (MHW) +15.8'
MEAN TIDE LEVEL (MTL) +6.7'
MEAN LOW WATER (MLW) +1.6'
MEAN LOWER LOW WATER (MLLW) 0.0'
ESTIMATED EXTREME LOW WATER (ELW) -6.0'

ESTIMATED DREDGE QUANTITIES

	VOLUME (CY)	AREA (SF)
DESIGN EL. -35.0'	9,620	39,545
1 FT OVERDREDGE	1,380	3,765
TOTAL (EL. -36.0')	11,000	43,310

STATE OF ALASKA
REGISTERED ENGINEER
CRAIG S. MIDDLETON
EXPIRES 12/31/2002

NO.	DATE	REVISION

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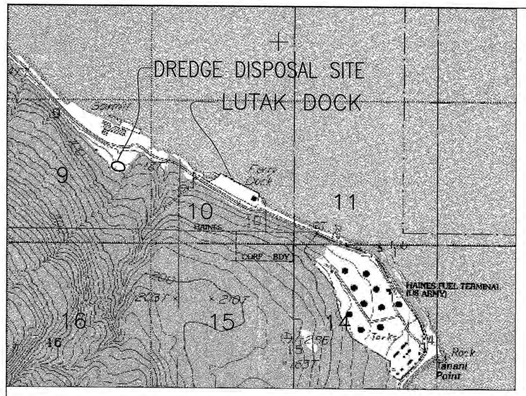
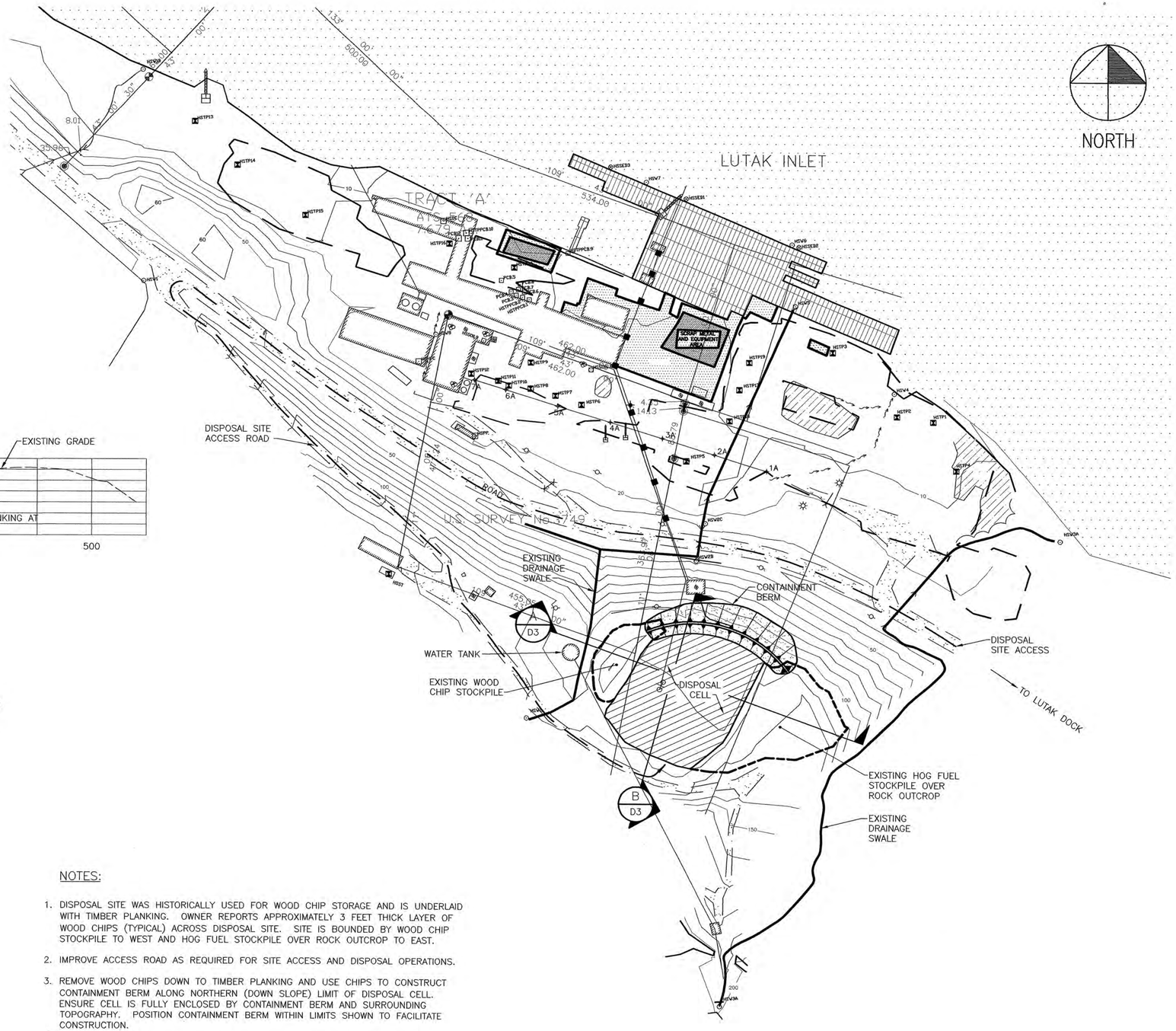
LUTAK DOCK REHABILITATION PROJECT
CITY OF HAINES - EDA PROJECT NO. 07-79-04967

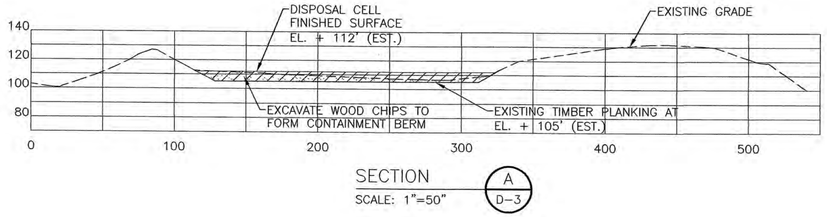
DREDGING PLAN AND SECTIONS

DWG

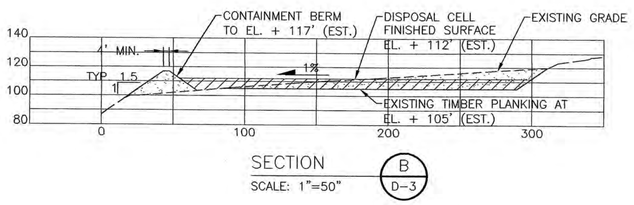
DES. SGM	SHEET NO.
DR. DJO	D2
CH.	
F.B. --	OF 16 SHEETS
DATE 05/02/02	
JOB NO. 40-02-010	







SECTION A / D-3
SCALE: 1"=50"



SECTION B / D-3
SCALE: 1"=50"

LEGEND

- EXISTING DRAINAGE SWALES (INDICATING DIRECTION OF FLOW)
- WOOD CHIPS
- DREDGE MATERIAL DISPOSAL

NOTES:

1. DISPOSAL SITE WAS HISTORICALLY USED FOR WOOD CHIP STORAGE AND IS UNDERLAID WITH TIMBER PLANKING. OWNER REPORTS APPROXIMATELY 3 FEET THICK LAYER OF WOOD CHIPS (TYPICAL) ACROSS DISPOSAL SITE. SITE IS BOUNDED BY WOOD CHIP STOCKPILE TO WEST AND HOG FUEL STOCKPILE OVER ROCK OUTCROP TO EAST.
2. IMPROVE ACCESS ROAD AS REQUIRED FOR SITE ACCESS AND DISPOSAL OPERATIONS.
3. REMOVE WOOD CHIPS DOWN TO TIMBER PLANKING AND USE CHIPS TO CONSTRUCT CONTAINMENT BERM ALONG NORTHERN (DOWN SLOPE) LIMIT OF DISPOSAL CELL. ENSURE CELL IS FULLY ENCLOSED BY CONTAINMENT BERM AND SURROUNDING TOPOGRAPHY. POSITION CONTAINMENT BERM WITHIN LIMITS SHOWN TO FACILITATE CONSTRUCTION.
4. SPREAD DREDGE MATERIAL EVENLY THROUGHOUT DISPOSAL CELL TO CREATE A CONSISTENT 1% DOWN SLOPE GRADE.
5. MAINTAIN CONTAINMENT BERM MINIMUM 5 FEET ABOVE FINISHED GRADE OF DISPOSAL MATERIAL CELL. BORROW FROM WOOD CHIP STOCKPILE AS NEEDED TO INCREASE CONTAINMENT DURING CONSTRUCTION.
6. INTENT OF CELL CONTAINMENT IS TO PREVENT FREE DRAINAGE OF DISPOSAL MATERIAL INTO EXISTING DRAINAGE SWALES. MODIFY AND MAINTAIN CONTAINMENT THROUGHOUT CONSTRUCTION AS REQUIRED.

NO.	DATE	BY	REVISION

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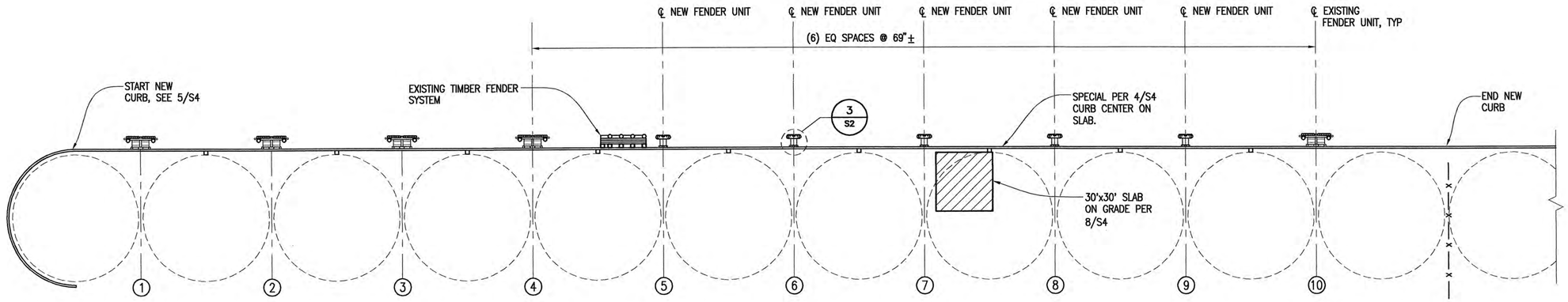
CITY OF HAINES – EDA PROJECT NO. 07-79-04967
LUTAK DOCK REHABILITATION PROJECT

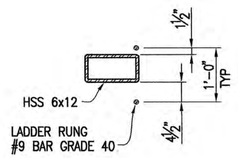
DREDGE DISPOSAL SITE – PLAN AND SECTIONS

DWG	
DES. SGM	SHEET NO. D3
DR. RR	
CH.	
F.B. — —	OF 16 SHEETS
DATE 05/02/02	
JOB NO. 40-02-010	

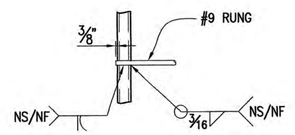


NORTH

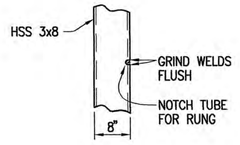
1 FENDER PLAN
SCALE 1" = 30 FEET



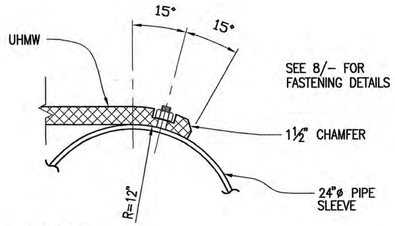
2 DETAIL
SCALE ¾" = 1'-0"



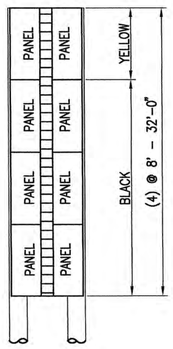
3 DETAIL
SCALE ¾" = 1'-0"



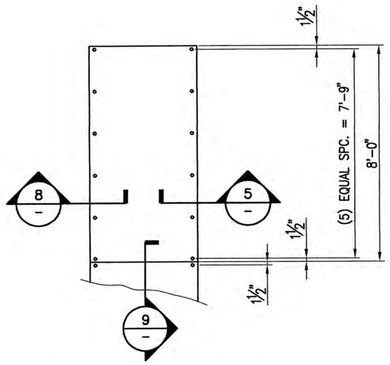
4 DETAIL
SCALE ¾" = 1'-0"



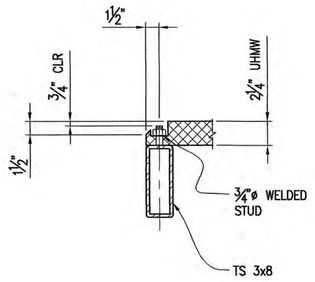
5 DETAIL
SCALE 1½" = 1'-0"



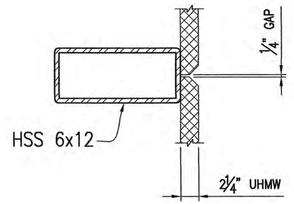
6 DETAIL–UHMW PANEL LAYOUT
SCALE ⅛" = 1'-0"



7 DETAIL–BOLT PATTERN
SCALE ⅜" = 1'-0"



8 SECTION
SCALE 1½" = 1'-0"



9 SECTION
SCALE 1½" = 1'-0"

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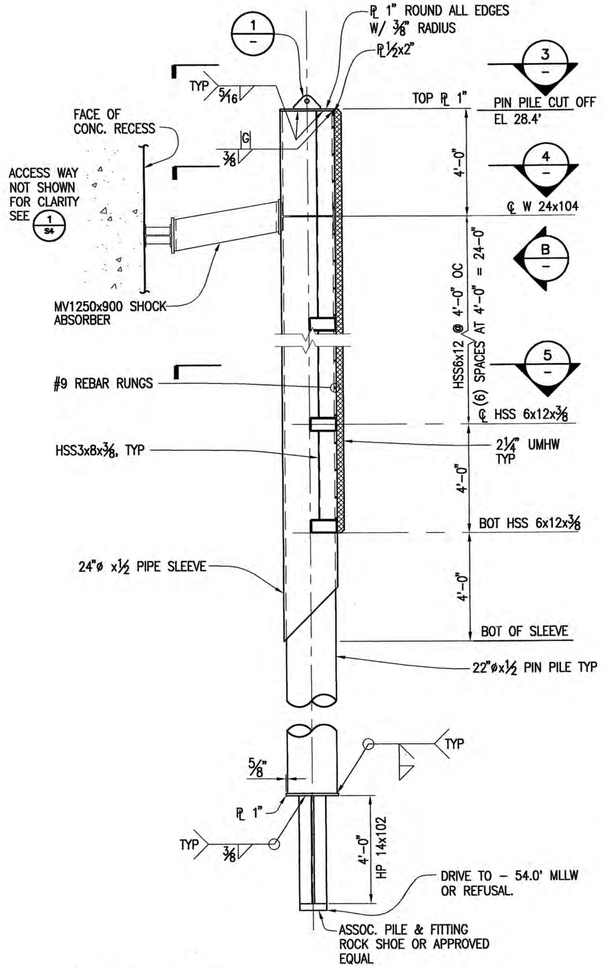
LUTAK DOCK REHABILITATION PROJECT
CITY OF HAINES – EDA PROJECT NO. 07-79-04967

FENDER LAYOUT PLAN AND DETAILS

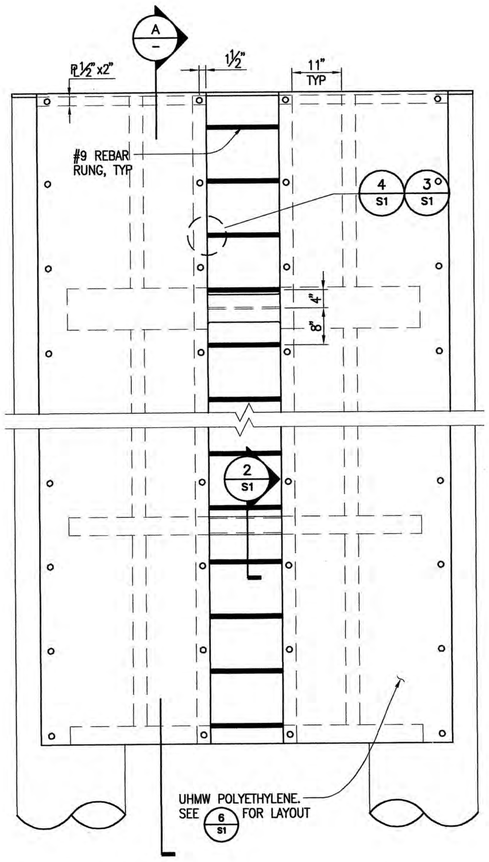
NO.	DATE	BY	REVISION

DWG

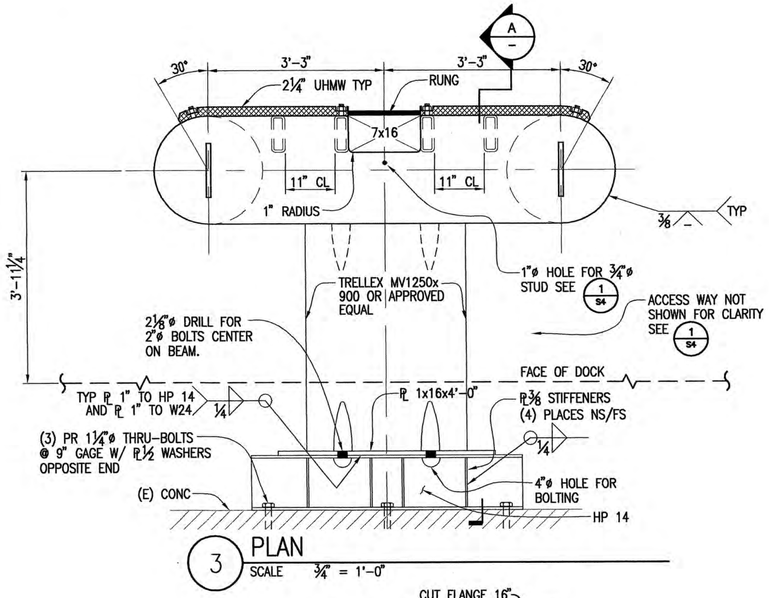
DES. CF/JJ	SHEET NO.
DR. CSB	S1
CH. CF/JJ	
F.B.	OF 16 SHEETS
DATE 05/02/02	
JOB NO. 40-02-010	



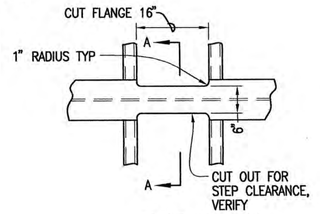
A SECTION
SCALE 3/8" = 1'-0"



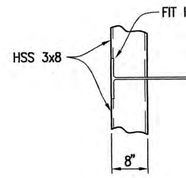
B FACE ELEVATION
SCALE 3/4" = 1'-0"



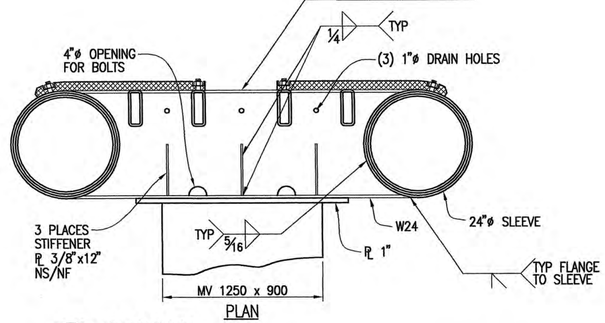
3 PLAN
SCALE 3/4" = 1'-0"



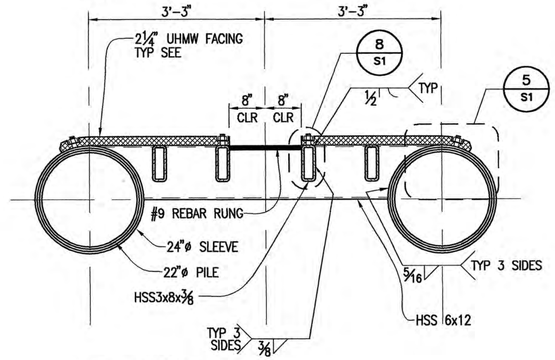
FRONT ELEVATION



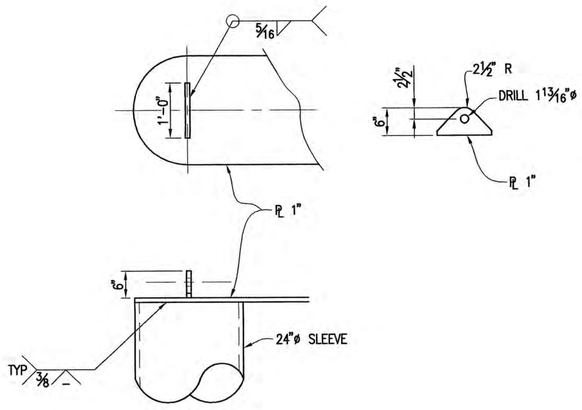
SECTION A–A



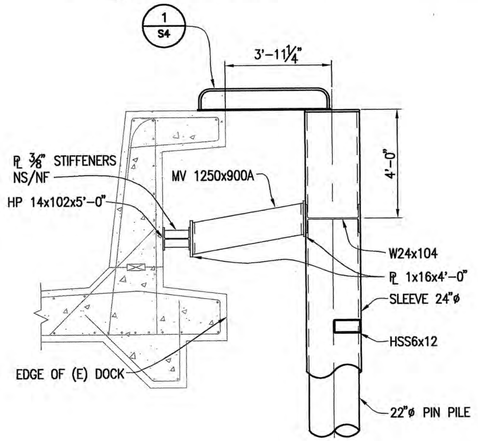
4 SECTION
SCALE 3/4" = 1'-0"



5 PLAN SECTION
SCALE 3/4" = 1'-0"



1 DETAIL–LIFTING PAD EYE
SCALE 3/4" = 1'-0"



2 SECTION
SCALE 3/8" = 1'-0"

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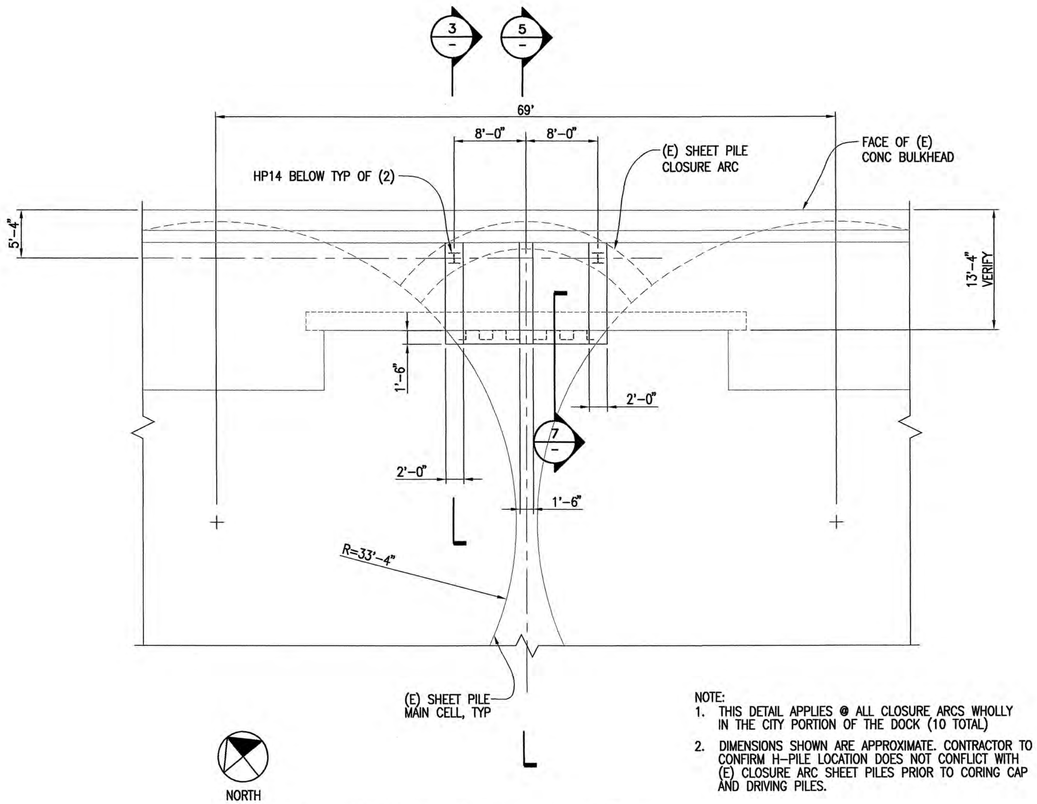
LUTAK DOCK REHABILITATION PROJECT
CITY OF HAINES – EDA PROJECT NO. 07-79-04967

FENDER DETAILS

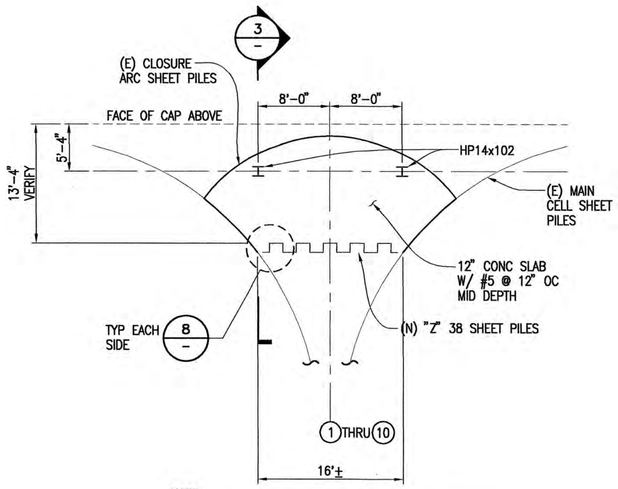
NO.	DATE	BY	REVISION

DWG	
DES. CF/JJ	SHEET NO.
DR. CSB	S2
CH. CF/JJ	
F.B.	OF 16 SHEETS
DATE 05/02/02	
JOB NO. 40-02-010	



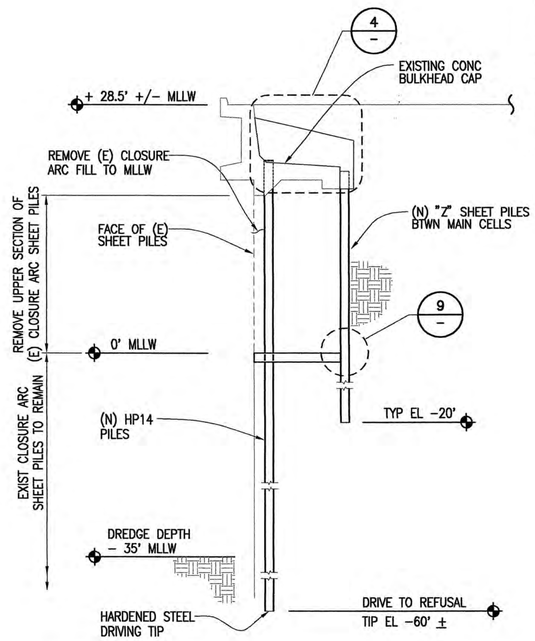
NOTE:
1. THIS DETAIL APPLIES @ ALL CLOSURE ARCS WHOLLY IN THE CITY PORTION OF THE DOCK (10 TOTAL)
2. DIMENSIONS SHOWN ARE APPROXIMATE. CONTRACTOR TO CONFIRM H-PILE LOCATION DOES NOT CONFLICT WITH (E) CLOSURE ARC SHEET PILES PRIOR TO CORING CAP AND DRIVING PILES.

1 PARTIAL PLAN @ EL 26' +/- TYP CLOSURE ARC MODIFICATIONS
SCALE 1/8" = 1'-0"



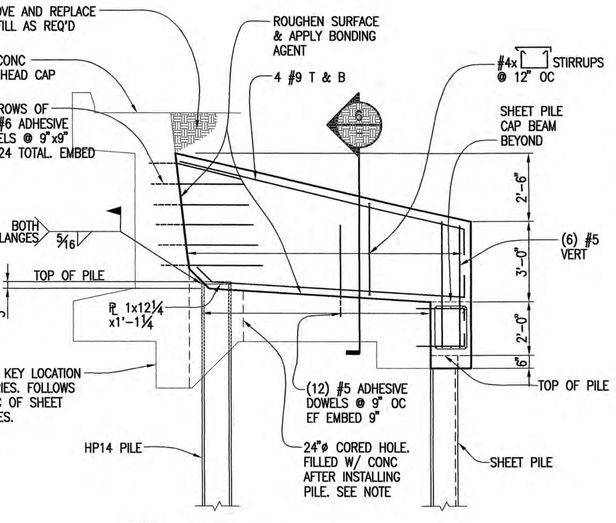
NOTE:
1. THIS DETAIL APPLIES @ ALL CLOSURE ARCS WHOLLY IN THE CITY PORTION OF THE DOCK (10 TOTAL)
2. DIMENSIONS SHOWN ARE APPROXIMATE. CONTRACTOR TO CONFIRM H-PILE LOCATION DOES NOT CONFLICT WITH (E) CLOSURE ARC SHEET PILES PRIOR TO CORING CAP AND DRIVING PILES.

2 PARTIAL PLAN @ EL 0.00'
SCALE 1/8" = 1'-0"



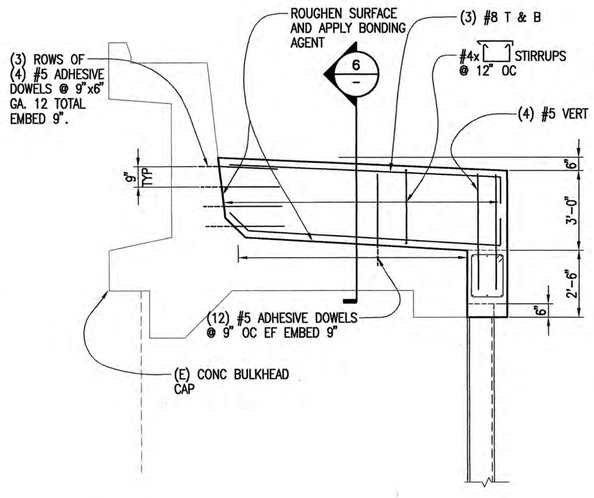
3 SECTION
SCALE 1/8" = 1'-0"

8 SHEET PILE CLOSURE
SCALE ¾" = 1'-0"

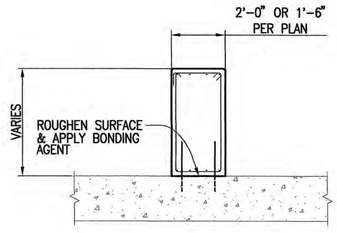


NOTE:
ALTERNATIVELY, HOLES FOR PILES MAY BE SAW-CUT 18" SQUARE. NO OVER CUTTING IS PERMITTED.

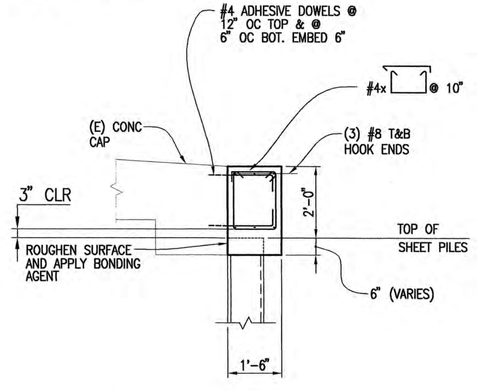
4 DETAIL
SCALE 3/8" = 1'-0"



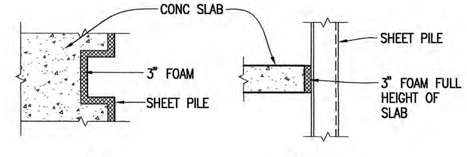
5 SECTION
SCALE 3/8" = 1'-0"



6 SECTION
SCALE 3/8" = 1'-0"



7 DETAIL
SCALE 1/2" = 1'-0"



9 SLAB @ NEW Z PILES
SCALE NTS

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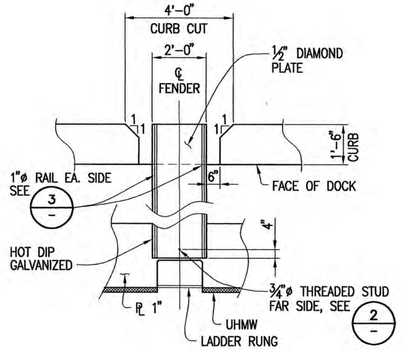
LUTAK DOCK REHABILITATION PROJECT
CITY OF HAINES - EDA PROJECT NO. 07-79-04967

CLOSURE ARC MODIFICATIONS

NO.	DATE	BY	REVISION

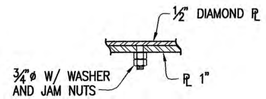
DWG	
DES. CF/JJ	SHEET NO. S3
DR. CSB	
CH. CF/JJ	
F.B.	OF 16 SHEETS
DATE 05/02/02	
JOB NO. 40-02-010	



1 ACCESS WAY PLAN

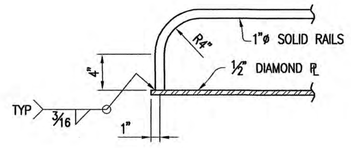
SCALE ⅜" = 1'-0"

REQ'D @ EACH (N) FENDER



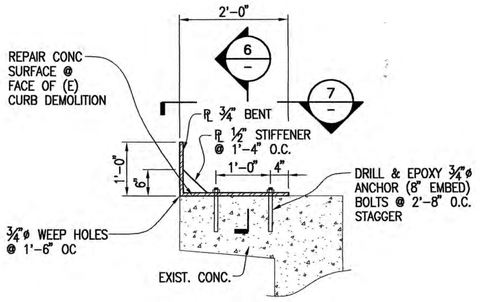
2 THREADED STUD

SCALE 1½" = 1'-0"



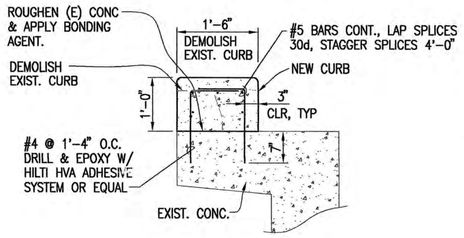
3 RAIL DETAIL

SCALE 1½" = 1'-0"

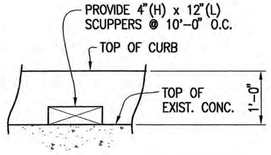


4 SPECIAL CURB

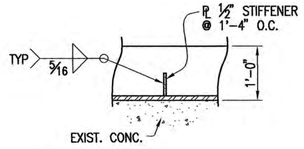
SCALE ¾" = 1'-0"

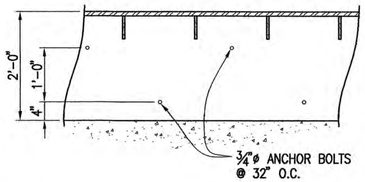


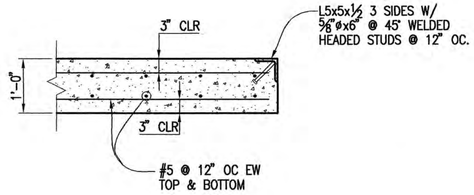
SECTION



ELEVATION







5 CAST-IN-PLACE CURB

SCALE ¾" = 1'-0"

6 SECTION

SCALE ¾" = 1'-0"

7 PLAN

SCALE ¾" = 1'-0"

8 SLAB DETAIL

SCALE ¾" = 1'-0"

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LUTAK DOCK REHABILITATION PROJECT
CITY OF HAINES - EDA PROJECT NO. 07-79-04967

MISCELLANEOUS DETAILS

NO	DATE	BY	REVISION

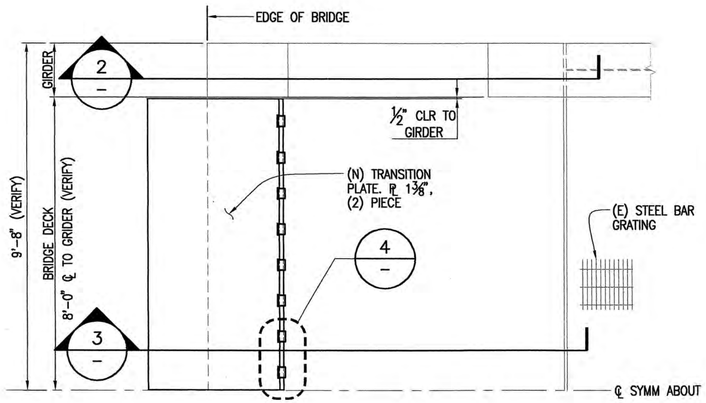
DWG

DES. CF/JJ
DR. CSB
CH. CF/JJ
F.B.

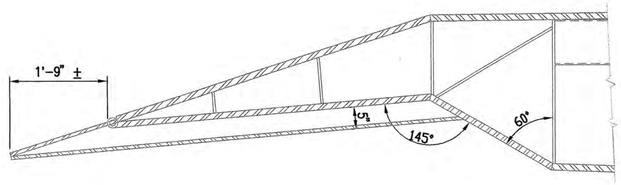
SHEET NO. S4 OF 16 SHEETS

DATE 05/02/02

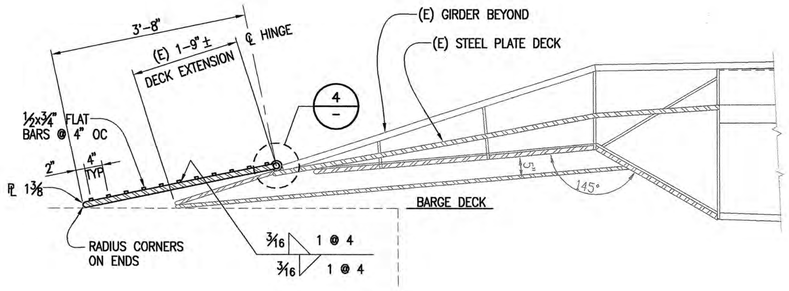
JOB NO. 40-02-010



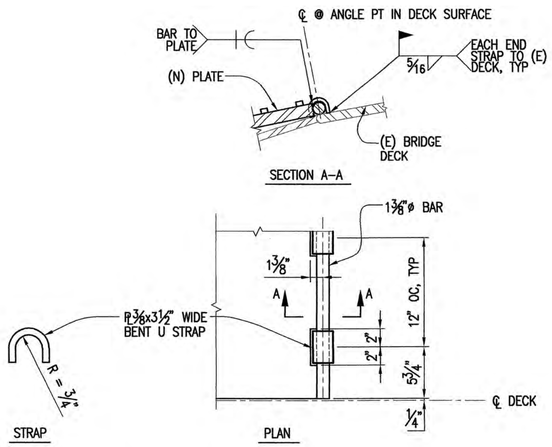
1 END PLAN
SCALE ½" = 1'-0"



2 (E) SECTION @ GIRDER
SCALE ¾" = 1'-0"



3 SECTION @ DECK
SCALE ¾" = 1'-0"



4 HINGE DETAIL
SCALE 1½" = 1'-0"

NOTE:
SEE SHEET C1 FOR TRANSFER BRIDGE (BARGE RAMP) LOCATION

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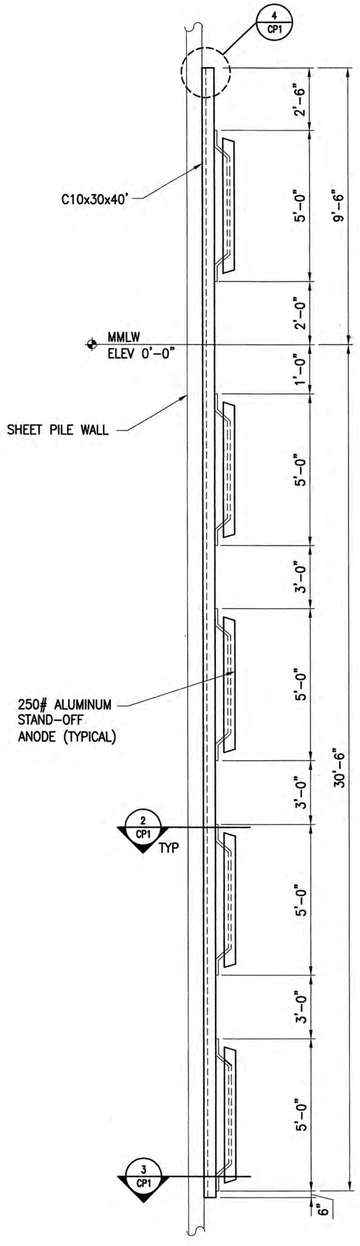
Email: cfress@reidmiddleton.com

LUTAK DOCK REHABILITATION PROJECT
CITY OF HAINES - EDA PROJECT NO. 07-79-04967

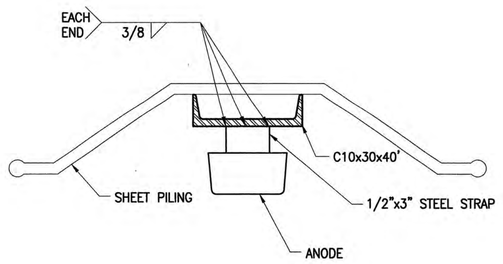
RO-RO TRANSFER BRIDGE

NO.	DATE	BY	REVISION

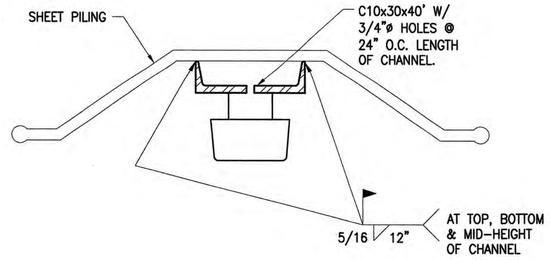
DWG
DES. CF/JJ
DR. CSB
CH. CF/JJ
F.B.
SHEET NO. S5 OF 16 SHEETS
DATE 05/02/02
JOB NO. 40-02-010



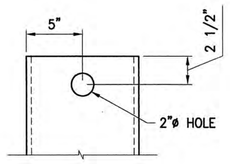
1 CP1 MAIN CELL ANODE ASSEMBLY ELEVATION
N.T.S.



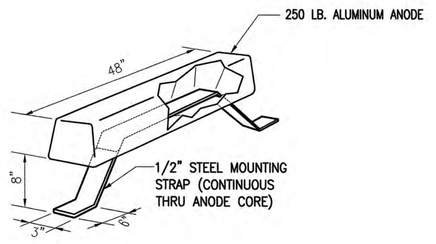
2 CP1 ANODE ASSEMBLY MOUNTING DETAIL
N.T.S.



3 CP1 CHANNEL MOUNTING DETAIL
N.T.S.



4 CP1 ANODE CHANNEL LIFT LUG
N.T.S.



5 CP1 STAND-OFF ANODE DETAIL
N.T.S.

COFFMAN ENGINEERS
800 E Dimond Blvd, Suite 3-450
Anchorage, Alaska 99501
907-276-6664 Fax 907-276-5042

Reid Middleton
4300 B St., Suite 403
Anchorage, AK 99503
Phone: 907.562.3439
Email: cfross@reidmiddleton.com

NO.	DATE	BY	REVISION

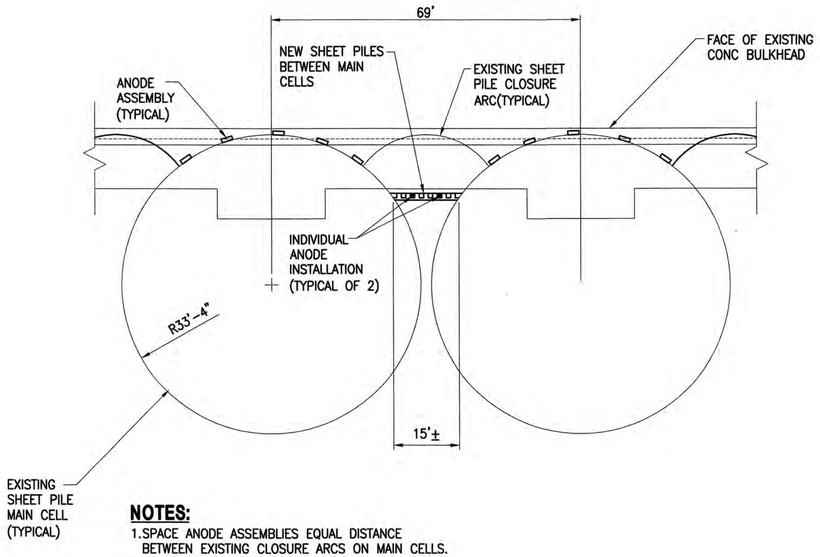
LUTAK DOCK REHABILITATION PROJECT
CITY OF HAINES – EDA PROJECT NO. 07-79-04967

CATHODIC PROTECTION DETAILS

DWG	
DES. CDS	SHEET NO.
DR. LAI	CP1
CH. TB	
F.B. — —	OF SHEETS
DATE 5/2/02	
JOB NO. 40-02-010	

STATE OF ALASKA
49TH
REGISTERED PROFESSIONAL ENGINEER
W.K. MARSH
NE 5469

P:\0 CORROSION PROJECTS\00058 Haines Lutak Dock\Drawings\00058CP1.dwg, 05/01/2002 02:26:47 PM

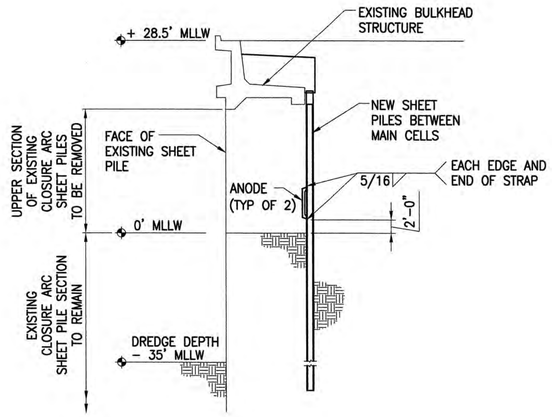


NOTES:

1. SPACE ANODE ASSEMBLIES EQUAL DISTANCE BETWEEN EXISTING CLOSURE ARCS ON MAIN CELLS.
2. START ASSEMBLY ORIENTATION AS CLOSE AS POSSIBLE TO EXISTING ARC

1 CP2 ANODE ASSEMBLY ORIENTATION

N.T.S.



NOTES:

1. INSTALL TWO(2) NEW ANODES AT EACH CLOSURE ARC LOCATION. SPACE ANODES EQUAL DISTANCE (HORIZONTALLY) BETWEEN MAIN CELLS.
2. INSTALL EACH ANODE APPROXIMATELY FIVE(5) FEET IN FROM THE MAIN CELL WALLS.

2 CP2 CLOSURE ARC ANODE INSTALLATION DETAIL

N.T.S.

CATHODIC PROTECTION NOTES

GENERAL

A. The Contractor shall provide all labor, equipment and materials required for a complete galvanic anode cathodic protection system. The cathodic protection system shall include the appropriate anodes, channels and all necessary items of equipment and materials for a completed system.

B. Corrosion control materials shall be submitted to the Engineer for approval. A complete list of equipment and materials, including name and manufacturer, catalog number, size, finish and other pertinent data shall be provided. The following items shall be included in the submittals:

1. Aluminum anodes
2. Epoxy Coating
3. Channels (Mill test certificates)
4. Welders qualifications & weld procedures

PRODUCTS

A. All materials shall conform to the requirements set forth herein or as shown. All materials must be new, free from defects, and shall be of the best commercial quality for the purpose specified. All necessary items and accessories not shown or specified herein, but which are required to fully carry out the specified intent of the work, shall be furnished without additional cost.

B. Aluminum stand-off anode shall be 250 lb. nominal weight and shall have a 1/2" steel mounting strap continuous through the core. Anodes shall conform to the following chemical composition:

Element	%
Indium	0.014 to 0.020
Zinc	3.0 to 5.0
Silicon	0.08 max.
Mercury	0.09 max.
Tin	0.001 max.
Others(each)	0.001 max.
Aluminum	Remainder

Anodes shall be Purity Casting Alloys aluminum anodes, Galvalum III or Engineer approved equal.

C. Coating shall be Sherwin-Williams Dura-Plate 235 or Engineer approved equal. Color shall be gray.

D. Channels shall be C10x30x40', ASTM A36 steel.

E. Welds shall use 60 KSI minimum rod. Welders and procedures shall be qualified per A.W.S.

EXECUTION

A. Anodes shall be installed as shown. Anodes shall be installed on channel assemblies on the main cells and directly attached to the new sheet pile closure arcs. Anodes, damaged as a result of improper handling, shall be replaced.

B. Remove all marine growth and clean all surfaces that the anodes and anode channel are being installed to.

C. Each end of the anode straps shall be welded to the anode channel or the new closure arc sheet pile. A continuous weld is required on each side and the end of the straps.

D. The anode channels shall be welded to the main cells at the top, bottom and mid-height of the channels. Both sides of the channel shall be welded at these locations.

E. Main cell anodes assemblies and individual anodes being installed on the new sheet piling shall be installed at equal distances between anodes/assemblies.

F. After the anodes have been installed on the channels or closure arcs, coat the channels, mounting straps and weld areas with 4-8 mils of coating per manufactures recommendations. Ensure that no coating is applied to the anode surface.

STATE OF ALASKA 49TH W.K. McNEAL ME 5489 REGISTERED PROFESSIONAL ENGINEER

NO.	DATE	BY	REVISION

COFFMAN ENGINEERS
800 F Street
Anchorage, Alaska 99501
907-276-6664 Fax 907-276-5042

Reid Middleton
4300 B ST., Suite 403
Anchorage, AK 99503
Phone: 907 562-3439
Email: cfreas@reidmiddleton.com

LUTAK DOCK REHABILITATION PROJECT
CITY OF HAINES - EDA PROJECT NO. 07-79-04967

CATHODIC PROTECTION NOTES AND DETAILS

DWG
DES. CDS
DR. LAI
CH. TB
F.S. --
SHEET NO. CP2 OF SHEETS
DATE 5/2/02
JOB NO. 40-02-010

REF #9

NOAA Tides and Current. Data from Skagway, AK recording station.

9452400 Skagway, AK

Datums for 9452400, Skagway AK

Certain geographic areas are experiencing rapid land movement due to uplift or subsidence, which results in anomalous relative sea level trends compared to most other coastal regions in the United States (see http://tidesandcurrents.noaa.gov/sltrends/ (/sltrends/)). In these areas, NOAA has adopted a modified procedure for computing accepted tidal datums for the National Water Level Observation Network (NWLON). The long-term control stations in these areas have tidal datums updated approximately every 5 years using the modified procedure, with the most recent update being the 2007-2011 Modified Procedure. The adoption of this procedure was necessary to ensure that these tidal datums accurately represent the existing stand of sea level relative to the land. A detailed report explaining this modified procedure can be found in the publications section of the CO-OPS Tides & Currents website at http://tidesandcurrents.noaa.gov/pub.html (/pub.html).

Elevations on Station Datum

Station: 9452400, Skagway, AK
Status: Accepted (Apr 29 2014)
Units: Feet
T.M.: 135 W
Epoch: (/datum_options.html#NTDE) 2007-2011
Datum: STND

Datum	Value	Description
MHHW (/datum_options.html#MHHW)	19.06	Mean Higher-High Water
MHW (/datum_options.html#MHW)	18.06	Mean High Water
MTL (/datum_options.html#MTL)	11.00	Mean Tide Level

Datum	Value	Description
MSL (/datum_options.html#MSL)	11.14	Mean Sea Level
DTL (/datum_options.html#DTL)	10.69	Mean Diurnal Tide Level
MLW (/datum_options.html#MLW)	3.95	Mean Low Water
MLLW (/datum_options.html#MLLW)	2.33	Mean Lower-Low Water
NAVD88 (/datum_options.html)		North American Vertical Datum of 1988
STND (/datum_options.html#STND)	0.00	Station Datum
GT (/datum_options.html#GT)	16.73	Great Diurnal Range
MN (/datum_options.html#MN)	14.11	Mean Range of Tide
DHQ (/datum_options.html#DHQ)	1.00	Mean Diurnal High Water Inequality
DLQ (/datum_options.html#DLQ)	1.62	Mean Diurnal Low Water Inequality
HWI (/datum_options.html#HWI)	9.75	Greenwich High Water Interval (in hours)
LWI (/datum_options.html#LWI)	3.51	Greenwich Low Water Interval (in hours)
Maximum	28.80	Highest Observed Water Level
Max Date & Time	10/22/1945 14:30	Highest Observed Water Level Date and Time
Minimum	-4.12	Lowest Observed Water Level
Min Date & Time	12/14/2008 04:42	Lowest Observed Water Level Date and Time
HAT (/datum_options.html#HAT)	23.38	Highest Astronomical Tide
HAT Date & Time	10/16/1993 22:06	HAT Date and Time
LAT (/datum_options.html#LAT)	-2.75	Lowest Astronomical Tide
LAT Date & Time	12/24/1999 04:42	LAT Date and Time

Tidal Datum Analysis Periods

01/01/2007 - 11/30/2008

02/01/2009 - 12/31/2011

To refer water level heights to NAVD88 (North American Vertical Datum of 1988), apply the values located at National Geodetic Survey ( ).

Datums for 9452400, Skagway, AK
All figures in feet relative to station datum

Showing datums for

9452400 Skagway, AK

Data Units ◉ Feet
○ Meters

Epoch ◉ Present (1983-2001)
○ Superseded (1960-1978)

Submit

Show nearby stations

Products available at 9452400 Skagway, AK

TIDES/WATER LEVELS

Water Levels (/waterlevels.html?id=9452400)

NOAA Tide Predictions (/noaatidepredictions/NOAATidesFacade.jsp?Stationid=9452400)

Harmonic Constituents (/harcon.html?id=9452400)

Sea Level Trends (/sltrends/sltrends_station.shtml?stnid=9452400)

Datums (/datums.html?id=9452400)

Benchmarks (/benchmarks.html?id=9452400)

Extreme Water Levels (/est/est_station.shtml?stnid=9452400)

Reports (/reports.html?id=9452400)

METEOROLOGICAL/OTHER

Meteorological Observations (/met.html?id=9452400)

Water Temp/Conductivity (/physocean.html?id=9452400)

PORTS®

This station is not a member of PORTS®

OPERATIONAL FORECAST SYSTEMS

This station is not a member of OFS

INFORMATION

Station Home Page (/stationhome.html?id=9452400)

Data Inventory (/inventory.html?id=9452400)

Measurement Specifications (/measure.html)

Information

About CO-OPS (/about.html)
Disclaimers (/disclaimers.html)
Contact Us (/contact.html)
Privacy Policy (/privacy.html)

Products

PORTS (/ports.html)
OFS (/models.html)
Tide Predictions (/tide_predictions.html)
Currents (/cdata/StationList?type=Current+Data&filter=active)
More about products... (/products.html)

Programs

Mapping and Charting Support (/mapping.html)
Maritime Services (/maritime.html)
COASTAL (/coastal.html)
More about programs... (/programs.html)

Partners

Hydrographic Survey Support (/hydro.html)
Marsh Restoration (/marsh.html)
GoMOOS (/gomoos.html)
TCOON (/tcoon.html)

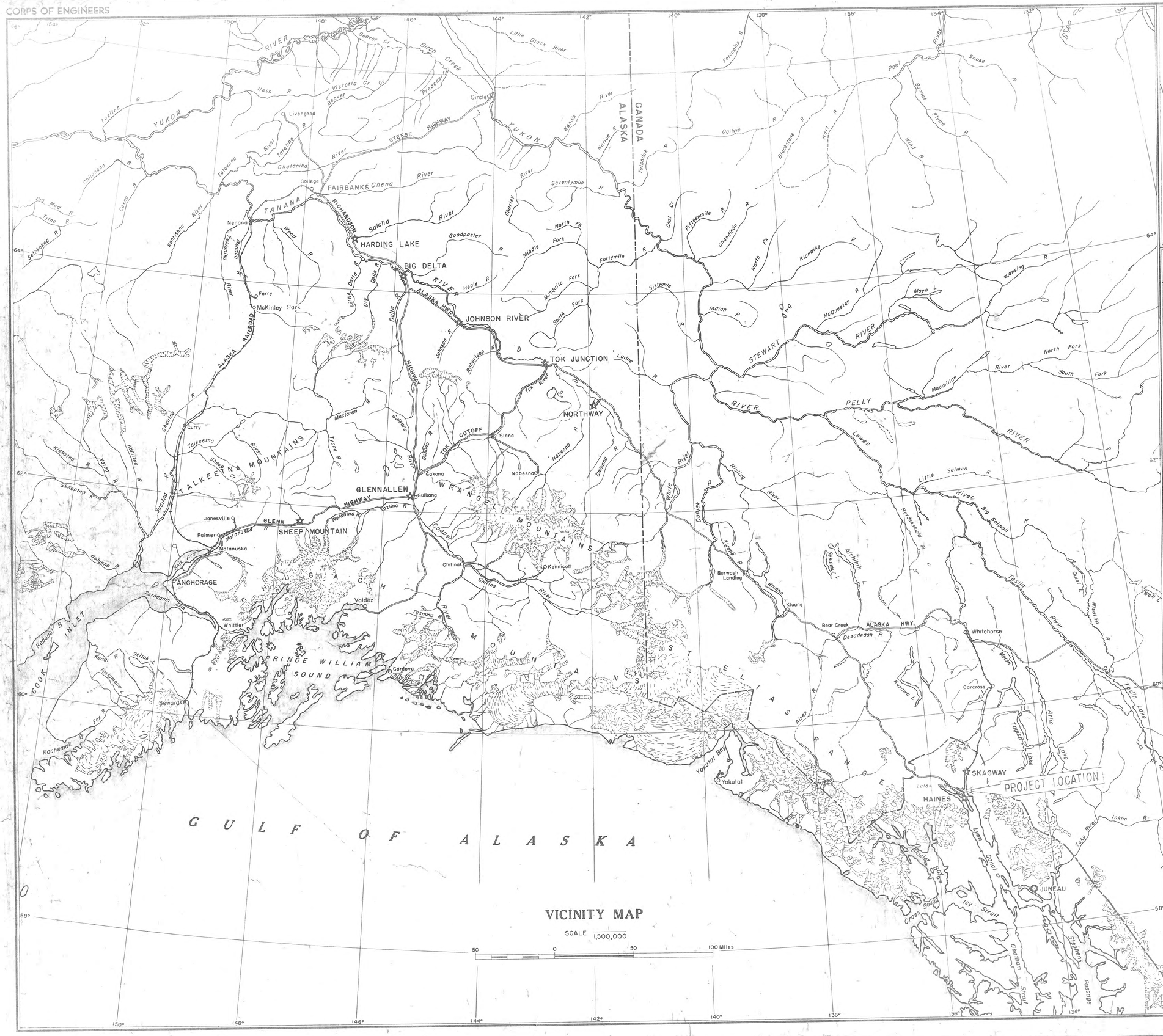
Revised: 10/15/2013
NOAA (http://www.noaa.gov) / National Ocean Service (http://oceanservice.noaa.gov)
Web site owner: Center for Operational Oceanographic Products and Services

REF #10

United States Geologic Survey Applet.

REF #11

1953 Haines Expansion Port Facilities Project. As-Built drawings of the Bulkhead.

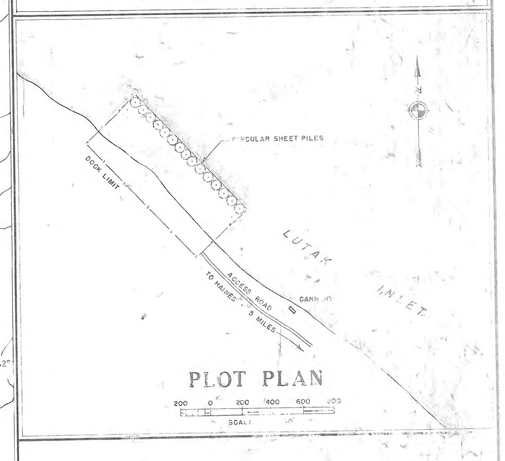


LOCATION MAP

VICINITY MAP

SCALE 1/1,500,000

50 0 50 100 Miles



NOTE:
For scope of work in this contract see drawings described in paragraph SC2 of specifications.

PROJECT LOCATION

BEEN
UI T
5 August 52
for Robert K. Dowell

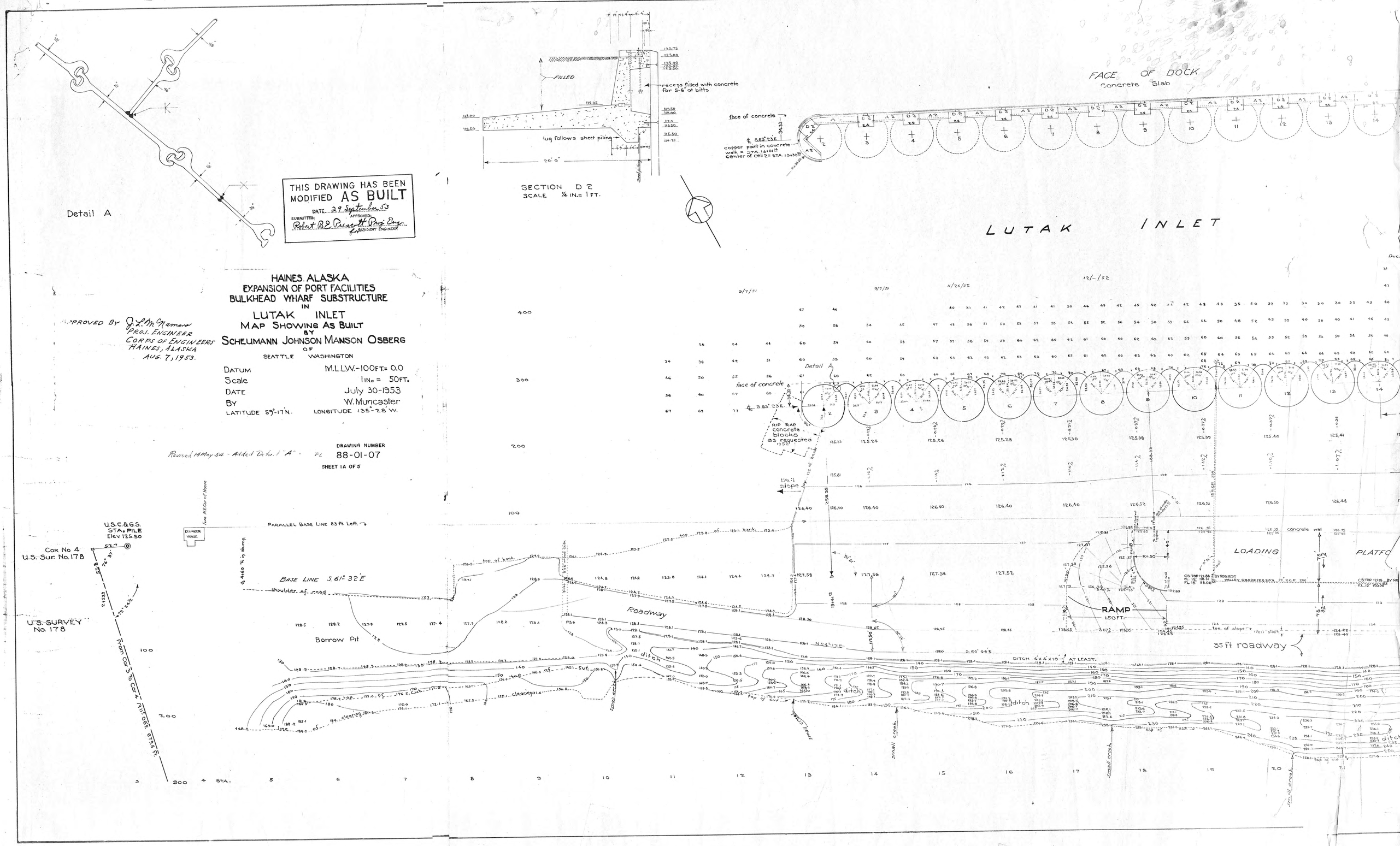
REVISION	DATE	DESCRIPTION	BY

CORPS OF ENGINEERS
U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ANCHORAGE, ALASKA

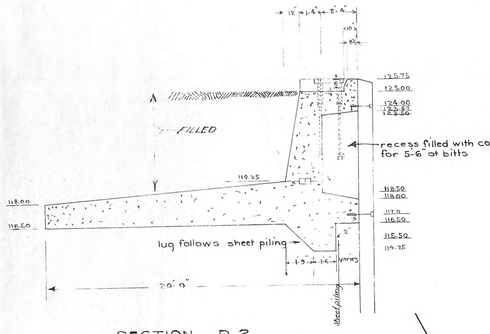
DRAWN: H M	HAINES, ALASKA
TRACED: H M	PORT FACILITIES, BULKHEAD WHARF
CHECKED: D C R	PLOT PLAN
PREPARED:	
SUBMITTED:	

RECOMMENDED: _____ CHIEF, DESIGN BRANCH
APPROVED: _____ CHIEF, ENGINEERING DIVISION
DATE: 2 May
APPROVED: _____ COLONEL, CORPS OF ENGINEERS, DISTRICT ENGINEER
DATE
SCALE
SPEC. NO.
DRAWING NUMBER 16-06-02
SHEET 1 OF 1

3-4



Detail A
THIS DRAWING HAS BEEN MODIFIED AS BUILT
DATE 29 September 53
SUBMITTED: APPROVED:
Robert B.E. Prescott Proj. Eng.
for RESIDENT ENGINEER
SECTION D 2
SCALE 1/4 IN.= 1 FT.
FILLED
recess filled with concrete for 5-6" at bitts
lug follows sheet piling
20'-0"
FACE OF DOCK
Concrete Slab
face of concrete
copper point in concrete walk = STA. 13+01.14
center of cell 2 = STA. 13+32.25
LUTAK INLET
HAINES ALASKA
EXPANSION OF PORT FACILITIES
BULKHEAD WHARF SUBSTRUCTURE
IN
LUTAK INLET
MAP SHOWING AS BUILT
BY
SCHEUMANN JOHNSON MANSON OSBERG
OF
SEATTLE WASHINGTON
APPROVED BY J.L. McNamara
PROJ. ENGINEER
CORPS OF ENGINEERS
HAINES, ALASKA
AUG. 7, 1953.
DATUM M.L.L.W.-100FT.= 0.0
Scale 1IN.= 50FT.
DATE July 30-1953
BY W. Muncaster
LATITUDE 59°-17'N. LONGITUDE 135°-28'W.
DRAWING NUMBER
88-01-07
SHEET 1A OF 5
Revised 14 May 54 - Added Detail "A" - VL
U.S.C.&G.S. STA. PILE Elev. 125.50
COR No 4
U.S. Sur. No. 178
U.S. SURVEY No. 178
From Cor 3 To Cor 4 N11°06'E 679.8 ft.
ELLINGEN HOUSE
PARALLEL BASE LINE 83 ft. Left.
BASE LINE S.61°32'E
shoulder of road
Borrow Pit
Roadway
ditch
small creek
RAMP
150 FT.
LOADING
PLATFO
35 ft roadway
DITCH 4'x4'x10' AT LEAST.



SECTION D 2
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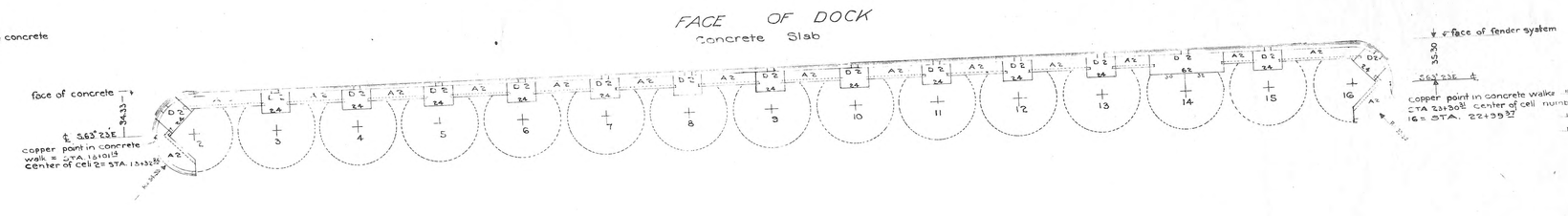
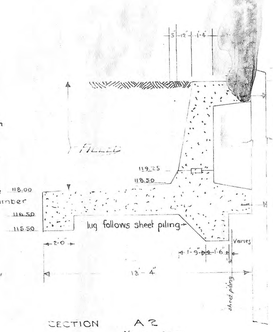
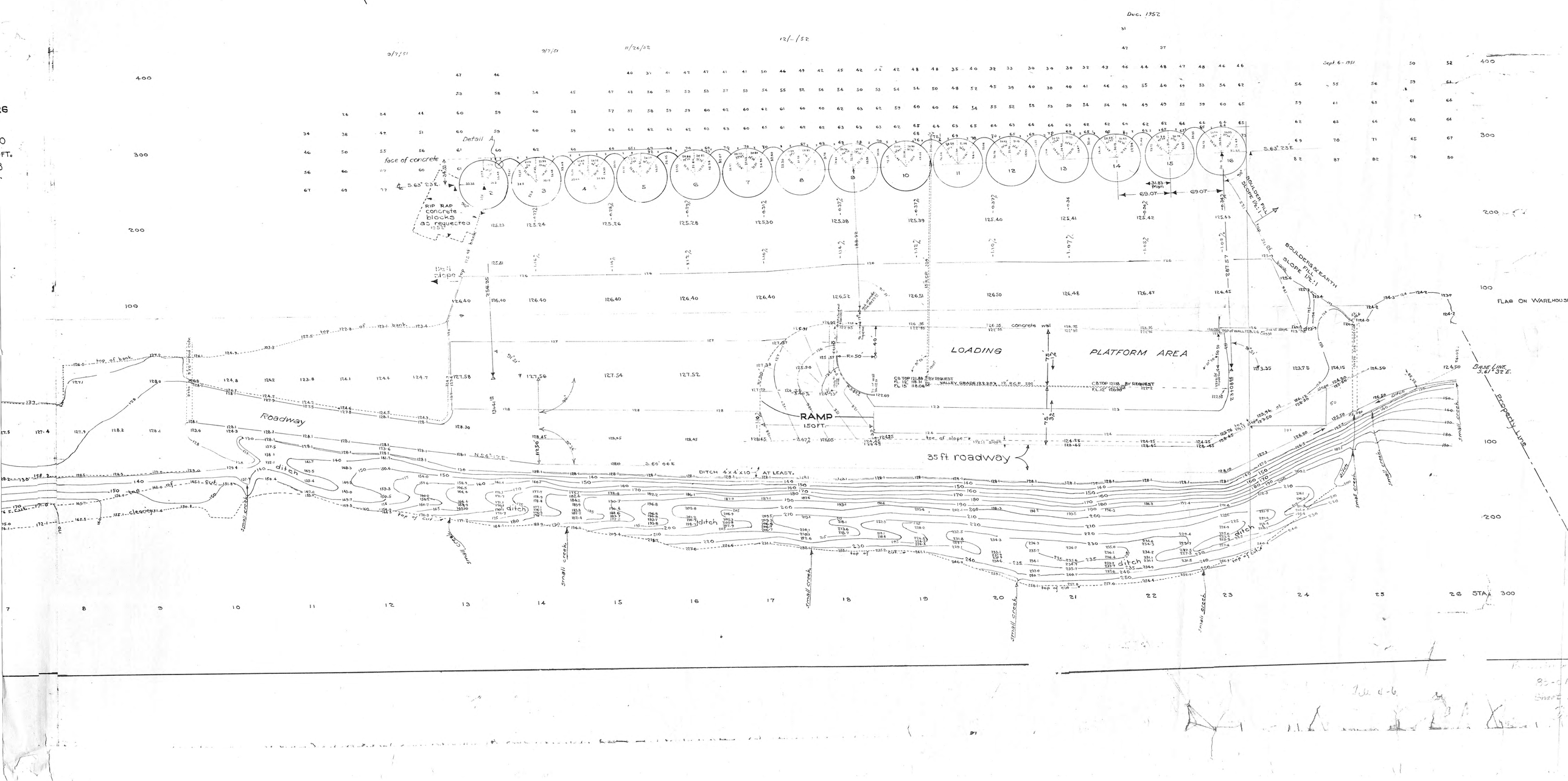


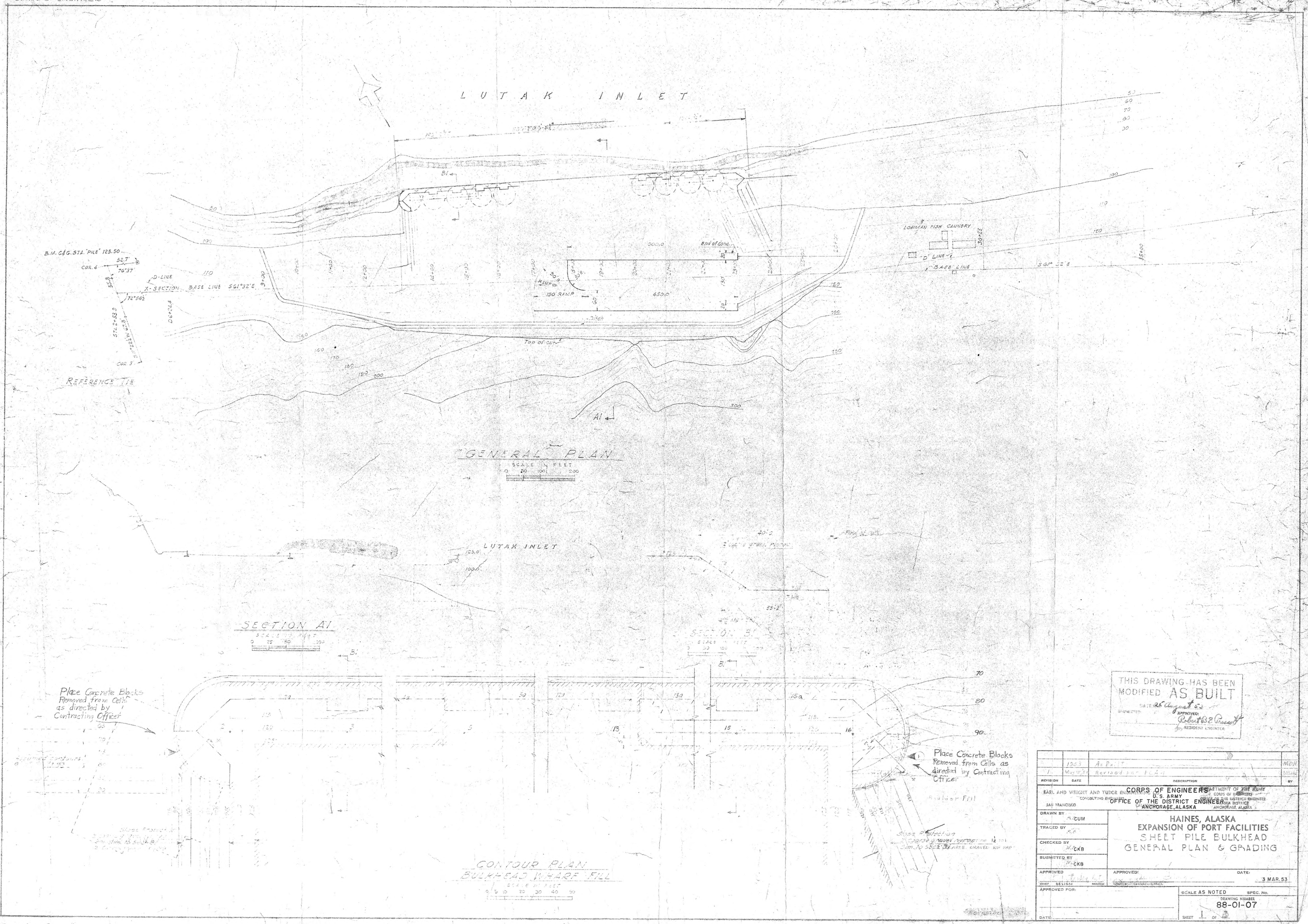


SECTION A 2
SCALE 1/4 IN = 1 FT.

LUTAK INLET



CORPS OF ENGINEERS



THIS DRAWING HAS BEEN MODIFIED AS BUILT
DATE 26 August 53
SUBMITTED
APPROVED: Robert B. E. Prescott
for RESIDENT ENGINEER

REVISION	DATE	DESCRIPTION	BY
	1953	As Built	MDH
1	May 15/53	Revised for PCA	MDH

EARL AND WRIGHT AND TUDOR ENGINEERING CO.
CONSULTING ENGINEERS
SAN FRANCISCO

CORPS OF ENGINEERS
U.S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ANCHORAGE, ALASKA

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
ALASKA DISTRICT
ANCHORAGE, ALASKA

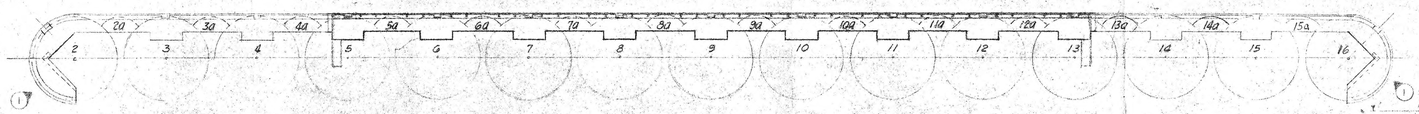
DRAWN BY: R. CUM
TRACED BY: R.B.
CHECKED BY: W.K. CKB
SUBMITTED BY: W. CKB

HAINES, ALASKA
EXPANSION OF PORT FACILITIES
SHEET PILE BULKHEAD
GENERAL PLAN & GRADING

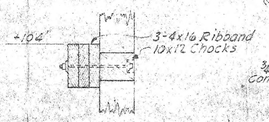
APPROVED: CHIEF DESIGN BRANCH
APPROVED: CHIEF, DESIGN & SERVICES DIVISION
DATE: 3 MAR. 53
APPROVED FOR:
DATE:
SCALE AS NOTED
SPEC. No.
DRAWING NUMBER 88-01-07
SHEET 1 OF 3

B-4-9

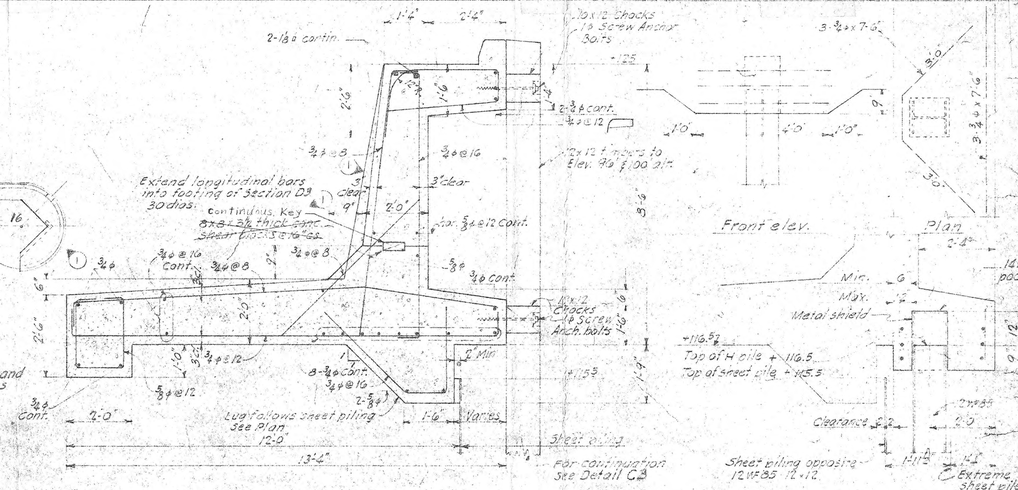
LUTAK INLET



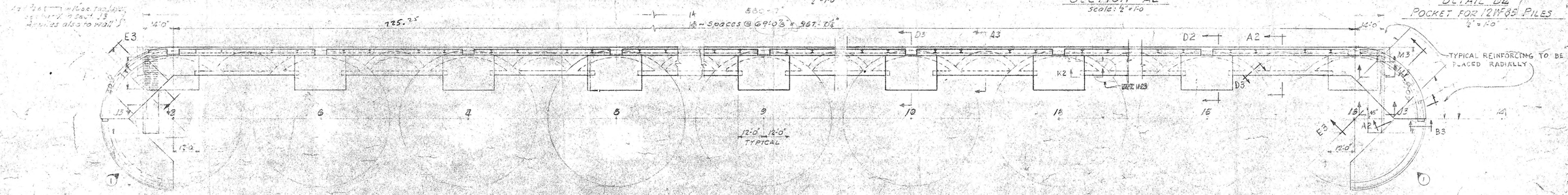
GENERAL PLAN
Scale: 1" = 50'



DETAIL C3
1/2" = 1'-0

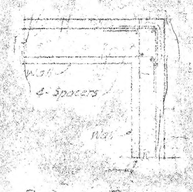


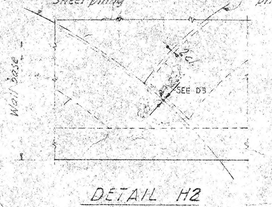
SECTION A2
Scale: ½" = 1'-0



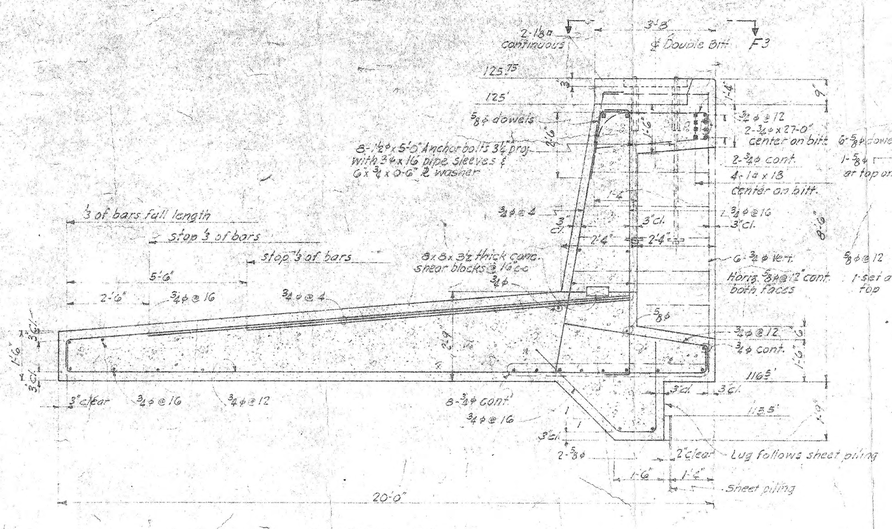
PLAN OF CONCRETE BULKHEAD WALL
Scale: 1" = 20'

DETAIL B2
POCKET FOR 12WF85 PILES
½" = 1'-0

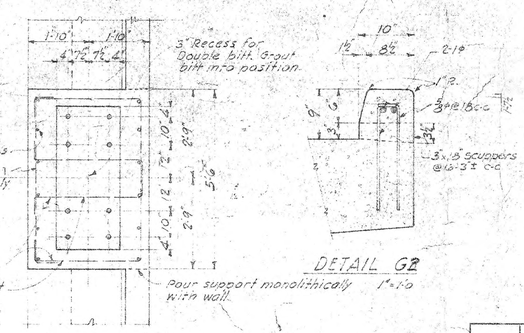




DETAIL H2
No scale



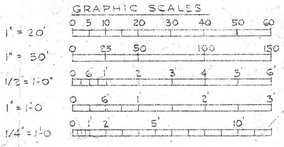
SECTION D2
Scale: ½" = 1'-0



PLAN F2
Scale: ½" = 1'-0

DETAIL G2
1" = 1'-0

GRAPHIC SCALES



THIS DRAWING HAS BEEN MODIFIED AS BUILT
DATE 20 August 53
SUBMITTED: APPROVED:
RESIDENT ENGINEER

GENERAL NOTES

WORKING STRESSES:
1. DESIGN STRESSES: F_c 1,200,000 P.S.I.
F_{cl} 3000 P.S.I.
F_{cc} 1,350 P.S.I.
2. CONCRETE TO BE POURED DIRECTLY ON FILL. OMIT PAPER.
3. ALL RIGHT ANGLE BENDS TO EXTEND 16 BAR DIAMETERS UNLESS OTHERWISE NOTED.
4. BARS SHOWN SQUARE SHALL BE ROUND BARS OF EQUIVALENT CROSS-SECTIONAL AREA AND WEIGHT.
5. ALL CONCRETE ON SHEET 2 TO BE CLASS A.
6. SPLICE ALL CONTINUOUS BARS BY LAPPING 30 DIAS.

REVISION	DATE	DESCRIPTION	BY
		As Built	MEH
		1:44	MEH

EARL AMD WRIGHT AND TUDOR ENGINEERS, CONSULTING ENGINEERS, SAN FRANCISCO

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
U.S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ANCHORAGE, ALASKA

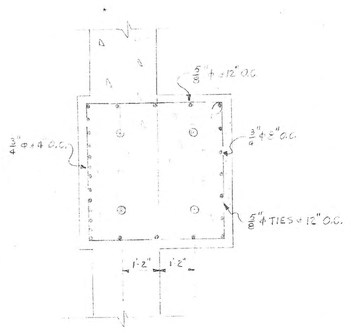
DRAWN BY: CJM
TRACED BY: R.S.
CHECKED BY: CKBH
SUBMITTED BY: CKB
APPROVED: CHIEF, DESIGN BRANCH
APPROVED: DATE: 3 MAR 53
APPROVED FOR:
DATE:

HAINES, ALASKA
EXPANSION OF PORT FACILITIES
SHEET PILE BULKHEAD RETAINING WALL

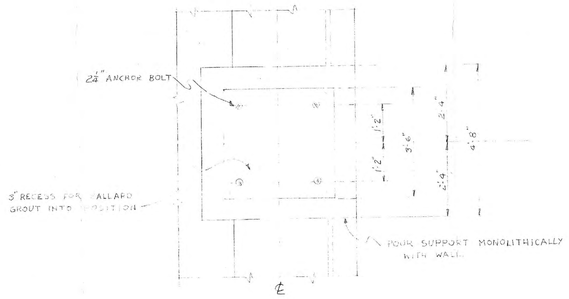
SCALE AS NOTED SPEC. No.
DRAWING NUMBER 88-01-07
SHEET 2 OF 5

Nov 7, 1952

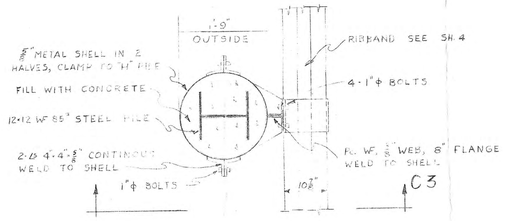
B-H-1



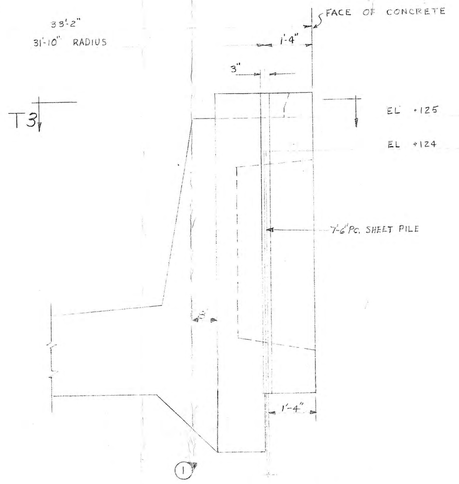
SECTION A3
SCALE 1/2" = 1'-0"



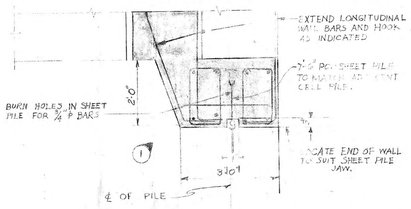
SECTION E3
SCALE 1/2" = 1'-0"



PLAN



SECTION B3



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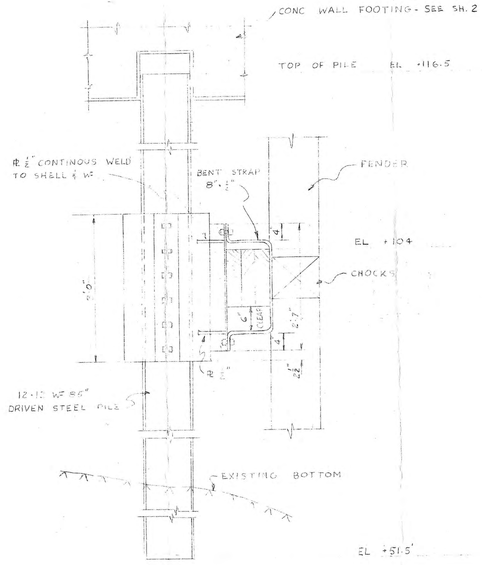
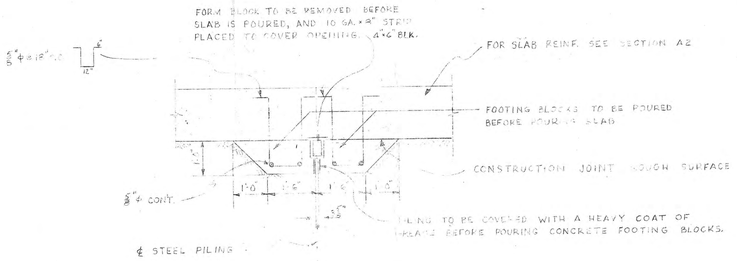


ELEVATION C3

DETAIL OF "H" PILE
SCALE 3/4" = 1'-0"



SECTION D3
SCALE 3/4" = 1'-0"

REV.	DATE	RCA NO.	ACTION	DESCRIPTION	BY	APP'D.
1	1953			Modified As Built	MDH	

CORPS OF ENGINEERS
U. S. ARMY
OFFICE OF THE DISTRICT ENGINEER
ANCHORAGE, ALASKA

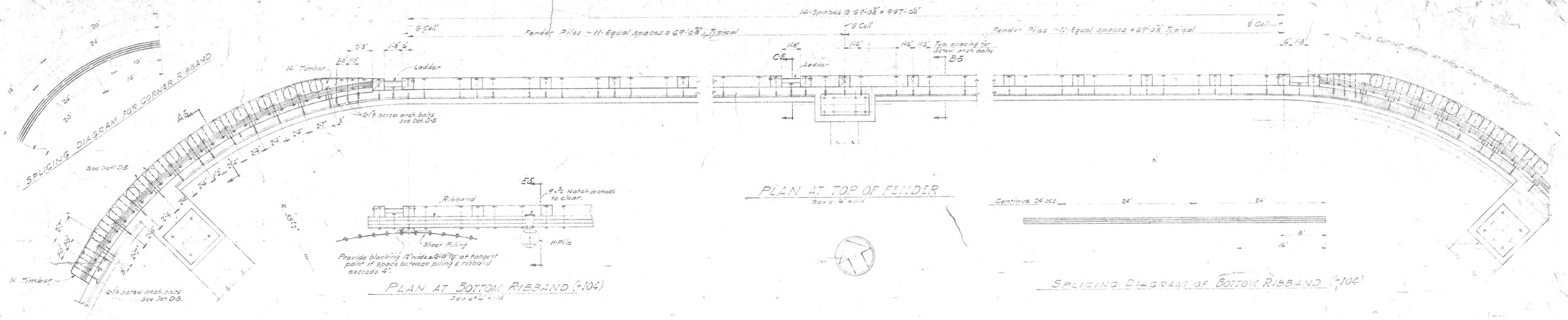
DESIGNED:
DRAWN: CUM
TRACED:
CHECKED: CKB
SUBMITTED: CKB

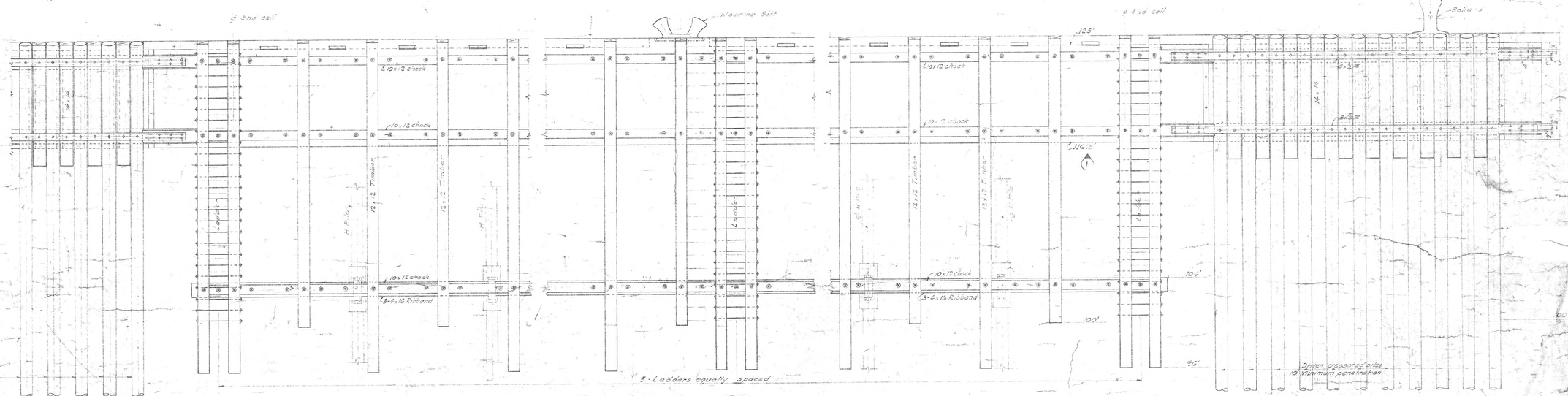
HAINES, ALASKA
EXPANSION OF PORT FACILITIES
STRUCTURAL
MISCELLANEOUS SECTIONS

RECOMMENDED: [signature] Act. CHIEF, DESIGN BRANCH
APPROVED: [signature] CHIEF, ENGINEERING DIVISION
DATE 3 MAR 53
APPROVED:
SCALE AS NOTED
SPEC. NO.
DRAWING NUMBER 88-01-07
SHEET 3 OF 5

3-4

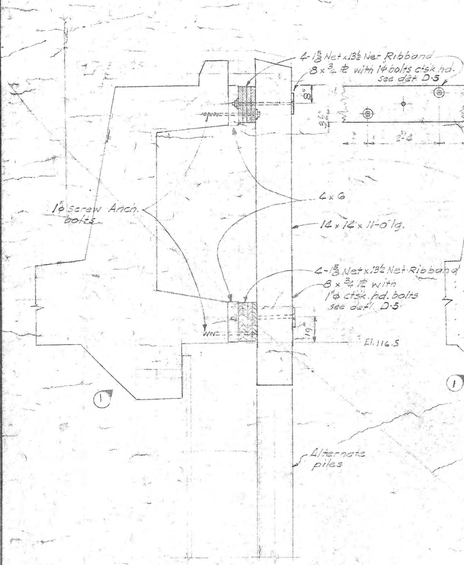
CORPS OF ENGINEERS



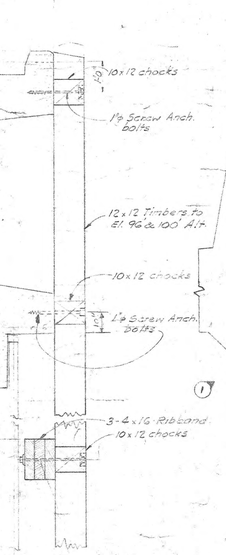


FENDER ELEVATION
Scale 1/4" = 1'-0"

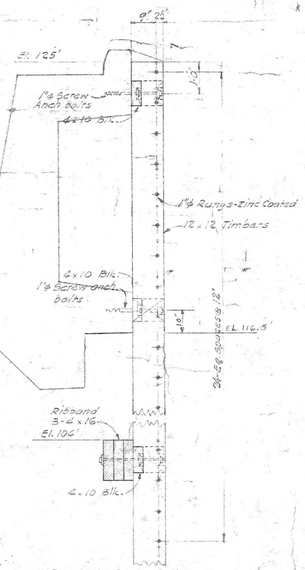
ELEVATION OF DEVELOPED CORNER



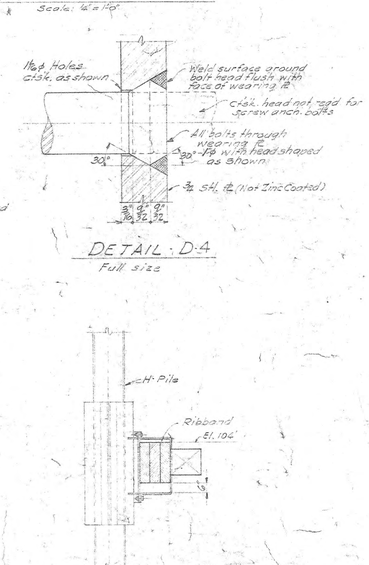
SECTION A-4
¼" = 1'-0"



SECTION B-4
¼" = 1'-0"



SECTION C-4
¼" = 1'-0"



DETAIL : D-4
Full size

SECTION E-4
¼" = 1'-0"

NOTES:
1- Material: All bolts shall be ℄ unless noted.
All piles and timbers shall be creosoted.
The 3-4x16 straps and face, 2" thick - rough hardware including bolts, nuts, washers, screw anchors, ladder rungs and nails and spikes shall be zinc coated.
2- Washers shall be rolled ends - the head and nut of all bolts bearing on wood or concrete. Additional lead washers required under washers bearing on concrete.
3- Holes in timbers for bolts and ladder rungs shall be drilled the same diameter as the bolt or rung.
4- The 4-16 Ribband timbers shall be fitted together with 7/8" ø holes in two rows staggered @ 4'-0 centers in each row in addition to the bolts shown. The laminations of the 16" x 3" Ribband timbers at the corners shall be bolted together with 20d spikes in two rows staggered spaced 9'-0" on ℄ in each row.
5- Chocks before treatment shall be framed ½" longer than required and shall have the holes and countersinks for bolts drilled in placing ends the ends shall be re-cut as necessary to obtain a close fitting tightly against adjacent fender timbers. If necessary to cut more than ½" from end of a chock the cut end, before placing, shall be treated as specified for cut and shaped surfaces of piles.

THIS DRAWING HAS BEEN MODIFIED AS BUILT
DATE 25 August 53
SUBMITTED: APPROVED: Robert R. Prescott for RESIDENT ENGINEER

REVISION	DATE	DESCRIPTION	BY
2	1949	As Built	MET
1	May 1951	Revised for RCA-3	MET

EARL AND WRIGHT AND TUDOR ENGINEERING CO.
CONSULTING ENGINEERS
SAN FRANCISCO CALIFORNIA

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
ALASKA DISTRICT
ANCHORAGE, ALASKA

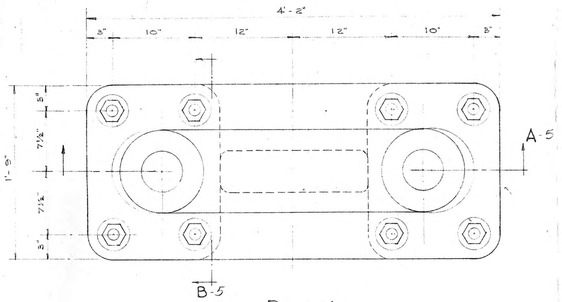
DRAWN BY R.S.
TRACED BY R.S.
CHECKED BY Jr.N.
SUBMITTED BY Earl & Wright Engrs.
APPROVED
APPROVED FOR:
DATE:

HAINES, ALASKA
EXPANSION OF PORT FACILITIES
BULKHEAD WHARF SUBSTRUCTURE
STRUCTURAL DETAILS
FENDER SYSTEM

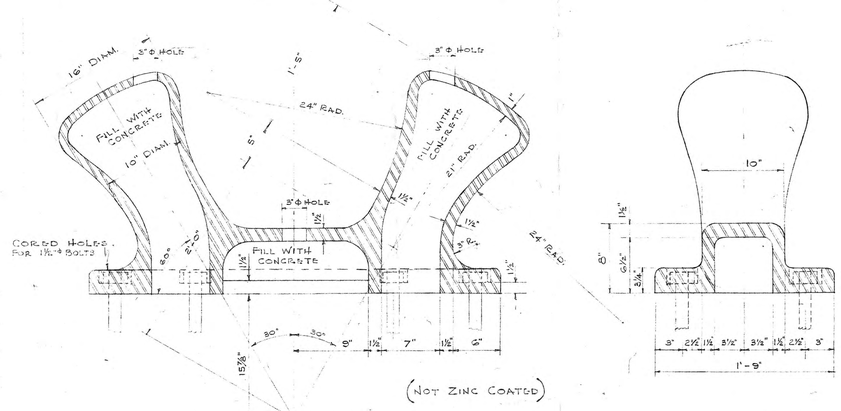
APPROVED: DATE: 3 Mar 1953
CHIEF, SEATTLE DISTRICT OFFICE, SEATTLE

SCALE As Noted SPEC. NO.
DRAWING NUMBER 88-01-07
SHEET 51 OF 53
INV. NO. S 35-507-51-109

17 April 1951



PLAN

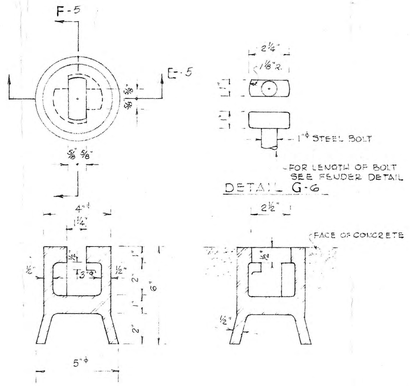


SECTION A-5

SECTION B-5

DETAIL OF C.S. DOUBLE MOORING BITT (4-REQUIRED)

SCALE - 1½" = 1'-0"



DETAIL G-6

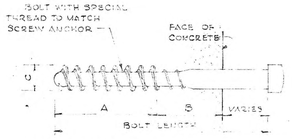
SECTION E-5

SECTION F-5

C.I. ANCHOR BOLT INSERT & T BOLT

SCALE 3" = 1'-0"

THIS DETAIL MAY BE USED AT THE OPTION OF THE CONTRACTOR IN LIEU OF THE SCREW ANCHOR BOLT.



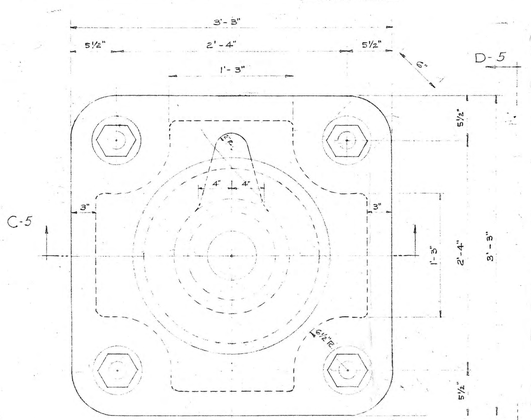
ASSEMBLY OF BOLT & SCREW ANCHOR

SCREW ANCHOR

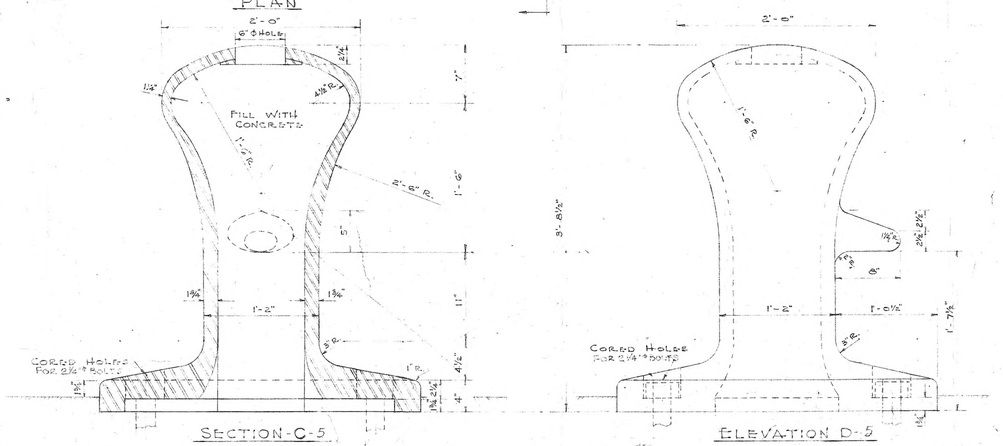
BOLT SIZE DATA			
SIZE	A	B	C
3/4" ϕ	4"	3½"	1¼"
1" ϕ	6"	4"	1½"

"RICHMOND" SCREW ANCHOR & BOLT

SCALE 3" = 1'-0"



PLAN



SECTION-C-5

ELEVATION D-5

DETAIL OF C.S. BOLLARD (NOT ZINC COATED) (2-REQUIRED)

SCALE 1½" = 1'-0"

SCALE IN INCHES 0 3' 6" 12" 18"

LEGEND

C.S. = CAST STEEL

C.I. = CAST IRON

THIS DRAWING HAS BEEN MODIFIED AS BUILT

DATE 29 September 53

SUBMITTED: APPROVED: Robert R.E. Prescott Proj. Eng. for DISTRICT ENGINEER

REVISION	DATE	DESCRIPTION	BY
1	1953	As Built	MDH

EARL AND WRIGHT AND TUDOR ENGINEERING CO.
CONSULTING ENGINEERS
SAN FRANCISCO CALIFORNIA

DEPARTMENT OF THE ARMY
CORPS OF ENGINEERS
OFFICE OF THE DISTRICT ENGINEER
ALASKA DISTRICT
ANCHORAGE, ALASKA

DRAWN BY H.L.G.
TRACED BY V.R.V.
CHECKED BY H.H. GILBERT
SUBMITTED BY

HAINES, ALASKA
EXPANSION OF PORT FACILITIES
BULKHEAD WHARF SUBSTRUCTURE
STRUCTURAL DETAILS
DECK FITTINGS

APPROVED: CHIEF, BRANCH
APPROVED: R.E. Chandler CHIEF, SEATTLE BRANCH OFFICE, SEATTLE
DATE: 18 Apr. 1951
APPROVED FOR:
DATE:
SCALE AS NOTED SPEC. No.
DRAWING NUMBER 88-01-07
SHEET 5 OF 5

17 APRIL 1951

INV. NO. S95-507-51-100

3-1

REF #12

EM 1110-2-2503 – Design of Sheet Pile Cellular Structures, Cofferdams and Retaining Structures.

CECW-EP Engineer Manual 1110-2-2503	Department of the Army U.S. Army Corps of Engineers Washington, DC 20314-1000	EM 1110-2-2503 29 September 1989
	Engineering and Design DESIGN OF SHEET PILE CELLULAR STRUCTURES COFFEDAMS AND RETAINING STRUCTURES	
	Distribution Restriction Statement Approved for public release; distribution is unlimited.	

CECW-ED 11 June 1990

Errata Sheet

No. 1

ENGINEERING AND DESIGN

Design of Sheet Pile Cellular Structures

EM 1110-2-2503

29 September 1989

Cover Letter: Replace unsigned letter with the enclosed signed letter.

Page A-l: Delete Reference 16. EM 1110-2-2501 has been superceded by EM 1110-2-2502.

REF #13

"Cellular Cofferdams – Developments in Design and Analysis," Clough and Martin, 1988.

Proceedings: Second International Conference on Case Histories in Geotechnical Engineering, June 1-5, 1988, St. Louis, Mo., Invited Paper

Cellular Cofferdams—Developments in Design and Analysis

G. Wayne Clough
Professor and Head, Civil Engineering Department, Virginia Polytechnic Institute and State University, USA

James R. Martin, II
Research Assistant, Civil Engineering Department, Virginia Polytechnic Institute and State University, USA

SYNOPSIS: Cellular cofferdams are reviewed from the point of view of recent developments, with a particular focus on movements. It is found that applications for cofferdams are expanding, with uses extended beyond the conventional temporary systems to permanent navigation and retaining structures. Results from instrumented cofferdams show that some conventional analysis procedures for predicting interlock force and internal stability are excessively conservative. Where conditions do not deviate far from the norm, movement patterns for cofferdams can be quantified and predicted. For more exotic cases, finite element procedures are available that yield reasonable predictions of behavior.

INTRODUCTION

Cellular cofferdams are normally temporary, serving to surround an area in a body of water so that it can be dewatered and used for construction. A plan view of this type of system is shown in Figure 1 for the Stage 1 cofferdam used for construction of a portion of the replacement Lock and Dam 26 on the Mississippi River. The cofferdam is formed by means of linked cells, each of which is created by driving interlocking steel sheetpiles. The cell may be circular, diaphragm, or cloverleaf in shape, although circular is the most common. As shown in Figure 1, the circular shape actually only applies to the main cells, with the circular cells connected by smaller arc cells. The wall that the main cell and arc cell share is termed the common wall. Where the common wall joins the converging main and arc cells, a special wye or tee element is used.

For many years, cofferdam technology was relatively static. Design methods remained untested, and in many cases gave conflicting answers. In recent years, this situation has changed. Cofferdams have found use in a wider variety of applications, including retaining structures, waterfront and harbor structures, and floodwalls. This has led to new forms of loading, increased concern about movements, and the use of the cofferdam as a permanent system.

The past 15 years have also seen a number of new investigations of cofferdam behavior. In a few, key cases, cofferdams have been instrumented, leading to data that allows behavior trends to be better understood, particularly as to movements. Also, several groups have adapted the finite element method to cofferdams, leading to insights into the nature of the soil-structure interaction process in these systems, and a linkage of movements in the structure and the soil. Finally, other workers have developed modifications to the conventional design technology. This paper reviews each of these subjects, and assesses the potential for future developments.

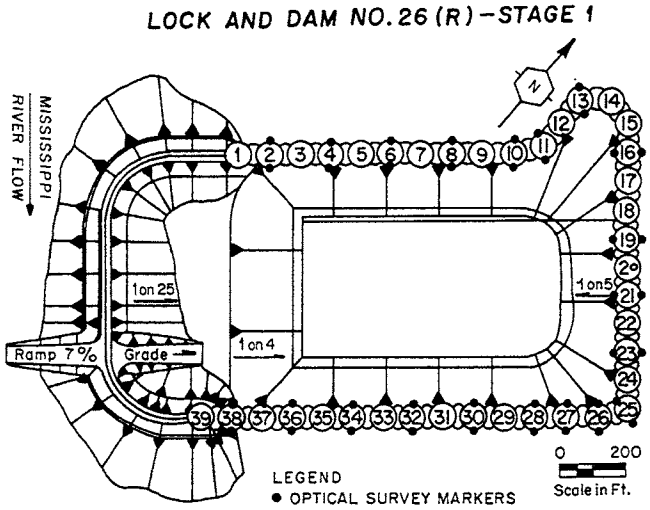


Fig. 1 - Layout and Instrumention of Stage 1 Cofferdam

CONVENTIONAL ANALYSIS AND DESIGN PROCEDURES

Conventional geotechnical design considerations for cofferdams include internal and external stability (Figure 2). Internal stability is concerned with the degree of loading of the sheetpile interlocks and connecting elements, and the possibility of a shear failure developing through the cell fill under the distortions that occur during lateral loading. External stability considers the possibility that the cofferdam behaves as a unit and is subject to bearing failure, sliding on the base, or overturning. Special considerations for cofferdams on rock relate to the possibility of sliding on weak planes in the rock. Broader aspects of cofferdam design involve overtopping, marine impact, ice loading, and construction related problems.

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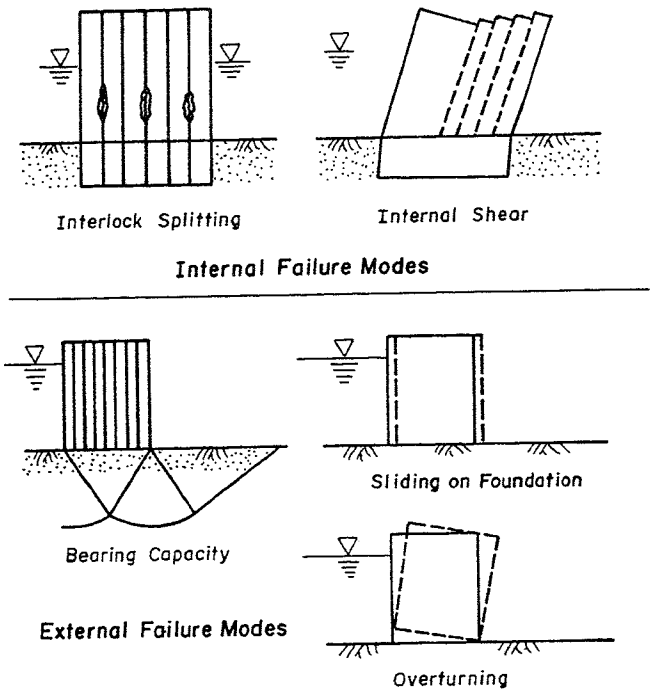

Fig. 2 - Typical Failure Modes of Cellular Cofferdams

Lacroix, et al. (1970) and Rossow, et al.(1987) have provided thorough reviews of conventional design approaches. These efforts will not be repeated in this section, rather only those areas where some new knowledge has been developed will be discussed. These include methods for calculating interlock loads for main or arc cell elements, interlock loads for the common wall, and internal stability.

There are a number of conventional methods for determining interlock loads. All are based upon the equation:

$$t = p \times r \quad \ldots\ldots\ldots\ldots\ldots \quad (1)$$

in which t = interlock load, p = pressure exerted by the fill, normally as applied to the inboard side of the cell, and r = radius of the cell. Differences in the methods lie in the distribution assumed for the fill pressure, and in the maximum value assigned for the pressure. In Figure 3 four commonly used alternatives are given: Terzaghi (1945), Tennessee Valley Authority (1957), Corps of Engineers, as outlined in NAVDOCKS Design Manual DM7 (1971), and Schroeder and Maitland (1979). For purposes of convenience, these will henceforth be referred to by the symbols TZ, TVA, COE, and S&M. The S&M approach represents the most recent development. As is seen in Figure 3, the TZ and COE methods assume triangular pressure distributions which continuously increase with depth, while the other two increase to a certain depth, and then decrease to zero. It is important to note that the TVA and S&M distributions are not intended to reflect the actual earth pressure distribution. Rather, they are designed to be used to generate the correct distribution of interlock load when using Equation (1). These methods reflect the fact that the interlock load must decrease near the dredge line because of the restraint offered by the embedded portion of the cell. Instrumentation data from cofferdams and finite element analyses support this concept. Further comment on this will be made later in this paper.

The common wall is subjected to a pull from both the main and arc cells. Thus, it is generally accepted that the common wall is subjected to a higher force than either the main or arc cell wall. The calculation of forces in the common wall is usually based on one of two approaches: (1.) The secant formula of the TVA:

$$t_{cw} = p\ L(\sec\ \theta) \quad \ldots\ldots\ldots\ldots\ldots \quad (2)$$

in which L = centerline distance between main and are cells; and θ = angle between centerline of main cell and wye connection point; and, (2.) The Swatek (1969) formula:

$$t_{cw} = p\ L \quad \ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

Of the two, the secant formula gives the larger force prediction, and as will be shown subsequently, field data support the Swatek approach.

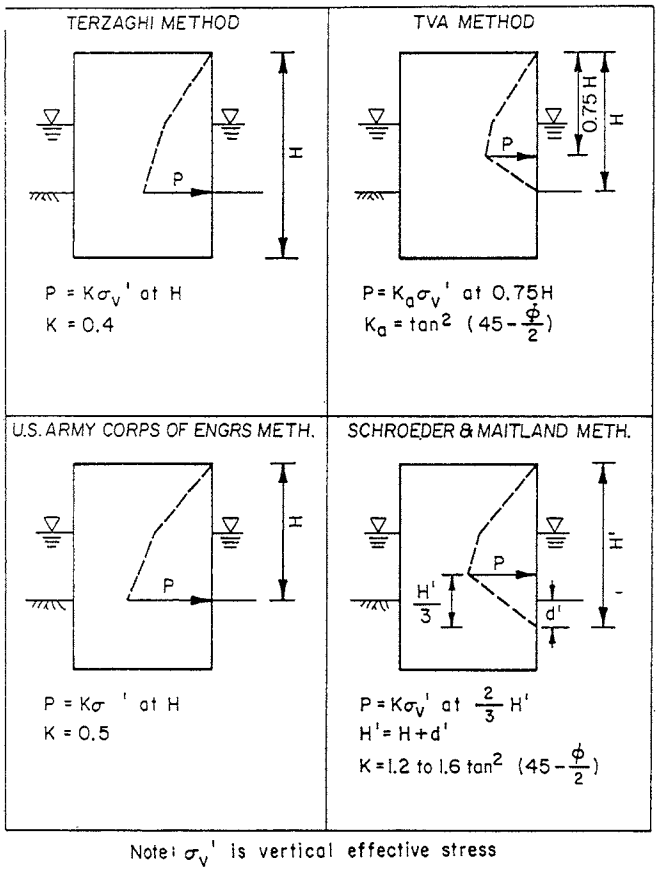

Fig. 3 - Pressure Distributions for Calculation of Interlock Forces

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A number of alternative methods have been proposed for analysis of internal shear failure under the action of lateral loading. Probably the most widely accepted procedure is that postulated by Terzaghi (1945). As shown in Figure 4, he assumes that the soil in the cell undergoes shear along a vertical plane due to the overturning moment generated by the resultant of the lateral loads acting on the cofferdam. The external shear force on the base of the cofferdam fill, Q, is a function of the external or driving moment, M. Q is related to the driving moment as Q = 3M/2b, where b = the diameter of the cell. The force Q is resisted by the internal soil friction on the assumed vertical shear plane, and the friction in the interlocks, and the factor of safety against internal shear failure by this technique is:

$$F = \gamma \frac{bH}{3M} K (\tan \phi + f)$$

where, f = coefficient of friction on the interlocks, K = the earth pressure coefficient acting on the vertical plane, γ = unit weight of the soil, and H = the height of the cell. The principal unknown in this approach is the value of the parameter K. Terzaghi assumed it to be 0.5. Schroeder and Maitland (1979) argue that this is too conservative, and that a more reasonable value is 1.0, thus doubling the allowable resultant lateral load. Finite element analyses have tended to support this finding (Singh and Clough, 1988).

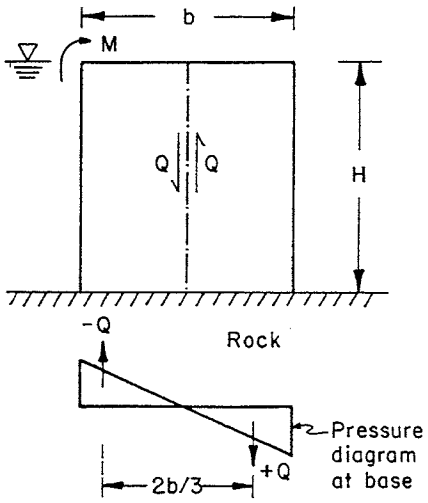

Fig. 4 - Terzaghi's Design Method for Cellular Cofferdams

Alternative methods for internal failure in the fill to that of Terzaghi were proposed by Krynine (1945), Hansen (1953), and Cummings (1957). Reasonable doubt still exists over which of the procedures is correct, and no cofferdam has been documented to have failed in this manner. Model tests by Maitland and Schroeder (1979) support the Terzaghi approach if the earth pressure coefficient used is 1.0, and not 0.5 as suggested by Terzaghi. However, in the model tests, the cofferdam had to be tilted 50 % of the cell height to generate the failure. Thus, it appears that a better approach to design would be to use limiting deformations as the control, since the large movements required to cause internal shear would obviously be unacceptable. In following sections of this paper, methods are suggested to allow calculation of the likely movements of a cellular cofferdam.

FINITE ELEMENT ANALYSIS PROCEDURES

Conventional analysis techniques for cofferdams are largely semi-empirical, and are limited to conditions similar to those for which we have an experience base. The concepts of soil-structure interaction upon which the conventional methods are based are elementary, and do not reflect the true nature of the problem in many instances. Further, they are unable to predict cofferdam movements and deflections. The latter drawback is important in where the cofferdam is deformation sensitive, or in the case deformations are to be monitored and used to judge the performance of the cofferdam.

Early efforts at finite element analyses of cofferdams were conducted without a full appreciation of the complexity of the system, particularly relative to the extra flexibility in the cells provided by the interlocks. Clough and Hansen (1977) first applied the method to a real cofferdam in the analysis of the Willow Island Cofferdam. This effort focused on the effects of lateral loading using a two-dimensional "vertical slice" model. The cell was represented by front and back bending elements connected by a series of flexible springs. The spring stiffnesses were derived on the basis that the cell acted as a perfect pressure vessel. The cell fill model incorcorporated allowances for nonlinear behavior, and provisions were made for slip between the cell walls and the cell fill. Deformation predictions using this model were reasonable for lateral loading, but were unrealistically small for the filling stage. Stevens (1980) later used the Clough and Hansen model to reanalyze the Willow Island Cofferdam, but he reduced the stiffness of the connecting springs for the front and back walls to allow for more flexibility, and hence to indirectly account for interlock induced deformations in the cell. This approach produced reasonable predictions of movements for both filling and lateral loading, but there was no general method available to determine the degree by which the flexibility should be reduced for interlock influences.

An intensive investigation into finite element analyses of cofferdams was triggered by the construction and instrumentation of the Lock and Dam 26 Replacement (R) cofferdam. Using the results of a series of tests on sheetpile assemblies, the senior author and his coworkers were able to develop three two-dimensional models which incorporated many of the key aspects of cofferdam behavior (Clough and Kuppusamy, 1985; Kuppusamy, et al., 1985). Each of the two-dimensional models addresses a different aspect of the problem. All incorporate allowances for interlock yielding, nonlinear soil behavior, slip between the cell fill and natural soils and the steel sheetpiles, and construction sequence. Further

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work by Mosher (1988) has extended these concepts to three-dimensional finite element analyses.

Figure 5 shows the results of an axisymmetric analysis of the filling of one of the main cells of Lock and Dam 26 (R). The results are given in terms of radial deflections of the cell, and interlock forces as a function of the degree of flexibility of the cell. The flexibility of the cell is described in terms of the parameter "E-Ratio," which is the ratio of the modulus used for the cell in the horizontal direction to that used for the vertical direction. To allow for interlock stretching, the E-Ratio must be less than one. It can be shown from interlock tests, and by comparison of predicted results to observed that the appropriate E-Ratio for the cell filling stage is in the range of 0.03-0.05. Importantly, with this level of E-Ratio, the cells typically expand enough on filling to cause active conditions within the cell fill, and lead to the lowest possible interlock forces.

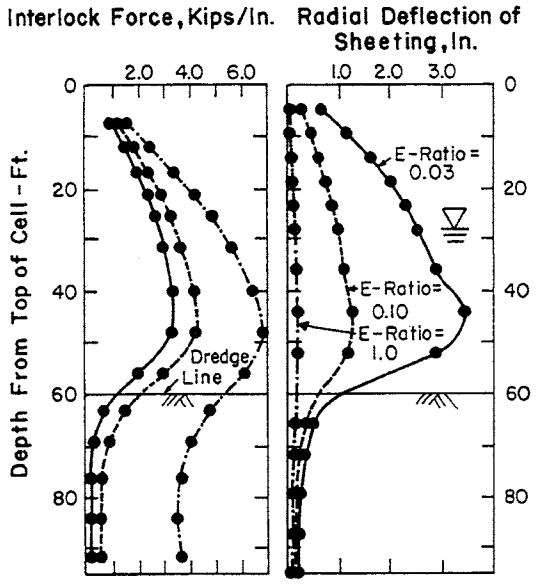

Fig. 5 - Results of Axisymmetric Analysis- End of Cell Filling - Lock & Dam 26 (R)

The contours of lateral stresses predicted for the cell fill for Lock and Dam 26 (R) Stage 1 cofferdam after completion of filling are given in Figure 6. The stress contours reveal a pattern of arching, caused by significant portion of the vertical weight of the fill being picked up by the shear between the cell sheetpiles and the fill. As reported by Clough and Goeke (1986), the arching was reflected in differences in insitu test results performed near the sheetpiles and near the center of the cells for the Lock and Dam 26 (R) Stage 1 cofferdam.

Figures 7 and 8 show comparisons of results predicted by the axisymmetric and three-dimensional finite element models for cell filling, and those obtained in the instrumentation program for Lock and Dam 26 (R) cofferdam. Results from both of the finite element models reasonably match the averages of the observed data for deflection and interlock loads. Movements of the instrumented cells are also closely predicted for differential loading caused by dewatering and flooding, Clough and Kuppusamy (1985). Further analyses of other cofferdams by Singh and Clough (1988) using the finite element tools support the idea that if proper parameters are defined for the models, realistic predictions of cofferdam behavior can be obtained.

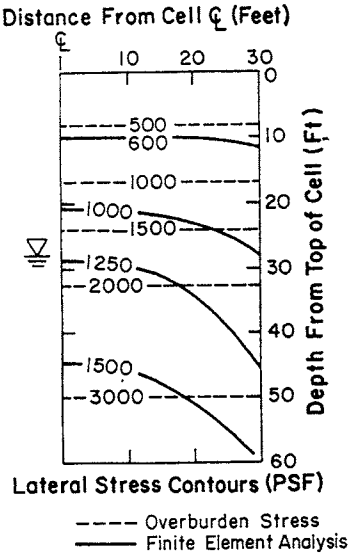

Fig. 6 - Lateral Stress Contours - End of Cell Filling - Lock & Dam 26 (R)

Recent design applications of the finite element models have been made for cofferdams used as floodwalls (Peters, 1987), and waterfront structures. Figure 9 shows predicted deflections for a cell used for a proposed offloading facility in Alaska which is subjected to strong ice loading just at the water line. The ice loading causes the cell to be distorted at the water line, and also to be translated as a unit by the ice loading. Analyses of this type were used to help select cell fill properties and foundation preparation techniques for the offloading facility so as to hold cell deflections within tolerable limits. Instrumentation results will be available in the near future for several projects to check the validity of the finite element predictions. However, the methods have already proven much of their merit in providing information to aid in critical design decisions.

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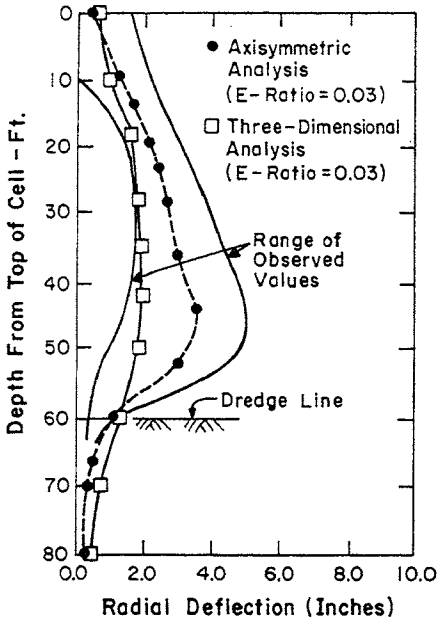

Fig. 7 - Comparison of Radial Deflection from Axisymmetric and Three-Dimensional Analyses - End of Filling

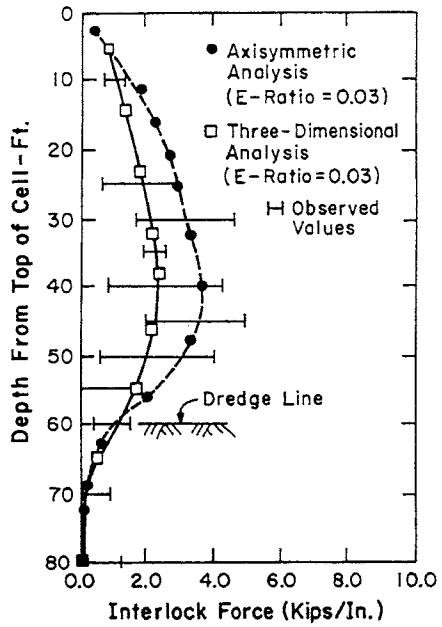

Fig. 8 - Comparison of Predicted Interlock Force from Axisymmetric and Three-Dimensional Analyses - End of Filling

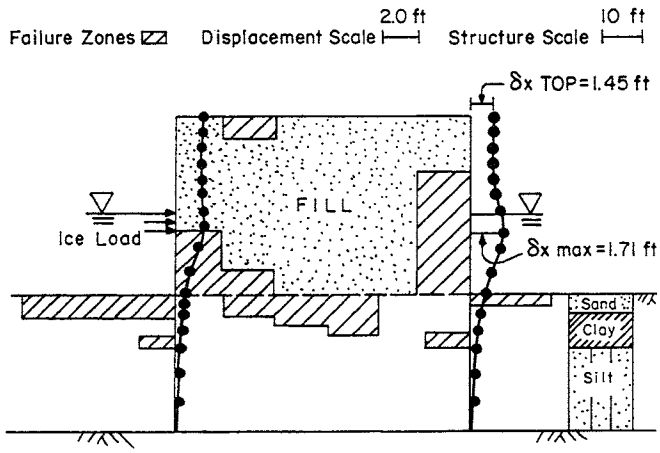

Fig. 9 - Predicted Deflections for Proposed Alaskan Offloading Facility

DESCRIPTION OF COFFERDAMS USED IN CASE HISTORY REVIEW

In Table 1 the key aspects are given for the six cofferdams used in this case history review. The first four are more conventional cases where the cofferdam serves as a temporary system to allow dewatering for construction within a body of water. The latter two are permanent systems, serving as waterfront and soil retaining structures. Instrumentation to monitor the cell filling stage was successfully installed for only two of the cases, the Lock and Dam 26 Stage 1 cofferdam, and the cofferdam for the Trident Drydock.

The diameter of the six cofferdams ranged from 62 to 76 ft, and their height above the dredge line varied from 36 to 80 ft. Sheetpile embedments varied widely, with the smallest at the Willow Island cofferdam where a rock foundation was present, to the largest at the Lock and Dam 26 (R) Stage 1 cofferdam with 35 ft of embedment.

Fill for all of the cofferdams was sand, although in the Seagirt case a 10 ft layer of soft silt soil remained in the cell beneath the fill. In the case of the Trident and Fulton Terminal 6 cofferdams the cell fills were densified using vibratory compaction after they were placed. The Trident case is also unique in that with unwatering of the interior of the cofferdam, the cells themselves were dewatered to increase the weight of the cell fill. In other cases, the cell fill remains partially saturated with water seeping from outboard to inboard side. The Lock and Dam 26 (R) cofferdams used special wellpoints just on the interior of the cofferdam to insure that the seepage water level was kept below dredge line.

Foundations for all of the case histories were generally sound, with the materials composed of dense sands, rock, or hard clays. The poorest foundation was for certain areas of the Seagirt cofferdam, where in some places it was composed of a fissured stiff clay.

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TABLE 1. CASE HISTORY INFORMATION

Cofferdam	Cell Ht. (Ft.)	Cell Diam. (Ft.)	Foundation Conditions	F_{mc}* (K/in.)	F_{cw}* (K/in.)	Ht.of Fmc (Ft.)	Max. Diff'l Load	Max.Diff'l Diff'l Load	Avg.Defl. Max.Diff'l Load	References
Lock & Dam 26 Stage 1	60	63	Sand, med. to dense	4 (1.3 Ka)	5.8	H/4	60 ft. Water	5.7 in. (.8% H)	3.8 in. (.5% H)	Shannon and Wilson (1983)
Lock & Dam 26 Stage 2	60-80	63	Sand, med. to dense	NA	NA	NA	80 ft. Water	5 in.** (.6% H)	3.5 in.** (.4% H)	Martin and Clough (1988)
Trident Drydock	80	76	Dense Sand & gravel	8 (2.1 Ka)	8	H/8	80 ft. Water	4 in. (.4% H)	3.0 in. (.3% H)	Sorota et al., (1981)
Willow Island	55	65	Rock & Hard Clay	NA	NA	NA	40 ft. Water	1.5 in. (.2% H)	NA	Stevens (1980)
Seagirt Terminal	36	62	Sand & Stiff Clay	NA	NA	NA	55 ft. Slurry	Several Feet	NA	Clough and Duncan (1986)
Fulton Terminal 6	60	66		NA	NA	NA	60 ft. Compacted Backfill	11 in. (1.5% H)	5.5 in. (.8% H)	Schroeder (1987)

*F_{mc}, F_{cw} = Max. Interlock Force after filling: Main Cell, Common Wall, respectively; NA = Not Available
**As of December 1986

Loading was provided by water for most of the cofferdams, except for the Seagirt and Fulton Terminal 6 cases. The Seagirt cofferdam, was designed to support dredged spoil, and the Fulton Terminal 6 cofferdam carried the load of the sand fill used to form the base for a wharf.

REVIEW OF CASE HISTORY DATA FOR CELL FILLING

Relatively few cofferdams have been instrumented and monitored for the cell filling stage. This is attributable to the problems of maintaining workable instrumentation on the sheetpiles as they are driven, and in obtaining reasonable zero readings on them before they are stabilized by filling the cells. Fortunately, as noted earlier, in two instances instrumentation has been successfully implemented during cell filling. In the case of both the Trident drydock (Sorota, et al., 1981) and the Lock and Dam 26 (R) Stage 1 cofferdams (Moore and Kleber, 1985), strain gages were applied to cells to allow determination of interlock forces during filling. Only in the case of Lock and Dam 26 (R) Stage 1 cofferdam were the inclinometers in place to measure cell movements during filling. A section through the Lock and Dam 26 (R) Stage 1 cofferdam is shown in Figure 10. As can be seen from the date in Table 1, the two cofferdams are similar in many characteristics, although the Trident cells had a greater height, less sheetpile embedment, a smaller diameter to height ratio, and was supported by a better foundation material. Notably, after fill placement, the cell fill in the Trident case was densified using a vibratory technique.

Interlock Forces on Filling

In Figure 11 the ranges of the measured main cell interlock forces for the two cases immediately after fill placement are shown. The trend in the data is for the interlock forces to increase with depth to a point slightly above the dredge line, and then to decrease after that. With compaction of the fill for the Trident cofferdam, the interlock stresses showed a clear increase over the placement values. For purposes of comparison with predicted interlock forces the measured values at the completion of cell filling are shown with those determined using the the TVA

Fig. 10 - Typical Cell Schematic of Lock & Dam 26 (R)

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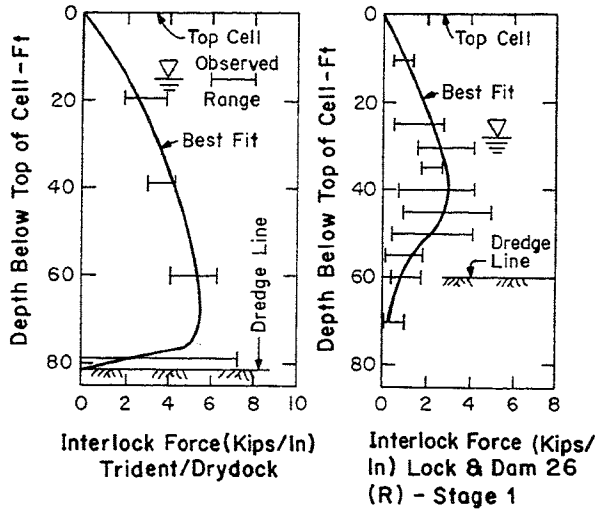

Fig. 11 - Measured Interlock Forces for Trident and Lock & Dam 26 (R) Cofferdams

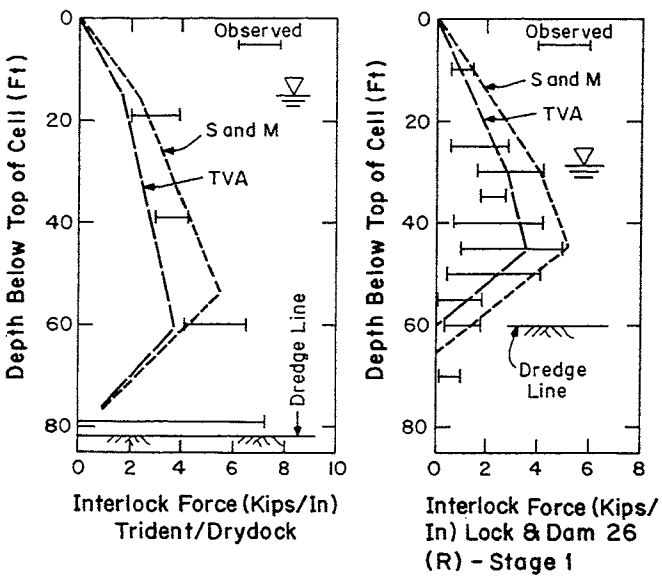

Fig. 12. - Comparison of Predicted TVA and S&M Interlock Forces with Measured Values

and S&M methods in Figure 12. General conclusions that are drawn from the measured interlock forces from the filling stage are:

1.) The observed interlock force distributions follow a similar trend to that defined by the TVA or S&M methods.

2.) The TVA method tends to predict interlock forces for cell placement which fit the average observed values for the Lock and Dam 26 (R) case well, but are on the low side for the Trident case. Conversely, the S&M method is on the high side for Lock and Dam 26 (R), and fits the average well for the Trident case.

3.) The maximum interlock force typically occurs at distances of 0.125H and 0.25H above the dredge line for the Trident and Lock and Dam 26 (R) cofferdams respectively. The higher location for the maximum interlock force is associated with the greater embedment of the Lock and Dam 26 (R) cofferdam versus that of the Trident case.

4.) The earth pressure coefficient for the compacted fill in the Trident cofferdam is higher than could be reasonably predicted by either the TVA or S&M methods. This is likely due to either built-in compaction stresses, or break down of arching effects in the fill as a result of the disturbance in the fill by the compaction.

As would be expected from conventional theory, the measured values of interlock forces in the common walls of the two cofferdams are higher than those in the main cells. However, the measured values are less than would be expected by the TVA secant formula, and more consistent with the predictions of the Swatek approach.

Cell Displacements on Filling

The different measurements of cell displacements generated by filling for Lock and Dam 26 (R) Stage 1 cofferdam inclinometers yielded the same general trends, but there was a rather wide spread in magnitudes (Figure 7). This is likely due to the extreme flexibility of the cofferdam before and during filling. The deflected shape indicates increasing cell bulging down to a point about 0.25H above the dredge line, and decreasing movements after this. The location of the maximum bulge is consistent with the location defined for the measured maximum interlock force, as it should be. This location of maximum bulge is also consistent with that suggested by Lacroix et al. (1970) for other cofferdams with similar conditions.

REVIEW OF CASE HISTORY DATA FOR COFFERDAMS SUBJECTED TO DIFFERENTIAL LOADING BY WATER

The data available for the four temporary cofferdams varies in extent and consistency. In some cases, there was only a survey net to measure movements of the top of the cells (Stage 2 cofferdam of Lock and Dam 26), while in others data were obtained from inclinometers and strain gages as well.

Interlock Forces

In the application of differential loads to the two cofferdams instrumented for interlock force, conditions differed. At Lock and Dam 26 (R) Stage 1, a restraining berm was placed on the interior of the cofferdam before unwatering began. During unwatering, a dewatering system was activated which controlled the underseepage beneath the cofferdam. The Trident cofferdam was unusual in that upon unwatering of the interior, the cell fills themselves were dewatered.

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For Lock and Dam 26 (R), the maximum interlock forces showed decreases during berm placement, unwatering, and flooding by percentages of 3, 40, and 27 respectively. These decreases are all counter to conventional theory which would predict no change for berm placement, and an increase on initial unwatering and flooding. Interestingly, the finite element analyses of this cofferdam correctly predicted these trends (Clough and Kuppusamy, 1985). The reason for the lack of agreement between conventional theory and the actual behavior is shown in the finite element analyses to lie in the effects of the stabilizing berm. The initial berm loading works against the pressures exerted by the cell fill, and lowers the net load at a point near the maximum interlock force. Upon unwatering and flooding, the cell walls are pushed against the berm which then mobilizes the strength of the soil in the berm, and generates additional resistance to lower the net loading on the sheetpiles.

In the Trident cofferdam, where no berm was present, unwatering led to an increase of interlock forces on the unwatered or inboard side, and a decrease on the outboard side. This response is consistent with conventional theory, and reflects the fact that there is no element requiring further consideration of soil-structure interaction beyond the simplest concepts. The contrast between the Trident and Lock and Dam 26 (R) Stage 1 cofferdams shows the importance of soil-structure interaction in cofferdam behavior.

Cell Movements

The inclinometer-based movements of the instrumented cells for the Trident and Lock and Dam 26 (R) Stage 1 cofferdams allow the cell wall displacement patterns to be determined. Figures 13 and 14 show the measured profiles for the outboard and inboard sides for two of the typical cells for each cofferdam. The profiles reflect the effects of the unwatering isolated from the previous displacements induced by filling. The Trident cell tended to rotate in a relatively uniform pattern, while the Lock and Dam 26 (R) cell underwent more of a distortion effect with the lower portions of the cells restrained. This difference can be attributed to: (1) The differences in cell embedments, with those for the Trident cell small, and those for the Lock and Dam 26 (R) cell large; (2) The presence of the restraining berm against the inboard sheetpiles for Lock and Dam 26 (R); and, (3) the especially dense condition of the fill at the Trident cofferdam. One notable common trend in the results is that the outboard side moved more than the inboard side, a behavior that is confirmed in other measurements shown subsequently.

The inclinometer measurements are interesting, but they only represent the movement of one cell. Thus, it is also useful to study the survey data for top of cell movement since this is available for many cells, and allows an insight into the scatter of behavior as well as other aspects of response. In Figure 15 the survey data obtained during the unwatering of Lock and Dam 26 (R) Stage 1 cofferdam for

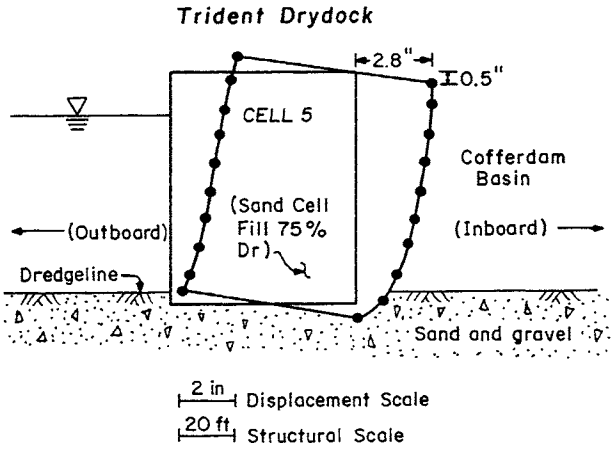

Fig. 13. - Movement of Cell 5 of Trident Drydock Cofferdam during Dewatering

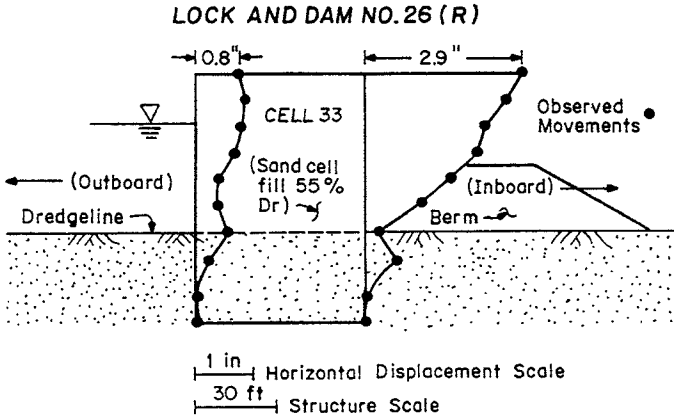

Fig. 14 - Movement of Cell 33 of Lock & Dam 26 (R) during Dewatering

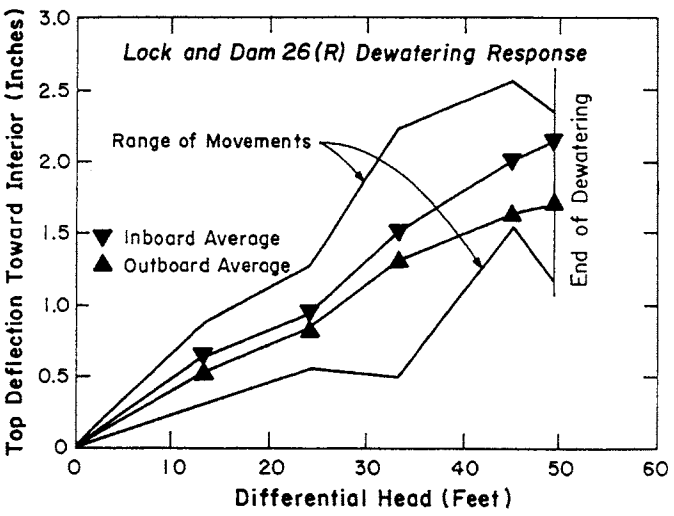

Fig. 15 - Movements of 11 Typical Cells during Dewatering (optical survey)

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movements of the tops of 11 typical upstream and downstream cells are plotted versus the differential head between the outboard and inboard side of the cells. The differential head is a simple measure of the loading applied to the cells. The range in cell movements as well as the averages are shown. The following trends stand out:

1.) Although there is scatter in the data, there is a consistent trend for the cell displacements to increase almost linearally with differential head.

2.) The average of the inboard cell movements is uniformly larger than those of the outboard cell movements.

Review of the data from other cofferdams shows the same response as indicated in these conclusions, and a similar response is seen in finite element analyses (Martin and Clough, 1988).

Figure 16 extends the movement data for the 11 typical cells of Lock and Dam 26 (R) Stage 1 cofferdam to include the behavior that was observed as the water levels fluctuated after completion of unwatering. The changes in differential water head first involved a small decrease. This was followed by a flood event in December of 1982. As the head reached essentially 60 ft and threatened to overtop the cells, the Corps of Engineers began voluntary flooding of the interior of the cofferdam using an emergency spillway. This immediately led to a decrease of the differential head acting on the cofferdam. As is seen in Figure 16, the response of the cofferdam to these loadings was not the same as that which developed on initial unwatering. Each event led to cumulative increases in movements. The response to the December high water was particularly sharp with deformations increasing with differential head faster than during unwatering. It is believed that this is caused by the fact that the moment arm of the high water about the cofferdam centerline is greater than that of the normal pool, and that the high water leads to more submergence of the cell fill, thus reducing the effective stresses and stiffness of the fill. It is also notable that the outboard sheetpiles moved more than the inboard sheetpiles during the high water loading, where the opposite was true on unwatering. This reversal is caused by the fact that the high water loading is directly applied to the outboard sheetpiles.

To compare the four temporary cofferdam movements, the measured displacements of the tops of the cells are normalized by dividing by the free height of the cell, and then plotted against the differential head which is also normalized by the free cell height (Figure 17). The amount of data tabulated for each project varies; only limited inclinometer information was available for Willow Island, but for the others, each point represents the average of many survey measurements. As is seen in the figure, the nondimensionalized movements follow the loading directly, and in an almost linear fashion until the very high levels of loading

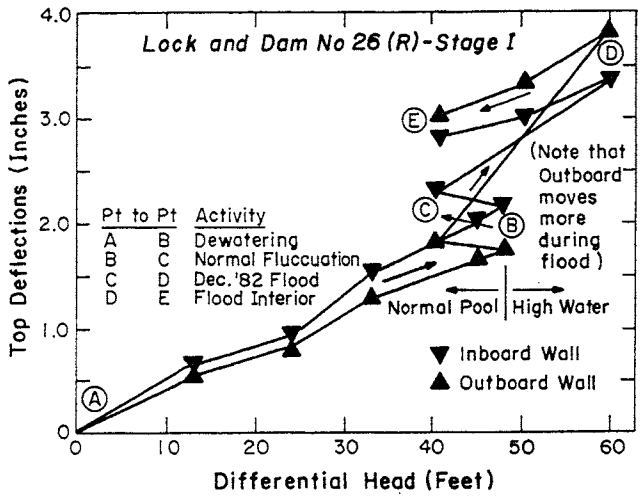

Fig. 16 - Dewatering and Flood Response 11 Typical Cells (optical survey)

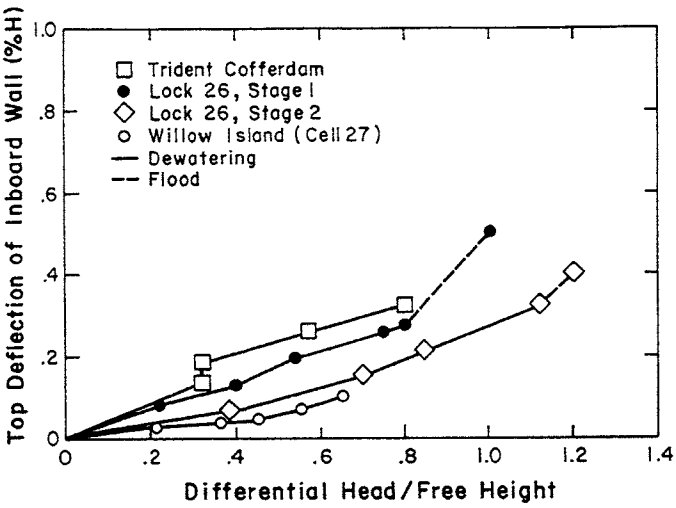

Fig. 17 - Normalized Loads and Movements for Cofferdams under Water Loadings

are reached. In order of increasing nondimensionalized movements, the cofferdams are ranked as Willow Island, Lock and Dam 26 (R) Stages 1 and 2, and Trident. This ranking is consistent with the relative conditions for the cofferdams. Willow Island has the largest diameter to height ratio, the strongest foundation, and a stabilizing berm, and it shows the smallest displacements. The Trident cofferdam has one of the smallest diameter to height ratios, no interior stabilizing berm, and the greatest height, and it shows the largest displacements. The Lock and Dam (R) 26 cofferdams have conditions which fall between the other two, and displacements which do likewise.

The general level of the nondimensionalized movements are of interest since the consistency of the trends shown for the cofferdams suggest that they can be used to help predict the likely level of movement of cofferdams with

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similar conditions. There are two other reference points of this type worthy of mention. Swatek (1967) reports that in his experience the typical temporary cofferdam moves 1 % of its free height at the top of the cell. This would be high for the average movement data in Figure 17, where none of the values exceeded 0.6 %. However, if the extreme movements were considered at each cofferdam, then the 1 % figure would not be far from the observed value. In finite parametric studies, Singh and Clough (1988) report that the movements should not exceed 0.5 % for a conservatively designed cofferdams, particularly if there is a stabilizing berm present. It would appear from all of the information that for prudent design of a cofferdam on good foundation conditions, a value of 0.5 to 1 % of the free cell height could be used to estimate displacements.

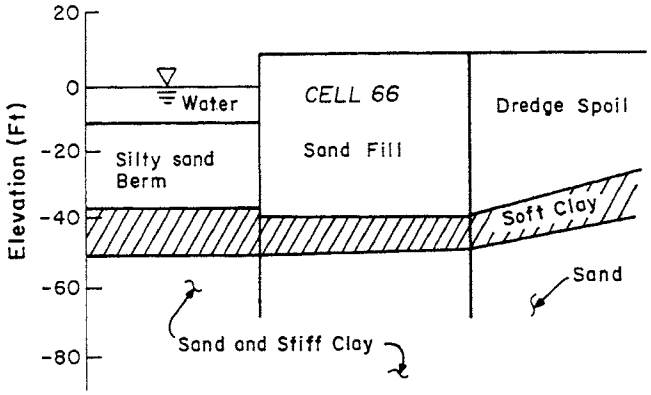

Fig. 18 - Typical Cell Schematic of Seagirt Cofferdam

REVIEW OF CASE HISTORY DATA FOR COFFERDAMS SUBJECTED TO OTHER THAN WATER LOADINGS

The Seagirt and Fulton Terminal 6 case histories represent unique situations in that these cofferdams are subjected to loading other than by water. Figure 18 shows typical section for the Seagirt cofferdam in the area where it was founded on sand. In other areas, this cofferdam was founded on a fissured hard clay. The Seagirt cofferdam was originally intended as a temporary system for the containment of the slurry created by excavation for the I-95 Baltimore Harbor tunnel. Several years subsequent to the construction of the tunnel, it was decided to use the cofferdam and its retained fill as a waterfront terminal. This review focuses on the cofferdam performance before the time changes were instituted to upgrade the system for its permanent use. The upper soils at the site were soft organic clays, and these were underlain by either stiff fissured clays, or dense sands. The cells were driven through the upper soils to a shallow embedment in the underlying stiffer soils. A sand fill was placed into the cells without removing the soft soils that remained inside the cells.

The Fulton Terminal 6 cofferdam case history is described by Schroeder (1987). This structure serves as a container wharf, and is founded on dense sand. Before driving the sheeting, an upper layer of soft silt was dredged. With the sheets in place, fill was dredged into the cells, and in the area behind the cells to create a level working surface. The fills in the cells and behind the cells was vibrated into a dense condition, with an estimated relative density of about 80 %.

Both the Seagirt and Fulton Terminal 6 cofferdams underwent significant deformations as a result of the lateral loads applied to them. In Figure 19, the lateral movements measured at the tops of selected cells for both cofferdams are plotted versus date, with key activities at the sites noted. For the Seagirt case, two sets of data are shown, one for a section of the cofferdam on a clay foundation, and one for a section on the dense sand. The movements for the Seagirt cofferdam do not represent the entire deformation history of this system, since some movement occurred before measurement benchmarks were installed.

Maximum lateral movements for the Seagirt case reached about 5 ft for the cell on founded on clay, and 1 ft for the cell founded on dense sand, values equal to 13 and 3 % respectively of the free cell height. While most of the displacements of the cells occurred during active construction, a significant portion of those for both foundation condition are time-dependent. Because no inclinometers were in place during the construction, the exact sources of the movements cannot be determined. However, it seems reasonable that the large deformations are in part related to the presence of the soft organic silts in the cell and in the soil on either side of the cell. Also, where the hard clay formed the foundation material, it is likely that the sheetpiles in the front of the cells plunged into the clay causing a rotation of the cells forward.

As is seen in Figure 19, lateral displacement due to placement of fill behind the cell was only 1.0 ft, or about 1.5% of the cell height. Inclinometers installed on the cofferdam cells allowed a definition of sources of movements. A significant amount of displacement was observed at the dredge line and below, indicating movements of the embedded portion of the sheetpiles.

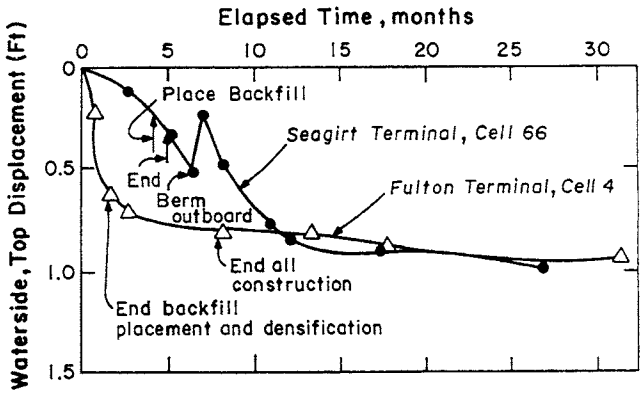

Fig. 19 - Lateral Displacements of Cells Loaded by Soil

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The performances of the Seagirt and Fulton Terminal 6 cofferdams lead to the following observations:

1). Despite large deformations of the cofferdams, they continued to perform adequately, showing the resilence of these flexible structures.

2.) The lateral movements of the cofferdams loaded by soil were larger than those of the cofferdams loaded by water.

3.) The movements of the Seagirt cofferdam are larger than those for the Terminal 6 cofferdam, presumably due to poorer foundation conditions and the presence of the weak silt in the fill.

SUMMARY AND CONCLUSIONS

Until the early 1970's, the technology for cellular cofferdams was relatively static. Design methods were largely empirical, and the main use for cofferdams was as a temporary structure to allow construction in the dry in a body of water. Since that time, a number of studies of the subject have taken place in the form of model and full-scale instrumentation projects. Also, investigators have developed finite element models with realistic allowances for many of the important aspects of the complex cofferdam problem. Conclusions from the work to date include:

1.) Model studies have led to new methods for calculating interlock forces and internal stability. These generally lead to less conservative answers than are obtained by the early design techniques.

2.) For finite element methods to successfully model the cellular cofferdam, they must account for the nature of the construction process, the nonlinear soil behavior, flexibility added to the system through sheetpile interlock deformations, and slip between the sheetpiles and the soils.

3.) Finite element analyses show that the soil-structure interaction process in the cofferdam is complicated because of the flexibility of the system and the large deformations that occur.

4.) Earth stability berms placed on the interior of the cofferdam have an impact on the behavior both in reducing overall cell deformations, and in limiting the development of interlock forces in the cell by providing a restraint at the location where the interlock forces are usually the largest.

5.) For cells with significant embedment, the maximum interlock force and cell bulge occurs at a distance of about 0.25H above the dredge line.

6.) For cells with small embedments, the maximum interlock force and cell bulge occurs at a distance less than 0.25 H above the dredge line.

7.) The maximum interlock force can be reasonably calculated using either the TVA or S&M methods when a earth berm is not present. If an earth berm is present, then these methods will lead to conservative predictions of interlock forces for conditions after filling.

8.) Average lateral displacements at the top of conservatively designed cofferdams under the action of water loading will be in the range of 0.5 % of the cell height. Repeated loading by high water can lead to cumulative increases of cofferdam movements.

9.) Lateral displacements of cofferdams are controlled by the magnitude of the lateral loading, and follow the loading magnitude almost linearally so long as the factor of safety of the system is well above one.

10.) Densification of the cell fill causes increases in interlock forces and cell displacements over those induced during cell filling.

11.) In the cases considered herein, cofferdams loaded by soil moved more than those loaded by water.

12.) Cellular cofferdams are resilient structures, and are able to sustain large displacements and distortions without failure.

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CELLULAR STRUCTURE FAILURES*

ROBERT GRAYMAN**

This paper discusses the prime causes and location of structural failures in cellular structures. The author suggests that failures can be mitigated by using 30° riveted wyes, designed for internal load of saturated fill and considering load from the connecting arc.

INTRODUCTION

The validity and adequacy of the design and construction of steel sheet pile cellular structures can be partially judged and evaluated by the study of known failures. However, presenting information on such failures is difficult because conclusions may be drawn which fix the responsibility of the various parties concerned when many interrelated factors and litigation may be involved.

Much has been written and published on the stability aspect of cellular cofferdam design but little has been published on the structural aspects. There have been failures which can be attributed to structural deficiencies in the "state of the art," that have had little publicity. These failures most of which are classed as minor because there was no loss of life can be costly. This does not mean that there have not been stability failures such as those caused by sliding of a cellular structure. This presentation will be primarily devoted to those structural aspects that are of principal concern and pictures shown that illustrate these features. Although a cellular structure is usually considered a relatively simple structure, the failures show that it is actually a complicated one with many complex interactions.

TEES

The prime location of structural failures of circular cells has been at the connecting tees both welded and, to a considerably lesser degree, riveted tees. A pull on the outstanding leg of the welded tee with a 3/8 in. thick web of 300-400 pounds per lineal inch can cause a theoretical bending stress in the web of the main sheet in the yield point range. A riveted tee can carry a somewhat higher load because of the extra metal in the connecting angles. A section of a welded tee cut and reassembled from a cell that failed is shown in Fig. 1. The figure was traced from a photograph. Note the extreme deflection which is many times the deflection in the plastic range. It is this deflection in the plastic range which permits the tee to adjust to and carry its load. There are few other structures in which such an extreme use is made of the plastic range of structural steel. The use of 30° connecting riveted wye piles in lieu of connecting tees will mitigate this type of failure because the load is applied more nearly tangential rather than at right angles.

Four photographs illustrate the failure of the cellular structure associated with the failure of the tee shown in Fig. 1.

Figure 2 illustrates the general appearance of a cellular structure when a connecting tee ruptures in three pieces.

Figure 3 shows a closer view of the same cellular structure after the rupture of a tee.

Figure 4 shows the three pieces of the same ruptured tee after it has been removed to a viewing area for inspection. The outstanding leg of the 3/8 in. welded tee is shown in the center.

Figure 5 shows one half of the same ruptured tee, approximately 15 ft. from the bottom of the sheet pile. Note the bow in the web above the zone cut out for metallurgical inspection. The pieces are reconstructed in Fig. 1.

SATURATION

Saturation of the internal fill is associated with many failures. It is the high lateral pressure of the water when added to the pressure of the fill that can produce the magnitude of hoop tension in the main cell and also the pull in the outstanding leg of the connecting tee resulting in the condition shown in Fig. 1. The saturation of the fill in the connecting arc is a particularly potent danger because of the magnitude of the tension that can be created on the outstanding leg of a connecting tee. It is suggested that the cells and connecting arcs should be investigated for a saturated condition unless specific measures are taken and enforced to prevent saturation. It should be noted that saturation can be caused by things other than the common leakage through the interlocks, holes, splices and filling by hydraulic dredge method. Waves splashing over the top of the cofferdam cells, leakage or breaks in the discharge lines from unwatering pumps over the cells can quickly cause saturation of the cofferdam cells.

Figure 6 illustrates the fill in a cellular structure being saturated from two causes simultaneously, namely, waves breaking over the top of the structure and a broken discharge line spilling water into the top of the cell fill.

*The views expressed in this paper are those of the author and not necessarily those of the Corps of Engineers

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INTERLOCK TENSION

It is common practice to calculate the interlock tension in the sheet piling based solely on the internal pressure in the main cells. However, the tension from the internal pressure in the connecting arc should also be added, taking into account the location and type of connecting member that is either tee or wye. This is particularly true when the connecting arcs are filled by hydraulic dredging which increases the tension on the outstanding leg of the connecting tee or wye. The use of the wye will tend to mitigate the tension on the main cell because the load is applied more nearly tangential rather than at right angles. The pull on the outstanding leg of a tee can, in addition to the previously mentioned load, induce an indeterminate tension in the adjacent pile of the main cell. This is somewhat analogous to the pull induced in a horizontal string when a weight is suspended vertically at its mid-point.

Figure 1 illustrates the results of the pull on the outstanding leg of a welded tee. Figure 7 shows a similar effect of the pull on the outstanding leg of a riveted connecting tee. The pull having been induced by a saturated load from a hydraulically placed fill in the connecting arc between circular cells. In each case note the severe bow in the web of the main sheet of the tee. The force that produces this bowing also induces an extra, but indeterminate, hoop tension in the main cell, primarily near the connecting tee.

SPLICES

A source of stress concentration can occur at a splice in a sheet pile. The precise width of a specific type of sheet pile may vary from pile to pile and particularly so from manufacturer to manufacturer. Thus at a splice the narrower pile would pick up considerably more load than the unit tension at a particular depth of fill. This can be avoided by not splicing piles or by controlling the width of sheets. Dependence cannot be placed on A.S.T.M. A328, the specification for steel sheet piling. This specification has very few controls on the quality of sheet piling as to size, tolerances, interlock shape or strength.

Figure 8 shows a failure in the web of a sheet pile immediately below a splice of steel sheet pile MP101 at the bottom and a WS 1 on top. The MP101 is approximately 1/4 in. narrower than the WS 1, as shown on Fig. 9 traced from a sample of each of the two piles. This failure occurred in a sheet pile, a couple of piles away from the tee shown in Fig. 7 previously described.

This failure shown in Fig. 8 may be said to illustrate the combination of all the potential weaknesses previously described, that is—tees, saturation, induced interlock tension and splices.

STABILITY

The author has no knowledge of a failure which may have been caused by a stability inadequacy such as overturning or sliding except the one illustrated by Figs. 10 and 11. The cells are believed to have slid approximately 5 ft. toward an open excavation on a strata below the bottom of the cells. The fill behind the cells was level with the top of the cells prior to the movement.

INVESTIGATION OF CELLULAR FAILURES

A checklist may be of assistance when making an investigation of a failure of a steel sheet pile cellular structure. It has been found helpful in making a systematic investigation when the cause is unknown and to help in preventing a premature conclusion. Items to be considered:

1. Stability
   a. Overturning
   b. Tilting
   c. Sliding
   d. Loading

2. Stresses
   a. Interlock tension
   b. Connecting tees or wyes
   c. Web tension
   d. Web bending

3. Material
   a. Steel
      New or used
      Type
      Size and manufacturer
   b. Interlock gage
   c. Welded or riveted tees or wyes
   d. Laminations

4. Fill
   a. Degree of saturation
   b. Type and Action
   c. Cell cap

5. Construction
   a. Layout
   b. Height
   c. Size of cell
   d. Overburden
   e. Toe-in into rock
   f. Berm
   g. Driving Conditions
   h. Interlock arrangement

6. Miscellaneous
   a. Impact
   b. Temperature
   c. Failure chevrons
   d. Location of discharge pipes
   e. Splice detail

7. Pictures (Show following items on sheet piles prior to taking pictures)
   a. Number
   b. Top or bottom
   c. Inside or outside
   d. Distance from top or bottom
   e. Size and manufacturer

AREAS OF INADEQUATE OR INSUFFICIENT KNOWLEDGE

Also recognizing that there are unresolved and controversial features in the design and construction of a cellular structure, a list of such items is presented in order to highlight and thus help in obtaining the answers to these problems:

1. Fabricated Connections
   a. Stresses in both welded and riveted tees
   b. Strength of tees, both welded and riveted
   c. Stresses and strength of wyes
   d. Comparison of tees and 30 wyes for circular cells

2. Stresses induced in piles adjacent to tees and wyes

3. Saturation of cell fill
   a. Leakage
   b. Wave overtopping
   c. Breaks in discharge lines
   d. Sandbagging

4. Omission of connecting arc load when determining hoop tension in main cell.

5. Splicing sheets from different manufacturers
   a. Stresses induced when widths are different
   b. Offset in webs

6. External impact

7. Strength of interlocks
   a. Under varying conditions of junction
   b. Joining sheets from different manufacturers
   c. Gaging of interlocks and strength in relation to tolerances
   d. "Murphy" or reversed interlock

8. Internal fill in cells
   a. Loading conditions
   b. Effect of method of filling

9. Construction practices
   a. Driving out of interlocks
   b. Effect of welding such as at splices
   c. Driving past a bulge
   d. Driving impact when load is applied to part of an assembly

10. Bending stresses in webs of straight sheet piling in a circular cell

11. ASTM A328 does not adequately cover requirements for sheet piling

12. How and when is "brittle" steel a problem (notch sensitive steel)

13. Omission of a tee during initial construction of a cell

SUMMARY

The prime causes and location of structural failures of cellular structures are:

1. Pull on outstanding leg of welded connecting tees and to a lesser extent riveted connecting tees.

2. Saturation of the internal fill

3. Neglect of the tension induced in main cell sheets by loads in the connecting arcs and additive tension induced by pull on outstanding leg of connecting tee.

Failures of cellular structures can be mitigated by using 30° riveted wyes, designing for internal load of saturated fill and considering load from connecting arc.

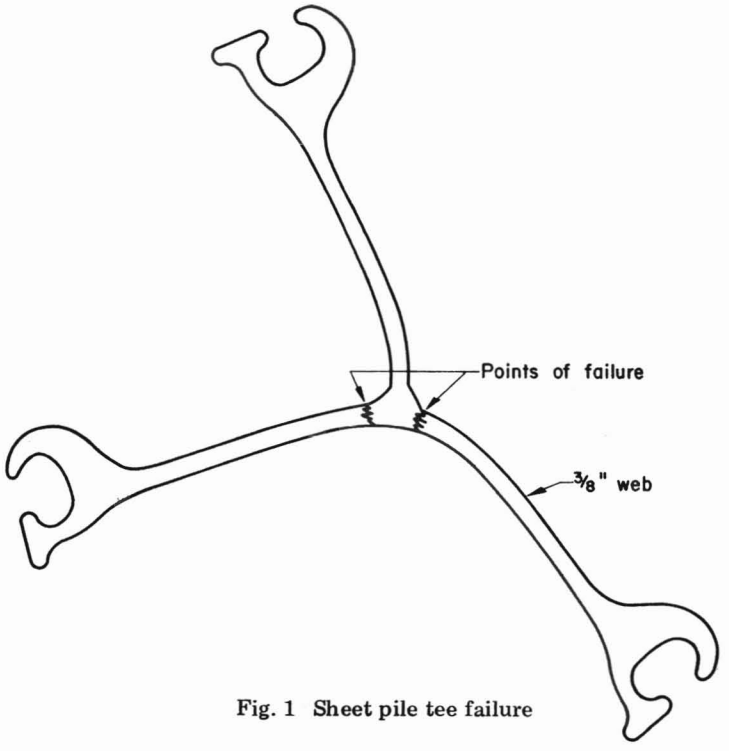

Fig. 1 Sheet pile tee failure

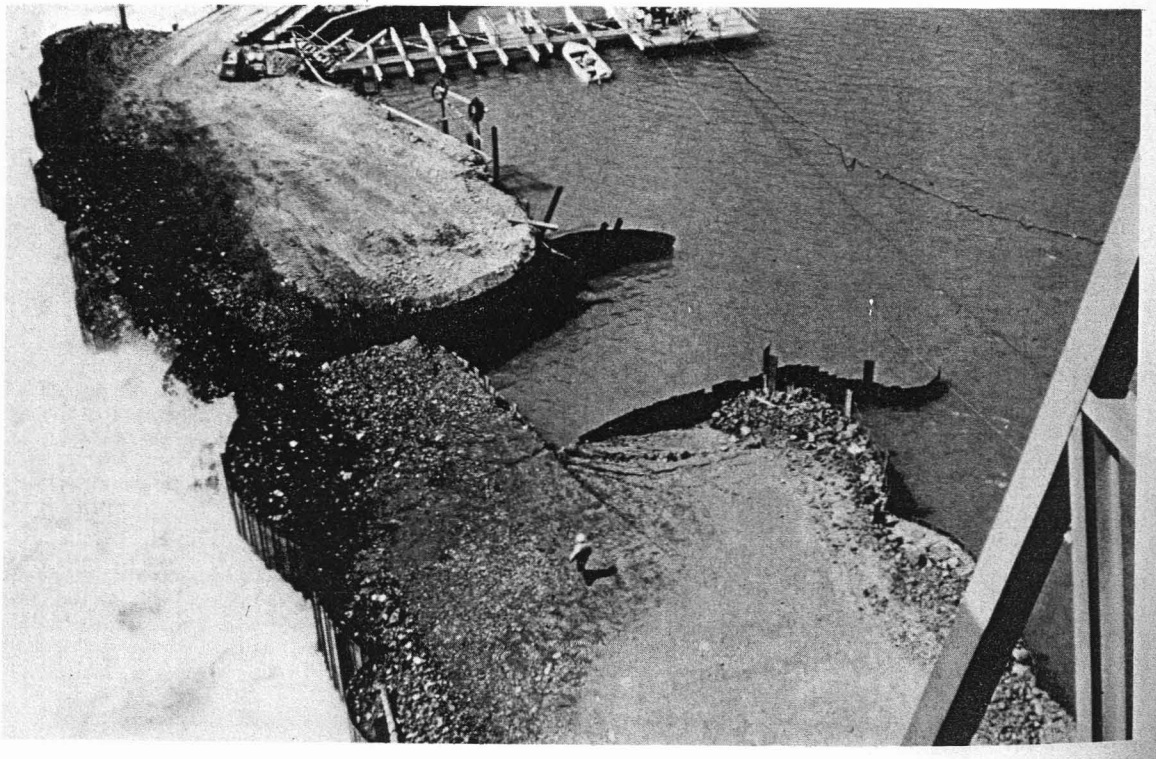

Fig. 2 General appearance of a cellular structure when a connecting tee ruptures in three pieces

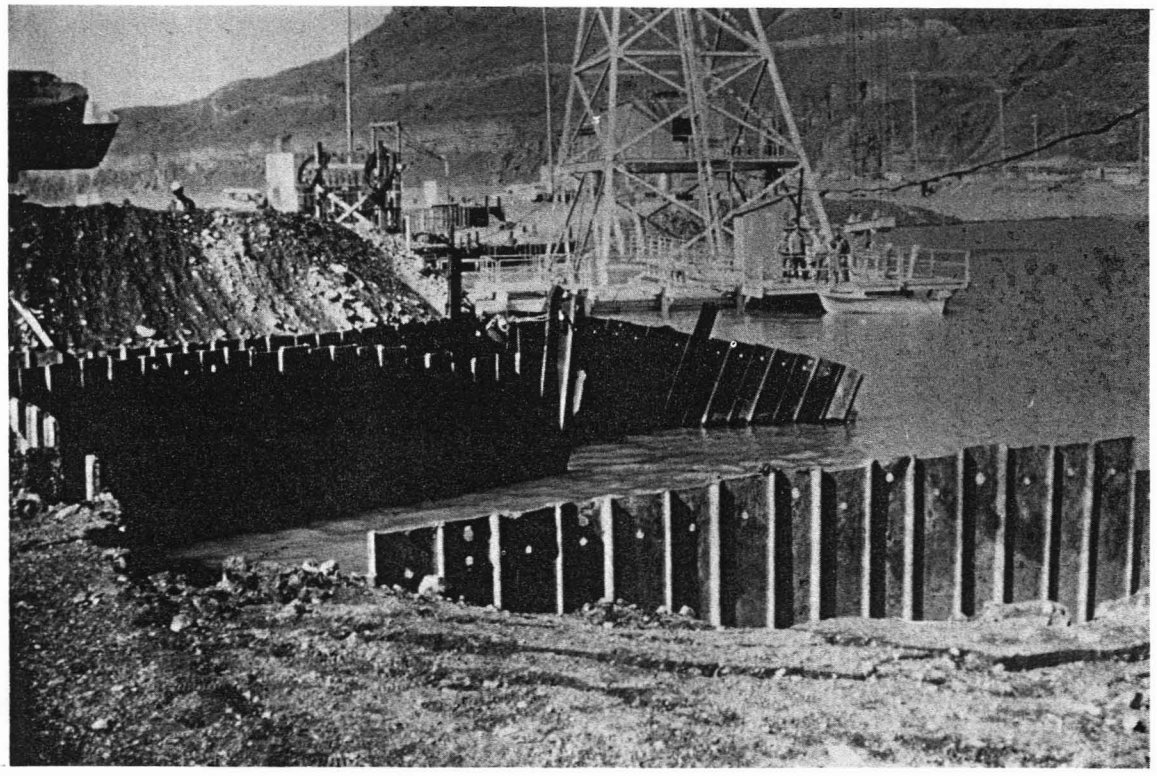

Fig. 3 Closer view of cellular structure

Fig. 4 The three pieces of the ruptured tee shown in Fig. 1. Outstanding leg of the 3/8 in. welded tee in center

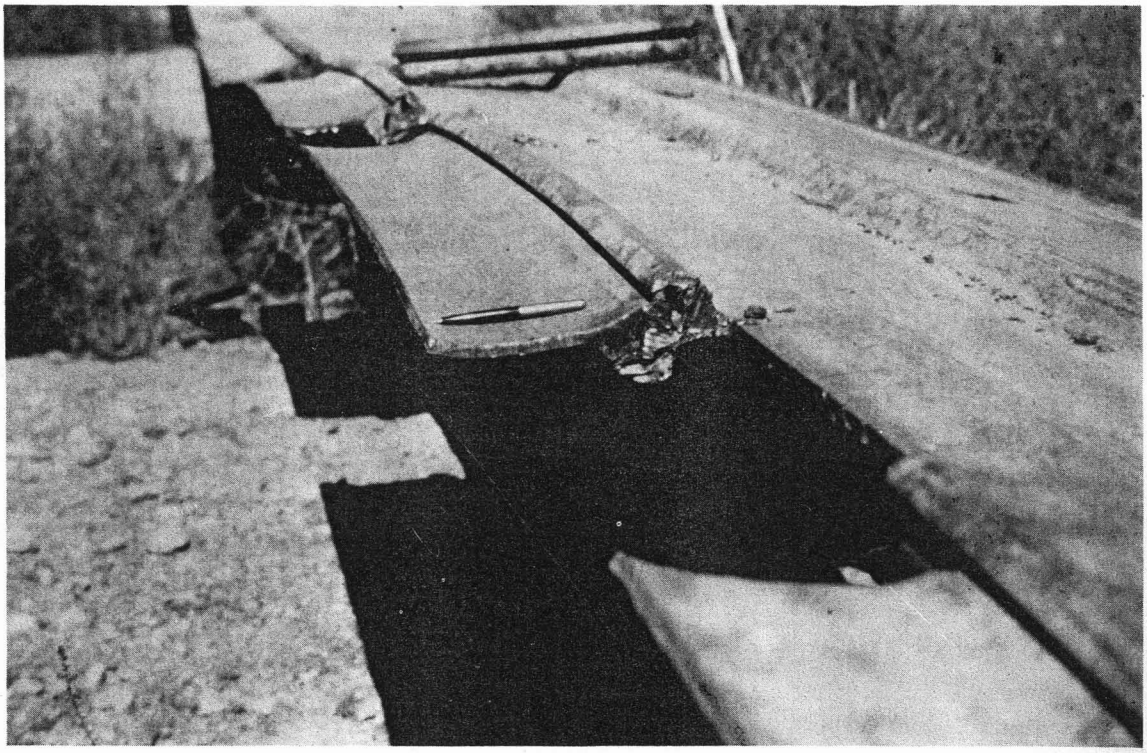

Fig. 5 Half of the ruptured tee approximately 15 ft. from bottom. Note the bow in the web above the cut out

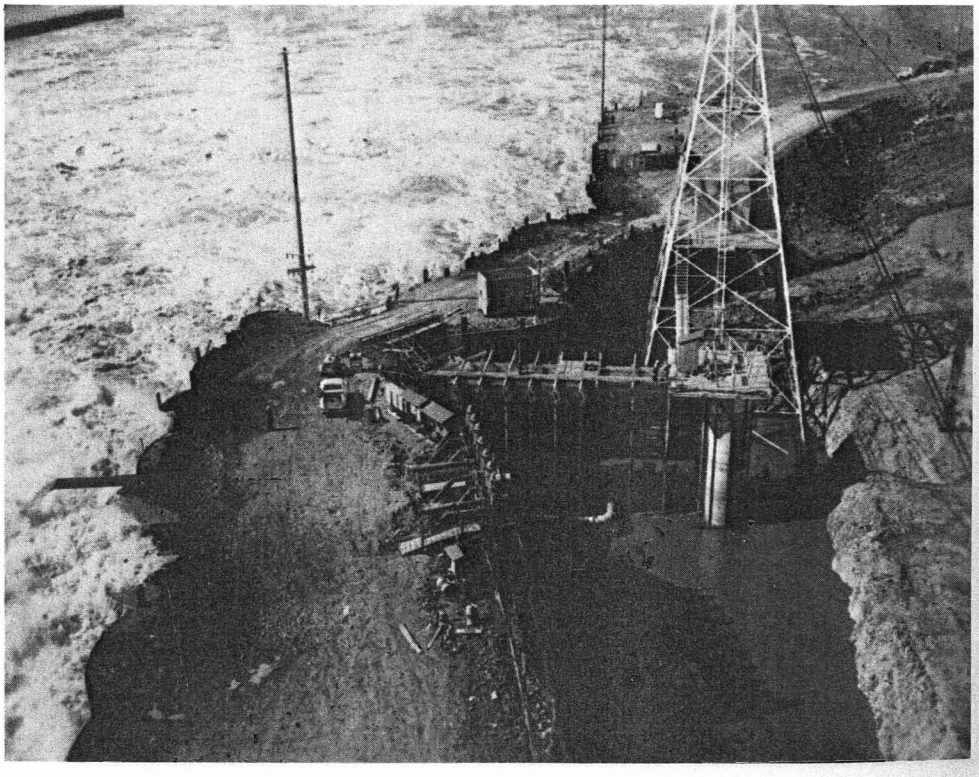

Fig. 6 Fill in cellular structure being saturated by waves and discharge from broken pipe

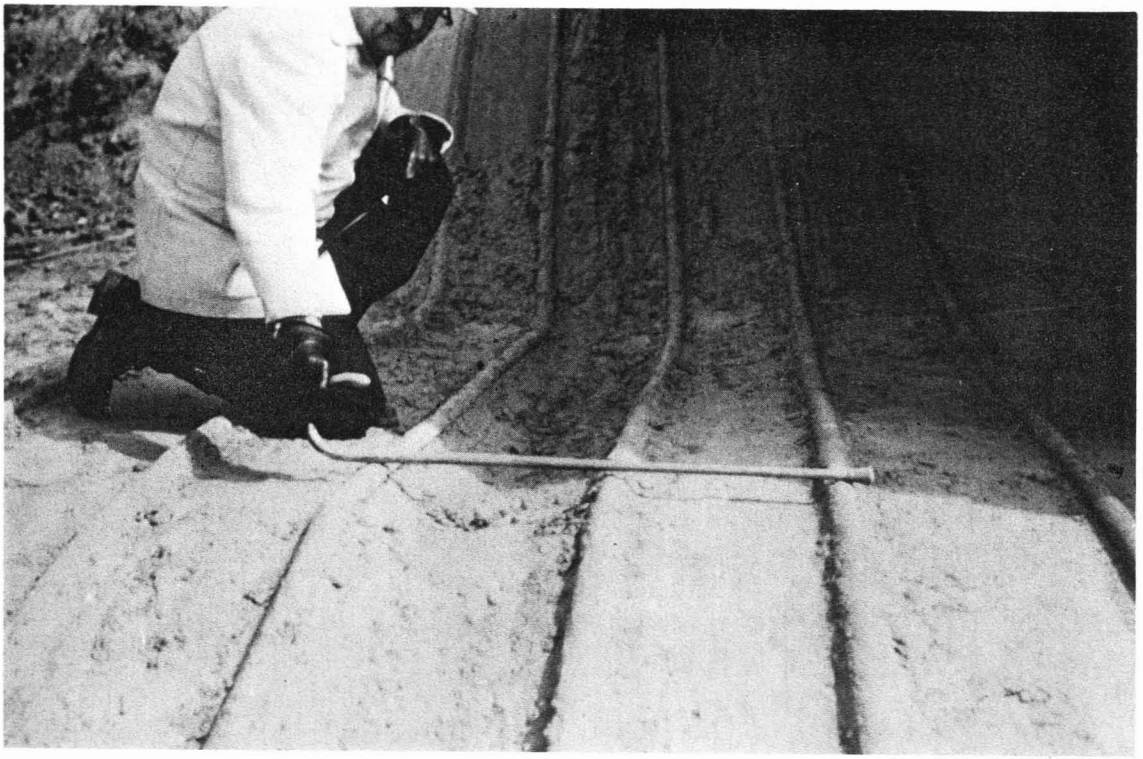

Fig. 7 Bow in web of a riveted tee caused by pull on outstanding leg

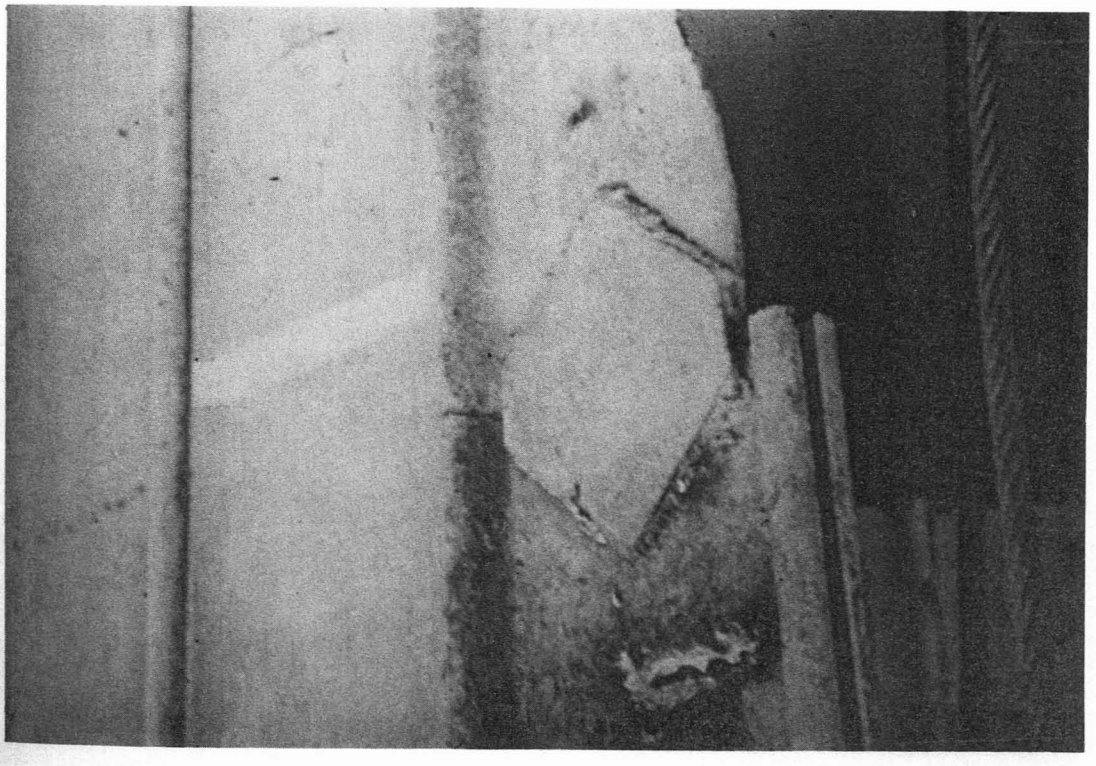

Fig. 8 Failure in web of a pile below a splice of an MP101 on bottom and WSI on top. Near the connecting tee shown in Fig. 7

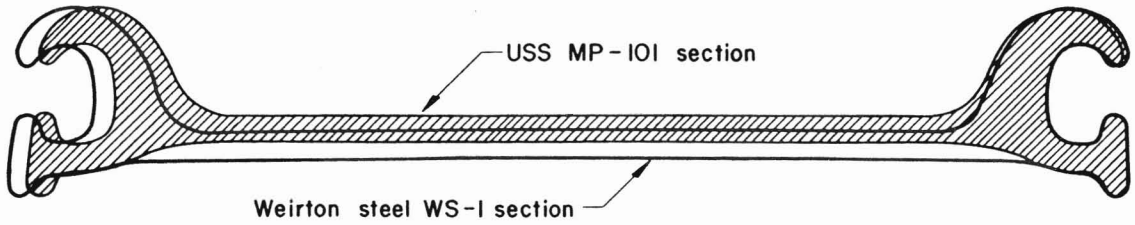

Fig. 9 Tracing of MP101 superimposed on WSI

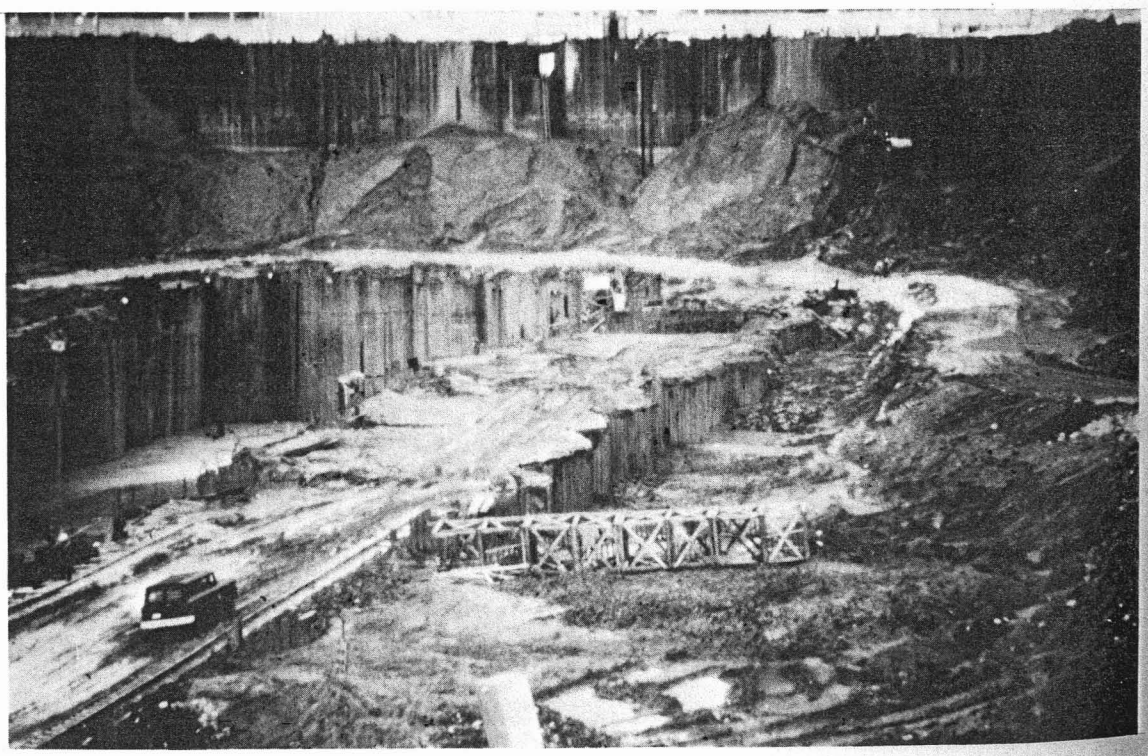

Fig. 10 Cellular structure on rock after slide of foundation. Fill behind the cells was level with top of cells prior to the slide

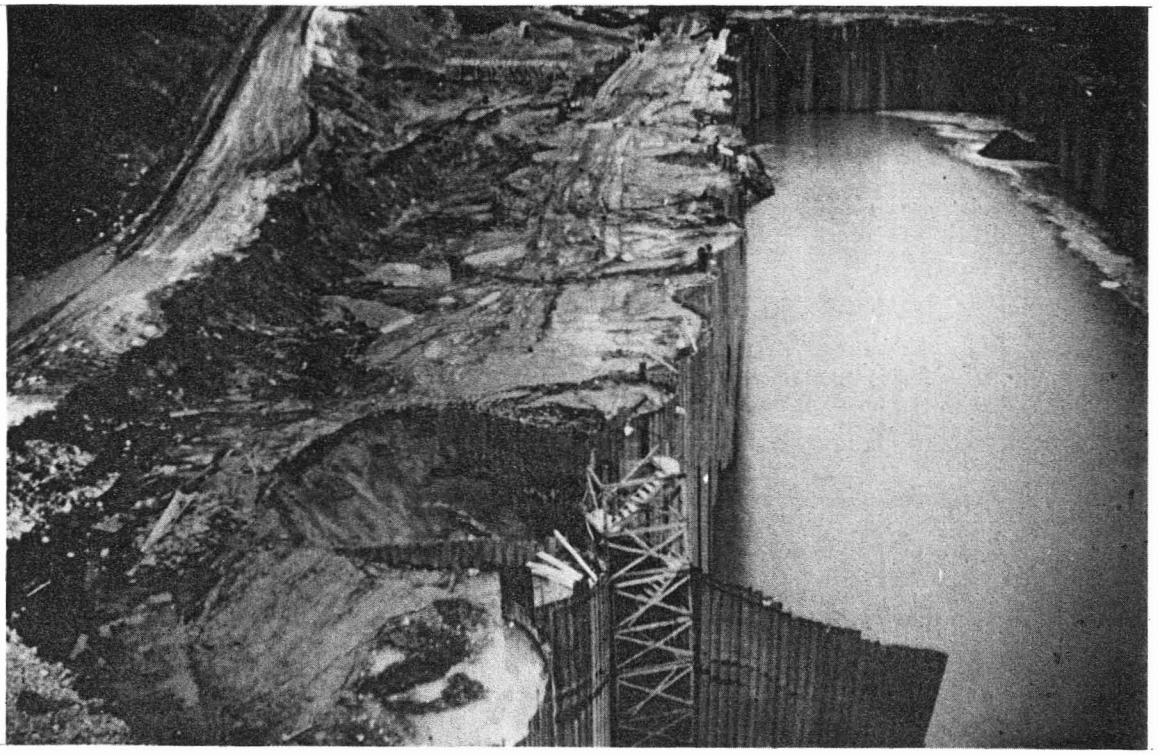

Fig. 11 Cellular structure on rock after slide of foundation. Fill behind the cells was level with top of cells prior to the slide

REF #15

EM 1110-2-2504 – Design of Sheet Pile Walls.