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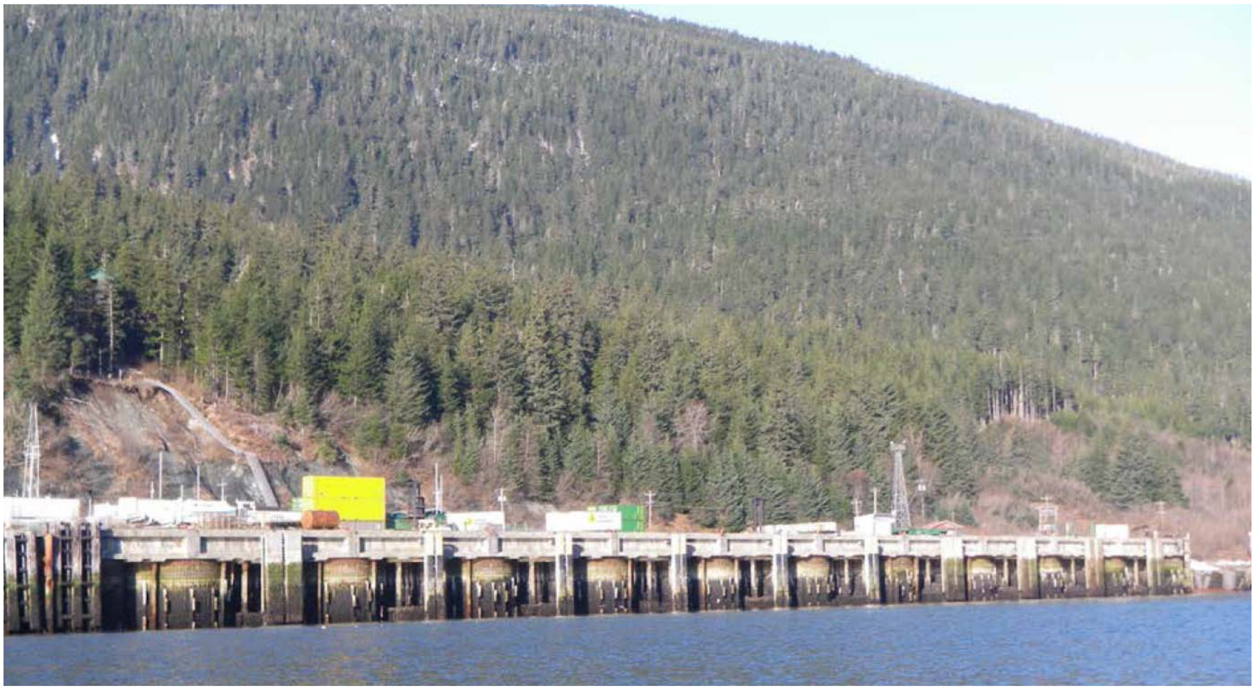
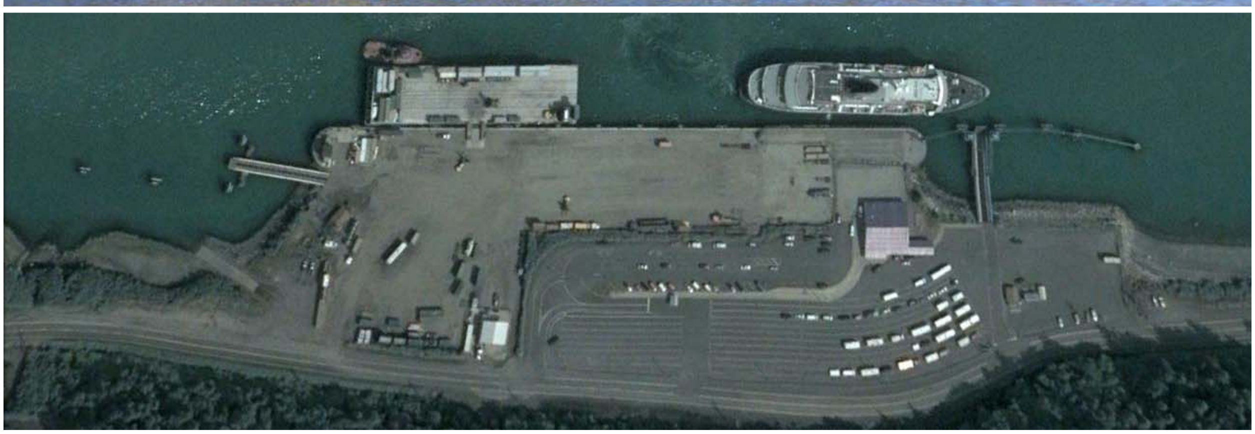
Lutak Dock Structural Assessment

City of Haines, Alaska





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1. Executive Summary

The City of Haines, Alaska has commissioned this report to document results of the 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 work is assumed to have been done 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.

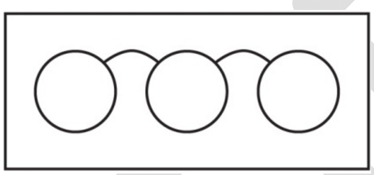
The seismic forces arising in Haines are modest, and the structure meets criteria USACE critera for low intensity earthquake with ground acceleration of .072g (50% likelihood of occurrence in 50 years). Above this level of ground shaking the structure cannot withstand earthquakes at the current "design 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 the town of Haines, Alaska. 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 City of Haines (City) and the Alaska Marine Highway (AMH). Lutak Dock is comprised of 15 interlocking circular closed cells; the City'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.



The cells are built from interlocking flat web sheet pile. The sheet pile in the closure arcs have web thicknesses of 3/8". The main cells have sheet pile with web thickness of 1/2". PND was not able to ascertain the original manufacturer of the sheet pile. PND has assumed that that these sheet piles have 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 pile over the last 61 years. Corrosion of the sheet pile 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 pile have a thicker web than the closure arc sheet pile.

Lutak Dock was most recently modified sometime after 2002 according to the Shannon and Wilson Report (Ref. #1) by and 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 is a detail excerpted from the 2002 repair drawings depicting repairs to the closure arcs (Ref. #8).

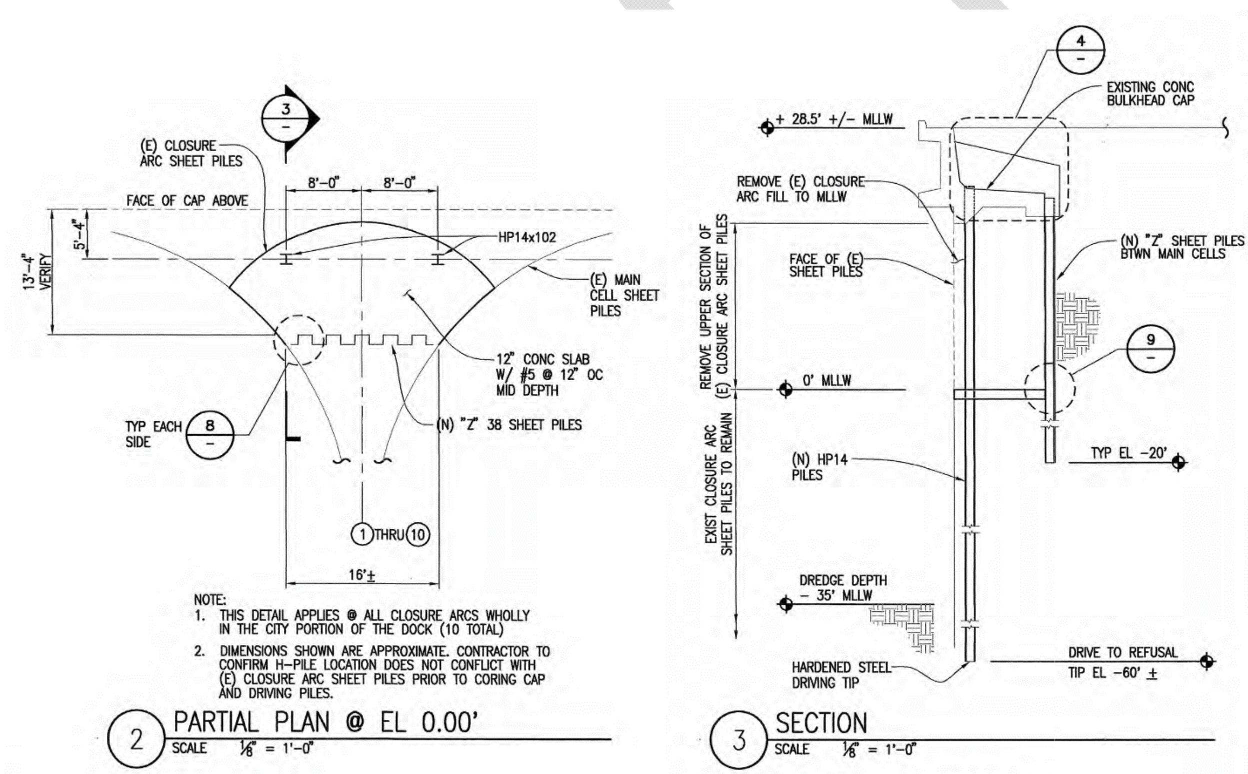


Figure 3-1. Closure Arc Repair Detail

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 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

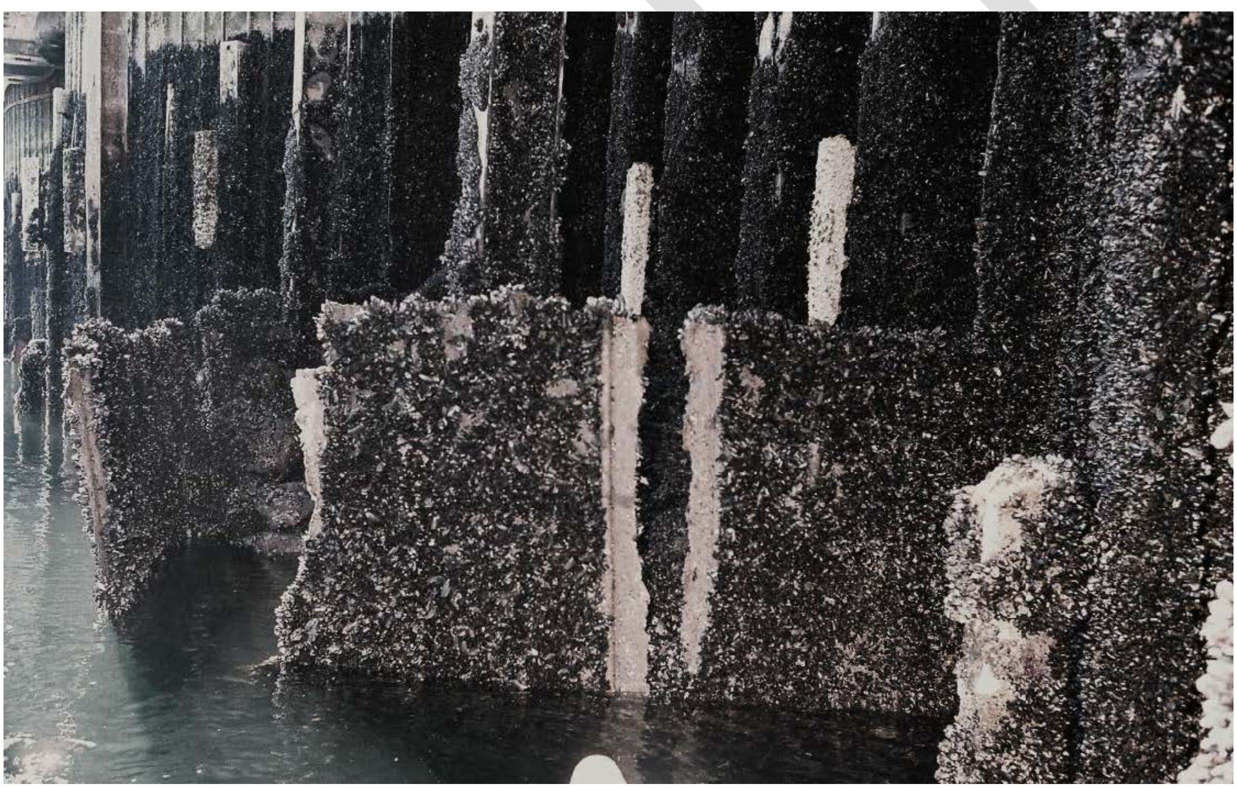


Figure 3-3. Failure at Closure Arc 7.5

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 2002 as the sheet pile 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

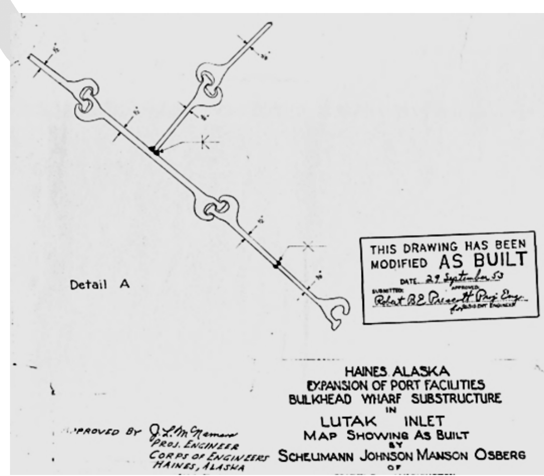


Figure 3-6. Connection Detail at Lutak Dock, 1953

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 after 2002, 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 2002, based on inspection report 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 city of Haines 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 pile driving.

4. Geotechnical Conditions

The dock site is located in a fjord formed via glacier 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 condition assessed using boring 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 AMH 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): 19.06’

Mean Sea Level (MSL): 11.14’

Mean Lower Low Water (MLLW): 2.33’

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 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 Acceleration	0.010 g	0.070 g	0.188 g	0.438 g

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 pile 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 pile 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 pile 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 2002. 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. 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 2002 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 sheet pile. Table 8-2 summarizes the result 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 support 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 sheer 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 crack 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 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 like 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.

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.

8.5. 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 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.

Given the failure of two closure arcs, and the poor conditions of 4 others, it is prudent to halt vehicle operations in areas defined by the closure arc zee-sheet 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 were 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.

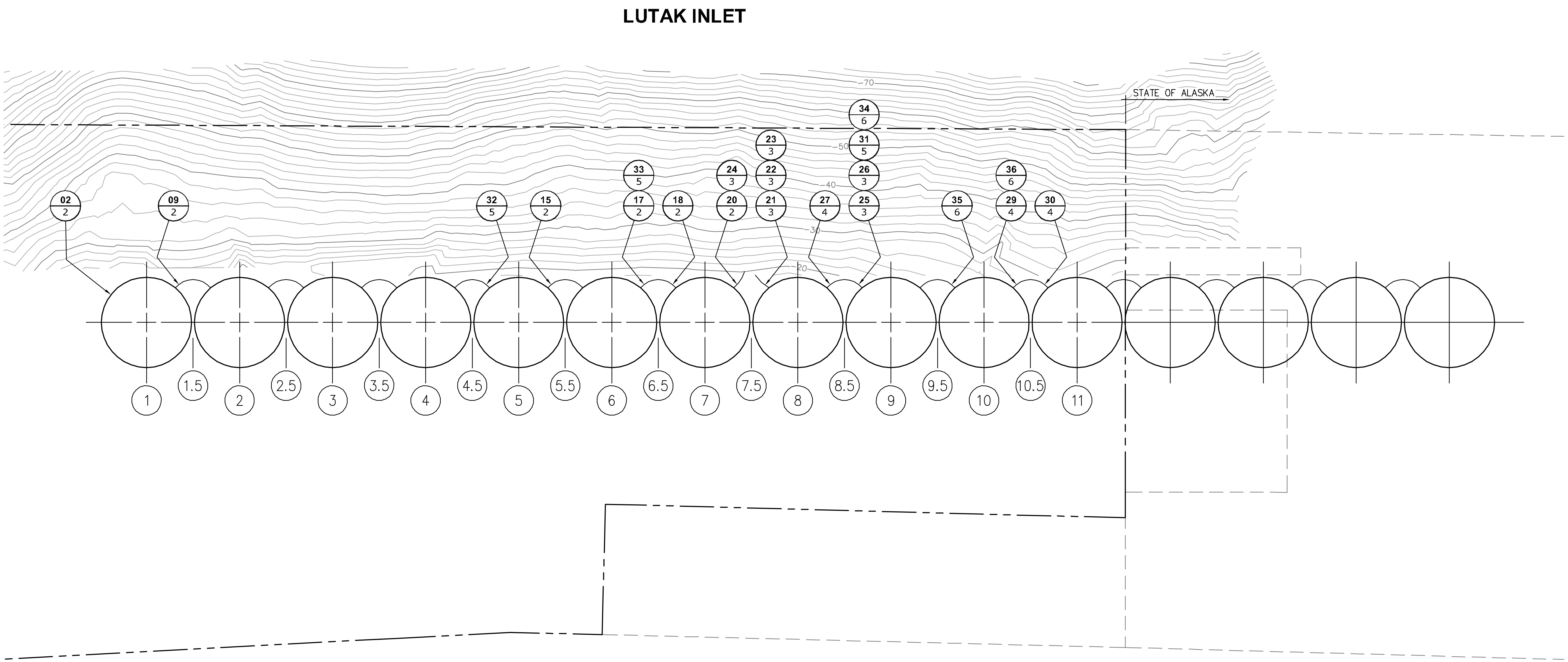
Ref. #13: "Cellular Cofferdams – Developments in Design and Analysis," Clough and Martin, 1988.

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.

Photographs of Damage

17 — PHOTOGRAPH # FROM ECHELON REPORT
3 — SHEET #



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LUTAK DOCK HAINES ALASKA

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PROJECT NO:	142010.01
SHEET NO:	1 OF 6

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PHOTOGRAPHS FROM ECHELON ENGINEERING REPORT
– PND REFERENCE #4

02
1
PHOTO NO. 02

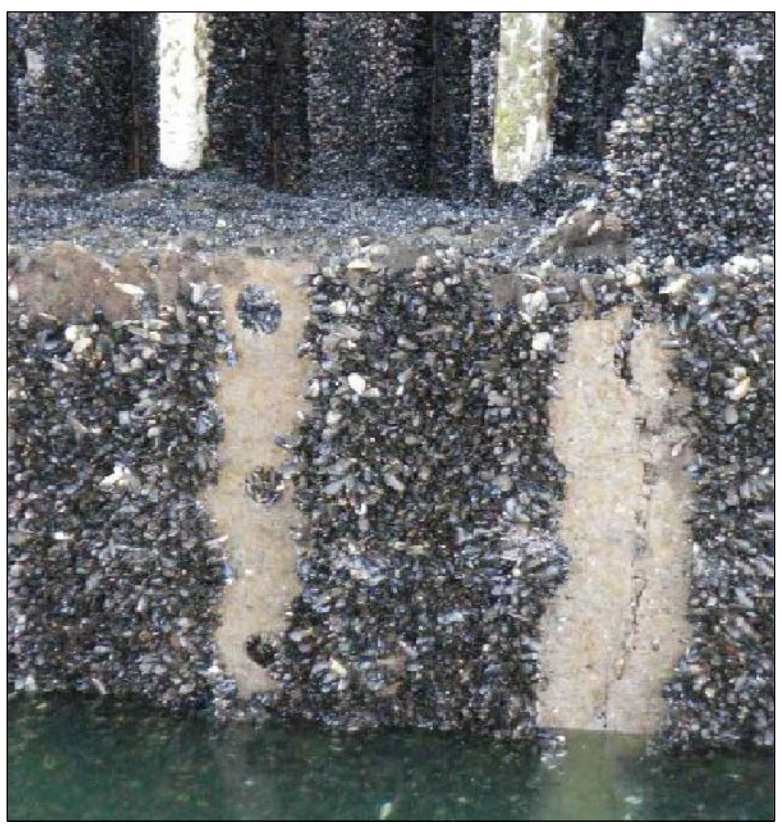
MAIN CELL NO. 1, LOOKING EAST – INSPECTION NOTED THAT THIS PORTION OF THE CELL HAS NOT BEEN OUTFITTED WITH ANY SACRIFICIAL ANODES. THE INSPECTION DID NOTE ON-GOING SURFACE CORROSION IN THIS AREA.

09
1
PHOTO NO. 09

CLOSURE ARC NO. 1.5, WEST SIDE – NOTE THE SHEET PILES WHICH HAVE BEEN CLEANED SHOWING AREAS OF PERFORATION.

15
1
PHOTO NO. 15

CLOSURE ARC NO. 5.5, WEST SIDE – NOTE THE LEVEL II CLEANED AREA REVEALING PERFORATION OF THE SHEET PILE.



17
1
PHOTO NO. 17

CLOSURE ARC NO. 6.5, WEST SIDE – NOTE THE RIP ALONG THE RIGHT SHEET PILE AND THE MULTIPLE CONSTRUCTION / WEEP HOLES IN THE SHEET PILE ON THE LEFT WHICH HAVE BOTH BEEN SUBJECTED TO LEVEL II CLEANING.

18
1
PHOTO NO. 18

CLOSURE ARC NO. 6.5, EAST SIDE – INSPECTION ALSO FOUND THESE TWO SHEET PILES WHICH HAVE ALSO FAILED DUE TO CORROSIVE SECTION LOSS.

20
1
PHOTO NO. 20

CLOSURE ARC NO. 7.5, WEST SIDE – INVESTIGATION FOUND THAT THIS CLOSURE ARC HAS FAILED DUE TO CORROSIVE SECTION LOSS. ALSO NOTE THE LOSS OF FILL AT THIS LOCATION. INVESTIGATION OF THE FILL AGAINST THE STRAIGHT CLOSURE ARC REPAIR FOUND NO EVIDENCE OF MIGRATION OF THE BACKFILL FROM UNDER THE SHEET PILING.

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– PND REFERENCE #4

21/1 **PHOTO NO. 21**

CLOSURE ARC NO. 7.5, FAILED SHEET PILE – NOTE THE KNIFE EDGE THINNING OF THE FAILED SHEET PILING LOCATED ON THE EAST EDGE OF THE FAILURE.

22/1 **PHOTO NO. 22**

CLOSURE ARC NO. 7.5, FAILED SHEET PILE – NOTE THE, THIN AND UNEVEN EDGE OF THE STEEL RESULTING FROM THE PLASTIC DEFORMATION AND ULTIMATE TENSILE FAILURE OF THE PILE'S WEB. VISIBLE CORROSION AND THINNING WERE EVIDENT ON THE PILES. INVESTIGATION OF THE FAILED PILES FOUND THE TEAR TO EXTEND TO THE MUDLINE.

23/1 **PHOTO NO. 23**

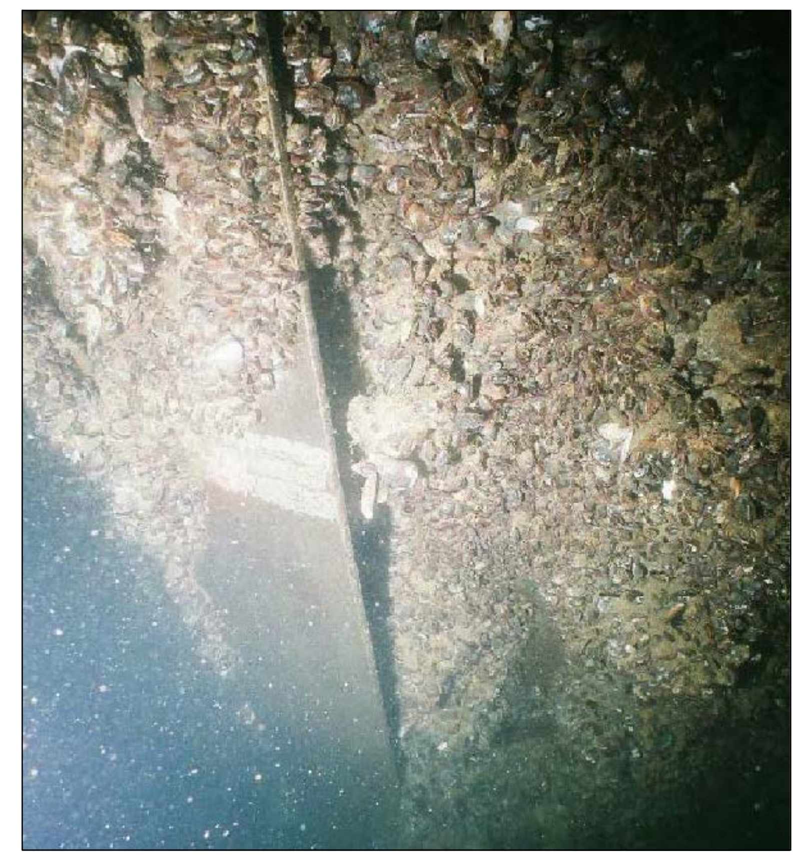
CLOSURE ARC NO. 7.5, EAST SIDE – PILES WITH ADDITIONAL PERFORATION AND FAILURE WERE ALSO IDENTIFIED ON THIS SIDE OF THE CLOSURE ARC.

24/1 **PHOTO NO. 24**

CLOSURE ARC NO. 7.5, LOOKING WEST – NOTE THE LOSS OF BACKFILL FROM BEHIND THE CIRCULAR CLOSURE ARC AND THE OVERALL GOOD CONDITION OF THE ANODES ATTACHED TO THE STRAIGHT, REPAIR PORTION OF THE CLOSURE ARC.

25/1 **PHOTO NO. 25**

CLOSURE ARC NO. 8.5, EAST SIDE – NOTE THE LEVEL II CLEANED AREA IN FRONT OF THE H PILE EXPOSING THE PERFORATION OF THE SHEET PILE. ALSO NOTE THE FAILURE OF THE SHEET PILE TO THE RIGHT INDICATED BY THE ARROW.



26/1 **PHOTO NO. 26**

CLOSURE ARC NO. 8.5, EAST SIDE – INSPECTION FOUND THAT THE H-PILE THAT SUPPORTS THE CONCRETE DECK, AND WAS INSTALLED AS PART OF THE REPAIR WORK, HAS BEEN DRIVEN THROUGH THE ORIGINAL CIRCULAR CLOSURE ARC IN THE SUBMERGED ZONE. NOTE THE FLANGE OF THE H PILE WHICH IS VISIBLE OUTSIDE OF THE SHEET PILING AT THIS LOCATION.

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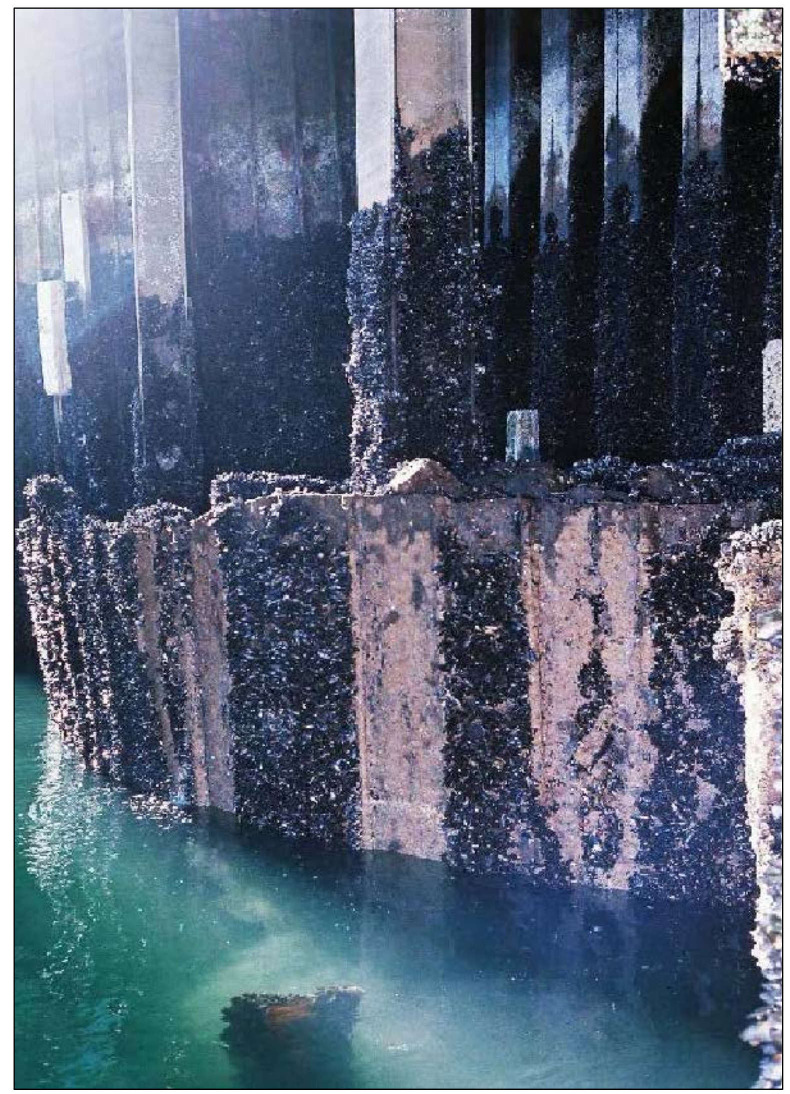
PHOTOGRAPHS FROM ECHELON ENGINEERING REPORT – PND REFERENCE #4

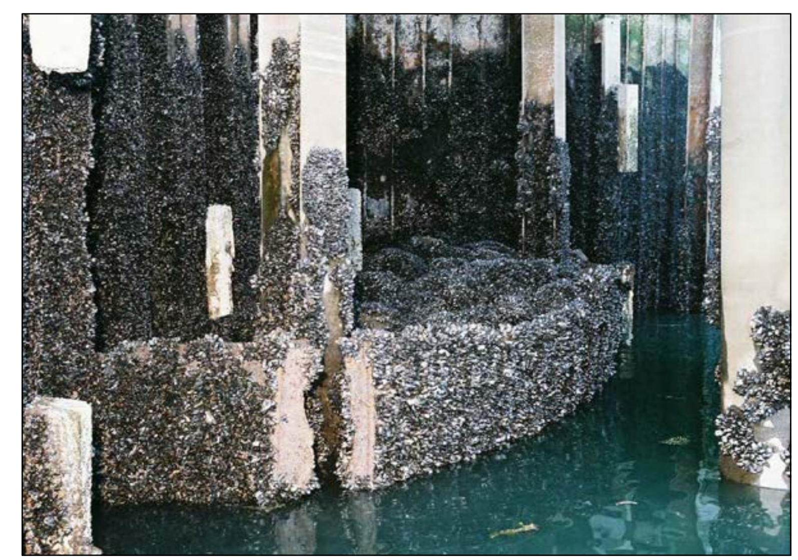
27 / 1 **PHOTO NO. 27**

CLOSURE ARC NO. 8.5, LOOKING WEST – NOTE THE LEVEL OF THE BACKFILL BEHIND THE CIRCULAR CLOSURE ARC. NO EVIDENCE OF ANY LOSS OF FILL WAS FOUND ASSOCIATED WITH THIS CLOSURE ARC.



29 / 1 **PHOTO NO. 29**

CLOSURE ARC NO. 10.5, WEST SIDE – NOTE THE PERFORATION DAMAGE TO THE THREE SHEET PILES WHICH HAVE RECEIVED LEVEL II CLEANING. INSPECTION BELOW WATER FOUND THAT THIS WESTERN H-PILE HAS ALSO BEEN DRIVEN THROUGH THE SHEET PILING IN THE SUBMERGED ZONE.



30 / 1 **PHOTO NO. 30**

CLOSURE ARC NO. 10.5, EAST SIDE – INVESTIGATION OF THE EAST SIDE OF THIS CLOSURE ARC ALSO IDENTIFIED DAMAGE AND FAILURE OF THE SHEET PILING. INVESTIGATION OF THE BACK FILL DID NOTE IT TO BE SLIGHTLY LOWER, BUT NO VISIBLE EVIDENCE OF LOSS WAS APPARENT.

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PHOTO NO. 31 (31/1)

CLOSURE ARC NO. 8.5 REPAIR, EAST END – NOTE THE REPAIRED SECTION OF THE 8.5 CLOSURE ARC TO THE RIGHT OF THE STADIA ROD AND THE MAIN CELL NO. 9 ON THE LEFT. THE ARROWS INDICATE THE GAP BETWEEN THE MAIN CELL AND THE EAST END OF THE REPAIR SECTION. NOTE A CLEANED PORTION OF THE STEEL H-PILE SHOWN ON REFERENCE DRAWING S-3, DETAIL 8, CAN BE SEEN JUST ABOVE THE LOWER RIGHT THE UPPER RIGHT ARROW INDICATES A LARGE (~2' DIA.) ROCK IN THE BACKFILL. NO EVIDENCE OF ANY FINES OR INTACT GEOTEXTILE FABRIC WAS VISIBLE THROUGH THIS GAP. REMNANTS OF THE GEOTEXTILE FABRIC WAS FOUND AT THE BOTTOM OF THE GAP. THIS CONDITION IS TYPICAL OF THE LOCATIONS SHOWN IN THE FOLLOWING SERIES OF PHOTOS NO. 32-36.

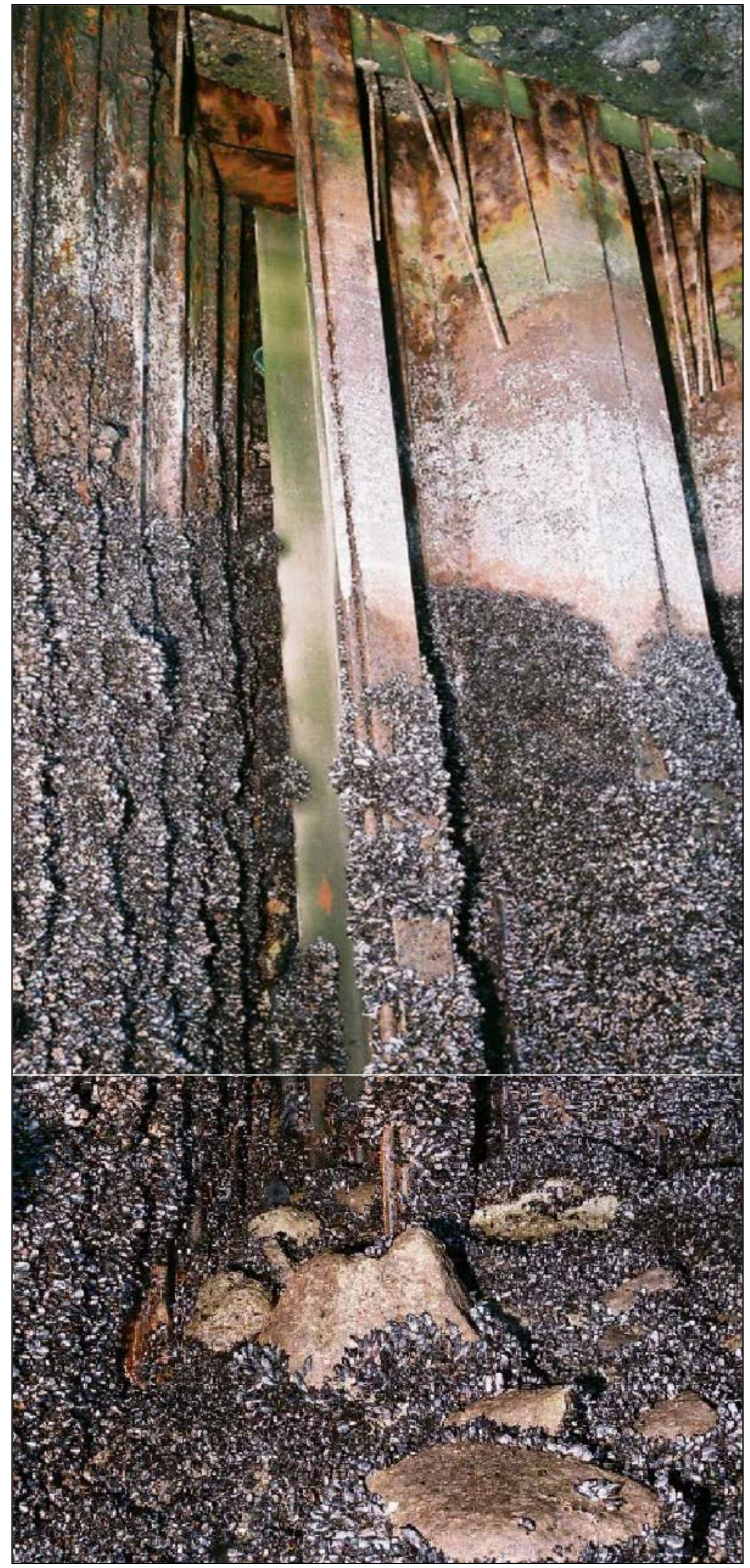

PHOTO NO. 32 (32/1)

CLOSURE ARC NO. 4.5 REPAIR, EAST END – NOTE THE GAP BETWEEN THE STEEL H-PILE, DESIGNED TO TERMINATE AND SEAL THE END OF THE CLOSURE REPAIR, AND THE MAIN CELL. NO EVIDENCE OF ANY INTACT GEOTEXTILE FABRIC OR ANY FINES WAS VISIBLE THROUGH THE GAP.

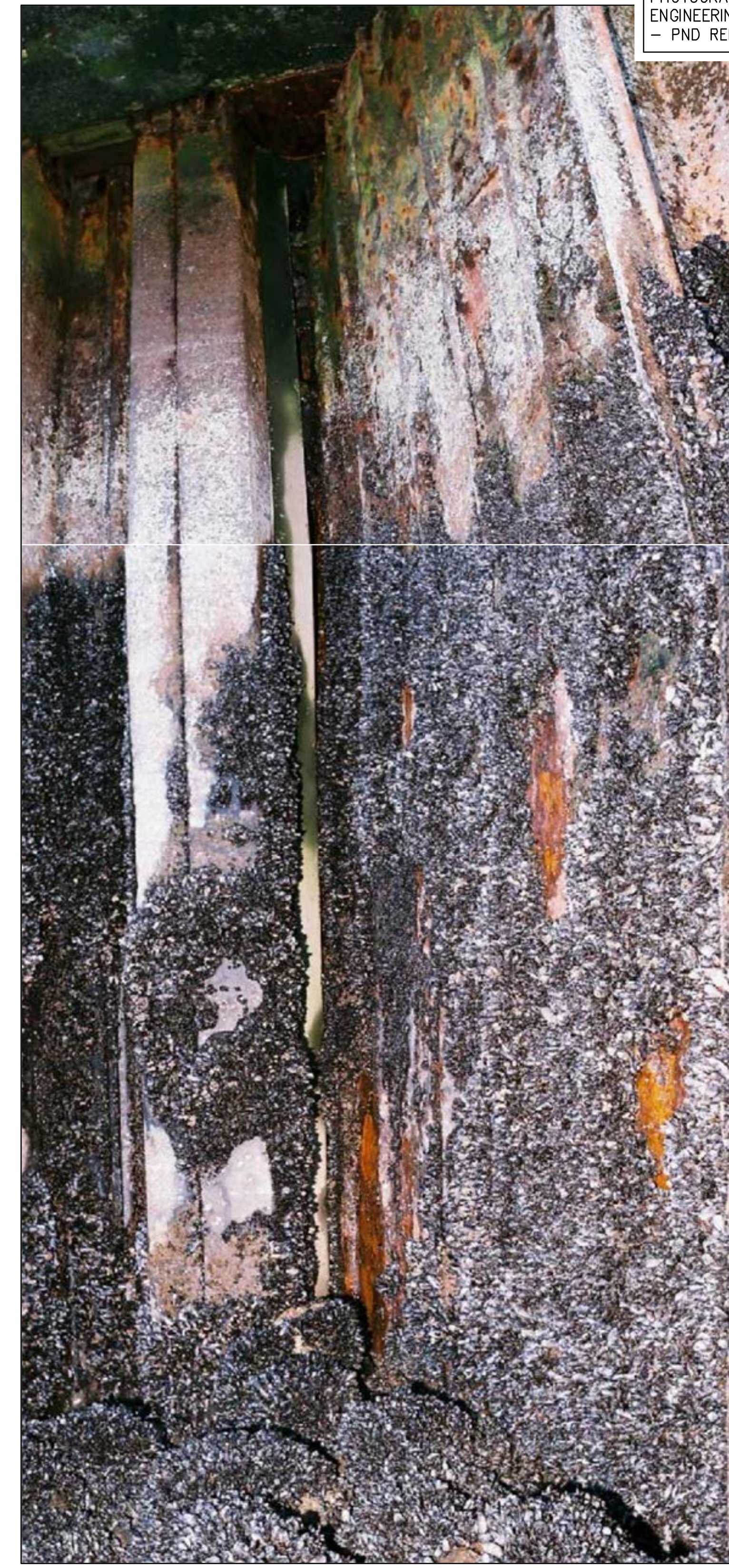

PHOTO NO. 33 (33/1)

CLOSURE ARC NO. 6.5 REPAIR, WEST END – NOTE THE GAP BETWEEN THE STEEL H-PILE, DESIGNED TO TERMINATE AND SEAL THE END OF THE CLOSURE REPAIR, AND THE MAIN CELL. NO EVIDENCE OF ANY INTACT GEOTEXTILE FABRIC OR ANY FINES WAS VISIBLE THROUGH THE GAP.

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PHOTO NO. 34 (34/1)

CLOSURE ARC NO. 8.5 REPAIR, EAST END – NOTE THE GAP BETWEEN THE STEEL H-PILE / END SHEET PILE AND THE MAIN CELL. NO EVIDENCE OF ANY INTACT GEOTEXTILE FABRIC OR ANY FINES WAS VISIBLE THROUGH THE GAP. (ALSO SHOWN IN PHOTO NO. 31).

PHOTO NO. 35 (35/1)

CLOSURE ARC NO. 9.5 REPAIR, EAST END – NOTE THE GAP BETWEEN THE STEEL H-PILE, DESIGNED TO TERMINATE AND SEAL THE END OF THE CLOSURE REPAIR, AND THE MAIN CELL. NO EVIDENCE OF ANY INTACT GEOTEXTILE FABRIC OR ANY FINES WAS VISIBLE THROUGH THE GAP.

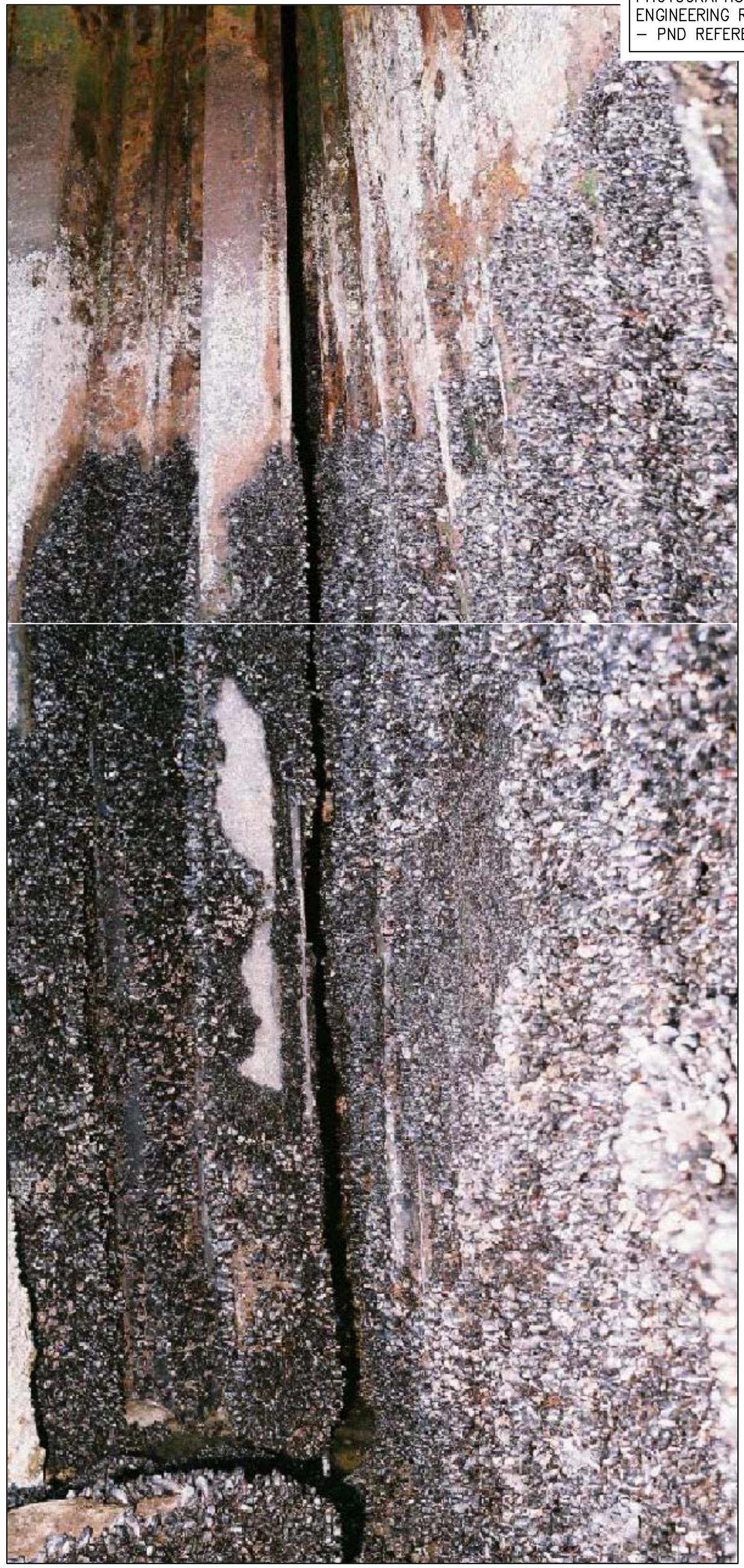

PHOTO NO. 36 (36/1)

CLOSURE ARC NO. 10.5 REPAIR, WEST END – NOTE THE GAP BETWEEN THE LAST STEEL PILE AND THE MAIN CELL. NO EVIDENCE OF ANY INTACT GEOTEXTILE FABRIC OR ANY FINES WAS VISIBLE THROUGH THE GAP.

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Analysis

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Project: 142010

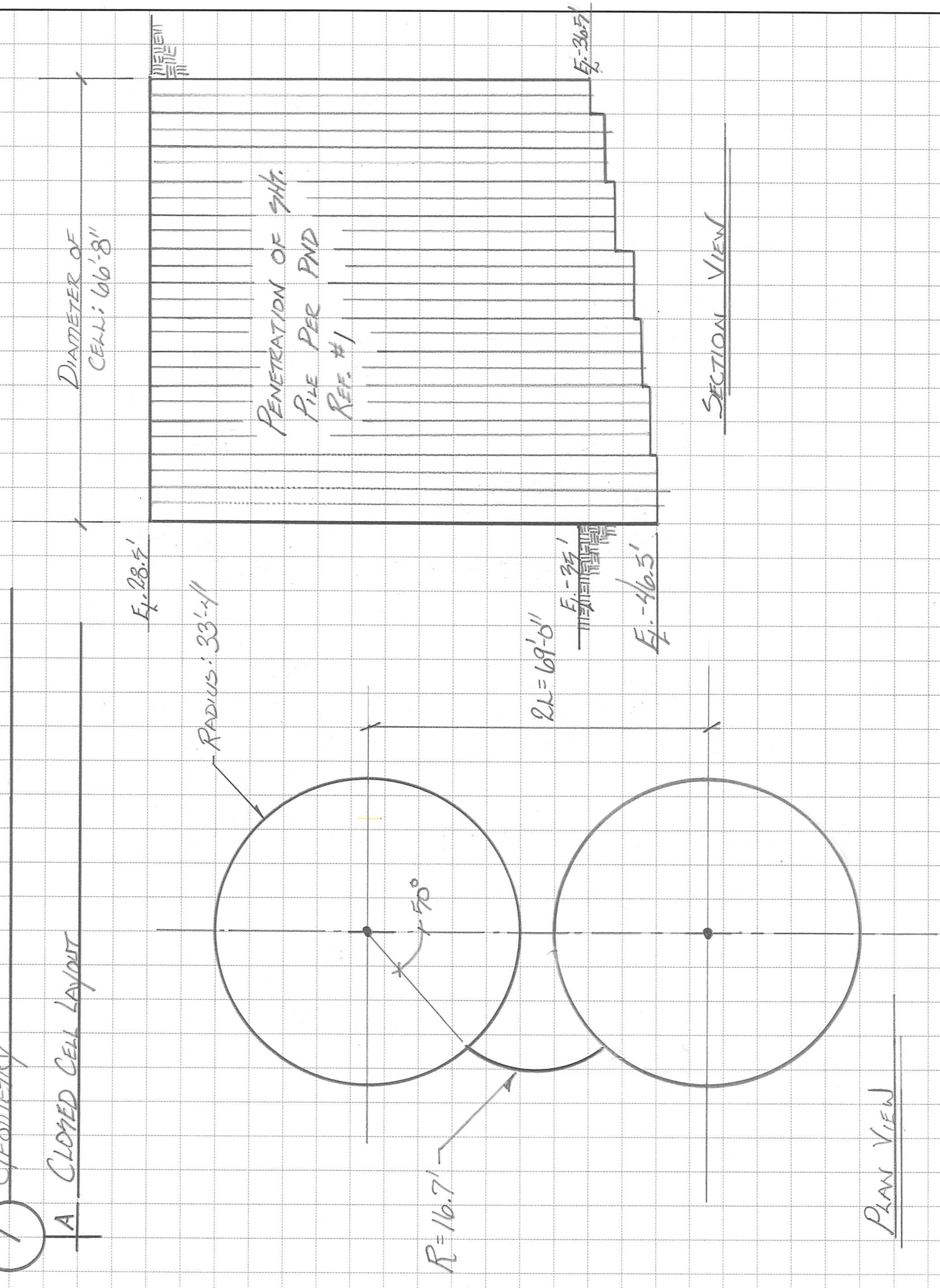
Sheet Number: ______ Of: ______

Calculated by: MJA Date: 9/14

Checked by: CF Date: 9/14

1 A GEOMETRY

CLOSED CELL LAYOUT



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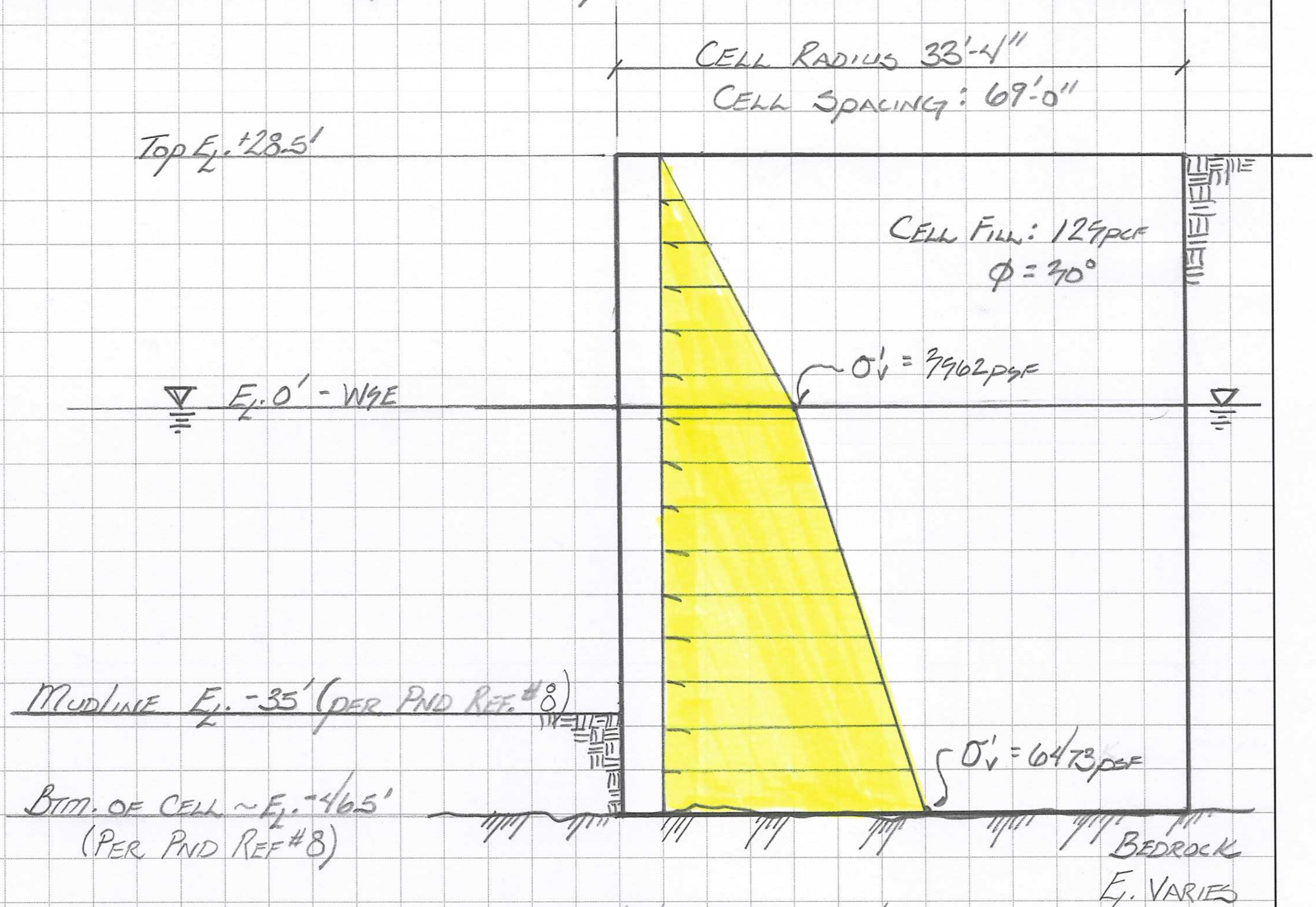
2 Loading / Demand

A Cell Geometry Analyzed / Dead Load Force & Moment



- Internal Force ($K = 1.0$) = 284 K/FT @ E_L. -19.24'
- Calc Forces w/ $K = 1.0$ since different lateral pressure coeff. are applicable to each limit state.

- Demand on cell occurs from active state: $K_A = 0.33$
- $F_D = 0.33(284\ K/FT) = 94.67\ K/FT$
- $M_D = 0.33(284\ K/FT)(E_L.\ -19.24 - E_L.\ -46.5) = 2580^{K\text{-}FT}$

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2 LOADING / DEMAND

B CHECK INTERLOCK TENSION (DEAD LOAD)

- TOTAL HEIGHT OF CELL: $H = E_L.28.5' - E_L.-46.5 = 75'$

- PEAK INTERLOCK TENSION FOR CELLS FOUNDED ON ROCK OCCURS AT H/4 FROM BTM. OR $E_L. -27.75$

- LATERAL COEFF. OF PRESSURE FOR CLOSE CELL INTERLOCK TENSION: $K = 1.4 \tan^2(45 + \phi/2)$
  $= 0.467$

- $T_{D_L} = \sigma_v'(E_L.-27.75')\cdot K \cdot r_{CELL}$ – FOR MAIN CELL

  $= \left(125pcf\,(E_L.28.5 - E_L.0') + 62.6pcf\,(E_L.0 - E_L.-27.75')\right)(0.467)(33.33')$

  $= 82.5^K/FT = 6.9^K/IN$ ← MAIN CELL DEMAND – D_L – ONLY

- $T_{D_L} = \sigma_v'(E_L.-27.75')\cdot K \cdot r_{CELL} \cdot \sec\theta$ – FOR CONNECTING ARC

  θ = CONNECTING ARC ANGLE: 30°

  $= (5300psf)(0.467)(33.3')\sec(30°) = 128.4^K/FT$
  $= 10.7^K/IN$ ← MAIN CELL / CONNECTING ARC DEMAND D_L – ONLY

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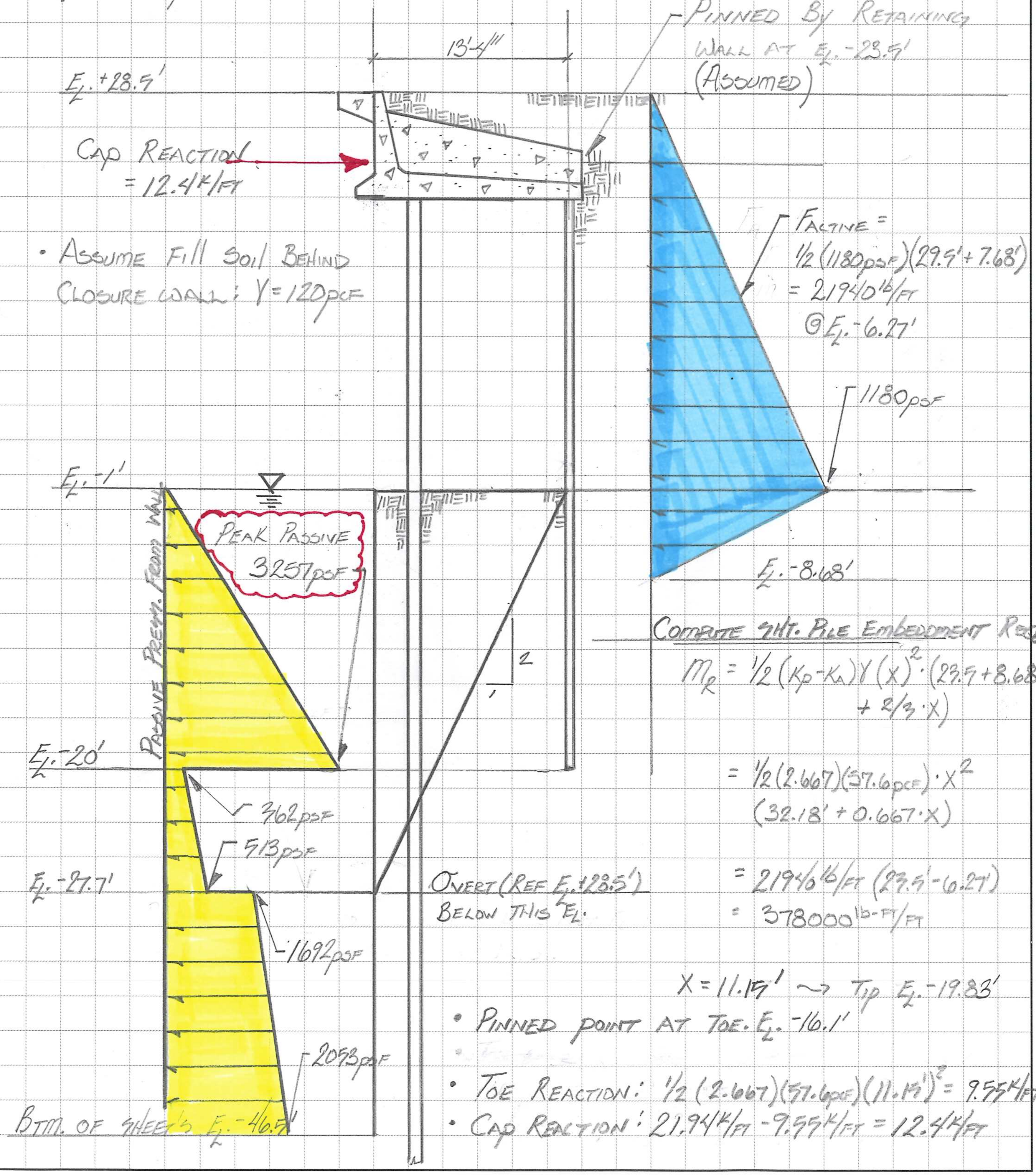
2 Loading / Demand

C Compute Wall Pressure On Connecting Arc (Dead Load)



Compute Sht. Pile Embedment Req.

$$M_R = 1/2\,(K_p - K_A)\,\gamma\,(X)^2 \cdot (23.5 + 8.68' + 2/3 \cdot X)$$

$$= 1/2\,(2.667)(57.6\,pcf) \cdot X^2\,(32.18' + 0.667 \cdot X)$$

$$= 21940\,lb/ft\,(23.5' - 6.27')$$

$$= 378000\,lb\text{-}ft/ft$$

$X = 11.15' \rightarrow$ Tip EL. -19.83'

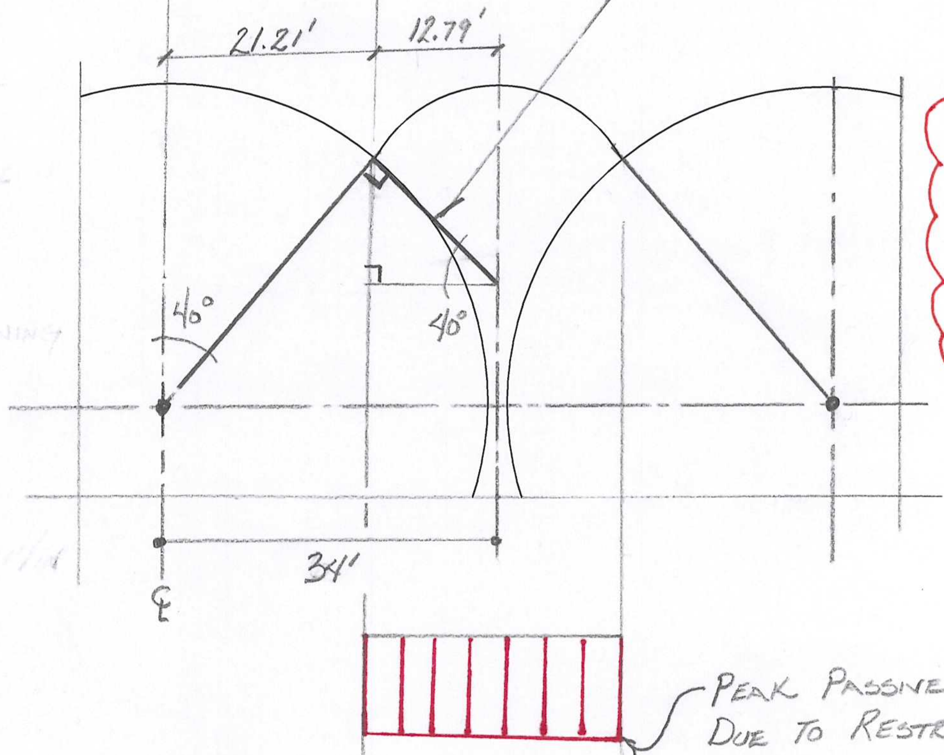
- Pinned point at Toe. EL. -16.1'
- Toe Reaction: $1/2\,(2.667)(57.6\,pcf)(11.15')^2 = 9.55\,K/ft$
- Cap Reaction: $21.94\,K/ft - 9.55\,K/ft = 12.4\,K/ft$

(2) LOADING/DEMAND

✓CF 9/14

℄ SHEET PILE INTERLOCK STRESS AT CONNECTING ARC (DEAD LOAD)

RADIUS OF ARC: $12.79'/\cos(40°) = 16.7'$



INTERLOCK TENSION

$T_D = p \cdot r$

$= (4.56\,\text{ksf})(16.7') = 76.2\,^{K}/_{FT}$

$= 6.35\,\text{ksi}$

PEAK PASSIVE PRESSURE DUE TO RESTRAINING TOE OF REPAIR WALL: 3257psf

PER EM 1110-2-2503 - WHERE COMPUTING INTERLOCK TENSION USE $K = 1.2$ TO $1.6\,K_A$. SINCE LATERAL PRESSURE BASED ON PASSIVE CONDITION USE $K = 1.4\,K_P$:

PEAK CELL PRESSURE: $1.4(3257\,\text{psf}) = 4560\,\text{psf}$

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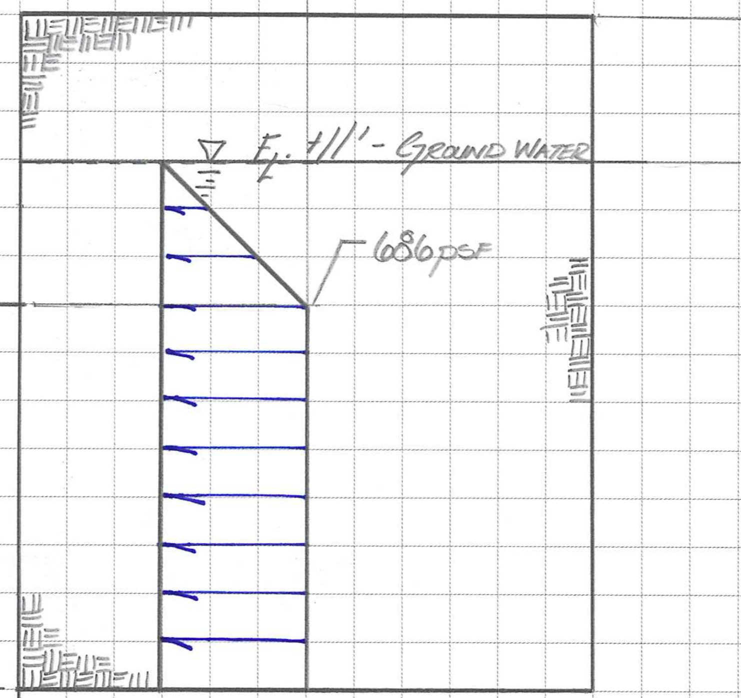
2 Load / Demand

D Differential Water Surface Elevation Load

- Compute forces due to differential water surface elevation

- $F_W = \frac{1}{2}(11')(686\,psf) + (46.5')(686\,psf) = 35.7^{k}/FT$



EL. 0'

- M_W (about EL. $-46.5'$)

$$= \tfrac{1}{2}(11')(686\,psf)\left(\tfrac{11'}{3}+46.5'\right) + \tfrac{1}{2}(46.5')^2(686\,psf)$$
$$930^{k\text{-}FT}/FT$$

EL. -46.5

- Interlock tension from differential WSE

$$T_{W/E} = (11')(62.4\,pcf)(33.33') = 22.9^{k}/FT = 1.9^{k}/IN$$

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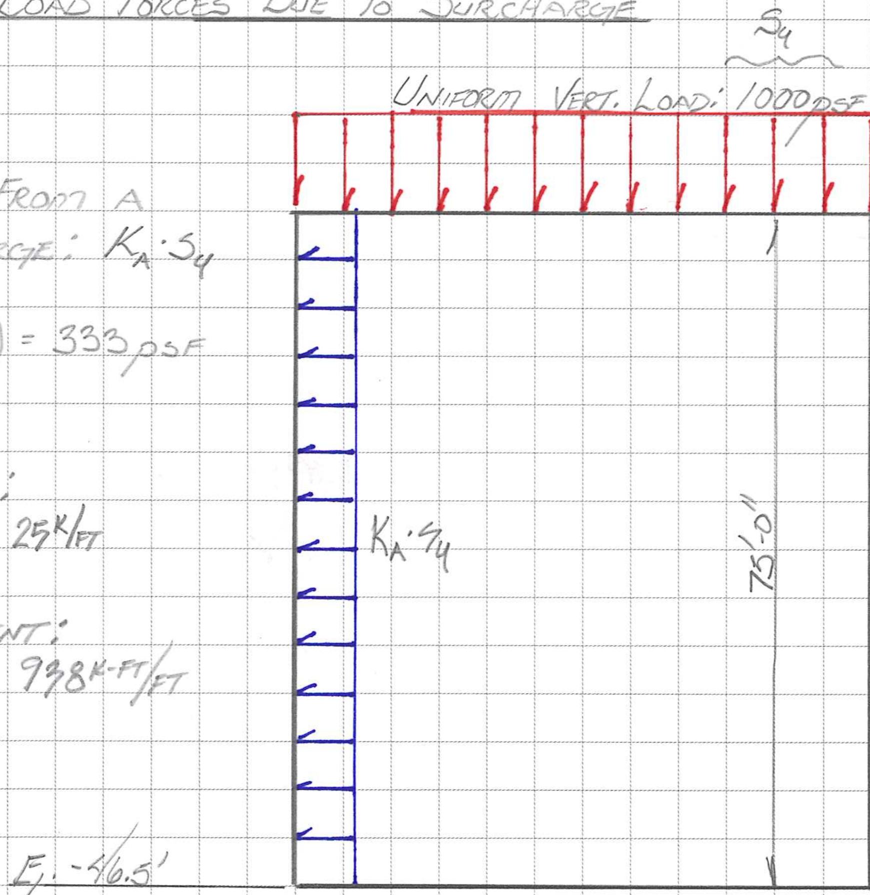
2 LOAD / DEMAND

E ESTIMATE LIVE LOAD FORCES DUE TO SURCHARGE



- LATERAL LOAD FROM A 1000 PSF SURCHARGE: $K_A \cdot S_4$

  $= 0.33(1000\text{psf}) = 333\text{psf}$

- LIVE LOAD FORCE:
  $(75') \cdot (K_A \cdot S_4) = 25^K/\text{FT}$

- LIVE LOAD MOMENT:
  $1/2(K_A \cdot S_4)L^2 = 938^{K\text{-FT}}/\text{FT}$

- INTERLOCK TENSION: $(1.4 \cdot K_A \cdot S_4)r_{CELL}$

  $= 1.4(0.333)(1000\text{psf})(33.33') = 15.6^K/\text{FT}$ OR
  $1.3^K/\text{IN}$

✓CF 9/14

TREAT AXLE LOAD AS A POINT LOAD

Point Load	130 kip	MAX AXLE LOAD - PER SPEC'S
Poisson's ratio	0.35	TYP. VALUES RANGE FROM 0.25 TO 0.49
Load Offset from Wall	10 ft	
Out of Plane Offset	0 ft	
Top El. Of Wall	28.5 ft	
Btm El. Of Wall	-46.5 ft	
Wall Height	75 ft	

Increment	Vert. Position (ft)	Elev. (ft)	Radius (ft)	Lateral Pressure (ksf)	Force (k/ft)	Moment (kip-ft/ft)
1	0.00	28.50	10.00	0.00		
2	3.13	25.38	10.48	0.22	0.34	9.27
3	6.25	22.25	11.79	0.28	0.78	18.68
4	9.38	19.13	13.71	0.20	0.75	15.62
5	12.50	16.00	16.01	0.12	0.50	8.83
6	15.63	12.88	18.55	0.07	0.30	4.27
7	18.75	9.75	21.25	0.04	0.17	1.91
8	21.88	6.63	24.05	0.02	0.10	0.79
9	25.00	3.50	26.93	0.01	0.06	0.28
10	28.13	0.38	29.85	0.01	0.03	0.06
11	31.25	-2.75	32.81	0.00	0.02	-0.02
12	34.38	-5.88	35.80	0.00	0.01	-0.04
13	37.50	-9.00	38.81	0.00	0.01	-0.04
14	40.63	-12.13	41.84	0.00	0.00	-0.02
15	43.75	-15.25	44.88	0.00	0.00	-0.01
16	46.88	-18.38	47.93	0.00	0.00	0.00
17	50.00	-21.50	50.99	0.00	0.00	0.00
18	53.13	-24.63	54.06	0.00	0.00	0.00
19	56.25	-27.75	57.13	0.00	0.00	0.00
20	59.38	-30.88	60.21	0.00	0.00	0.00
21	62.50	-34.00	63.29	0.00	0.00	0.00
22	65.63	-37.13	66.38	0.00	0.00	0.00
23	68.75	-40.25	69.47	0.00	0.00	0.00
24	71.88	-43.38	72.57	0.00	0.00	0.00
25	75.00	-46.50	75.66	0.00	0.00	0.00
					3.1	19.4

LOAD MAGNITUDE (K/FT)

ELEV. OF LOAD CENTROID

- EQUATION FOR WALL PRESSURE: $\frac{P}{\pi \cdot R^2}\left(\frac{3ZX}{R^3} - \frac{R(1-2\nu)}{R+Z}\right)$

P = POINT LOAD

X = HORIZONTAL OFFSET FROM WALL FACE

Z = DEPTH BELOW POINT

ν = POISSON'S RATIO

y = OUT-OF-PLANE OFFSET FROM LOAD

- POINT LOAD MOMENT (ABOUT EL. -46.5'): $3.1^{K}/_{FT}$ (EL. 19.4' - EL. -46.5') = $204^{K\text{-}FT}/_{FT}$
- POINT LOAD WILL HAVE MINIMAL EFFECT ON INTERLOCK TENSION.

PEAK INTENSITY DOES NOT OCCUR AT SAME EL. AS PEAK P_L TENSION

2 LOAD/DEMAND

F 130 KIP AXLE LOAD (LIVE LOAD)

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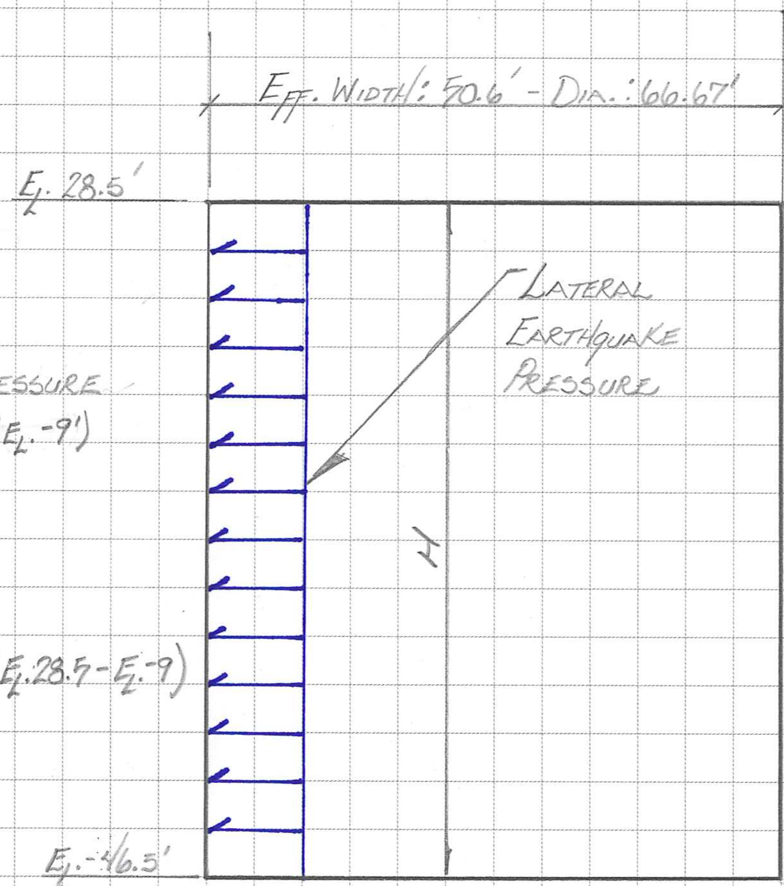
2 Load/Demand

9 Seismic Load on Bulkhead



- Seismic Wall Pressure Equal To: $K_H \cdot \sigma_v (E_L. -9')$

  $K_H = 1/2 \cdot PGA$

- $W_{EQ} = 1/2 \cdot (PGA) \cdot (\gamma)(E_L.28.5 - E_L.-9)$

- For 10yr Return Period Eq.: $PGA = 0.016$, $W_{Eq} = 1/2(0.016)(125pcf)(37.5')$
  $= 37.5psf = 0.5H$

- For 72yr Return Period Eq.: $PGA = 0.072$, $W_{Eq} = 1/2(0.072)(125pcf)(37.5')$
  $= 169psf = 2.25H$

- Interlock Tension: (No Liquefaction): $W_{Eq} \cdot r_{cell} = (169psf)(33.33')$
  $= 5.6^k/ft$ or $0.47^k/in$

- $F_{EQ}(72yr.) = W_{EQ} \cdot H = (169psf)(E_L.28.5' - E_L.-46.5) = 12.6^k/ft$

- $M_{EQ}(72yr.) = 1/2 \cdot W_{EQ} H^2 = 1/2(169psf)(E_L.28.5' - E_L.-46.5)^2 = 475^k/ft$

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3 CAPACITIES

A ESTIMATE CAPACITY OF SHEET PILE INTERLOCKS

- ORIGINAL SHEET PILE INSTALLED HAVE WEBS WITH 1/2"t STEEL FOR THE MAIN ARCS, AND 3/8"t STEEL FOR THE CLOSURE ARCS.

- ON THE BASIS OF DOCUMENTATION PROVIDED/COLLECTED BY PND ENGINEERS APPROXIMATELY 0.16" OF METAL HAS CORRODED SINCE INSTALLATION IN 1953.

- ORIGINAL INTERLOCK CAPACITY (IN 1953) IS EST. TO BE 16K/IN.

- EST. CURRENT CAPACITY BY RATIO OF PRESENT BASE METAL TO ORIGINAL BASE METAL.

- FOR 1/2" SHT. PILE (MAIN CELL): $16^K/\text{IN} \cdot \left(\frac{0.5'' - 0.16''}{0.5''}\right) = 10.9^K/\text{IN}$
- FOR 3/8" SHT. PILE (CONNECTING ARCS): $16^K/\text{IN} \cdot \left(\frac{0.375'' - 0.16''}{0.375}\right) = 9.17^K/\text{IN}$

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3) Capacities

B Compute Vertical Shear Capacity

- Demand moment - Active soil only: $2980^{K\text{-}FT}$

- Moment capacity (vertical shear)

$$M_{VS} = \left(\tfrac{2}{3}\right) B_{EFF} \cdot \left(f \cdot P_{INT.} + \tan\phi \cdot P_S\right)$$

B_{EFF} = Eff. width of cell = 50.6' = $\pi \cdot r^2_{CELL}$ / (Cell to Cell)

f = Interlock friction = 0.3

P_S = Soil pressure in cell ~> $K = \cos^2\phi / (2 - \cos^2\phi) = 0.6$

P_F = Interlock force in interlock

$P_S = 0.6(284^{K}/_{FT}) = 170.4^{K}/_{FT}$

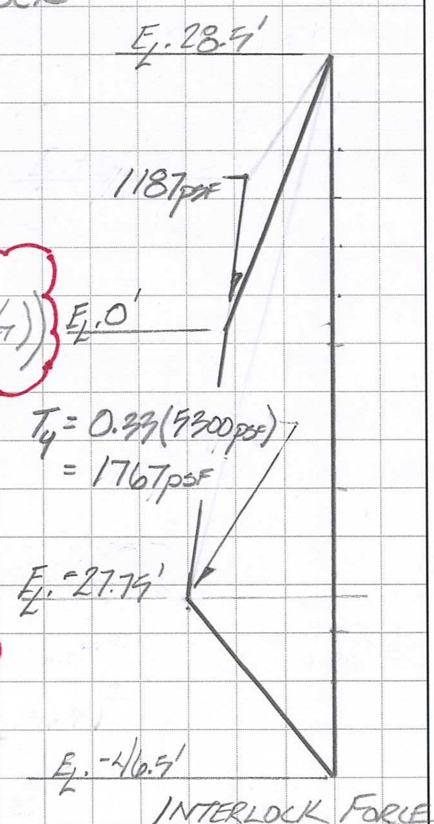
P_F = = $74.47^{K}/_{FT}$

$$M_{VS} = \tfrac{2}{3}(50.6')\left(0.3(74.5^{K}/_{FT}) + \tan(30°)(170.4^{K}/_{FT})\right)$$

$$= 4065^{K\text{-}FT}$$

Factor of safety against OL moment

$$FOS = 4065^{K\text{-}FT} / 2580^{K\text{-}FT} = 1.57$$



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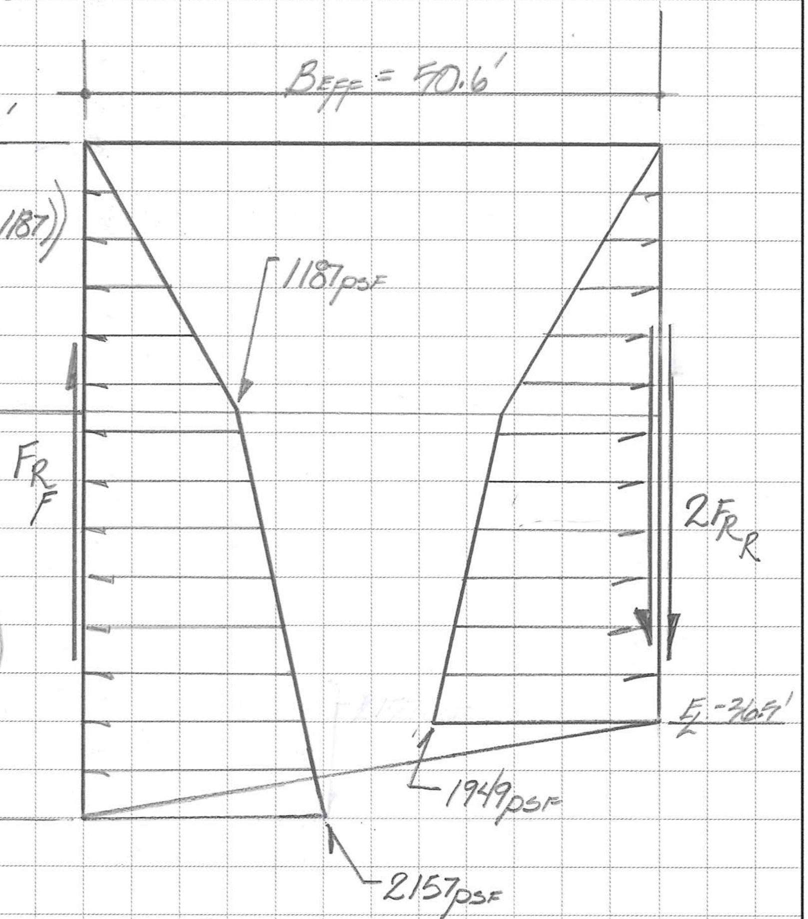
3 CAPACITIES

c ESTIMATE SHEET PILE PULLOUT

- SOIL/STEEL FRICTION ANGLE: 20°

- $F_R = \tan\delta \cdot F_A$

- $F_{A_R} = \frac{1}{2}\left((28.5')(1187\,psf) + (36.5')(1949 + 1187)\right)$
$= 74.15\,K/FT$



$F_{A_F} = \frac{1}{2}\left((28.5')(1187\,psf) + (46.5')(1187\,psf + 2157\,psf)\right)$
$= 94.7\,K/FT$

$F_R = (94.7\,K/FT)\tan(20°)$

$F_{R_R} = 27.0\,K/FT \quad F_{R_F} = 34.5\,K/FT$

- MOMENT CAPACITY OF SHEET PILE PULLOUT

$$M_{SPP} = (2F_{R_R} + F_{R_F}) \cdot B_{EFF} = (2(27\,K/FT) + 34.5\,K/FT)(50.6') = 4478\,K\text{-}FT$$

- MOMENT CAPACITY OF SHEET PILE PULLOUT GREATER THAN EITHER VERTICAL SHEAR OR HORIZONTAL SHEAR.

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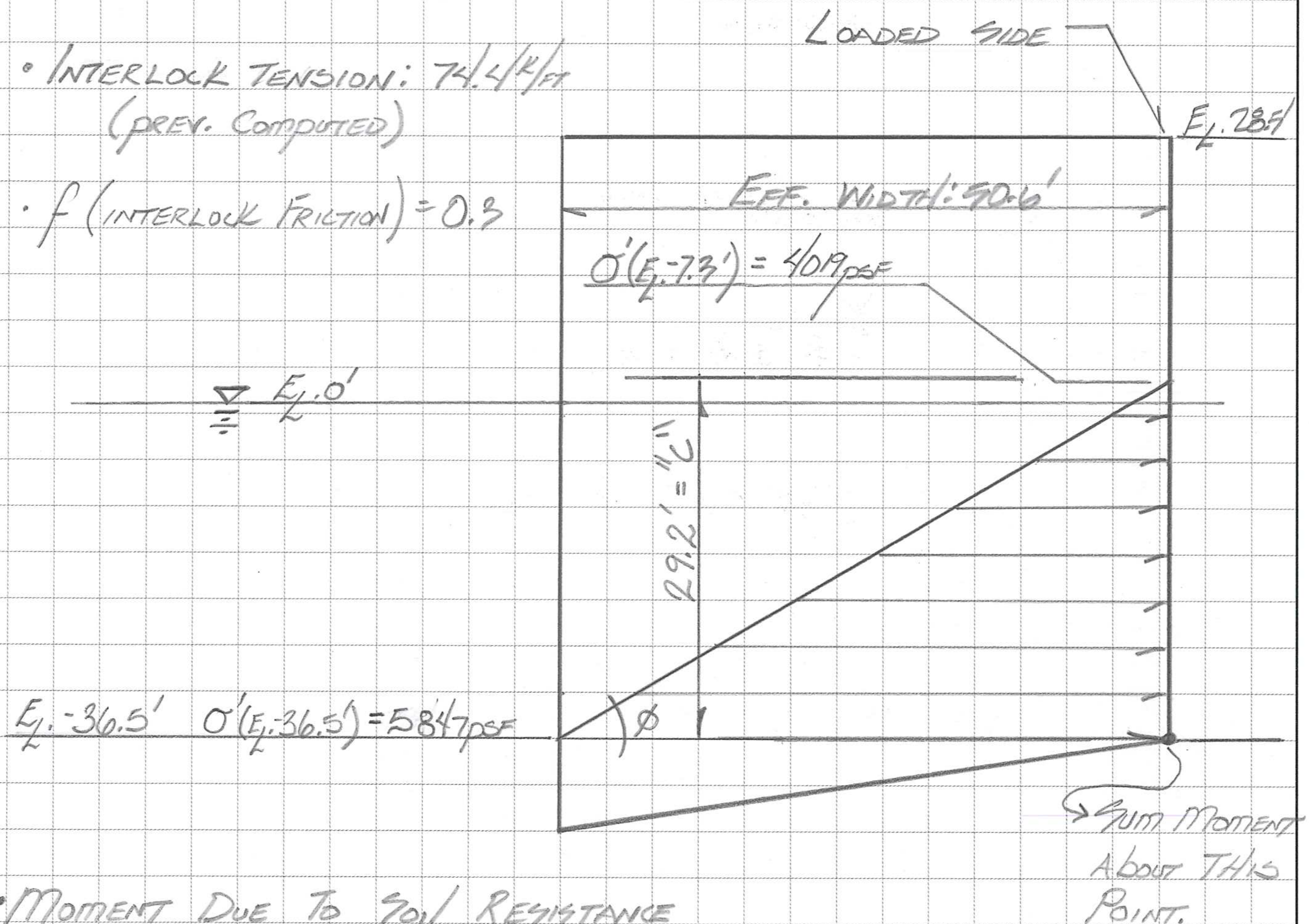
4 (HORIZONTAL SHEAR)

A TILTING CAPACITY

- INTERLOCK TENSION: 74.4 K/FT (PREV. COMPUTED)
- f (INTERLOCK FRICTION) = 0.3



- MOMENT DUE TO SOIL RESISTANCE

$$= \sigma'(E_L\,-7.3)\cdot C^2/2 + \sigma'(E_L\,-46.5')\cdot C^2/3$$

$$= (4019\,psf)(29.2')^2/2 + (5847\,psf)(29.2')^2/3 = 3375\ ^{K\text{-}FT}/_{FT}$$

- MOMENT DUE TO INTERLOCK FRICTION

$$= f\cdot P_T\cdot B = 0.3\,(74.4\ ^{K\text{-}FT}/_{FT})(50.6') = 1129\ ^{K\text{-}FT}/_{FT}$$

TOTAL RESIST 4504 K-FT/FT

- RECOMPUTE DEMAND LOADS BASE ON SUMMING MOMENT ABOUT E_L. -36.5

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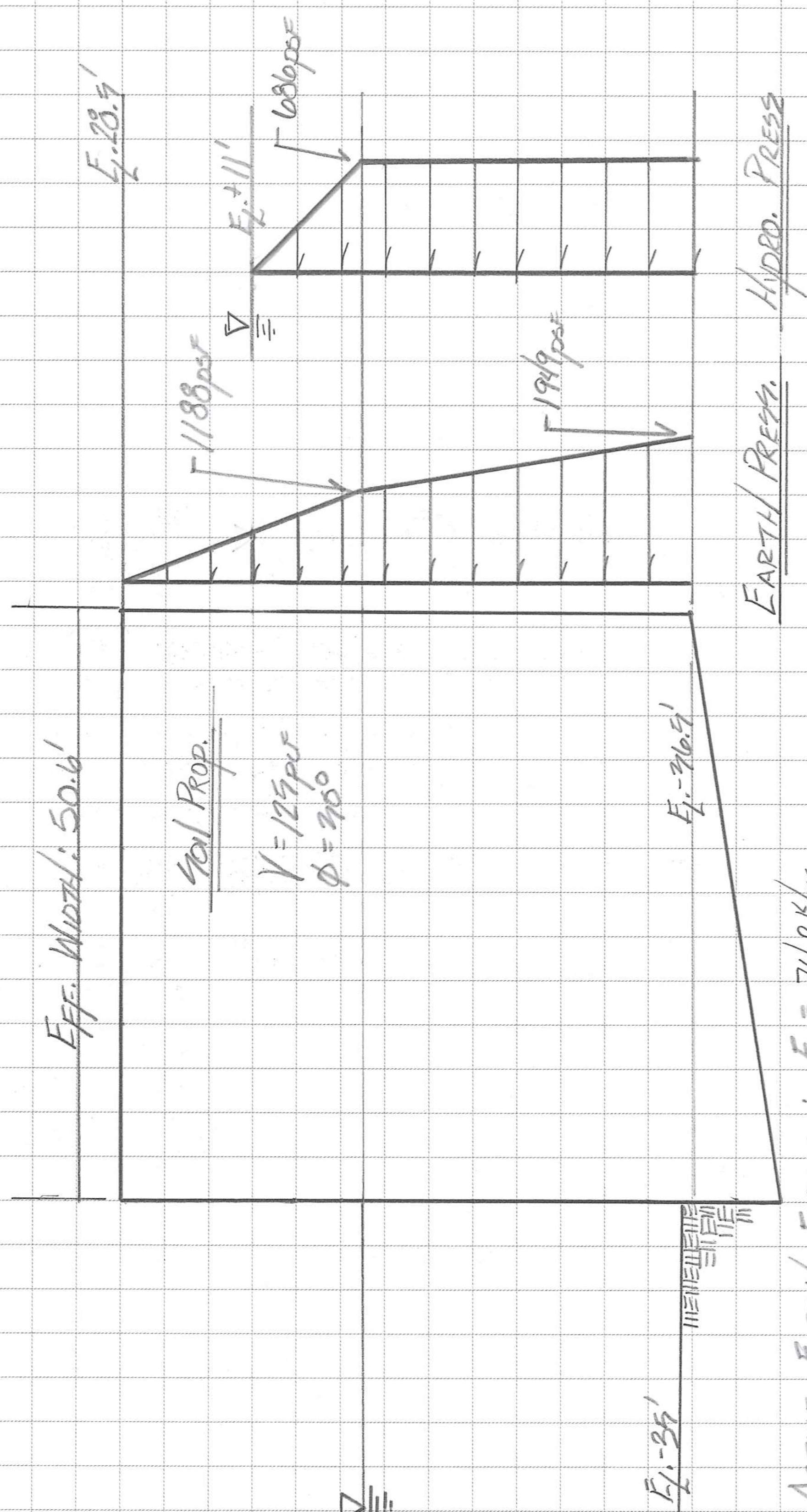
4 Tilting

B Tilting Moment (Driving)



Active Earth Forces: $F_A = 74.2$ k/ft
$M_A = 1739$ k-ft/ft

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4 TILTING

B TILTING MOMENT (DRIVING)

- EST. LIVE LOAD MOMENT ON CLOSED CELL
- HEIGHT OF CELL: 65'
- LATERAL LIVE LOAD PRESSURE: 333 PSF
- LIVE LOAD MOMENT: $\frac{1}{2} W \cdot L^2 = \frac{1}{2}(333\,\text{psf})(El.28.5' - El.-36.5')^2$

$$= 703^{\text{K-FT}}/\text{FT}$$

- ESTIMATE HYDROSTATIC MOMENT ON CELL

WSE IN FRONT OF BULKHEAD IS AT EL. 0'
WSE BEHIND BULKHEAD IS AT EL. 11'

$$F_W = \frac{1}{2}(62.4\,\text{psf})\left((47.5')^2 - (36.5')^2\right) = 28.8^{\text{K}}/\text{FT}$$

$$M_W = \frac{1}{6}(62.4\,\text{psf})\left((47.5')^3 - (36.5')^3\right) = 609^{\text{K-FT}}/\text{FT}$$

- EST. SEISMIC MOMENT ON CELL (475 YR EVENT)

$$W_{Eq} = 169\,\text{psf}$$

$$M_{EQ} = \frac{1}{2}(169\,\text{psf})(El.28.5' - El.-36.5')^2 = 357^{\text{K-FT}}/\text{FT}$$

Excerpt – PND Reference #1

5.0 SUBSURFACE CONDITIONS

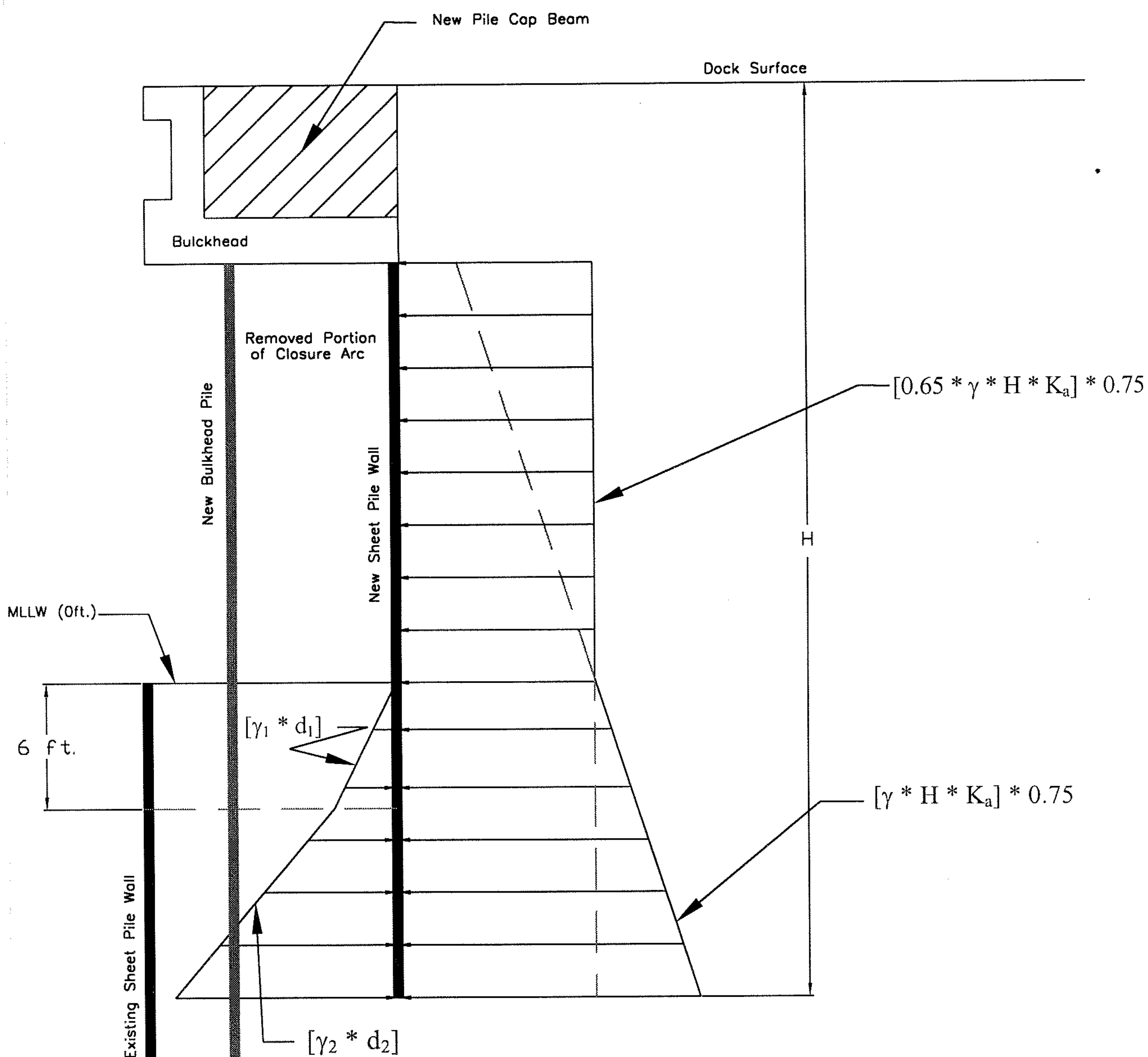
5.1 Soil and Groundwater Conditions

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



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

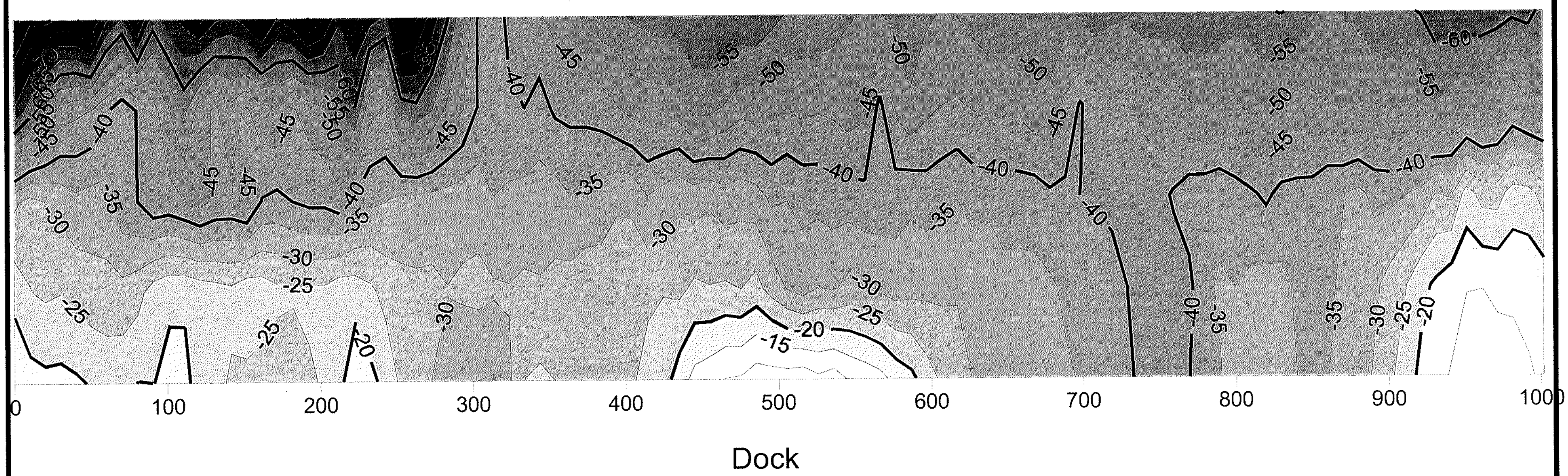
SHEET PILE WALL EARTH PRESSURE DETAIL

April 2002 32-1-01482

SHANNON & WILSON, INC.
Geotechnical & Environmental Consultants

FIG. 8

Bathymetric Map



0 100 200

Approximate Horizontal Scale 1" = 100'

0 20 40

Approximate Scale 1" = 20'

APOLLO GEOPHYSICS CORPORATION ENGINEERING, GEOLOGY, ENVIRONMENTAL CONSTRUCTION & MINING	Lutak Dock Rehabilitation Haines, Alaska		FIGURE 4
	FILE NO. 02.1003	DATE April 2002	

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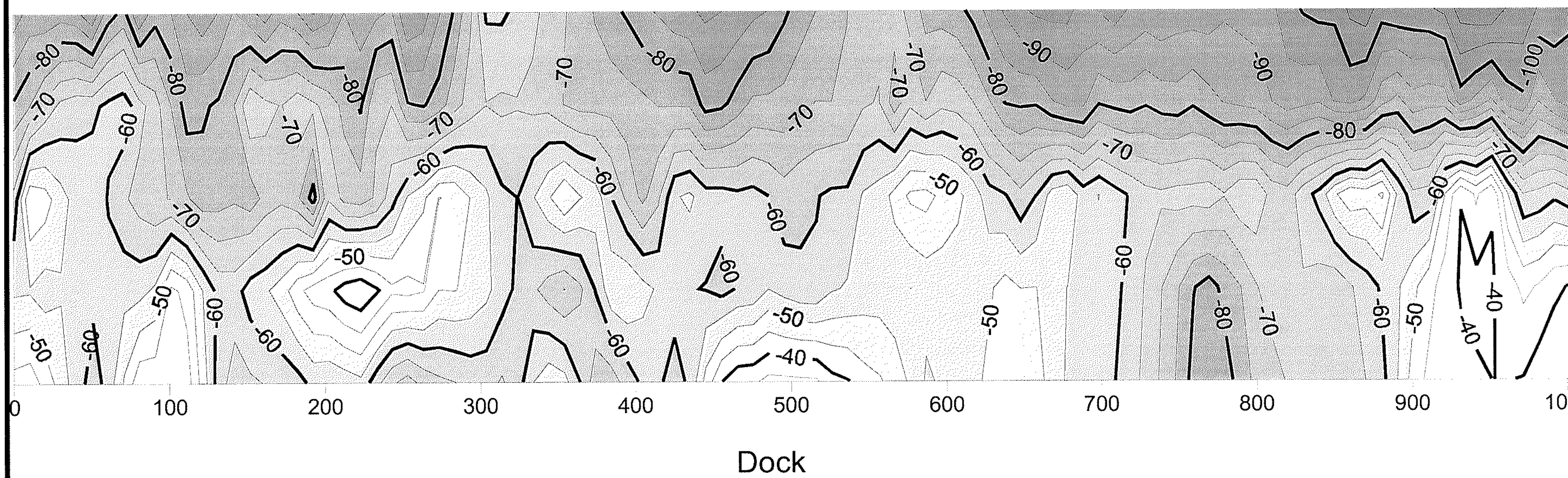
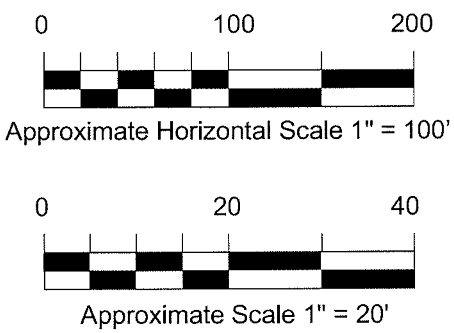
Geophysics\2002 Project Reports & Data\02.1003\02.1003 bathymetry

Sub-Bottom Elevation Map





APOLLO GEOPHYSICS CORPORATION ENGINEERING, GEOLOGY, ENVIRONMENTAL CONSTRUCTION & MINING	Lutak Dock Rehabilitation Haines, Alaska	FIGURE 5
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Geophysics\2002 Project Reports & Data\02.1003\02.1003 sub-bottom 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 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

Excerpt – PND Reference #7

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.

Excerpt – PND Reference #12

CHAPTER 4

ANALYSIS AND DESIGN

Section I. Characteristics

4-1. Structural Behavior. The stability of a sheet pile cell results from the composite action of the soil fill and the interlocking steel piling. The structural behavior of a cellular structure is governed by the engineering properties of the cell fill and the steel pile shell that contains and stiffens the cell fill. Because of this composite action, cells cannot be classified as a traditional concrete gravity monolith or a flexible earth embankment.

4-2. Forces.

a. Applied External Forces. Steel sheet pile cells are subject to external forces resulting from static water head, wave action, lateral earth pressure, and surcharge due to live load, earthquake, etc. These forces should be computed and applied as specified in the various engineer manuals referenced in Appendix A.

b. Reactive Berm Force. The passive force developed by a berm should be determined by a wedge analysis that accounts for the intersection of the failure wedge with the back slope of the berm. The Coulomb method of analysis or a Culmann graphical solution can be used when appropriate. The resistance provided by the berm should be limited to a value consistent with the berm reaction resulting from a sliding analysis.

4-3. Equivalent Cell Width. The equivalent width B of a sheet pile cellular structure is defined as the width of an equivalent rectangular section having a section modulus equal to that of the actual structure. For design purposes this definition can be simplified to equivalent areas as follows:

$$B = \frac{A}{2L}$$

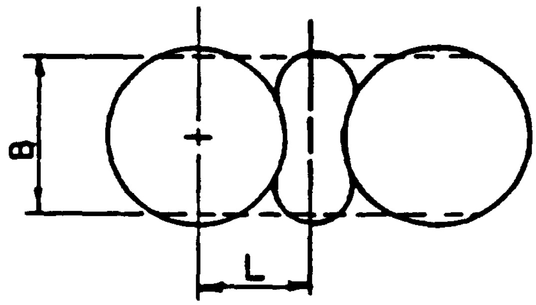
where

B = equivalent width

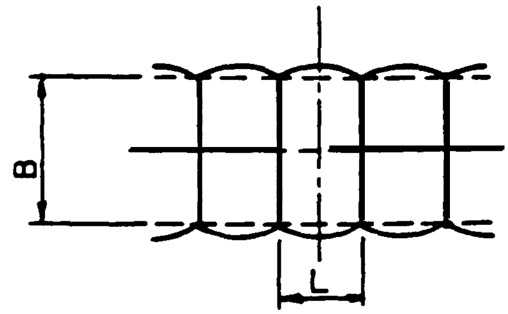
A = area of main cell, plus one connecting cell

2L = center-to-center distance between main cells

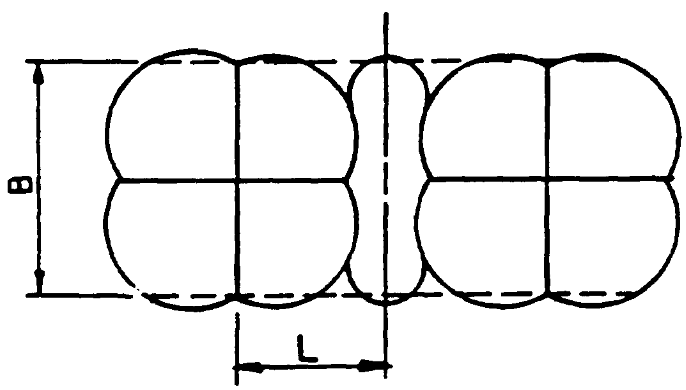
See Figure 4-1.



a. Plan circular cell



b. Plan arc and diaphragm cell



c. Plan clover leaf cell

Figure 4-1. Typical cellular cofferdam geometry can be pumped individually by turbine pumps or connected to a collector pipe with a centrifugal wellpoint pump system. Details of design of the relief well system have been discussed by Mansur and Kaufman and in EM 1110-2-1905. Details of dewatering are also included in Chapter 6 of this manual.

4-14. Internal Cell Stability.

a. Pile Interlock Tension. A cell must be stable against bursting pressure, i.e., the pressure exerted against the sheets by the fill inside the cell must not exceed the allowable interlock tension. The FS against excessive interlock tension is defined as the ratio of the interlock strength as guaranteed by the manufacturer to the maximum computed interlock tension. The interlock tension developed in a cell is a function of the internal cell pressure. The internal horizontal pressure p at any depth in the cell fill is the sum of the earth and water pressures. The earth pressure is equal to the effective weight of the cell fill above that depth times the coefficient of horizontal earth pressure K . This coefficient should ideally vary with the loading condition and the location within the cell; however, the actual variation is erratic and impossible to predict. It is recommended that a coefficient in the range of $1.2K_a$ to $1.6K_a$ is the coefficient of active earth pressure. The coefficient is dependent upon the type of cell fill material and the method of placement. See Table 4-2 for recommended values.

Table 4-2

Coefficients of Internal Pressure

Method of Placement	Type of Material: Crushed Stone	Coarse Sand and Gravel	Fine Sand	Silty Sand and Gravel	Clayey Sand and Gravel
Hydraulic dredge	$1.4K_a$		$1.5K_a$		$1.6K_a$
Placed dry and sluiced		$1.4K_a$		$1.5K_a$	
Wet clammed	$1.3K_a$		$1.4K_a$		$1.5K_a$
Dry material placed in dry		$1.3K_a$		$1.4K_a$	
Dumped through water	$1.2K_a$		$1.3K_a$		$1.4K_a$

Interlock tension is also proportional to the radius of the cell. The maximum interlock tension in the main cell is given by

$$t = pr$$

where

p = maximum inboard sheeting pressure

r = radius

The interlock tension at the connections between the main cells and the connecting arcs is increased due to the pull of the connecting arcs, as illustrated in Figure 4-15, and can be approximated by

$$t_{max} = pL \sec$$

where

t_{max} = interlock tension at connection

p = as previously defined

L = as shown in Figure 4-15

It must be emphasized that the above equation is an approximation since it does not take into account the bending stresses in the connection sheet pile produced by the tensile force in the sheet piles of the adjacent cell. Consequently, for critical structures, special analyses such as finite element should be used to determine interlock tension at the connections. In computing the maximum interlock tension, the location of the maximum unit horizontal

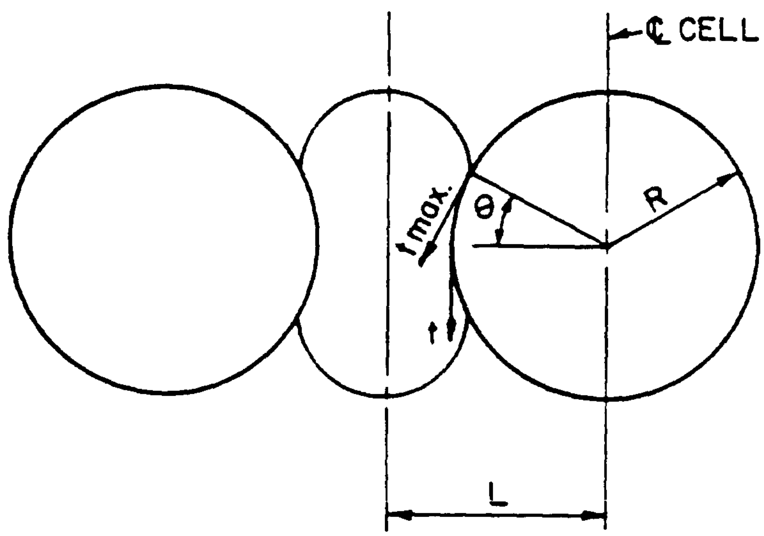


Figure 4-15. Interlock stress at connection

pressure p should be assumed to occur at a point one fourth of the height of the cell above the level at which cell expansion is fully restrained. Full restraint can be assumed to be where the external passive forces, due to overburden or a berm, and hydrostatic forces equal the internal cell pressures. In this case, it is generally sufficiently accurate and conservative to assume the point of maximum pressure to be at the top of the overburden or berm. When there is no overburden or berm, full restraint can be assumed to be at top of rock if the piling is seated on and bites into the rock. Maximum pressure should be assumed to occur at the base of cells which are neither seated in rock nor fully restrained by overburden or berm. See Figure 4-16 for typical pressure distributions. As stated previously, future changes in the depth of overburden, removal of berms, changes in saturation level in the cell fill, rate of dewatering, etc., must be anticipated when determining the maximum interlock tension.

b. Interlock Tension. In order to minimize interlock tension, the following details should be considered:

(1) Adequate weep holes should be provided on the interior sides of the cells in cofferdams to reduce the degree of saturation of the cell fill. The weep holes should be adequately maintained during the life of the cofferdam.

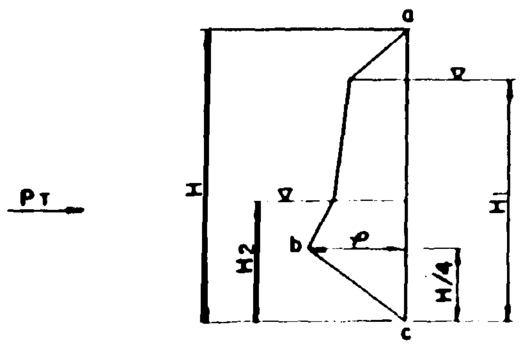
(2) Interlock tension failure has often occurred immediately after filling of the cells and can usually be traced to driving the sheets out of interlock. This results from driving through excessive overburden or striking boulders in the overburden. Overburden through which the piling must be driven should be limited to 30 feet. If the overburden exceeds this depth, consideration should be given to removing the excess prior to pile driving. The degree to which boulders may interfere with watertightness and driving of the cells can be estimated after a complete foundation exploration program.

(3) In an effort to reduce the effect of the connecting arc pull on the main cells, wye connectors are preferable to tees since the radial component of the pull on the outstanding leg is less for arcs of equal radius.

(4) Pull on the outstanding leg of connector piles can be reduced by keeping the radius of the connecting arc as small as practicable. The arc radius should not exceed one half of the radius of the main cell.

(5) Since tees and wyes are subjected to high local bending stresses at the connection, strong ductile connections are essential. Welded connections do not always meet this requirement because neither the steel nor the fabrication procedure is controlled for weldability. Therefore all fabricated tees, wyes, and cross pieces shall utilize riveted connections. In addition, the piling section from which such connections are fabricated shall have a minimum web thickness of one-half inch.

(6) Only straight web pile sections shall be used for cells as the hoop-tension forces would tend to straighten arch webs, thus creating high bending stresses.



H1 = AVERAGE HEIGHT OF SATURATION LINE

H2 = HEIGHT OF SATURATION LINE AT INBOARD FACE

FIG. 4-16a PILING SEATED ON ROCK, NO OVERBURDEN OR BERM, p max @ H/4

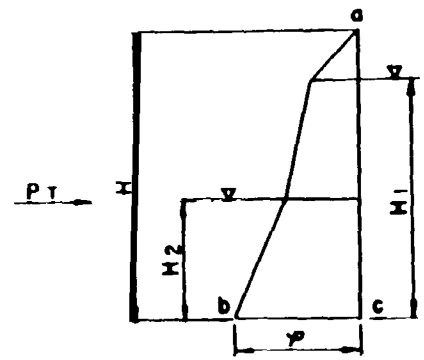
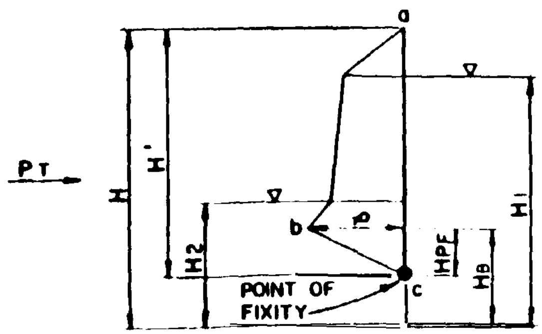


FIG 4-16c PILING NOT SEATED IN ROCK, p max @ BASE OF CELL. PILING NOT FULLY RESTRAINED BY BERM OR OVERBURDEN, p max @ TOP OF BERM OR OVERBURDEN



HB = HEIGHT OF BERM OR OVERBURDEN

HPF = DISTANCE FROM TOP OF BERM OR OVERBURDEN TO POINT OF FIXITY

H' = HEIGHT OF CELL ABOVE POINT OF FIXITY

FIG 4-16b PILING FULLY RESTRAINED BY EXTERNAL PASSIVE AND HYDROSTATIC FORCES, p max @ H'/4 OR TOP OF BERM OR OVERBURDEN

Figure 4-16. Resultant interlock pressure and point of maximum horizontal pressure

(7) Used piling is often utilized with little regard to the manufacturer. Because of small differences in interlock configuration and dimensional tolerances, sheets from different manufacturers may not be compatible and may not develop the assumed interlock strength. Splices have been made without considering the dimensions of the sheets joined. Splicing two sheets that do not have exactly the same width can cause a stress concentration in the narrower sheet. Where previously used piling is employed, care should be taken to ensure that the sheets are gaged and will interlock and that the sheets are compatible for splicing.

c. Shear Failure Within the Cell (Resistance to Tilting). Tilting of cofferdam cells is resisted by both the vertical and horizontal shear resistance of the soil in the cell, to which the frictional resistance of the steel sheet piling is added. Vertical shear resistance is determined by the theory developed by Terzaghi (item 81). The horizontal shear resistance is determined by the theory proposed by Cummings (item 19). Both of these methods of analysis should be used independently to determine the adequacy of the cell to resist tilting. Additionally, tilting resistance of cells founded in overburden should be investigated by the theory proposed by Schroeder and Maitland (item 66).

(1) Vertical Shear Resistance. Excessive shear on a vertical plane through the center line of the cell is a possible mode of failure by tilting. For stability, the shearing resistance along this plane, together with the frictional resistance in the interlocks, must be equal to or greater than the shear due to the overturning forces. The frictional resistance in the interlocks must be included since shear failure cannot occur without simultaneous slippage in the interlocks. Figure 4-17a shows the assumed stress distribution on the base due to the net overturning moment. The total shearing force on the neutral plane at the center line of the cell is equal to the area of the triangle. Therefore

$$Q = \left(\frac{1}{2}\right)\left(\frac{B}{2}\right)\left(\frac{6M}{B^2}\right) = \frac{3M}{2B}$$

where

Q = total shearing force

M = net overturning moment

To prevent rupture, the shear resistance on the neutral plane must be equal to the shearing force Q on this plane. The shear resistance on the neutral plane is due to the lateral pressure of the cell fill and is equal to this pressure times the coefficient of internal friction of the cell fill. Thus, as illustrated in Figure 4-17b

$$Ps = \frac{1}{2}\gamma K(H - H_1)^2 + \gamma K(H - H_1)\,H_1 + \frac{1}{2}\gamma K H_1^2$$

where

Ps = total lateral pressure, per unit length of cofferdam, due to cell fill

= unit weight of cell fill above saturation line

= submerged unit weight of cell fill

$K = \frac{\cos^2 \phi}{2 - \cos^2 \phi}$, empirical coefficient of earth pressure as suggested by Kryine

ϕ = angle of internal friction of cell fill

The total center-line shear resistance per unit length of cofferdam is

$$Ss = Ps \tan \phi$$

where

Ss = total vertical shear resistance

$\tan \phi$ = coefficient of internal friction of cell fill

The frictional resistance in the sheet pile interlock is equal to the interlock tension times the coefficient of friction of steel on steel. The resistance against slippage per unit length is therefore

$$S_F = fP_T$$

where

S_F = frictional resistance against slippage

f = coefficient of friction of steel on steel at the interlock = 0.3

P_T = resultant interlock pressure (area abc on Figure 4-16)

The total shearing resistance S_T along the center line of the cell is then

$$S_T = Ss + S_F = Ps \tan \phi + fP_T$$

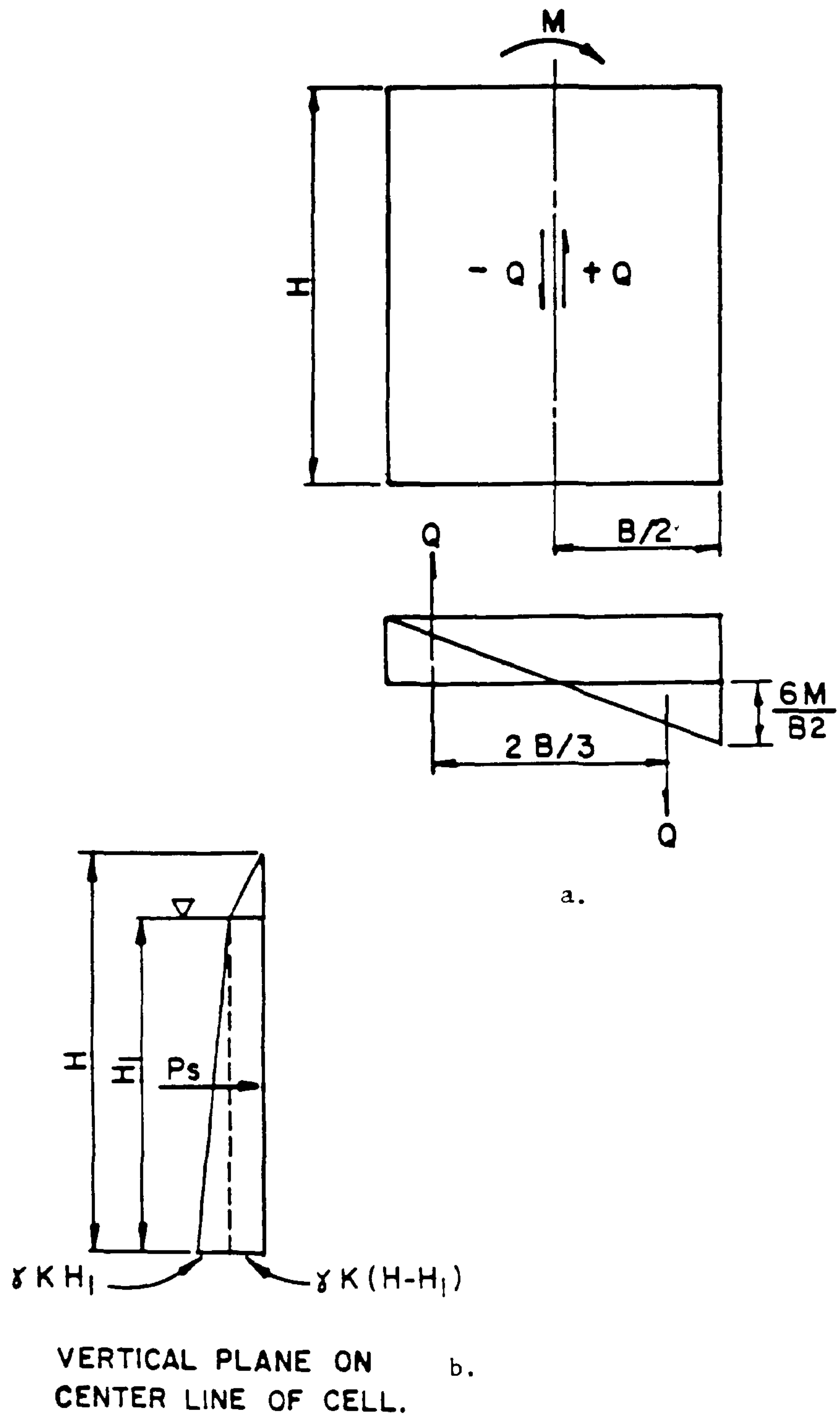


Figure 4-17. Vertical shear resistance,
Terzaghi method

and the FS against tilting by vertical shear is thus

$$= \frac{S_T}{Q} = \frac{(Ps \tan \phi + fP_T)2B}{3M}$$

The foregoing is applicable to cells founded on rock, sand, or stiff clay. The determination of P_T is dependent upon whether the piling is seated on rock, the presence of a berm or overburden, and the degree of restraint provided thereby, as discussed previously. In the case of cells on soft to medium clay, a relatively small overturning moment will produce an unequal distribution of pressure on the base of the fill in the cell causing it to tilt. The stability of the cell is virtually independent of the strength of the cell fill since the shear resistance through vertical sections offered by the cell fill cannot be mobilized without overstressing the interlocks. Therefore, for cells on compressible soils, the shear resistance of the fill in the cells is neglected, and the factor of safety against a vertical shear failure is based on the moment resistance mobilized by interlock friction as follows:

$$FS = \frac{PRf\left(\frac{B}{L}\right)\left(\frac{L + 0.25B}{L + 0.50B}\right)}{M}$$

where

P = pressure difference on the inboard sheeting

R = radius

f = coefficient of interlock friction

B and L = as shown in Figure 4-1

M = net overturning moment

(2) Horizontal Shear Resistance. The stability of a cell against failure by tilting is also dependent on the horizontal shear resistance of the cell fill and on the resisting moment due to the frictional resistance of the pile interlock. This theory, as proposed by Cummings (item 19), is based on the premise that the cell fill will resist lateral distortion of the cell through the buildup of soil resistance to sliding on horizontal planes. This resistance will be developed in a triangle forming an angle ϕ to the horizontal as shown in Figure 4-18a. The triangle of soil will be in a passive pressure state and will be surcharged by the overlying fill. The magnitude of the resisting force F is

$$F = \gamma HB \tan \phi$$

where

$H = a + c$

$B = c/\tan \phi$

therefore

$$F = ac\gamma + c^2\gamma$$

The lateral force F is represented graphically by Figure 4-18b, the area of this diagram being equal to F . The total moment of resistance M_r about the base of the cell is

$$M_r = F_1\left(\frac{c}{2}\right) + F_2\left(\frac{c}{3}\right)$$

where

$F_1 = ac\gamma$

$F_2 = c^2\gamma$

therefore

$$M_r = \frac{ac^2\gamma}{2} + \frac{c^3\gamma}{3}$$

Interlock friction also provides shear resistance equal to the maximum interlock tension times the coefficient of interlock friction, with the maximum interlock tension being determined in accordance with the criteria set forth in paragraph 4-14a. Thus, the resisting moment M_f against tilting due to interlock tension is

$$M_f = P_T fB$$

where

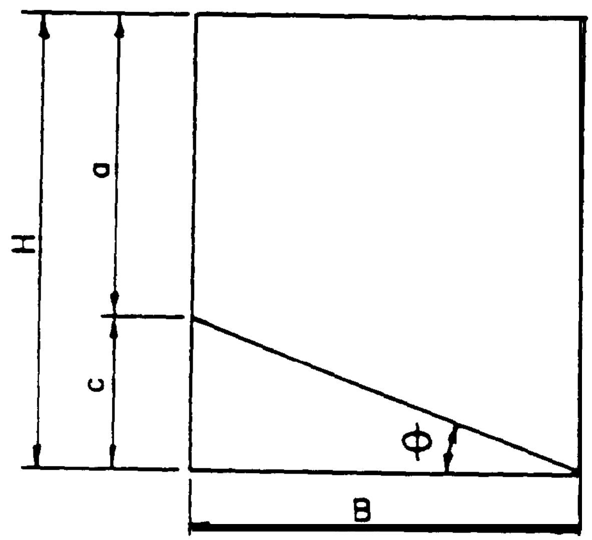
P_T = area abc as shown in Figure 4-16

B and f = as previously defined

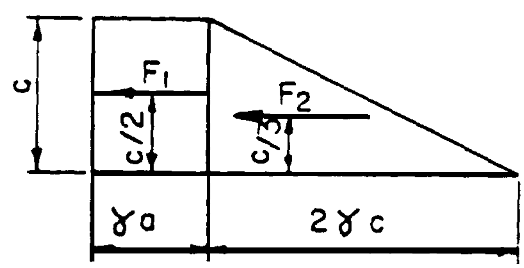
The FS against tilting due to horizontal shear is defined as

$$FS = \frac{M_r + M_f}{M_o}$$

where M_o = driving moment. Excessive tilting results from the use of weak cell fill; therefore, the fill should be well graded and free draining to the maximum extent possible. Further, since the shear resistance of the cell is



a.



b.

Figure 4-18. Horizontal shear resistance, Cummings method

derived from the material in the lower portion of the cell, it may be necessary to excavate any weak material encountered in the overburden. Should the shear resistance of the cell fill material be inadequate to withstand the external forces, consideration should be given to the use of a berm to assist in stabilization of the cell. If a berm is used, the resisting moment due to the effective passive pressure of the berm should be included. Thus, the FS against tilting due to horizontal shear is

$$FS = \frac{M_r + M_f + P_R(H_B/3)}{M_o}$$

All variables are as previously defined.

Ca = adhesion

perimeter = interior and exterior surfaces of a l-foot-wide strip, i.e., $1 \times 2 = 2$ feet

D = embedded length

Q_u granular = $(1/2\ Ka\ \gamma e\ D^2 \tan \delta)$ (perimeter)

Ka = coefficient of active earth pressure by Coulomb

γe = effective unit weight of underlying soil

$\tan \delta$ = coefficient of friction for steel against underlying soil. See Table 4-3 for recommended values.

Q_p = average pile reaction due to overturning moment on outboard piling = $\dfrac{P_w H_w + P_a H_s - P_R H_B}{3B(1 + B/4L)}$, where all variables are as shown in Figure 4-5.

Table 4-3

Wall Friction

Steel Sheet Piles Against the Following Soils	$\tan \delta$
Clean gravel, gravel-sand mixtures, well-graded rock fill with spalls	0.40
Clean sand, silty sand-gravel mixture, single size hard rock	0.30
Silty sand, gravel or sand mixed with silt or clay	0.25
Fine sandy silt, nonplastic silt	0.20

e. Penetration of Inboard Sheets. The penetration of the sheet piles on the inboard side must be sufficient to prevent further penetration. The FS against sheet pile penetration is defined as the ratio of the shear resistance on both sides of the embedded portion of the piles on the unloaded side to the internal downward shear force on the unloaded side as follows:

$$FS = \frac{F_1}{M}(D)$$

where

$F_1 = P_T \tan \delta$

p_T = area abc as shown in Figure 4-16

$\tan \delta$ = coefficient of friction between steel sheet piling and cell fill

M = net overturning moment

D = embedded length

Section IV. Design Criteria

4-15. Factors of Safety. The required FS for the various potential failure modes described in paragraph 4-4 are listed in Table 4-4. As previously stated in Chapter 1 cofferdams are not classified as temporary structures, nor are the loads imposed upon them generally considered temporary as far as FS's are concerned. However, some loading conditions can be classed as temporary where failure would not result in loss of life, severe property damage, or loss of the navigation pool, e.g., initial dewatering of a cofferdam which does not maintain a navigation pool.

4-16. Steel Sheet Piling Specifications. Steel for sheet piling should conform to the requirements of the following American Society for Testing and Materials (ASTM) standards (item 4):

A328 Steel Sheet Piling

A572 High-Strength Low-Alloy Columbium Vanadium Steels of Structural Quality

A690 High-Strength Low-Alloy Steel H-Piles and Sheet Piling for Use in Marine Environments

A328 is the basic sheet piling specification and is satisfactory for most installations. A572 specifies high-strength sheet piling and is applicable for use in large diameter (>70 feet) cells where high interlock strength is required. A690 steel sheet piling provides greater corrosion resistance than other steels and should be considered for use in permanent structures in corrosive environments. The mechanical properties of the steel sheet pile grades are shown in Table 4-5. Cold-formed steel sheet piling is also available. Presently, there is no ASTM specification covering this piling. Although this piling has limited applicability, it may be used subject to the approval of Headquarters, US Army Corps of Engineers (CEEC-ED). An extruded

Table 4-4

Design Criteria--Factors of Safety

Failure Mode	Required Factor of Safety — Loading Condition: Normal	Temporary	Seismic
Sliding[1]	1.5	1.5	1.3
Overturning (gravity block)[1,2]	Inside Kern	Inside Kern	Inside Base
Rotation (Hansen)[2]	1.5	1.25	1.1
Deep seated sliding	1.5	1.5	1.3
Bearing capacity			
Sand	2.0	2.0	1.3
Clay	3.0	3.0	1.5
Seepage control			
Interlock tension[3]	2.0	1.5	1.3
Vertical shear resistance (Terzaghi)	1.5	1.25	1.1
Horizontal shear resistance (Cummings)	1.5	1.25	1.1
Vertical shear resistance (Schroeder-Maitland)[2]	1.5	1.25	1.1
Pullout of outboard sheets[2]	1.5	1.25	1.1
Penetration of inboard[2] sheets	1.5	1.25	1.1

Notes

1. These FS's/criteria are for cofferdams only. Refer to the appropriate engineer manual for the required FS for other installations or applications.

2. Design should not be based on these modes of failure, but rather these analyses should be employed as sensitivity checks only.

3. The FS against interlock tension failure should be applied to the interlock strength value guaranteed by the manufacturer for the particular grade of steel. The guaranteed value for used piling should be reduced as necessary depending upon the condition of the piling.

Table 4-5

Mechanical Properties

ASTM Grade	Minimum Yield Point, psi	Minimum Tensile Strength, psi	Interlock Strength, p1i.[1]
A328	38,500	70,000	16,000
A572(Gr. 50)	50,000	65,000	28,000
A690	50,000	70,000	28,000

Note

1. As guaranteed by the manufacturer.

wye, using A572, Grade 50 steel, is available on a limited basis. These wyes have a small cross section and are extremely flexible, thus creating handling and driving difficulties. As a result of this characteristic, together with their limited availability, the use of extruded wyes is not recommended.

4-17. Corrosion Mitigation. Permanent sheet pile structures located in polluted, brackish, or salt water should be protected against corrosion. A690 steel sheet piling, which offers greater corrosion resistance than A328 piling, should be considered for corrosive environments. A328 steel sheet piling with a protective coating in the splash zone, such as a coal-tar epoxy, should also be considered. For maximum protection, coatings can be applied to A690 piling.

Section V. Finite Element Method (FEM) for Analysis and Design

4-18. Background. The application of FEM analysis to date has been to develop its state of the art to the point where it can be used to refine existing design techniques and to analyze potential failure modes which cannot be checked by other methods. All studies so far have been made by researchers or engineers who are extremely familiar with the FEM techniques using specialized FEM programs for soil and structure modeling. The FEM analysis does not yet lend itself to application by typical design engineers working with currently available general-use programs. Due to FEM techniques currently being used for research applications, the information provided by this section will be limited to a review of available literature and methods used for analysis. Relatively little has been published concerning finite element analyses of cellular cofferdam structures. Kittisatra (item 42) was one of the first to apply FEM to cellular cofferdams by using a linear elastic axisymmetric model. Clough and Hansen (item 18) were the first to utilize FEM soil-structure interaction techniques in the analyses of cellular cofferdams. They developed a vertical slice model which was used to analyze the US Army Corps of Engineers Willow Island Cofferdam. Later, Dr. Clough used this model along with two others, axisymmetric and horizontal slice models, to analyze the US Army