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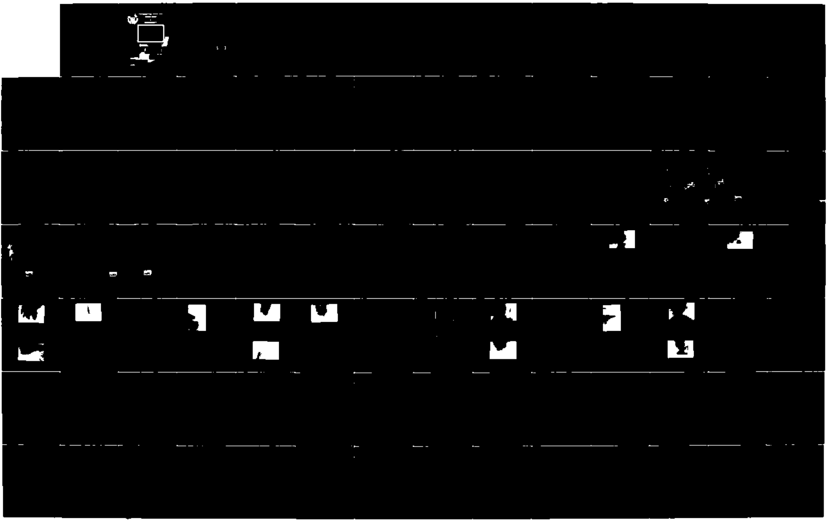

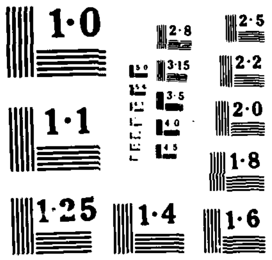
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UNDERWATER FACILITIES INSPECTIONS & ASSESSMENTS

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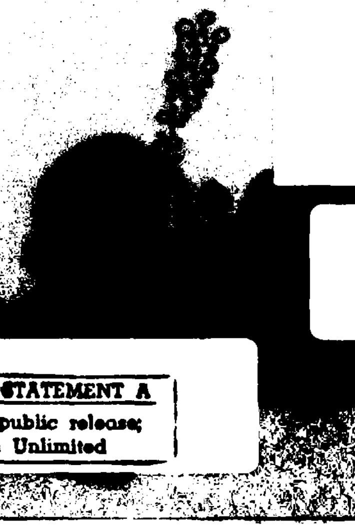
CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON, D.C. [illegible]



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UNDERWATER FACILITIES INSPECTIONS AND ASSESSMENTS AT

PHILADELPHIA NAVAL SHIPYARD

PHILADELPHIA, PA

VOLUME I

FILE COPY

FPO-1-83-(48) OCTOBER 1983

PERFORMED FOR: OCEAN ENGINEERING AND CONSTRUCTION PROJECT OFFICE
CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON, D.C. 20374

UNDER: CONTRACT N62477-81-C-0448
TASK 7

BY: CHILDS ENGINEERING CORPORATION
MEDFIELD, MA

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The objective of the Underwater Facility Assessments conducted at the Philadelphia Naval Shipyard, Philadelphia, Pennsylvania is to provide a generalized structural condition report on waterfront facilities within the Activity. The inspected facilities covered in Volume I are the Eastern (Con't)

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BLOCK 19 (Con't)

Seawall, Pier7, Pier 1, the Bulkhead to Pier 2, Pier 2, Wharves 4-A and 4-B, and Pier 4. Each facility was inspected by a team of engineer/divers using visual/tactile, non-destructive and destructive techniques. Typical and critical elements were photo-documented.

Conditions found throughout the facilities inspected ranged from excellent to marginal. Generally, the conditions were found to be good.

The Eastern Seawall was found to be in stable condition. In the past, possibly just following construction, there was some movement of the wall. This condition has apparently stabilized. We recommend that no change in the present live-load capacity of 0-PSF be made. If Shipyard Personnel desire an upgrading of the capacity of the seawall, we would recommend the placement of riprap along the south face of the seawall. Also in the portion of the seawall which is constructed of stone, there is a problem of the mortar between the stones being eroded away. In order to keep the stones in place, the wall should be re-pointed.

Pier 7 is in poor condition due to the softness found in the timber members. Crushed pile caps and failed or sagging deck planks were found throughout the facility. At the south end of the pier, there are three areas where deck planking had failed and the fill material above the relieving platform had been washed out. This loss of fill material created a void just below the existing top deck, essentially leaving the pier pavement unsupported. We recommend no live-loading be imposed on Pier 7. Major repairs are required to return the existing structure of Pier 7 to acceptable capacity.

Pier 1 and the bulkhead to Pier 2 are in good condition. There are some damaged perimeter piles which should be repaired and some areas in which settlement behind the seawall has occurred. The seawall surrounding the perimeter of Pier 1 is in poor condition and should be repaired. Although these conditions should be addressed, live-loading on Pier 1 and the adjacent bulkhead should be maintained at current levels except at the location of settlement along the approach to Pier 2 where there should be no live-loading until repairs are made.

Pier 2 is in good condition. There are fifty-two (52) damaged piles which should be repaired. Rotation and translation are occurring at the south end of Pier 2 leaving a large number of batter piles non-bearing. If this lateral movement is not stopped, there will be a failure of the structure at the south end of the pier. The general condition of the timber found throughout the facility is good. We recommend that no live-loading be imposed south of Bent 163 (the southernmost forty (40) feet of the pier), otherwise loading should be maintained at current levels (600 psf).

Wharves 4-A and 4-B are in excellent condition. There is some spalling of Wharf 4-A's superstructure that should be repaired. There is a gap in the sheet pile along Wharf 4-B that should be patched. The live-loading on both wharves should be maintained at current levels (300 psf).

Pier 4 is in excellent condition. There are twelve (12) damaged piles which should be repaired. A more detailed examination of the pier's superstructure is recommended due to the tension cracks noted that are propagating along the crane rail beams. This condition should be inspected more closely to determine the rate of corrosion of the reinforcing steel. Live-loading on Pier 4 can be maintained at current levels (11200 psf).

PREFACE

This report, Underwater Facilities Inspections and Assessments at the Philadelphia Naval Shipyard, Philadelphia, Pennsylvania, is divided into three (3) volumes covering a description of each facility, the inspected condition, a structural assessment, and recommendations for repair.

Volume I includes the Eastern Seawall, Pier 7, Pier 1 and the Bulkhead between Pier 1 and Pier 2, Pier 2, Wharves 4A and 4B and Pier 4.

Volume II includes Pier 5, the Barge Basin and the Bulkhead to the east of Pier 6, Pier 6, Pier 6A and the Bulkhead to the west of Pier 6, the Drydock Wharf and Wharves K, J, I, H and G.

Volume III includes Wharf F and Pier F, Wharf E, Rowan Avenue Wharf, Second Street Wharf, Preble Avenue Wharf, Broad Street Wharf, Wharf L and Wharf N.

EXECUTIVE SUMMARY
(Volume I)

The objective of the Underwater Facility Assessments conducted at the Philadelphia Naval Shipyard, Philadelphia, Pennsylvania is to provide a generalized structural condition report on waterfront facilities within the Activity. The inspected facilities covered in Volume I are the Eastern Seawall, Pier 7, Pier 1, the Bulkhead to Pier 2, Pier 2, Wharves 4-A and 4-B, and Pier 4. Each facility was inspected by a team of engineer/divers using visual/tactile, non-destructive and destructive techniques. Typical and critical elements were photo-documented.

Conditions found throughout the facilities inspected ranged from excellent to marginal. Generally, the conditions were found to be good.

The Eastern Seawall was found to be in stable condition. In the past, possibly just following construction, there was some movement of the wall. This condition has apparently stabilized. We recommend that no change in the present live-load capacity of 0-PSF be made. If Shipyard Personnel desire an upgrading of the capacity of the seawall, we would recommend the placement of rip-rap along the south face of the seawall. Also in the portion of the seawall which is constructed of stone, there is a problem of the mortar between the stones being eroded away. In order to keep the stones in place, the wall should be re-pointed.

Pier 7 is in poor condition due to the softness found in the timber members. Crushed pile caps and failed or sagging deck planks were found throughout the facility. At the south end of the pier, there are three areas where deck planking had failed and the fill material above the relieving platform had been washed out. This loss of fill material created a void just below the existing top deck, essentially leaving the pier pavement unsupported. We recommend no live-loading be imposed on Pier 7. Major repairs are required to return the existing structure of Pier 7 to acceptable capacity.

Pier 1 and the bulkhead to Pier 2 are in good condition. There are some damaged perimeter piles which should be repaired and some areas in which settlement behind the seawall has occurred. The seawall surrounding the perimeter of Pier 1 is in poor condition and should be repaired. Although these conditions should be addressed, live-loading on Pier 1 and the adjacent bulkhead should be maintained at current levels except at the location of settlement along the approach to Pier 2 where there should be no live-loading until repairs are made.

Pier 2 is in good condition. There are fifty-two (52) damaged piles which should be repaired. Rotation and translation are occurring at the south end of Pier 2 leaving a large number of batter piles non-bearing. If this lateral movement is not stopped, there will be a failure of the structure at the south end of the pier. The general condition of the timber found throughout the facility is good. We recommend that no live-loading be imposed south of Bent 163 (the southernmost forty (40) feet of the pier), otherwise loading should be maintained at current levels (600 psf).

Wharves 4-A and 4-B are in excellent condition. There is some spalling of Wharf 4-A's superstructure that should be repaired. There is a gap in the sheet pile along Wharf 4-B that should be patched. The live-loading on both wharves should be maintained at current levels (300 psf).

Pier 4 is in excellent condition. There are twelve (12) damaged piles which should be repaired. A more detailed examination of the pier's superstructure is recommended due to the tension cracks noted that are propagating along the crane rail beams. This condition should be inspected more closely to determine the rate of corrosion of the reinforcing steel. Live-loading on Pier 4 can be maintained at current levels (1200 psf).

Refer to the following Executive Summary Table to review each Facility's type of construction and recommendations.

PHILADELPHIA NAVAL SHIPYARD

PHILADELPHIA, PENNSYLVANIA

EXECUTIVE SUMMARY TABLE

VOLUME I

FACILITY	YEAR BUILT	TOTAL NO. OF PILES [1]	SIZE (LxW-FT.)	STRUCTURES	RECOMME
EASTERN SEAWALL	1899-1944	1,692	6689' in length	Low deck relieving platform structure with concrete or stone seawall.	1. Point w missing 2. To upgr place r of the 3. Reinspe 6 years
PIER 7	Circa 1931	592	328'x60'	Pile-supported, low deck, earth fill relieving platform structure.	1. Rebuild structu 2. Immedia load ca
PIER 1 AND BULKHEAD BETWEEN PIERS 1 AND 2	Circa 1890-1904	60	Finger Pier: 320'x100' Pier Head: 150'x70'	The Finger Pier is a pile-supported low deck, earth fill relieving platform structure. The Pier Head is a wood crib structure.	1. Repair 2. Patch l 3. Repair wall on 4. Reinspe 6 years
PIER 2	Circa 1930	6,300	900'x80'	Pile-supported low deck, earth fill, relieving platform structure.	1. Repair 2. Refaste 3. No live end of 4. Install 5. Reinspe 6 years

NOTE:
1. Approximate number of piles accessed by divers.
2. Costs exclude mobilization/demobilization and are based on 1983 East Coast prices.

PHILADELPHIA NAVAL SHIPYARD

PHILADELPHIA, PENNSYLVANIA

EXECUTIVE SUMMARY TABLE

VOLUME I

	STRUCTURES	RECOMMENDATIONS	ESTIMATED COST $(THOUSANDS)[2]
gth	Low deck relieving platform structure with concrete or stone seawall.	1. Point where needed and replace missing stones in seawall.	32
		2. To upgrade surcharge capacity, place rip-rap to the south of the seawall.	370
		3. Reinspect after repairs and in 6 years thereafter.	--
	Pile-supported, low deck, earth fill relieving platform structure.	1. Rebuild the pier as a solid fill structure.	2,500
		2. Immediate reduction of the live-load capacity to 0-psf.	--
:	The Finger Pier is a pile-supported low deck, earth fill relieving platform structure. The Pier Head is a wood crib structure.	1. Repair damaged perimeter piles.	2.8
		2. Patch large gap in sheeting.	3
		3. Repair spalling on concrete seawall on Pier 1.	70
		4. Reinspect after repairs and in 6 years thereafter.	--
	Pile-supported low deck, earth fill, relieving platform structure.	1. Repair damaged piles.	23
		2. Refasten tie-rods.	3
		3. No live-loading at the southern end of the pier.	--
		4. Install tie-back system.	40
		5. Reinspect after repairs and in 6 years thereafter.	--

PHILADELPHIA NAVAL SHIPYARD

EXECTUIVE SUMMARY TABLE, CONT'D.

VOLUME I

FACILITY	YEAR BUILT	TOTAL NO. OF PILES [1]	SIZE (LxW-FT.)	STRUCTURES	RECOM
WHARVES 4-A AND 4-B	1893-1969	200	896' in length	4-A: Pile-supported high deck structure 4-B: Pile-supported, earth fill, low deck, relieving platform structure with sheet pile face	1. Repai 2. Repai pile 3. Reins in 6
PIER 4	1917	4,000	1134'x100'	Pile-supported high concrete deck structure	1. Repai 2. Remov causi 3. Perfo inspe super 4. Reins in 6

NOTE:
1. Approximate number of piles accessed by divers.
2. Costs exclude mobilization/demobilization and are based on 1983 East Coast prices.

PHILADELPHIA NAVAL SHIPYARD

EXECTUIVE SUMMARY TABLE, CONT'D.

VOLUME I

STRUCTURES	RECOMMENDATIONS	ESTIMATED COST $ THOUSANDS
4-A: Pile-supported high deck structure	1. Repair spalling on Wharf 4-A.	3
4-B: Pile-supported, earth fill, low deck, relieving platform structure with sheet pile face	2. Repair large gap in the sheet pile on Wharf 4-B.	3
	3. Reinspect after repairs and in 6 years thereafter.	--
Pile-supported high concrete deck structure	1. Repair 12 damaged piles.	[illegible]
	2. Remove floating debris causing abrasion.	3
	3. Perform a more detailed inspection of the concrete superstructure.	--
	4. Reinspect after repairs and in 6 years thereafter.	--

EXECUTIVE SUMMARY
(Volume II)

The inspected facilities covered in Volume II are Pier 5, the Barge Basin and associated Bulkhead, Pier 6, Pier 6-A and associated Bulkhead, the Drydock Wharves and Wharves K, J, I, H and G.

Pier 5 has recently been rebuilt. There is a new concrete superstructure and some new piles have been driven. The pile foundation was found to be in excellent condition. Generally the concrete superstructure is in excellent condition, however, upon cursory inspection it is revealed that there is some deterioration of the concrete truss cantilever on the east and west sides of the pier. Spalling of the concrete is occurring at or near the elevation of mean low water exposing the reinforcing steel to the marine environment. If the reinforcing steel is not protected from the marine environment, corrosion will occur and eventually cause a reduction in the live-load capacity of the truss. Providing proper protection for the steel is recommended. Tension cracks were observed on the concrete crane rail beam. The observed vertical cracking is a common condition in concrete beams. However, when reinforcing steel is exposed to the marine environment it will corrode. This deterioration should be monitored. Live-loading on Pier 5 can be maintained at current levels (600 psf).

The pile foundation of the Barge Basin is in good condition. Anomalies are limited to damage caused by berthing forces at the perimeter of the basin. The bulkhead between the Barge Basin and Pier 6 has been partially reconstructed. The new structure, supported by steel H-piles, is in excellent condition. The older timber pile-supported structure is in good condition. The timber piles of a portion of the wharf near the Barge Basin are loaded eccentrically, a condition which is marginal and should be corrected. The seawall directly to the west of the Barge Basin is in poor condition and should be repaired. Live-loading on the Barge Basin and associated wharf can be maintained at current levels (200 psf).

Pier 6 is in good condition. There are 65 piles which have been damaged due to berthing forces generally occurring at the perimeter of the pier; these piles should be repaired. Otherwise, the timber pile foundation is in excellent condition. Live-loading on Pier 6 can be maintained at current levels (600 psf).

Pier 6-A and the associated Wharf to Pier 6 are in fair condition. There are 36 piles which have been damaged due to berthing forces, these piles should be repaired. Along the wharf from Pier 6-A to Pier 6 timber softness was detected to a depth of 4" in the structural timber. Due to the soft timber the live-load capacity of the structure should be reduced to 200 psf. The concrete seawall is beginning to deteriorate and should be repaired. At the southeast corner of Pier 6-A there are many broken piles in a concentrated area. As a result, the relieving platform is unsupported and in this area loading should be restricted to 50 psf until repairs have been made.

The Drydock Wharves have 242 piles with anomalies rendering them ineffective. These piles should be repaired. The general condition of the structural timber is good. Overall, live-loading can be maintained at current levels on the Drydock Wharves, however, a localized concentration of damaged piles occurs on Section A near Drydock No. 4 and until repairs are made, loading should be restricted to 100 psf in this area. The steel sheet pile along the inshore perimeter of Section A is showing signs of accelerated deterioration. Downgrading of the sheet piles' capacity to resist lateral earth pressure is not recommended at this time. Cathodic protection systems should be analyzed to determine possible sources of deterioration and protection alternatives. Possibly galvanic anodes could be installed to inhibit further deterioration of the sheet pile wall.

The structural timber observed on Wharves K, J, I, H and G is in excellent condition. In two locations (Wharves K and J), there has been a localized failure of the relieving platform due to overloading on the top deck. In these two locations loading should be restricted until repairs are made. Portions of this wharf structure are translating in the outshore direction due to excessive lateral earth pressure exerted on the sheet pile wall. This creates eccentric loading on the vertical piles which, in turn, reduces their column capacity. At this time the combined stress occurring in the vertical piles is not critical, however, if translation is allowed to continue, overstressing will occur. The sections of wharf which have been observed to be translating should be tied back and anchored. Until the wharf is stabilized, the area from the face of the wharf inshore 70' should be restricted to 300 psf live- loading. This will limit excessive lateral earth pressure due to live-loading from acting on the sheet pile wall.

Refer to the following Executive Summary Table to review each facility's construction, recommendations and repair cost estimates.

PHILADELPHIA NAVAL SHIPYARD

PHILADELPHIA, PENNSYLVANIA

EXECUTIVE SUMMARY TABLE

VOLUME II

FACILITY	YEAR BUILT	TOTAL NO. OF PILES	SIZE (LxW-FT.)	STRUCTURES	RECOMMENDATIONS
Pier 5	1912-1979	Approx. 3,000	790'x110'	Pile-supported high concrete deck structure	1) Replace broken 2) Repair non-bea piles. 3) Monitor tensio 4) Repair spallin and utility tu 5) Re-inspect aft
Barge Basin & Bulkhead to Pier 6	Barge Basin: Circa 1939 Bulkhead to Pier 6: 1903-1979	Barge Basin: approx. 850 Bulkhead: approx. 740	Barge Basin: 163'x60' Bulkhead: 747' in length	The Barge Basin is a timber pile-supported, low deck, earth fill, relieving platform structure. The original portion of the bulkhead is a timber pile-supported, low deck, earth fill relieving platform structure. The rebuilt section consists of steel H-piles arranged in bents with a low steel deck.	1) Replace broken 2) Repair non-bea and wild piles 3) Repair eccentr through 8 of b 4) Repair spallin 5) Re-inspect aft
Pier 6	Circa 1940	Approx. 8,500	970'x100'	Timber pile-supported, low deck, earth fill, relieving platform structure	1) Replace broken 2) Post and brace repair non-bea and displaced, 3) Re-inspect aft
Pier 6A & Bulkhead East to Pier 6	Circa 1903	Pier 6A: approx. 1,100 Bulkhead: approx. 84	Pier 6A: 235'x70' Bulkhead: 142.7' in length	Timber pile-supported, low deck, earth fill, relieving platform structure.	1) Replace or rep 2) Repair split, piles. 3) Repair spallin 4) Immediate rest 10' radius of made. 5) Limit live-loa 6) Re-inspect aft

*Cost estimates are based on 1983 U.S. East Coast prices. Mobilization/demobilization costs have been omitted.

PHILADELPHIA NAVAL SHIPYARD
PHILADELPHIA, PENNSYLVANIA

EXECUTIVE SUMMARY TABLE
VOLUME II

UCTURES	RECOMMENDATIONS	ESTIMATED COST* (THOUSANDS)
e-supported high concrete k structure	1) Replace broken pile.	1.5
	2) Repair non-bearing, partially bearing and split piles.	2.4
	3) Monitor tension cracks in crane rail beam on a yearly basis.	--
	4) Repair spalling on concrete truss cantilever and utility tunnel.	19.8
	5) Re-inspect after repairs and in 6 years thereafter.	--
e Barge Basin is a timber le-supported, low deck, earth ll, relieving platform structure. e original portion of the bulk- ad is a timber pile-supported, deck, earth fill relieving tform structure. e rebuilt section consists of el H-piles arranged in bents h a low steel deck.	1) Replace broken piles.	5.0
	2) Repair non-bearing, partially bearing, split and wild piles.	8.4
	3) Repair eccentric piles associated with Bents 4 through 8 of bulkhead. Re-inspect each year.	10.0
	4) Repair spalling on concrete seawall.	3.8
	5) Re-inspect after repairs and in 6 years thereafter.	--
nber pile-supported, low deck, th fill, relieving platform ucture	1) Replace broken perimeter piles.	15.0
	2) Post and brace broken interior piles, repair non-bearing, partially bearing, split and displaced, and wild piles.	20.0
	3) Re-inspect after repairs and in 3 years thereafter.	--
nber pile-supported, low deck, th fill, relieving platform ucture.	1) Replace or repair broken piles.	36
	2) Repair split, wild, and partially bearing piles.	1.6
	3) Repair spalling on concrete seawall.	7.1
	4) Immediate restriction of live-loading within 10' radius of broken piles until repairs are made.	--
	5) Limit live-load capacity to 200 psf.	--
	6) Re-inspect after repairs and in 2 years thereafter.	--

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PHILADELPHIA NAVAL SHIPYARD

EXECUTIVE SUMMARY TABLE, CONT'D.

VOLUME II

FACILITY	YEAR BUILT	TOTAL NO. OF PILES	SIZE (LxW-FT.)	STRUCTURES	RECOMMENDATIONS
Drydock Wharf	Circa 1941	Section "A" approx. 4,300 Section "B" approx. 2,800 Section "C" approx. 1,500	Section "A" 395'x600'x224' Section "B" 224'x280'x216' Section "C" 214'x184'	Timber pile-supported, low deck, earth fill, relieving platform structure	1) Replace or r 2) Repair split and partiall 3) Repair local 4) Repair damag 5) Investigate 6) Downgrade li north of Ben until repair 7) Enforce dred 8) Re-inspect a
Wharves K, J, I, H, and G	Circa 1943	12,060	3,315' in length	Timber pile-supported, low deck, earth fill, relieving platform structure.	1) Replace brok 2) Repair spli 3) Install tie- Wharves K, J capacity to wharves insh completed. 4) Repair damag capacity to repairs are 5) Sections of should be in repairs and 6) Shim non-bea

*Cost estimates are based on 1983 U.S. East Coast prices. Mobilization/demobilization costs have been omitted.

PHILADELPHIA NAVAL SHIPYARD

EXECUTIVE SUMMARY TABLE, CONT'D.

VOLUME II

ES	RECOMMENDATIONS	ESTIMATED COST (THOUSANDS)
ile-supported, low deck, ill, relieving platform re	1) Replace or repair broken piles.	49
	2) Repair split and displaced, wild, non-bearing, and partially bearing piles.	77.2
	3) Repair local pile cap-deck failure.	1.1
	4) Repair damaged pile caps.	12
	5) Investigate cathodic protection system.	--
	6) Downgrade live-load capacity of Section "A" north of Bent 220 and inshore 20' to 100 psf until repairs are completed.	--
	7) Enforce dredge limits.	--
	8) Re-inspect after repairs and in 6 years thereafter.	--
ile-supported, low deck, ill, relieving platform re.	1) Replace broken piles.	7.0
	2) Repair split piles, and wild piles.	14.4
	3) Install tie-back system along sections of Wharves K, J, and I. Restrict live-load capacity to 300 psf from the face of the wharves inshore 70' until repairs are completed.	605
	4) Repair damaged pile caps. Restrict live-load capacity to 0 psf in a radius of 10' until repairs are completed.	10
	5) Sections of wharf requiring tie-back system should be inspected yearly. Re-inspect after repairs and in 6 years thereafter.	--
	6) Shim non-bearing piles	26

EXECUTIVE SUMMARY
(Volume III)

The inspected facilities covered in Volume III are Wharf F and Pier F, Wharf E, Rowan Avenue Wharf, Second Street Wharf, Preble Avenue Wharf, Broad Street Wharf, Wharf L and Wharf N.

Wharf F and Pier F are both in good condition. There are 124 bearing piles which were found to be deficient. Generally the cause of these deficiencies can be attributed to berthing and mooring forces transmitted to the pile through the use of camels. These piles should be repaired. A slight crushing of the pile caps about the perimeter of the wharf and pier was noted. This condition is assumed to be caused by a weakening of the outer timber fibers (softness) and not overloading. At this time the softness is not a threat to the integrity of the structure. Live-loading on Wharf F and Pier F can be maintained at current levels (750 psf).

A partial collapse of the relieving platform of Wharf E has occurred. This collapse is a result of many forces acting in combination against the weakened (due to softness and eccentric loading) structural timber. Apparently, the imposition of a live-load on the top deck of the structure was the "straw that broke the camel's back". Live-loading on the structure from Bents 1 through 58 (newer construction) should remain at 200 psf. On the older structure, Bents 58 through 148, live-loading should be limited to 100 psf due to the timber softness found. In considering reconstruction of Wharf E, the extension of the Rowan Avenue steel sheet pile bulkhead would be the logical path to follow, particularly to increase the live-load capacity of the structure.

The Rowan Avenue Bulkhead has recently been rehabilitated with the construction of a new steel sheet pile bulkhead and soil anchor system. The steel sheetpiles are in excellent condition with no significant metal loss. There have been problems involving sinkholes behind the steel sheet pile. The sinkholes are caused by the repositioning or settlement of the fill material. The steel sheet pile wall has two locations where fill could be escaping. It should be noted that in review of the construction drawings and in discussions with shipyard personnel, it was determined that the timber deck had not been removed. The timber deck then prevents any loss of fill from on top of the deck through the steel sheet pile, unless the timber decking has failed or has been altered. To determine the cause of the sinkholes along Rowan Avenue, further investigation into the condition of the timber decking, the compactness of the sand fill below the timber decking and the presence of void space between the sand fill and timber deck will have to be made. Live-load levels on the Rowan Avenue Wharf can be maintained at current levels (600 psf).

The Second Street Wharf has 77 piles which were noted to be defective. Along with the piles are portions of the seawall and longitudinal pile cap which are also damaged. The majority of this damage is a result of berthing and fendering forces which are allowed to effect the bearing piles only through the lack of an adequate fender system. These piles should be repaired. Until repairs are made live-loading directly above broken piles should be restricted to 0 psf. Due to the good condition of the timber and upon completion of the repairs live-load levels on the Second Street Wharf can be maintained at current levels (200 psf).

The Preble Avenue Wharf has 116 piles which exhibit damage serious enough to be repaired. Along with the piles are portions of the seawall and longitudinal pile cap which are also damaged. The majority of this damage is a result of excessive berthing and mooring forces. Generally these berthing/mooring forces are transmitted to the bearing piles through the use of camels. In all locations of damage the fender system has also been rendered non-functional. Until repairs are made live-loading directly above broken piles should be restricted to 0 psf. Due to the sound condition of the timber and upon completion of the recommended repairs, live-load levels on the Preble Avenue Wharf can be maintained at current levels (200 psf).

The Broad Street Wharf has 56 piles which are damaged and in need of repair due to berthing and mooring forces. Soft timber is noted throughout the structure on the Broad Street Wharf. This condition along with the pile spacing of 6 feet on center (typical pile spacing of the relieving platforms throughout the PNSY is 3' to 4' o.c.) greatly reduces the capacity of the timber pile caps. The observed result of these factors is deflection of the pile caps (approximately 6") at the timber sheet pile wall. This is an indication that the ultimate stresses within the timber are being approached. Further investigation of the material characteristics of the structure should be made. The present live-load capacity of 100 psf on the Broad Street Wharf should be reduced to 50 psf until further investigation can determine the true capacity of the structure.

Wharf L has 117 piles which exhibit damage as a result of excessive berthing and mooring forces. These forces are generated by ships and are transferred to the bearing piles through camels. The fender system adjacent to locations where piles are damaged is generally non-functional. Until repairs are made live-loading directly above the broken piles should be restricted to 0 psf. Upon completion of the recommended repairs, live-loading on Wharf L can be maintained at current levels (200 psf).

Wharf N has 160 piles which are in need of repair. Generally these piles have been damaged by excessive berthing and mooring forces. These forces can only effect the structural piles when there has been a failure of the fender system. These forces are transmitted to the structural piles through camels. Until repairs are made, live-loading should be restricted to 0 psf directly above any broken pile. The steel sheet pile diaphram portion of the wharf is in excellent condition. There is very little loss of steel due to corrosion. Upon completion of the recommended repairs, live-loading on Wharf N can be maintained at current levels (200 psf).

Refer to the following Executive Summary Table to review each facility's construction, recommendations and cost repair estimates.

PHILADELPHIA NAVAL SHIPYARD
PHILADELPHIA, PENNSYLVANIA

EXECUTIVE SUMMARY TABLE

VOLUME III

FACILITY	YEAR BUILT	TOTAL NO. OF PILES	SIZE (LxW-FT.)	STRUCTURES	RECOMMENDATIONS
Wharf F & Pier F	Circa 1942	7,730	Wharf F 772'x40' Pier F 603'x79'	Timber pile-supported, low deck, earth filled, relieving platform structure	1. Replace or repair brok 2. Repair split and disp bearing and wild piles 3. Repair damaged pile ca 4. Re-inspect after repa
Wharf E	Bents 1-58 Circa 1942 Bents 59-148 Circa 1914-1915	N/A	730' in length	Timber pile-supported, low deck, earth filled, relieving platform structure	1. Replace or repair brok 2. Repair split and displ 3. Repair damaged pile ca 4. Limit live-load capac Bent 58-148 5. Enforce dredge limits 6. Consider steel sheet for collapsed portion 7. Re-inspect yearly
Rowan Ave. Bulkhead	Circa 1982	N/A	1,971' in length	Steel sheet pile wall and tie-back structure	1. Patch honeycombed port 2. Cut steel formwork flu 3. Repair separations in 4. Conduct 12 test boring 5. Investigate flume ass 6. Re-inspect after repai
Second St. Wharf	Circa 1902-1903	528	928' in length	Timber pile-supported, low deck, earth filled, relieving platform structure	1. Replace or repair brok 2. Repair longitudinal pi 3. Repair split and displ and partially bearing 4. Consider installing fe 5. Restrict live-loading until repairs are comp 6. Re-inspect in 2 years

*Cost estimates based on 1983 U.S. East Coast prices. Mobilization/demobilization costs have been omitted.

PHILADELPHIA NAVAL SHIPYARD
PHILADELPHIA, PENNSYLVANIA

EXECUTIVE SUMMARY TABLE

VOLUME III

		RECOMMENDATIONS	ESTIMATED COST * (THOUSANDS)
ow	1.	Replace or repair broken piles	7
eving	2.	Repair split and displaced, non-bearing, partially bearing and wild piles	58.3
	3.	Repair damaged pile caps	4.5
	4.	Re-inspect after repairs and in 6 years thereafter.	--
ow	1.	Replace or repair broken piles	6
eving	2.	Repair split and displaced piles	7.2
	3.	Repair damaged pile cap	0.5
	4.	Limit live-load capacity to 100 psf between Bent 58-148	--
	5.	Enforce dredge limits	--
	6.	Consider steel sheet pile and tie-back system for collapsed portion of wharf	--
	7.	Re-inspect yearly	--
	1.	Patch honeycombed portion of seawall	0.7
	2.	Cut steel formwork flush to concrete face	3
	3.	Repair separations in sheet pile wall	6
	4.	Conduct 12 test borings	4
	5.	Investigate flume associated with old Pier D	--
	6.	Re-inspect after repairs and in 6 years thereafter.	--
w	1.	Replace or repair broken piles	57
ving	2.	Repair longitudinal pile cap	9
	3.	Repair split and displaced, non-bearing, wild and partially bearing piles	8
	4.	Consider installing fender system	--
	5.	Restrict live-loading in areas of missing piles until repairs are completed	--
	6.	Re-inspect in 2 years	--

PHILADELPHIA NAVAL SHIPYARD EXECUTIVE SUMMARY TABL

VOLUME III

FACILITY	YEAR BUILT	TOTAL NO. OF PILES	SIZE (LxW-FT.)	STRUCTURES	RECOMMENDATIONS
Preble Ave. Wharf	Circa 1900	370	847' in length	Timber pile-supported, low deck, earth filled, relieving platform structure	1. Replace or repair 2. Repair longitudina 3. Repair split and d wild piles 4. Consider installin 5. Restrict live-load until repairs are 6. Re-inspect after r
Broad St. Wharf	Circa 1899	1,440	735' in length	Timber pile-supported, low deck, earth filled, relieving platform structure	1. Limit live-load ca and west lane of B 2. Consider long-term 3. Replace or repair 4. Repair split and d partially bearing 5. Monitor on 6-month
Wharf L	Circa 1900	400	930' in length	Timber pile-supported, low deck, earth filled, relieving platform structure	1. Replace or repair 2. Repair longitudin 3. Repair split and partially bearing 4. Consider installi 5. Restrict live-loa until repairs are 6. Re-inspect after r
Wharf N	Station 0+00 to 9+75, Circa 1903 Station 14+00 to 19+50, Circa 1941 Remaining Timber Circa 1943 Steel Sheet Piles Unknown	3,646	3,348' in length	Sta. 0+00 to 21+88, Timber pile-supported, low deck, earth filled, relieving platform structure Sta. 21+88 to 33+48, Steel sheet pile diaphrams	1. Replace or repair 2. Repair split and bearing and wild 3. Repair damaged pil 4. Monitor bulge at 5. Consider installi 6. Restrict live-loa until repairs are 7. Re-inspect timber Re-inspect remain

*Cost estimates based on 1983 U.S. East Coast prices. Mobilization/demobilization costs have been omitted.

ILADELPHIA NAVAL SHIPYARD EXECUTIVE SUMMARY TABLE - Cont'd.

VOLUME III

ES	RECOMMENDATIONS	ESTIMATED COST* (THOUSANDS)
le-supported, low rth filled, relieving structure	1. Replace or repair broken piles	87
	2. Repair longitudinal pile cap	12.6
	3. Repair split and displaced, non-bearing and wild piles	11.6
	4. Consider installing fender system	--
	5. Restrict live-loading in areas of missing piles until repairs are completed	--
	6. Re-inspect after repairs and in 2 years thereafter.	--
ile-supported, low rth filled, relieving structure	1. Limit live-load capacity to 50 psf on sidewalk and west lane of Broad Street	--
	2. Consider long-term reconditioning or reconstruction.	--
	3. Replace or repair broken piles	13
	4. Repair split and displaced, wild, non-bearing and partially bearing piles	17.2
	5. Monitor on 6-month intervals	--
pile-supported, low arth filled, relieving structure	1. Replace or repair broken piles	99
	2. Repair longitudinal pile cap	15.9
	3. Repair split and displaced, non-bearing, wild, and partially bearing piles	7.2
	4. Consider installing fender system	--
	5. Restrict live-loading in areas of missing piles until repairs are completed.	--
	6. Re-inspect after repairs and in 2 years thereafter.	--
00 to 21+88, Timber pported, low deck, illed, relieving m structure	1. Replace or repair broken piles	92
	2. Repair split and displaced, non-bearing, partially bearing and wild piles	27.2
+88 to 33+48, Steel ile diaphrams	3. Repair damaged pile cap	0.5
	4. Monitor bulge at Sta. 21+88	--
	5. Consider installing fender system	--
	6. Restrict live-loading in areas of missing piles until repairs are completed	--
	7. Re-inspect timber portion of wharf in 2 years. Re-inspect remainder of wharf on a 6-year basis	--

TABLE OF CONTENTS

VOLUME I

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LIST OF PHOTOGRAPHS
VOLUME I

VOLUME I LIST OF PHOTOGRAPHS Cont'd.

VOLUME I LIST OF PHOTOGRAPHS Cont'd.

LIST OF FIGURES

SECTION 1.0 INTRODUCTION

This report is a product of the Underwater Inspection Program conducted by the Ocean Engineering and Construction Project Office (FPO-1), Chesapeake Division, Naval Facilities Engineering Command (NAVFACENGCOM) under NAVFAC's Specialized Inspection Program.

This program sponsors task-oriented engineering services for the inspection, analysis and design, and monitoring of repairs for the submerged portions of selected Naval Waterfront Facilities. All services required to produce this report were provided by Childs Engineering Corporation of Medfield, Massachusetts under Task No. 7, P-00006 (PNSY) of Contract N62477-81-C-0448.

1.1 REPORT CONTENT

This report contains a description of inspection procedures, the results of the inspection and analysis of the findings, accompanied by pertinent drawings and photographs. Specifically, the inspection results include a description of the location, existing facilities, its observed condition and a structural assessment of that condition. Recommendations for each facility, including cost estimates (based on present local prices) for any repair work, are also included. Structural assessment calculations and cost estimate breakdowns can be found in the Appendix.

SECTION 2.0 ACTIVITY DESCRIPTION

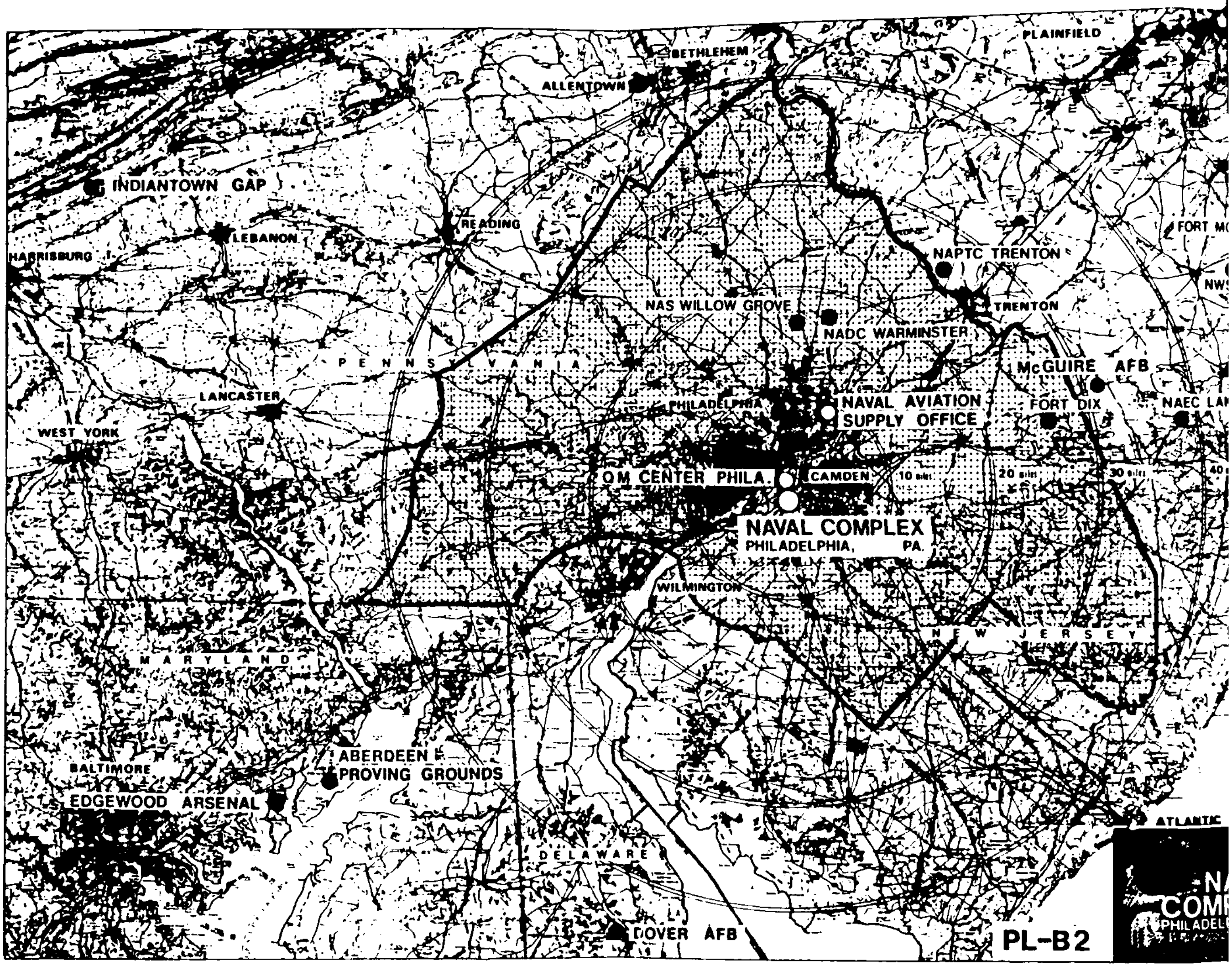
The purpose of this section is to provide a general description of the Philadelphia Naval Shipyard, Philadelphia, Pennsylvania. This section includes brief descriptions of the Naval Shipyard's location and existing facilities. The information is provided to aid in identification of the facility and to support all considerations necessary to accurately assess the condition of facilities inspected under this task.

2.1 LOCATION OF ACTIVITY

The Naval Shipyard, Philadelphia, PA is located about 123 miles northeast of Washington, DC and 83 miles southwest of New York City. The Shipyard is situated four miles south of the Philadelphia central business district at the confluence of the Delaware and Schuylkill rivers at 75° 10' 35.6" west longitude and 39° 53' 26.4" north latitude (see Figures 1, 2, & 3). The Shipyard is about 100 miles from the open sea but is accessible to the largest warships via the Delaware River which has a 40-foot deep channel and adequate bridge clearances. (Reference 1, see Appendix A-33)

2.2 EXISTING FACILITIES

The Naval Shipyard and its component activities comprise a self-sustaining Naval Complex, (see Figure 4) in the performance of the following services to the Fleet units: overhaul and repair of all assigned vessels; research and development, test and evaluation of shipboard systems; and provision of appropriate logistic support to units as assigned. Major support for the League Island activi-



REGIONAL MAP PHILADELPHIA STANDARD METROPOLITAN STATISTICAL AREA

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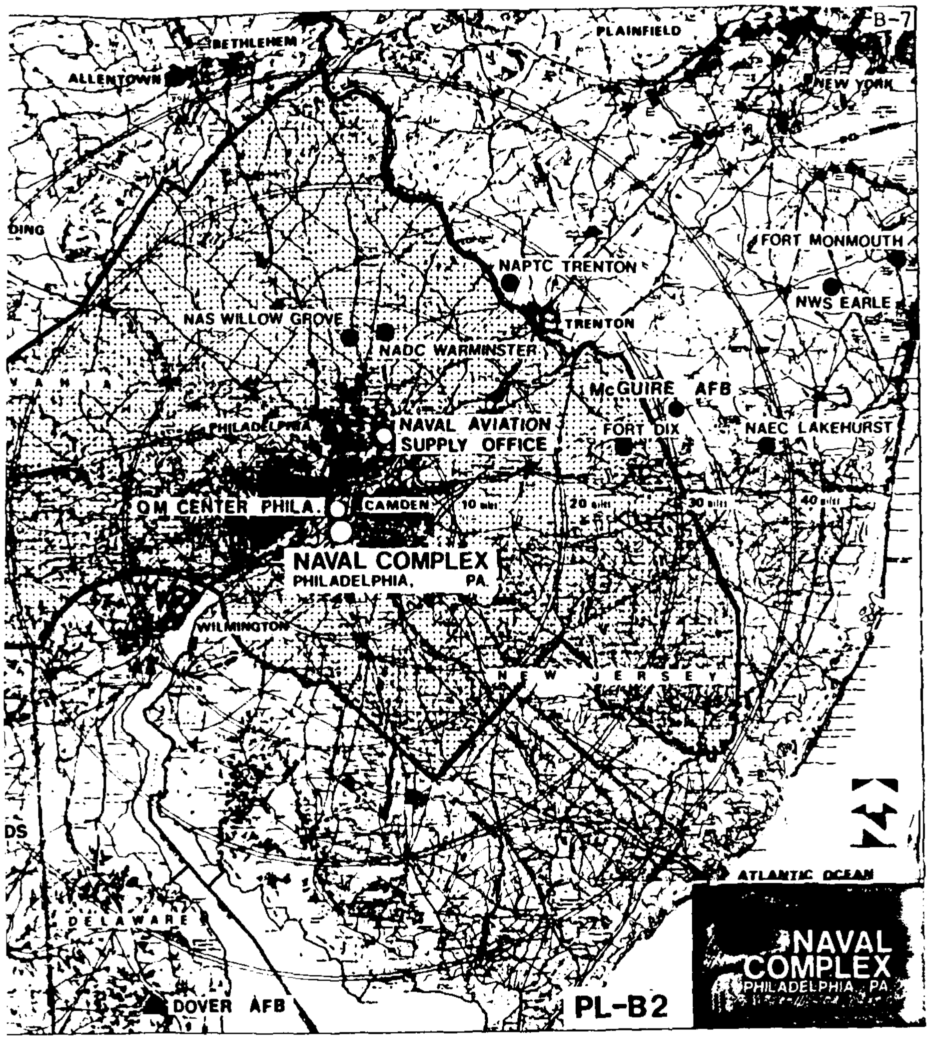

STANDARD METROPOLITAN STATISTICAL AREA

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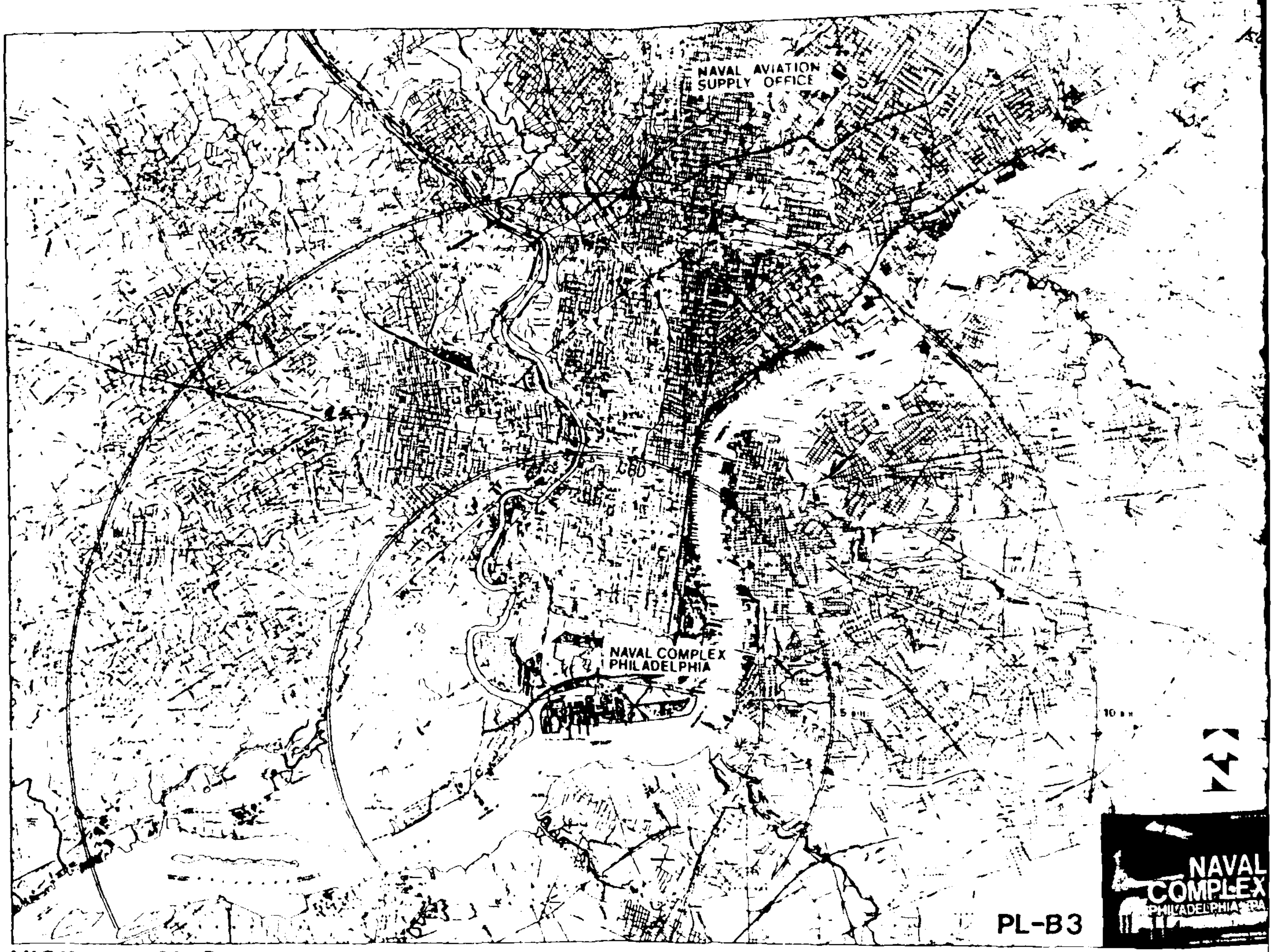
CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D.C.

PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA PA

REGIONAL MAP

FIG NO

1



VICINITY MAP

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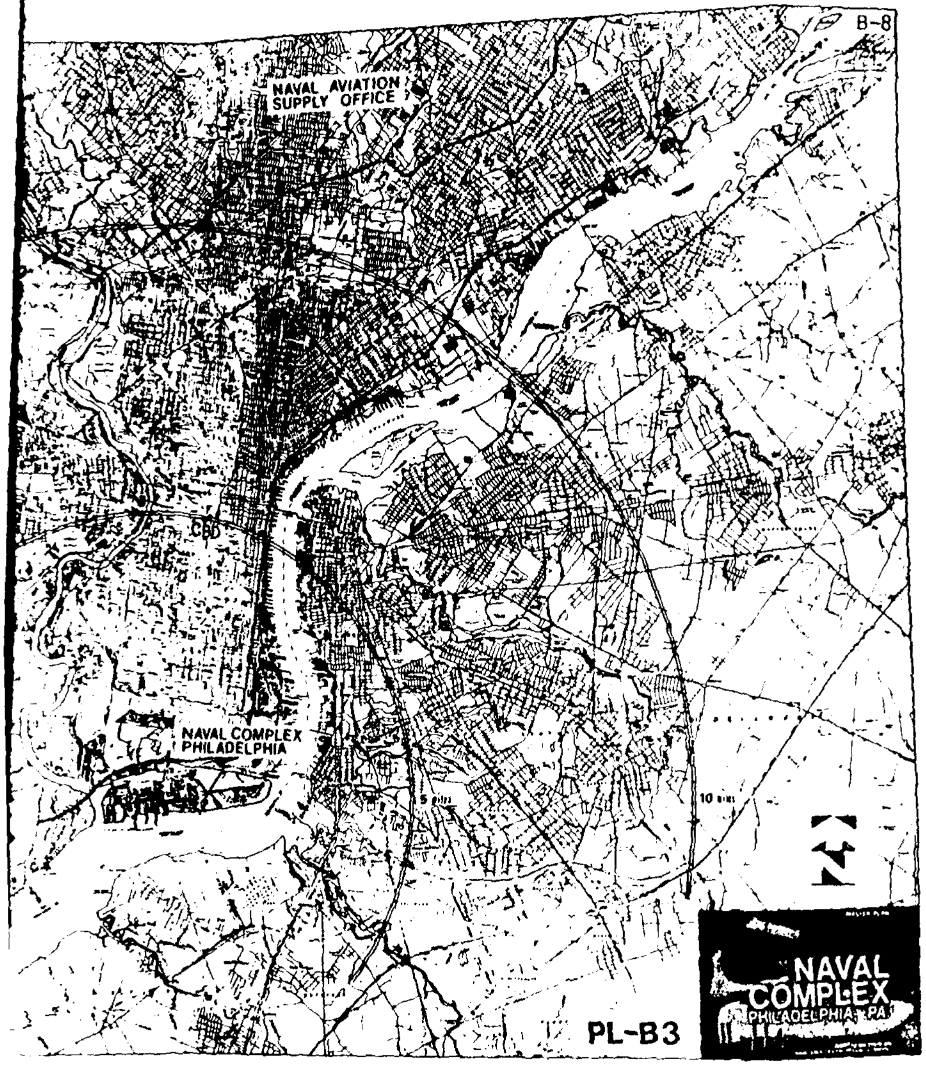

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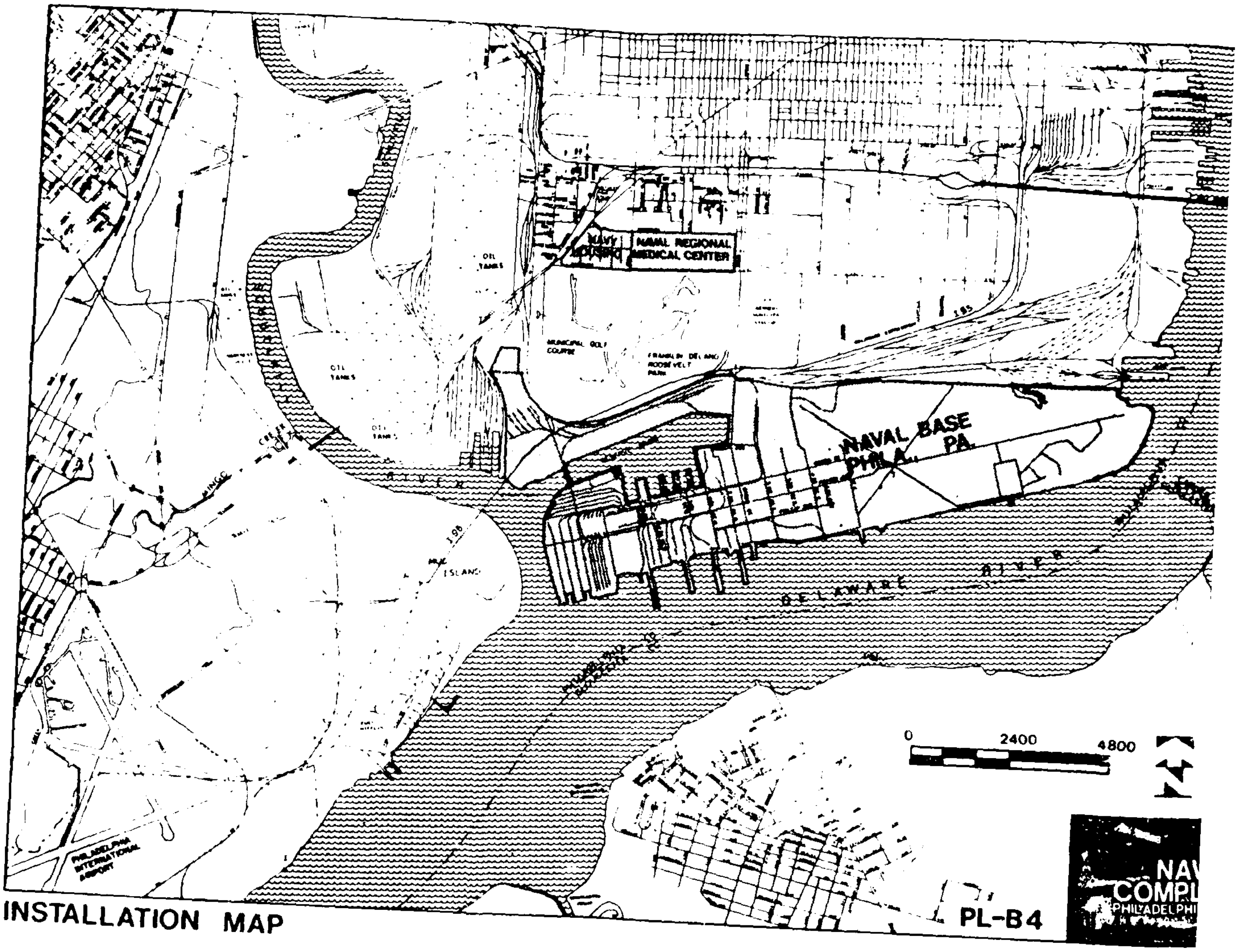

INSTALLATION MAP

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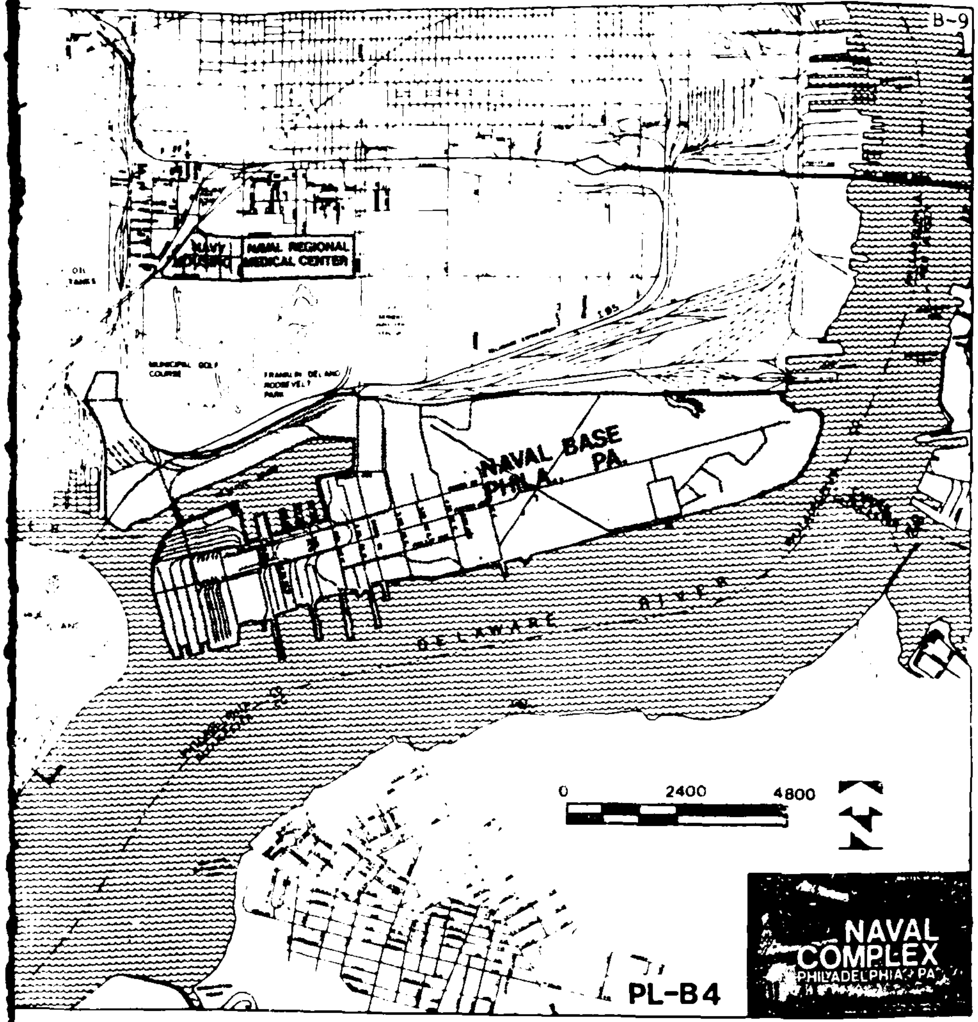

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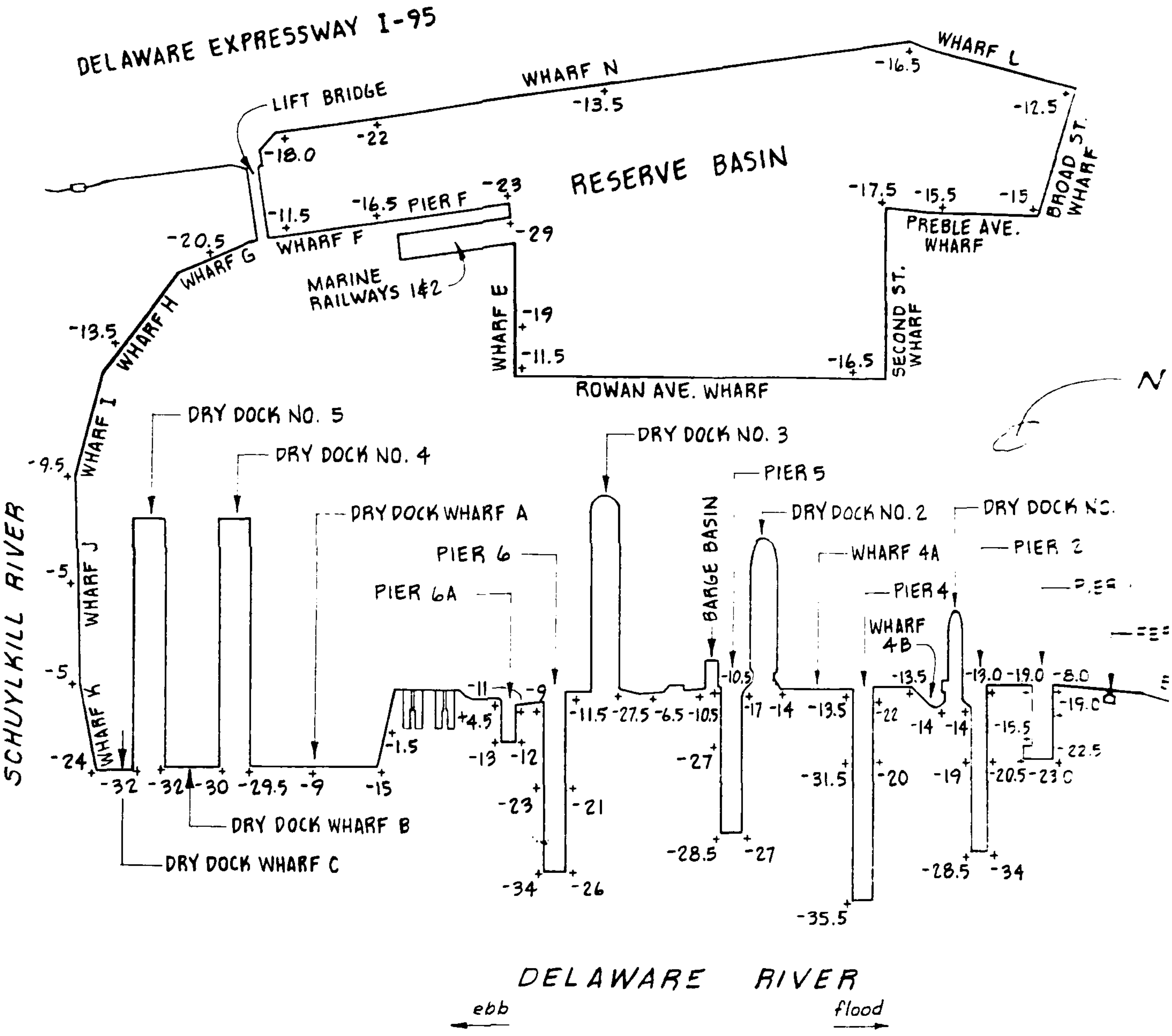
INSTALLATION MAP

FIG NO
3



PLAN

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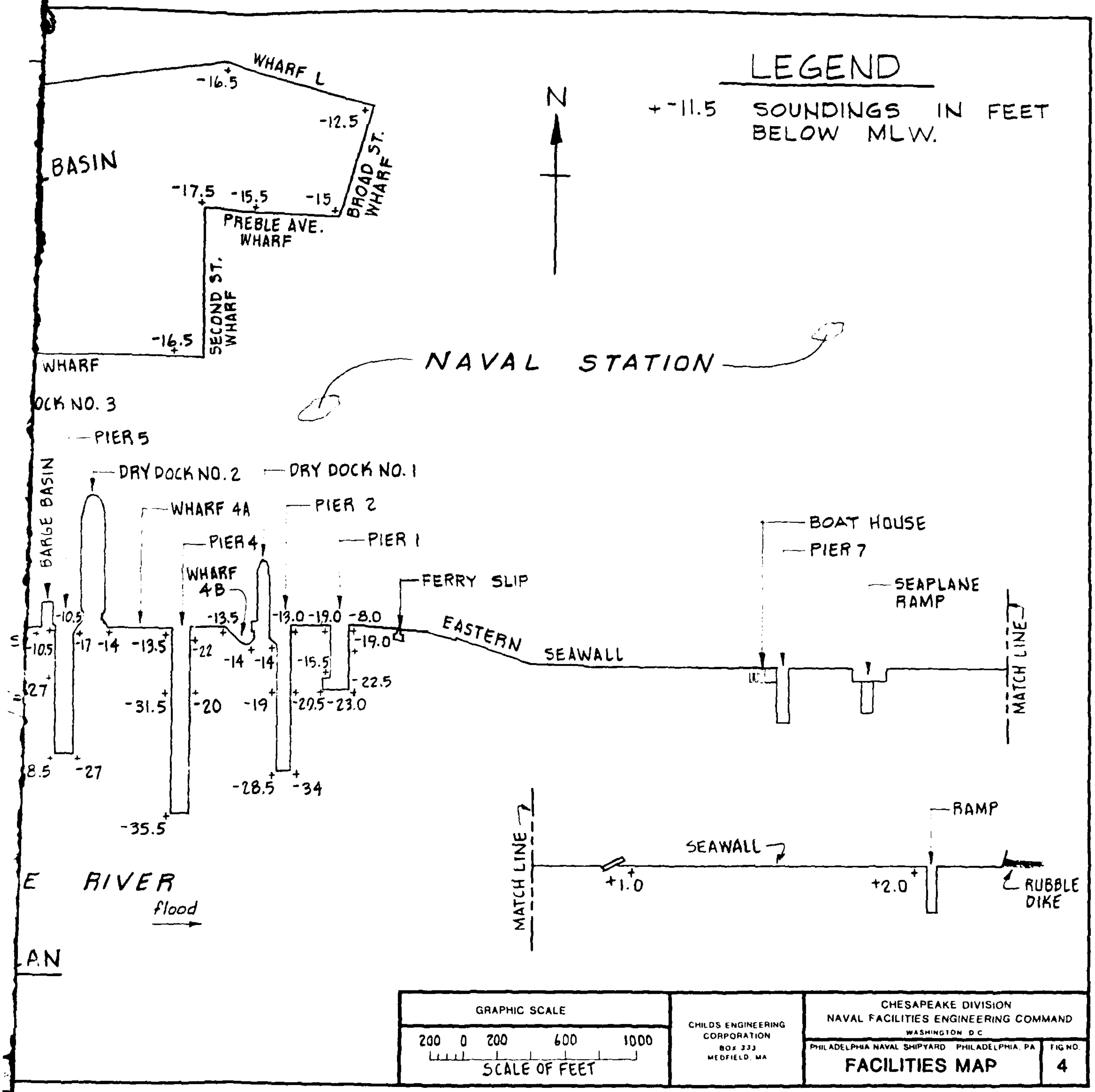

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WASHINGTON, D.C.

PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA

FIG. NO. 4

FACILITIES MAP

ties is furnished by the Shipyard and the Naval Support Activity; the Shipyard provides public works and family housing support and the Naval Support Activity provides general personnel support including berthing and messing.

Of the 534 buildings emcompassing nearly 11 million square feet of space, 442 (82%) are classified as permanent; 76 (14%) are semi-permanent and 16 (3%) are temporary. In addition, the Shipyard has two shipways, five drydocks, two marine railways, seven piers and approximately four miles of bulkheads and wharves.

(Reference 1, see Appendix A-33)

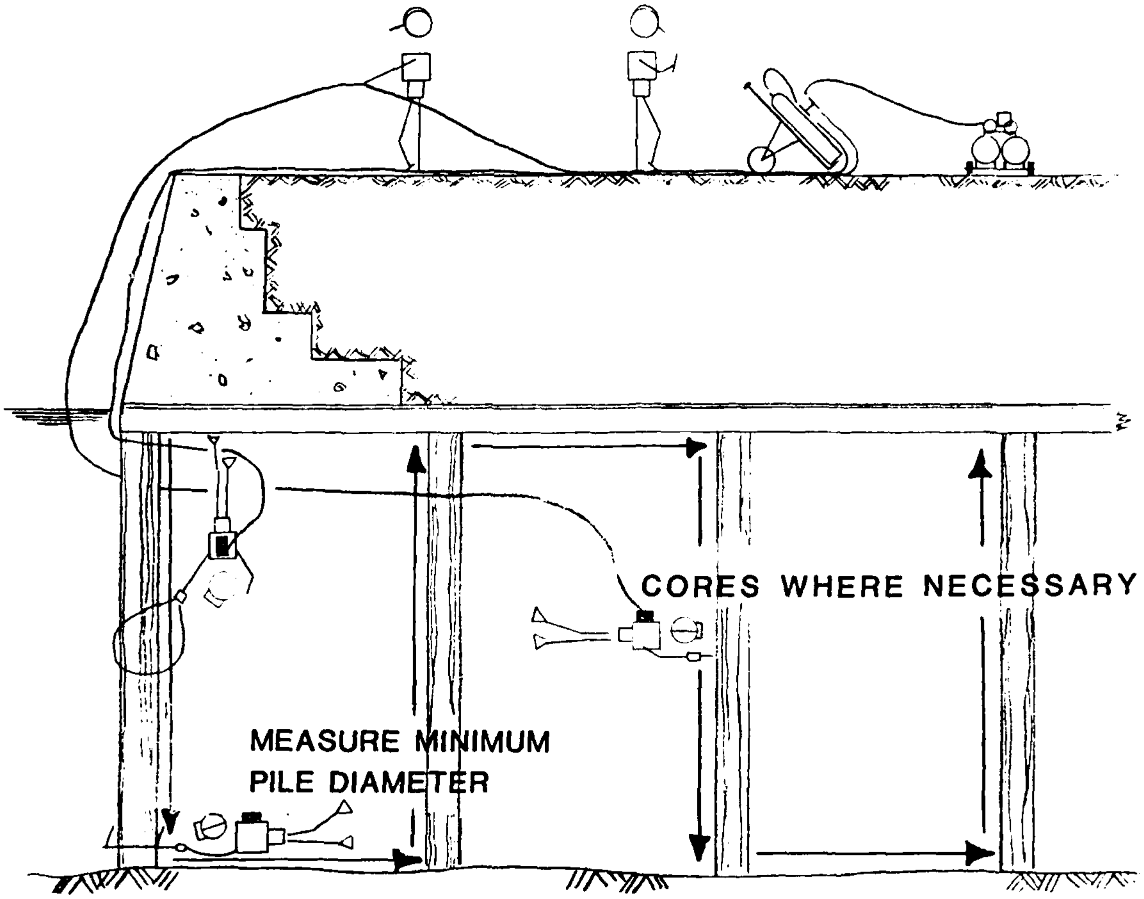
Between June 13 and October 7, 1983, a four-person Engineer/Diver, Technician/Diver inspection team performed an on-site Underwater Inspection of various piers, bulkheads and relieving platforms at the Philadelphia Naval Shipyard, Philadelphia, PA. The level of inspection to be performed, the type of structure being inspected, actual on-site conditions and past experience, combined with a thorough knowledge of engineering theory, dictated the inspection procedures that were followed.

3.1 LEVEL OF INSPECTION

The inspection techniques used had to be sufficient to yield information necessary to make a general condition assessment of the supporting structure of each facility, identify any areas that were mechanically damaged or in advanced states of deterioration and formulate repair and maintenance recommendations with cost estimates. In general, this means utilizing visual/tactile inspection techniques accompanied by occasional measurements using instruments such as calipers and an ultrasonic steel thickness gauge where appropriate. Core samples of the timber structural elements were also taken and evaluated. Photographic documentation of typical as well as notable or unusual conditions was also obtained.

3.2 INSPECTION PROCEDURE

A dive team consisting of two engineer/divers, an engineer/notekeeper and a tender performed the on-site inspection (see Figure



TYPICAL DIVER INSPECTION PATH
FOR TIMBER PILE INSPECTION

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D C	
NO SCALE		PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA INSPECTION PATH	FIG NO 5

5). Depending upon the layout of the individual pier or bulkhead, the divers would inspect one bent either across the pier or into a sheet pile wall and return on the next bent. Various levels of inspection were performed on selected piles as delineated below:

A Level I general inspection of the full length of the pile was performed on all perimeter piles and all piles within every third bent. A modified Level I inspection, which is a swim-by of the pile at Elevation 0.0 to Elevation -4.0, was conducted on the remaining piles. Structures such as bulkheads were inspected along their face at the mudline (ML), just below mean low water (MLW), and in the splash zone where accessible.

On all open type structures, a Level II inspection was performed on 5% of the piles. Along bulkheads the Level II inspection was conducted every 300 linear feet. The Level II inspection for timber-bearing piles involves band-cleaning the pile at two elevations (MLW and ML), and measuring the minimum pile diameter. For steel-bearing piles it involves band-cleaning on three sides at three elevations; mean low water (MLW), the mudline (ML) and halfway between MLW and ML. For wood and concrete sheet piles, the Level II inspection involved cleaning a 12-inch square area of bulkhead at three elevations; mean low water (MLW), mudline (ML), and midway between the ML and MLW. A 6-inch square area of steel sheet pile was cleaned on the web and flange at three elevations; MLW, ML, and midway between MLW and ML. On all areas cleaned during the Level II inspection, the condition of the cleaned surface was noted.

A Level III inspection, involving ultrasonic thickness measurements, was taken at every Level II location on steel-bearing piles and steel sheet piles. The Level III inspection for timber piles was performed on approximately one-half of the piles cleaned under the Level II inspection. This inspection involved the taking of three timber core samples at selected cleaned locations in piles, caps and deck, (see Photo #1).

The general pattern of inspection that was followed and the specific location of piles that were inspected were determined by mutual agreement between Childs Engineering Corporation and the on-site government representative.

3.3 INSPECTION EQUIPMENT

Equipment used for the inspection included a Minolta SRT200 camera with 28mm and 200mm lenses and strobe, Nikonos III, and IVA underwater cameras with Nikon 28mm lens and strobe, dive lights, 100-foot sounding tape, 200-foot fiberglass tape, 6-foot folding rules, large calipers, chipping hammers and dive knives. Also, to gauge steel thicknesses, a Krautkramer D-Meter ultrasonic thickness gauge with DMR probe was used. A pneumatic drill was used to take 1/2" core samples of the timber.

Choice of equipment was made as a result of past experience. Most of the equipment is straightforward, easy to implement, and has proven reliable under hard use.

PHOTO NO. 1: Wharf F, Bent 105, Pile B; illustrates typical timber core plug in pile cap. Plug diameter is 3/4".

Within this section of the report, each facility inspected at the Philadelphia Naval Shipyard is referenced separately. The discussion of each facility is presented in four parts: 1) a description of the construction and function of the structure, which is derived both from the on-site inspection and from the referenced government-furnished information; 2) an enumeration of general and specific conditions observed during the on-site inspection; 3) a qualitative assessment of the structural condition of the facility based on the inspection data; and 4) recommendations for actions to be taken to ensure long-term, cost-effective maintenance and utilization of the facility. Detailed breakdowns of cost estimates are included in the Appendix.

Marine growth profiles noted at each facility were similar throughout the shipyard. In general, the piles were covered with a soft algal growth from El. 0.0 to the mudline. (see Photo #2). This growth ranged in depth from 1/4" to 1". Living along with the algal growth there are various marine invertebrates, one specific marine invertebrate, <u>Limnoria</u> (marine borers) were found throughout each facility and in particular, the facilities bordering the Delaware River. The <u>Limnoria</u> found were not highly active and do not appear to present a serious threat to the structural integrity of any facility in the shipyard at this time. The presence of <u>Limnoria</u> is probably due to the advancement of the saltwater wedge up the Delaware River; this is usually associated with dry weather conditions.

PHOTO NO. 2: Preble Avenue, timber sheet pile wall between Bents 52-53; illustrates typical algal growth approximately 1/2" thick. Core plug diameter is 3/4".

Specific anomalies discovered range from broken piles to soft timber. In the following paragraphs we will try to define some of these conditions so reference can be made to them throughout the report.

Structural timber found throughout the shipyard was generally in sound condition. In some locations divers reported finding that they were able to probe into the timber with a sharp knife up to 5". This timber "softness" can be described as a weakening of the bonding agents holding the timber fibers together. Softness, therefore, reflects the overall strength of the timber member. In cases where softness is a significant factor, a reduced section is used in the analysis of the member (see Photos #3, 4, and 5).

A non-bearing pile is a pile which is not in contact with the pile cap and therefore there is no bearing between the pile and pile cap. The non-bearing pile is generally centered below the pile cap with the drift pin still in place. The wild pile is similar to the non-bearing pile in that both are not in contact with the pile cap. However, the wild pile differs in that it is out of alignment with the pile cap and the drift pin connection has failed. The difference between a wild pile and a split and displaced pile is the condition of the pile itself. Generally, the wild pile is in functional condition and a connection failure has occurred while the split and displaced pile itself has failed as well as the connection.

A non-bearing pile can be the result of a number of reasons. Occasionally the non-bearing condition is merely the loss of a

PHOTO NO. 3: Wharf N, Bent 201, Pile A; knife penetrating 3" into soft timber pile. Approximately 2" of knife blade is exposed.

PHOTO NO. 4: Pier 7, Bent 54, pile cap between Piles B and C; knife penetrating 3" into soft pile cap. Approximately 2" of knife blade is exposed.

PHOTO NO. 5: Broad Street Wharf, Bent 134, Pile F; knife penetrating 2" into soft deck plank. Approximately 3" of knife blade is exposed.

shim which was placed between the pile and pile cap during construction to attain full bearing (see Photo #6). In other instances, the non-bearing condition is related to an overall movement of the structure due to forces exerted on the structure. Settlement of the pile can also be a cause of the non-bearing pile. Detailed discussions of these conditions are included in the assessment of facilities where this condition occurs.

Broken piles are defined as piles which have suffered complete failure as columns. Typically, this condition is the result of horizontal loading of the pile (forces transferred through camels). However, in some rare cases the piles are overloaded in the vertical direction.

Broken or split piles occurring at the perimeter of a pier or wharf are generally thought to be caused by berthing impact. Piles which exhibit this type of damage in the interior of a pier are assumed to be damaged by floating objects under the pier unless otherwise noted (see Photos #7 and #8).

Typical corrosion profiles for the steel H-piles and steel sheet piles reveal that there are large orange-colored corrosion nodes (approximately 1" to 2" diameter) associated with pitting on the metal surface in the early stages of deterioration (see Photo #9). In an advanced state of deterioration, the surface of the steel is covered with a black corrosion by-product, approximately 1/4" to 3/8" thick. Accompanying this are pockets of trapped gas. Corrosion covering the surface of the metal with pits averaging

PHOTO NO. 6:

Wharf E, Bent 85, Pile A; non-bearing pile. Gap between pile and pile cap is approx. 3". Shim is loose and no longer transmitting load to the pile.

PHOTO NO. 7: Preble Avenue Wharf, between Bents 37-38, Pile A; pile broken approx. 10' below pile cap due to impact load. Pile diameter is approx. 12".

PHOTO NO. 8: Pier F, Bent 116, Pile A; top of pile split and knocked out from under pile cap due to impact load. Maximum split is approx. 5", pile 10% bearing.

PHOTO NO. 9: Barge Basin and Bulkhead to the east of Pier 6, Bent 65, Pile C; illustrates typical condition of steel H-pile, approximately El. -4'. Orange corrosion nodes 1"-2" diameter. To left is diver taking D-meter measurement.

1/16" deep and 1/4" in diameter will be noted as heavy pitting (see Photo #10 and Figure 6).

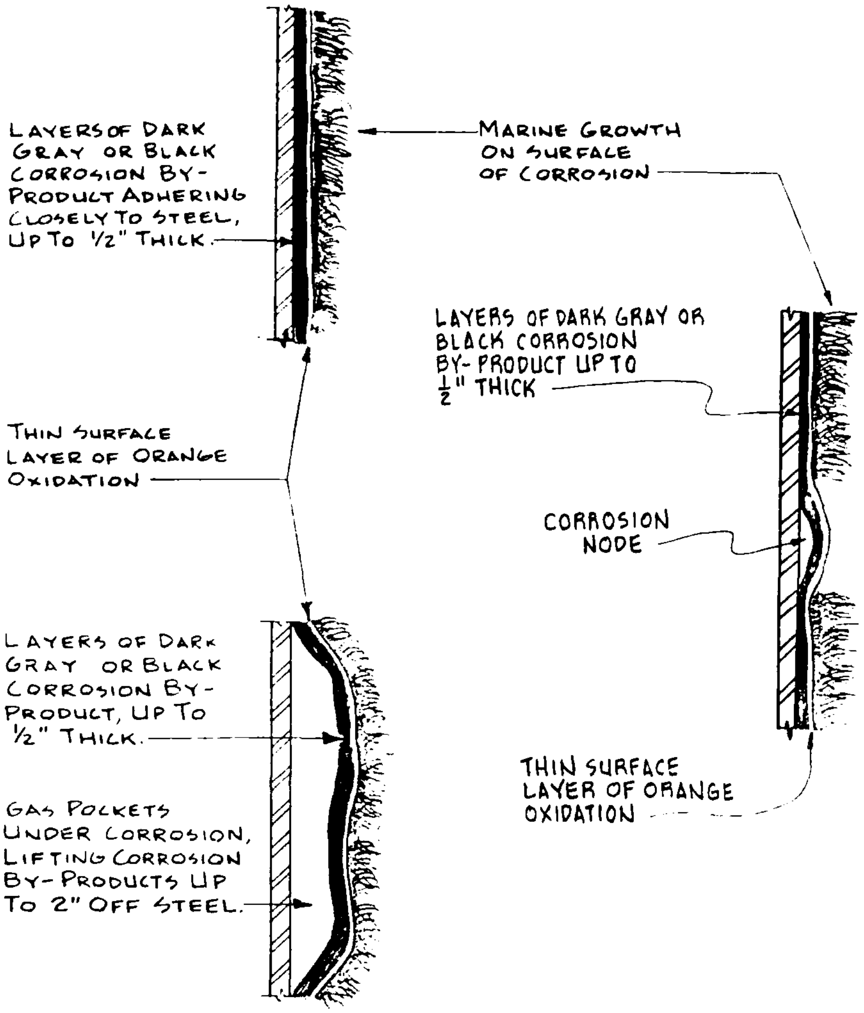
Crushing of the pile cap over a pile is a condition which is generally found at the perimeter of the pier, although it is not strictly limited to the perimeter and can be found throughout the pier (see Photo #11 and Figure 7).

Timber decking occasionally was found to be sagging and in some cases a deck plank had failed. This is attributed to overstressing due to a reduction of the strength of the timber's outer fibers. The decking is affected by the softness sooner than the pile caps or piles because of the lower cross-sectional area to surface area ratio, (see Photo #12).

The concrete poured near the elevation of mean low water generally exhibited the condition where sand and cement have been eroded away from the surrounding larger aggregate in the concrete (see Photo #13). This condition does not effect the overall strength of the concrete.

The timber sheet pile retaining walls occasionally have misaligned sheets. These sheets are generally 2"-6" out of line. In some places of misalignment there is also a 2"-3" gap between the sheets where fill material can be observed (see Photo #14). These conditions appear to be related to the original construction of the wall.

EXAMPLES OF CORROSION BUILD-UP

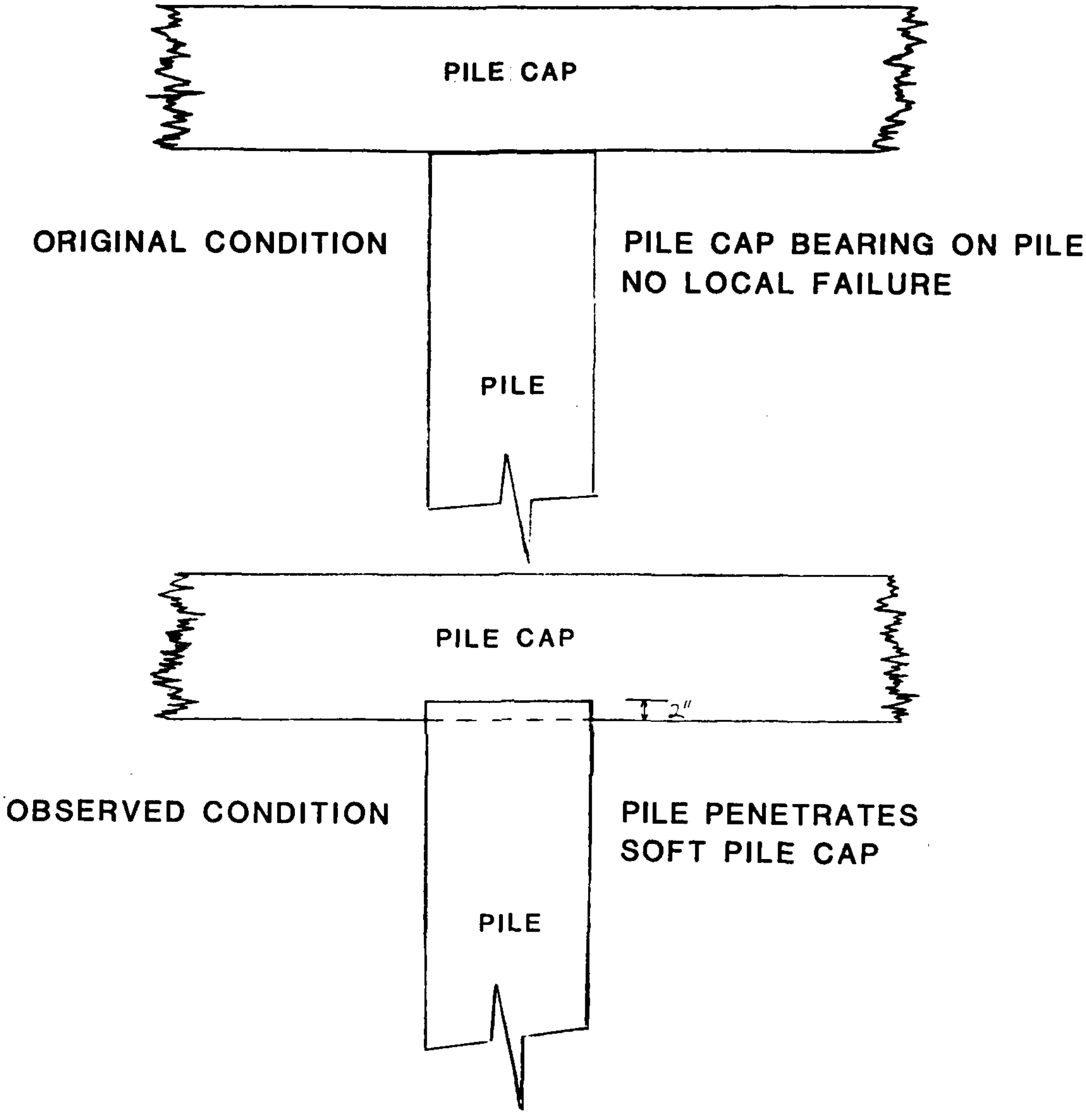


GRAPHIC SCALE NO SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D C PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA CORROSION BUILD-UP FIG NO 6

PHOTO NO. 10: Wharf 4-B, Sta. 4+20; illustrates typical condition of steel sheet pile, approx. El. +2.0. Pitting is approx. 1/16" deep and 1/4" in diameter.

PHOTO NO. 11: Wharf E, intermediate bent between Bents 26-27, Pile 1; local crushing of pile cap over the bearing pile due to timber softness and duration of dead load. Pile has penetrated 2" into pile cap.

PILE CAP SOFTNESS



ELEVATION

GRAPHIC SCALE

NO SCALE

CHILDS ENGINEERING
CORPORATION
BOX 333
MEDFIELD, MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D.C.

PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA

PILE CAP SOFTNESS

FIG NO. 7

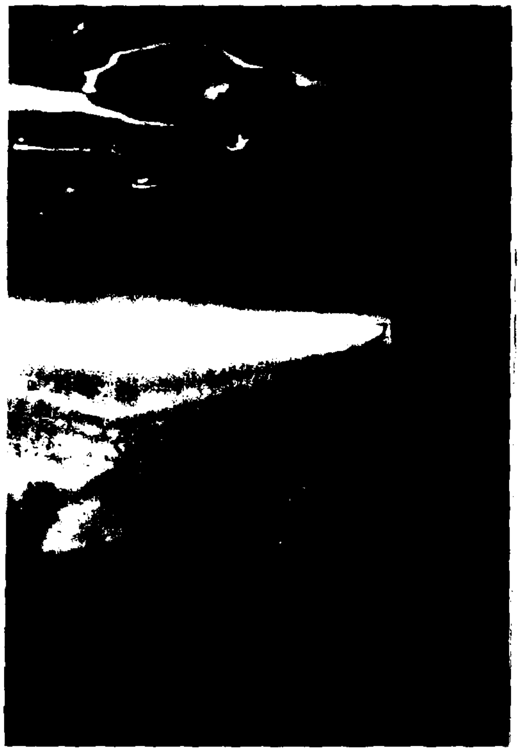


PHOTO NO. 12:
Wharf E, between Bents 92-93, adjacent to Pile A; broken deck plank due to soft timber.

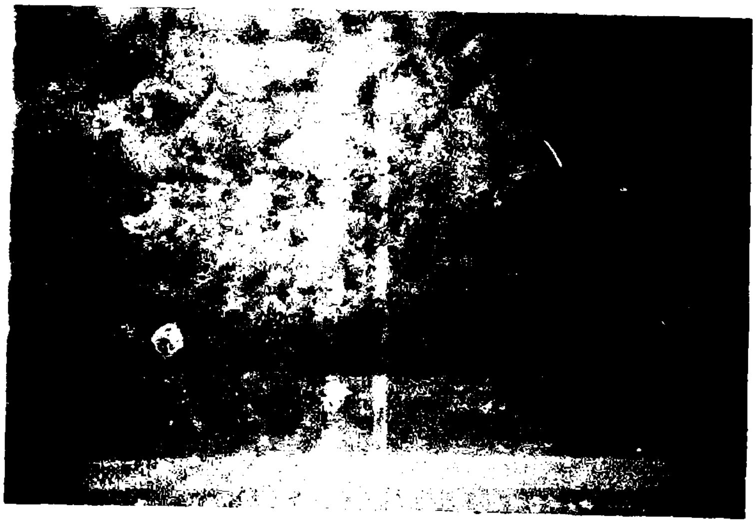
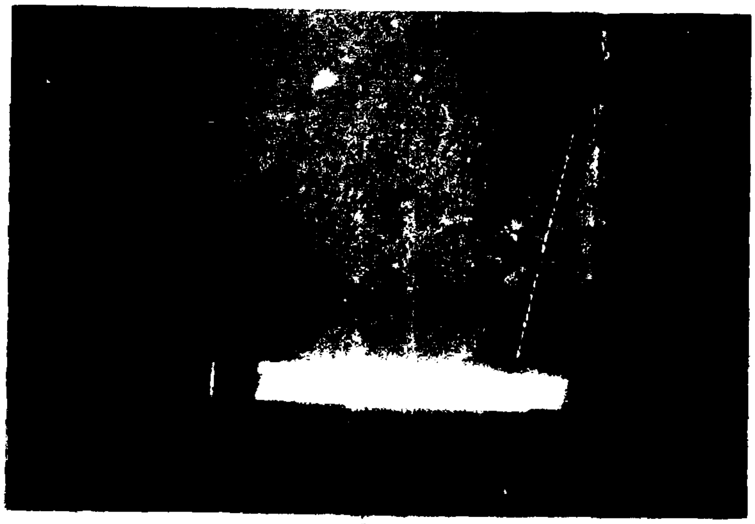


PHOTO NO. 13: Pier 5, Bent 43, Pile B; sand and cement eroded from concrete exposing aggregate. Timber pile cap is below.

PHOTO NO. 14: Eastern Seawall, Sta. 44+00 at the mudline; 2" separation between timber sheet piles with some loss of backfill.



The term "superstructure" is used throughout this report. This refers to that portion of the facility above the splash zone. In those facilities in which the superstructure is above the splash zone, only a cursory inspection was made of that area. In structures such as the relieving platform, the pile caps and timber deck material were closely examined along with the structural piles.

When considering dredging adjacent to any facility, particular attention should be paid to the design dredge limits. These limits should be determined prior to any dredging operation, and they should be followed. Over-dredging can create instability in the structure and possibly reduce its load-carrying capacity.

The live-loading limit recommendations pertain only to those portions of the structure which were accessed by the divers. Any specific recommendation stated can only pertain to those areas which were directly observed. The inference of live-load capacities beyond accessed areas is purely speculative, although it is based on past experience and sound engineering practice.

4.1 EASTERN SEAWALL

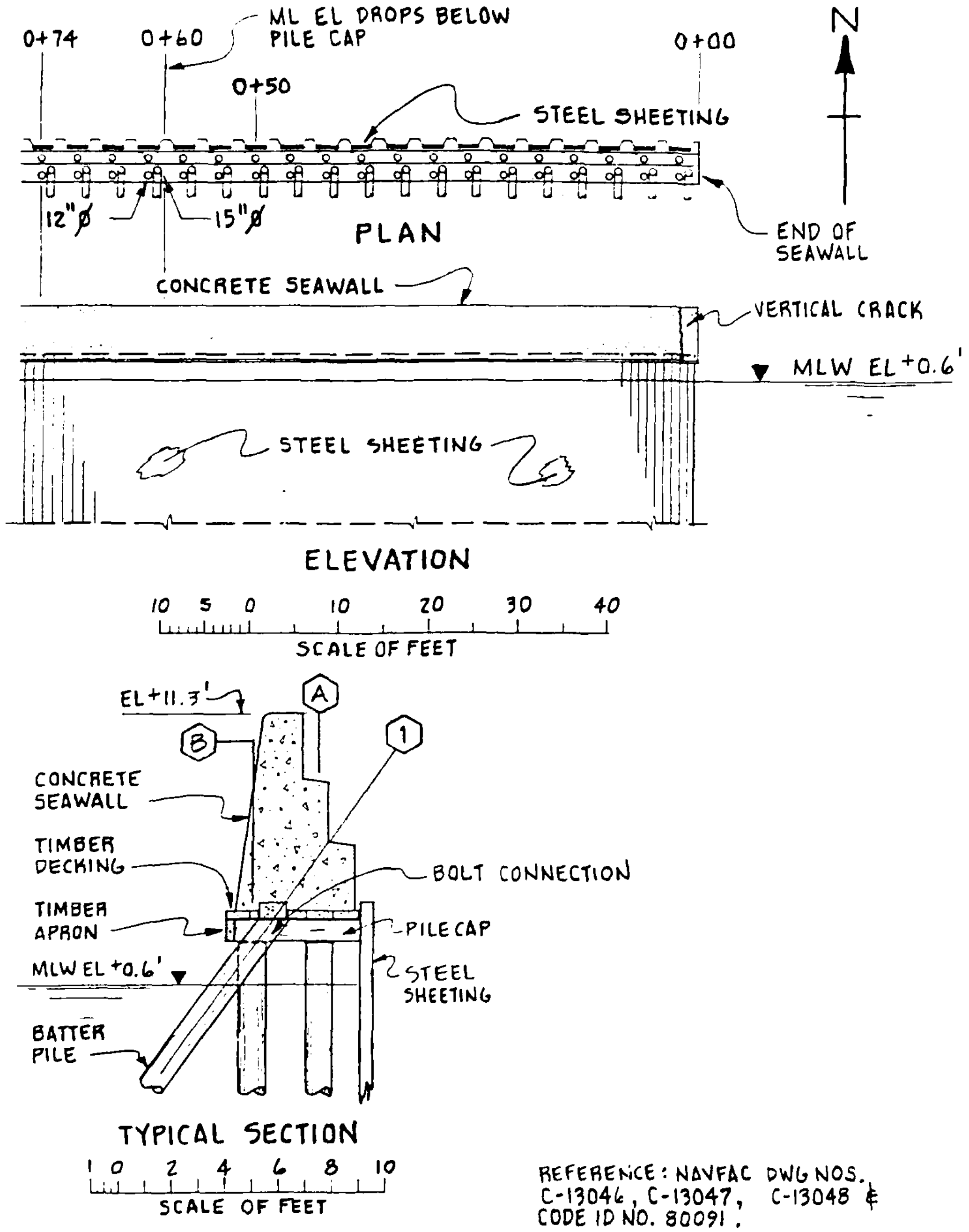
4.1.1 Description

The Eastern Seawall runs parallel to the southern shore of the Delaware River and is located to the east of and adjacent to Pier 1, (See Figure 4 and Figures 8 through 24). The portion of this timber pile-supported retaining structure between Station 0+00 through 10+10 was constructed circa 1943. The structure consists of two vertical piles and two batter piles arranged in bents spaced 5 feet on center with a steel sheet pile wall running along the inshore side of the "B" pile.

From Station 10+10 to 20+10 the timber pile-supported, earth fill, retaining wall structure was constructed circa 1944. From Station 20+10 to Station 52+35 the structure was constructed circa 1933. The structure consists of two vertical timber piles and one timber batter pile supporting a concrete seawall with a steel or timber sheet pile wall running behind the concrete seawall. The driven pile capacity of the bearing piles ranges from 5 tons to 20 tons.

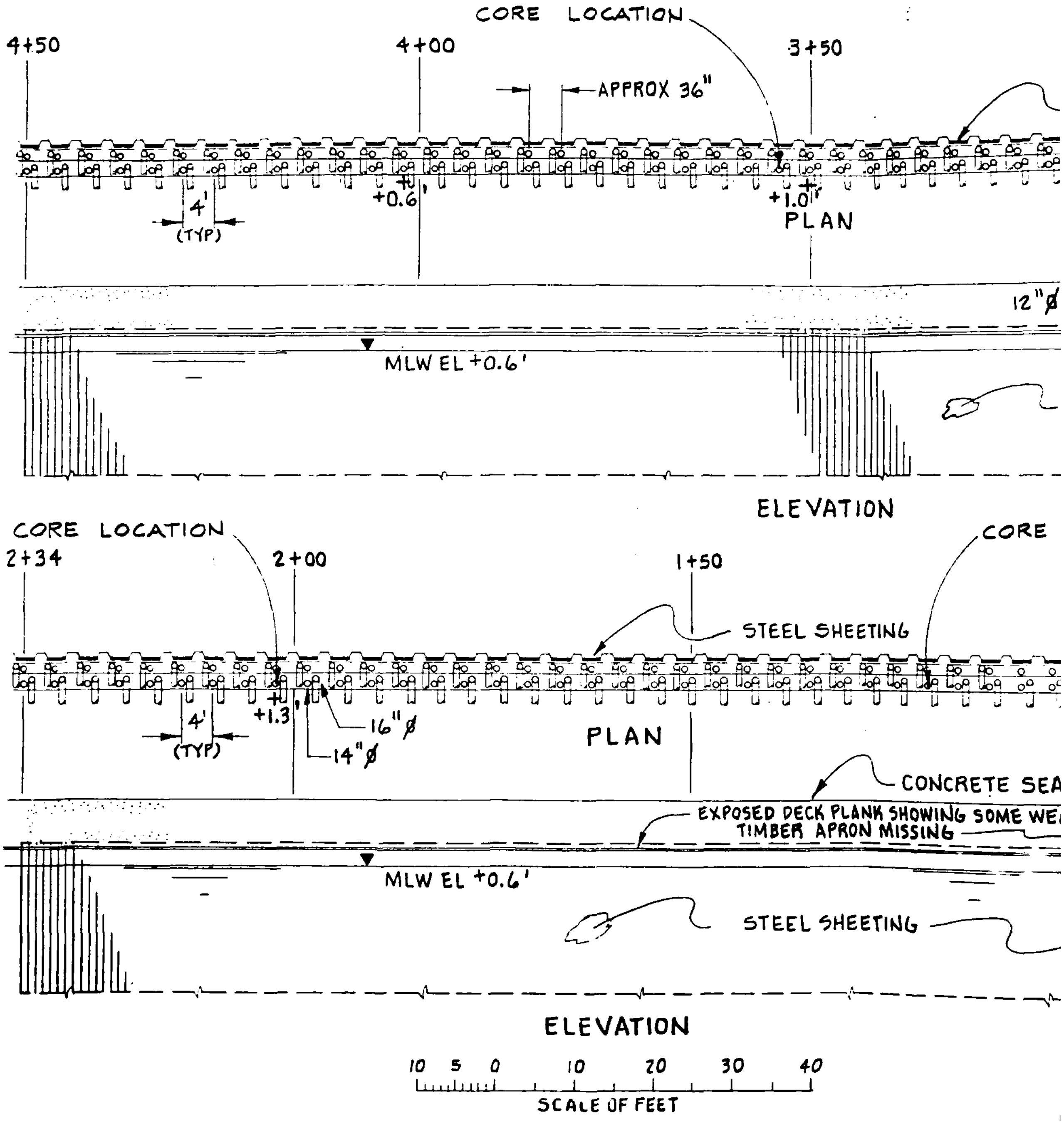
From Station 52+35 to Station 60+25 the seawall consists of a gravity type stone abutment, the date of construction is unknown. The bulkhead from Stations 60+25 to 61+90 has bents spaced 5' on center consisting of three vertical piles, timber clamps and concrete seawall with timber sheet pile driven inshore of the "C" pile. Date of construction was circa 1903. From Station 61+90 to Station 66+89 the portion of the bulkhead consists of bents with one vertical timber pile, clamps and a concrete seawall. Running along the inshore side of the vertical pile is a timber sheet pile wall, date of construction is circa 1899.

The deck elevation of the seawall ranges from +10' to +11.56' above mean low water. The original design capacity of the timber piles was between 3 tons and 20 tons (driven capacity). The overall length of the Eastern Seawall is 6689'.
(Reference 2, see Appendix A-33)

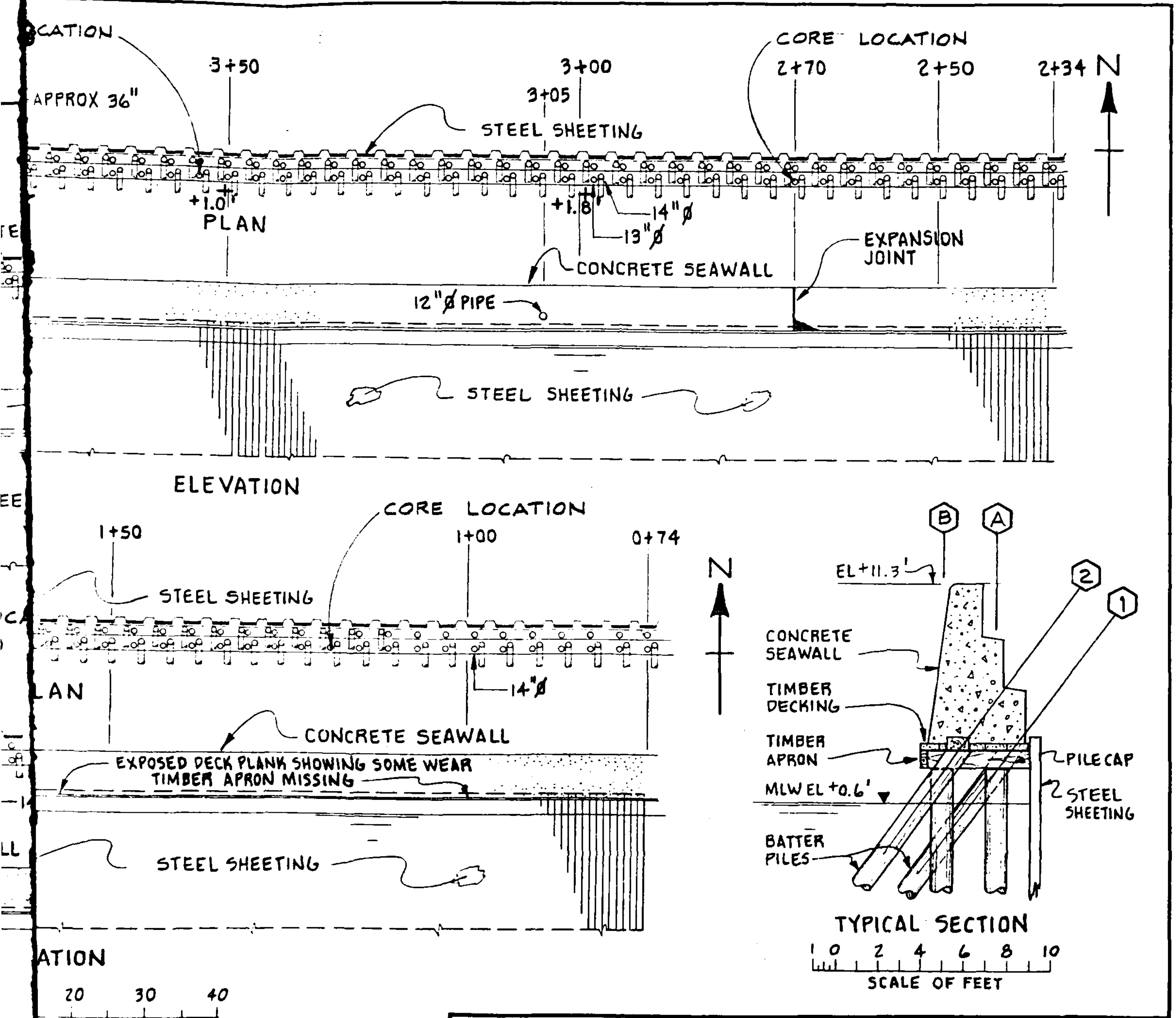


REFERENCE: NAVFAC DWG NOS.
C-13046, C-13047, C-13048 &
CODE ID NO. 80091.

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D.C.	
AS SHOWN		PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA EASTERN SEAWALL	FIG NO 8



REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWG NOS. C-13046, C-13047 & C-13048.



, C-13047 & C-13048.

GRAPHIC SCALE

AS SHOWN

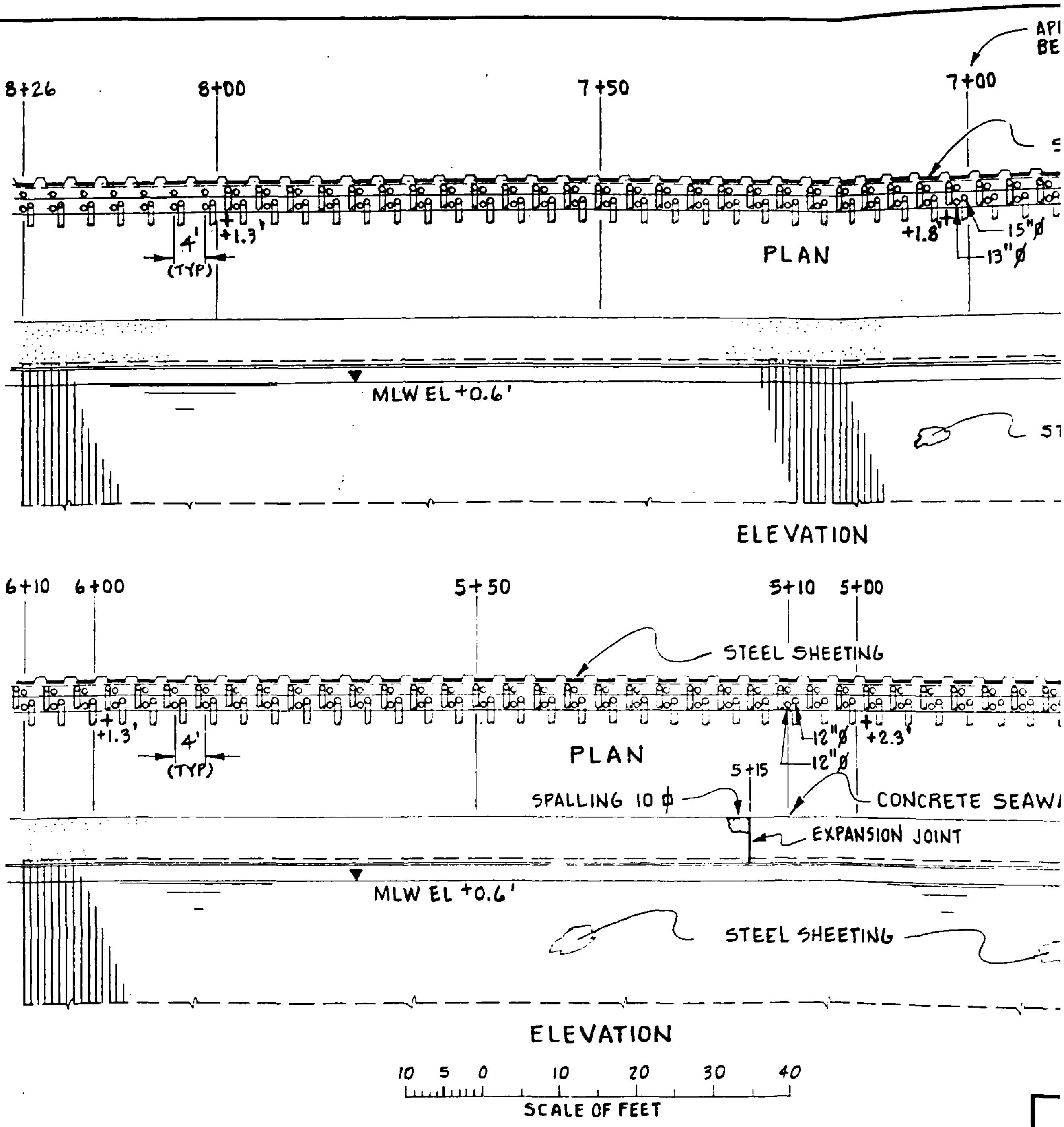
CHILDS ENGINEERING CORPORATION
BOX 333
MEDFIELD, MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D C

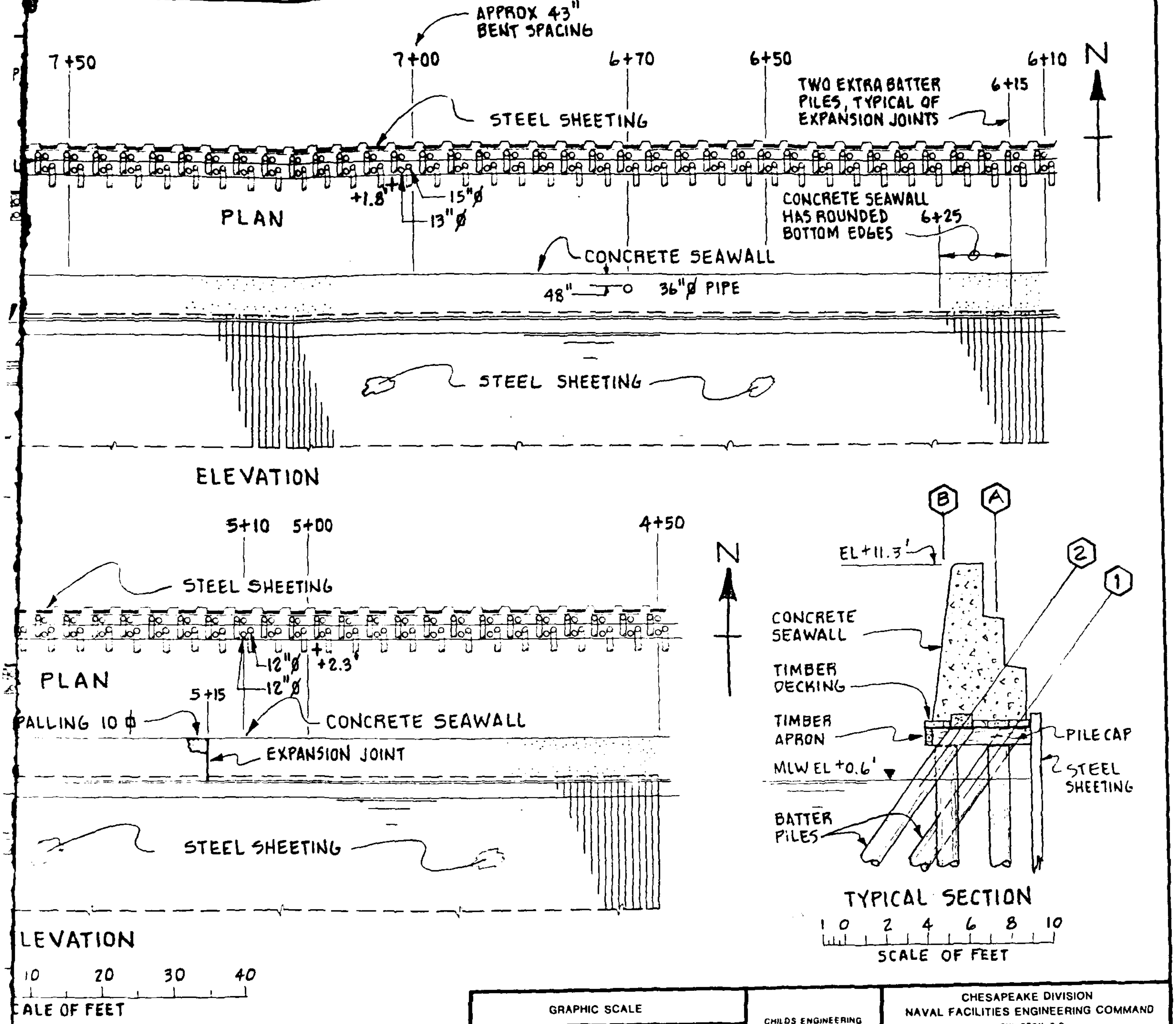
PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA

EASTERN SEAWALL

FIG. NO. 9



REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWG NOS. C-13046, C-13047 & C-13048.



URES
-13046, C-13047 & C-13048.

GRAPHIC SCALE

AS SHOWN

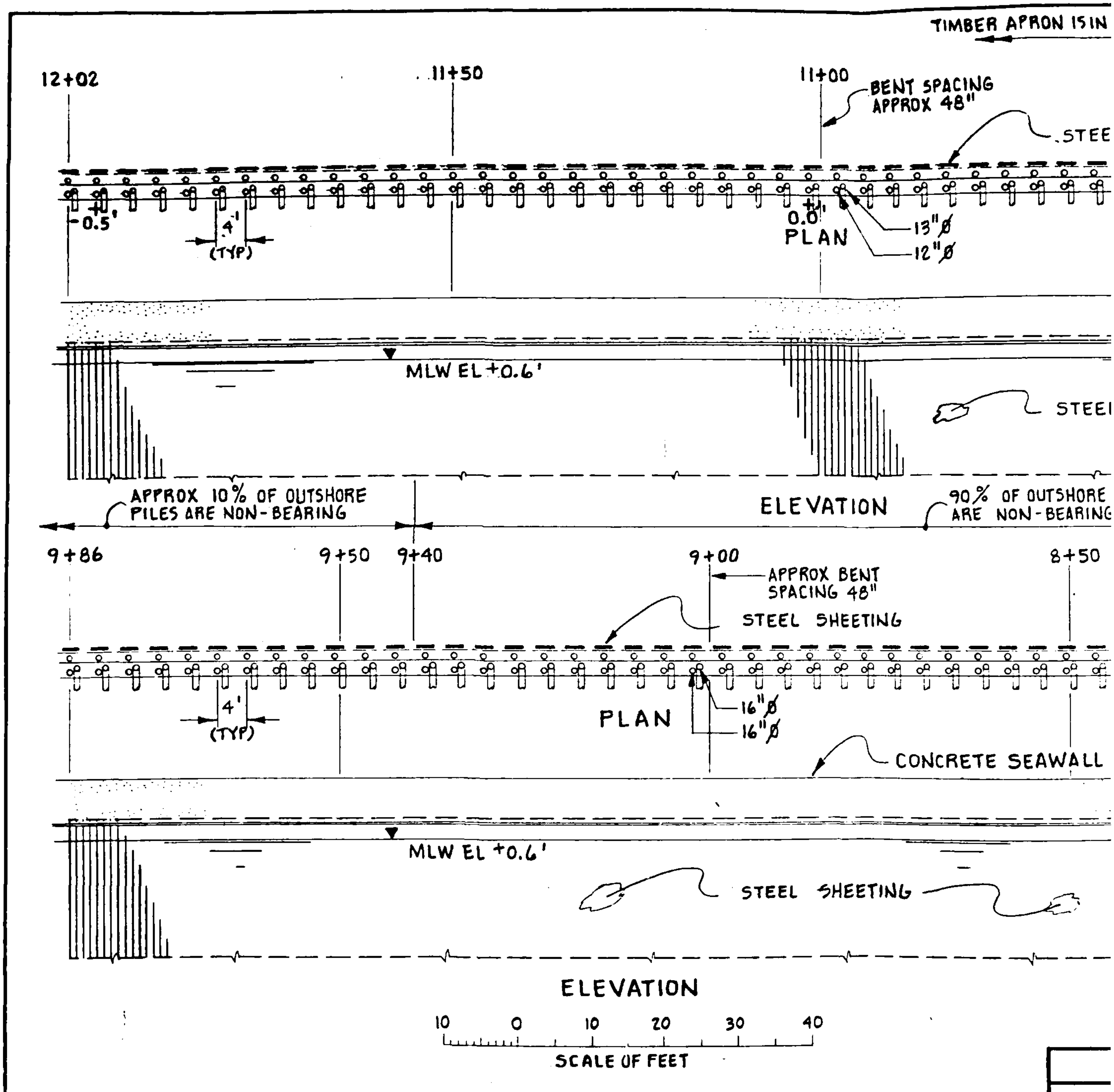
CHILDS ENGINEERING CORPORATION
BOX 333
MEDFIELD, MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON, D.C.

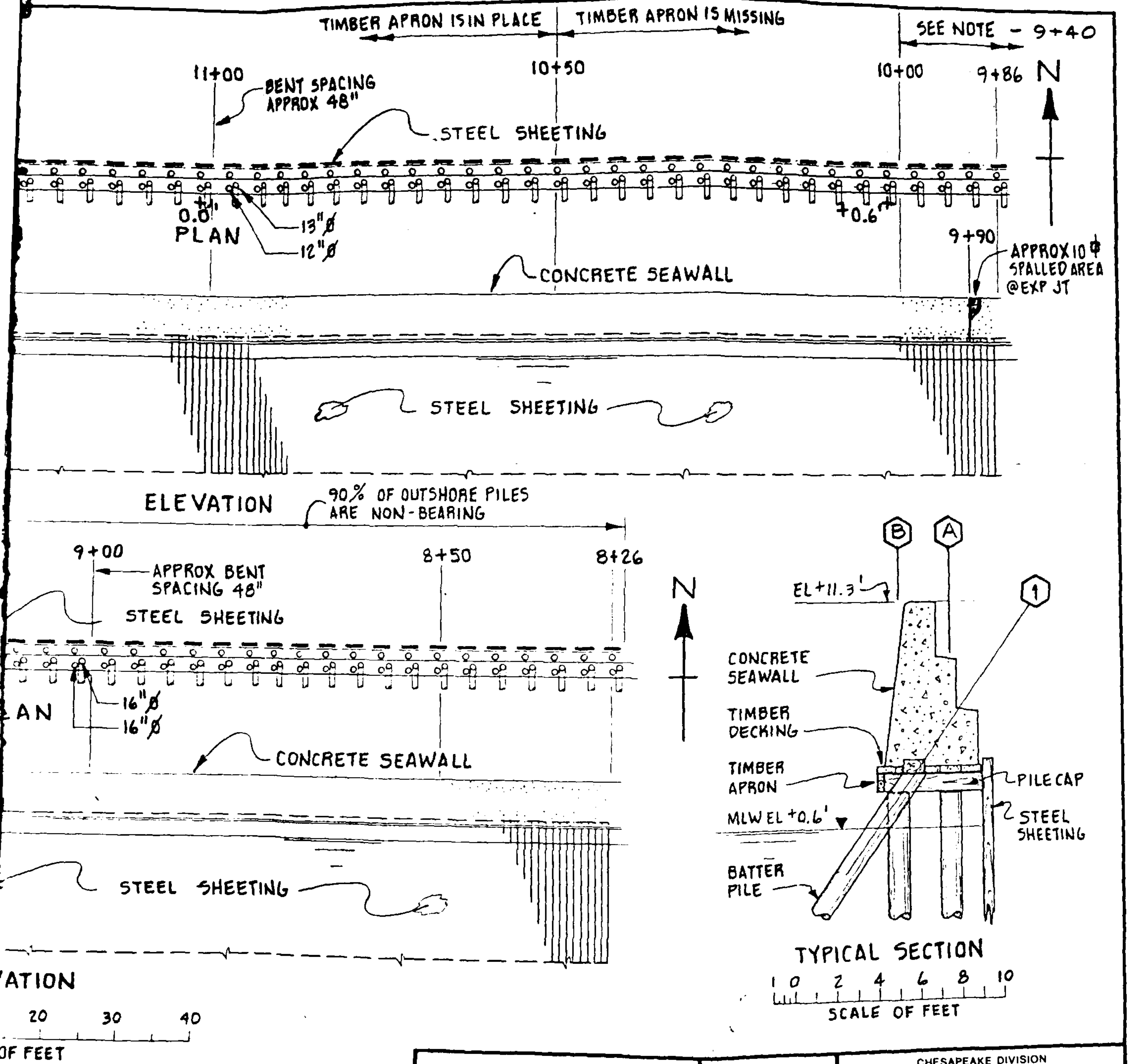
PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA

EASTERN SEAWALL

FIG NO
10



REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWG NOS. C-14724, C-14725, C-13046, C-13047 & C-13048.



C-14725, C-13046, C-13047 & C-13048.

GRAPHIC SCALE

AS SHOWN

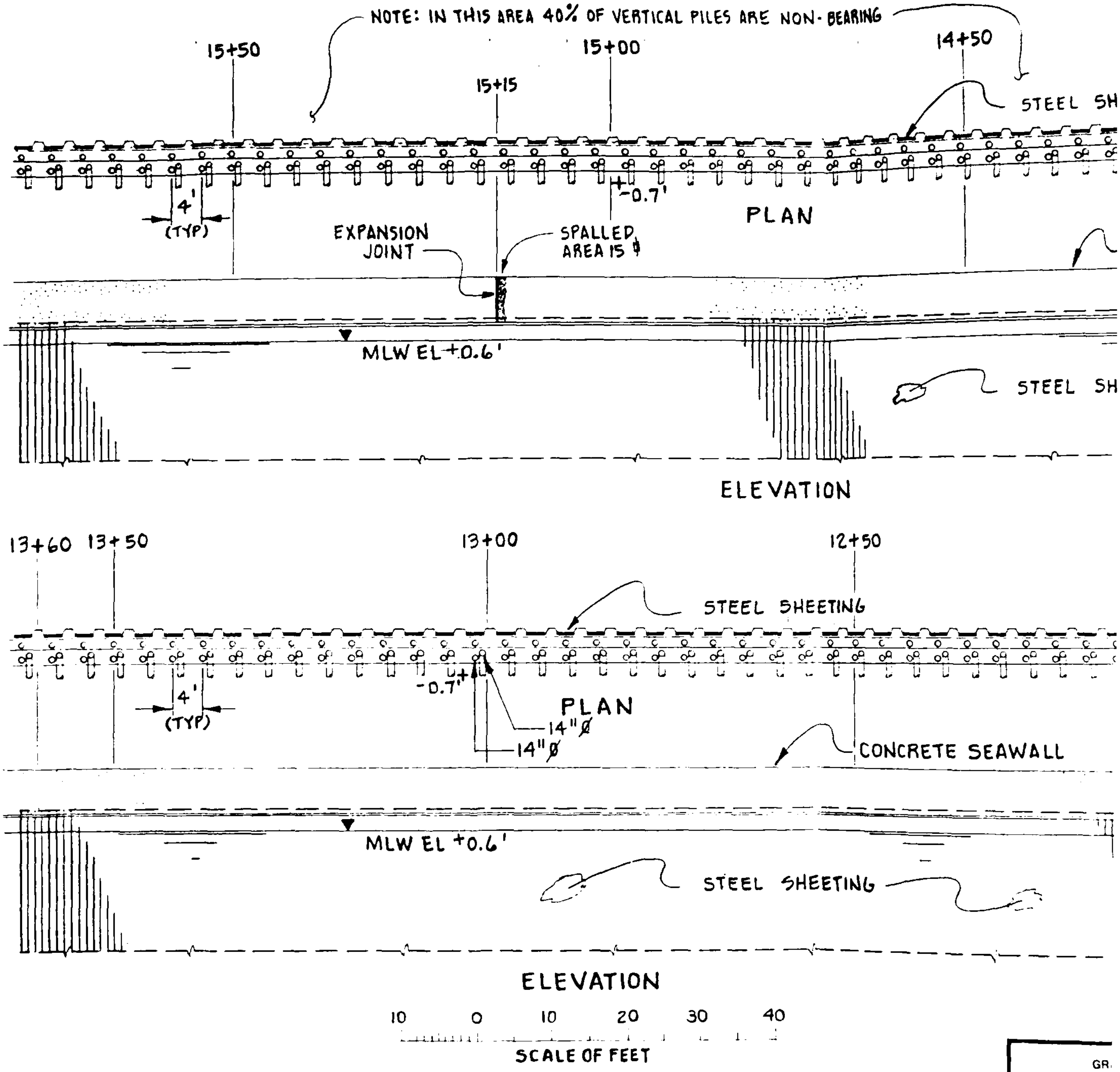
CHILDS ENGINEERING CORPORATION
BOX 333
MEDFIELD, MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON, D.C.

PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA

EASTERN SEAWALL

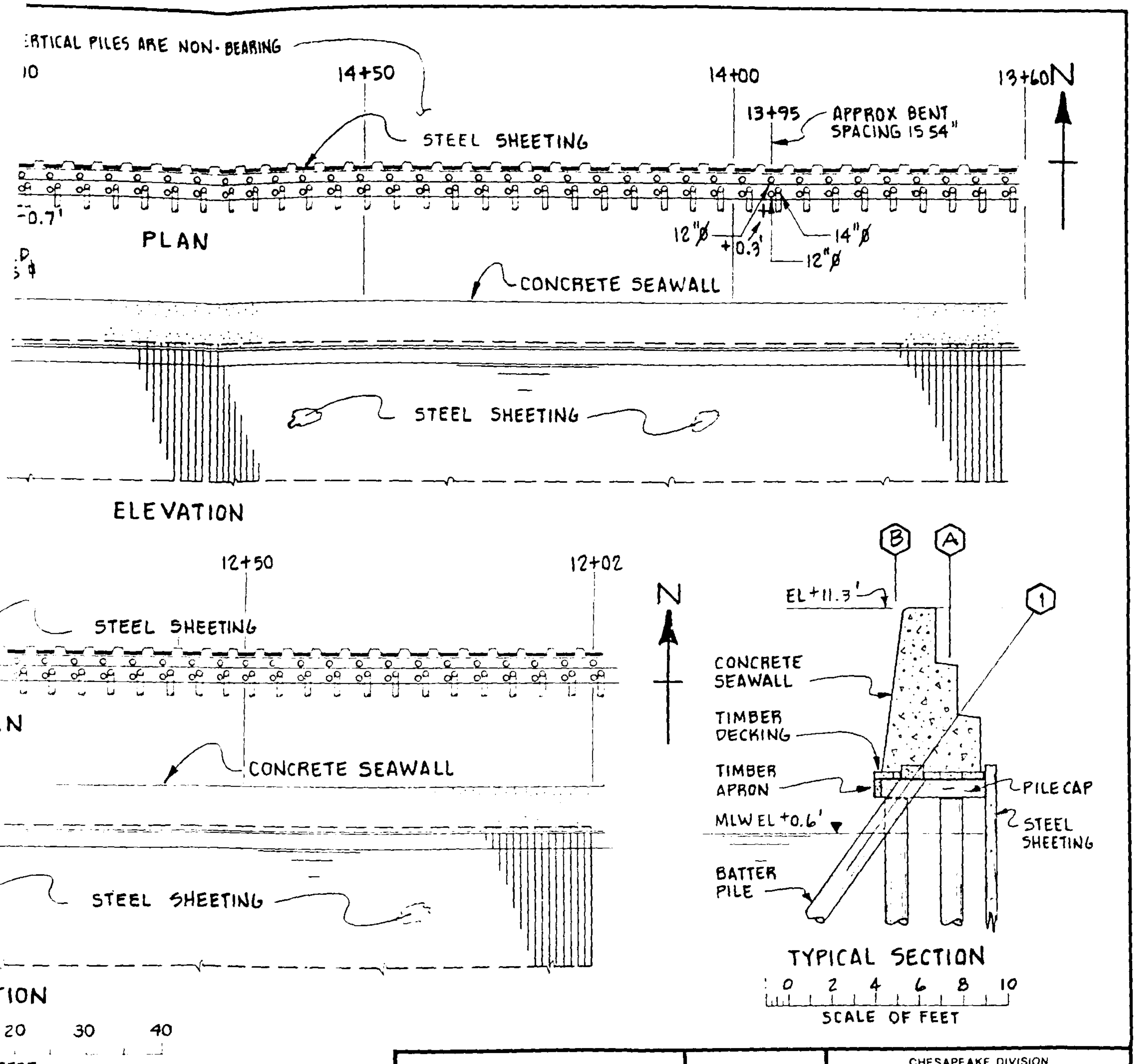
FIG. NO. 11



REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWG NOS. C-14724, C-14725, C-13046, C-13047 & C-13048.

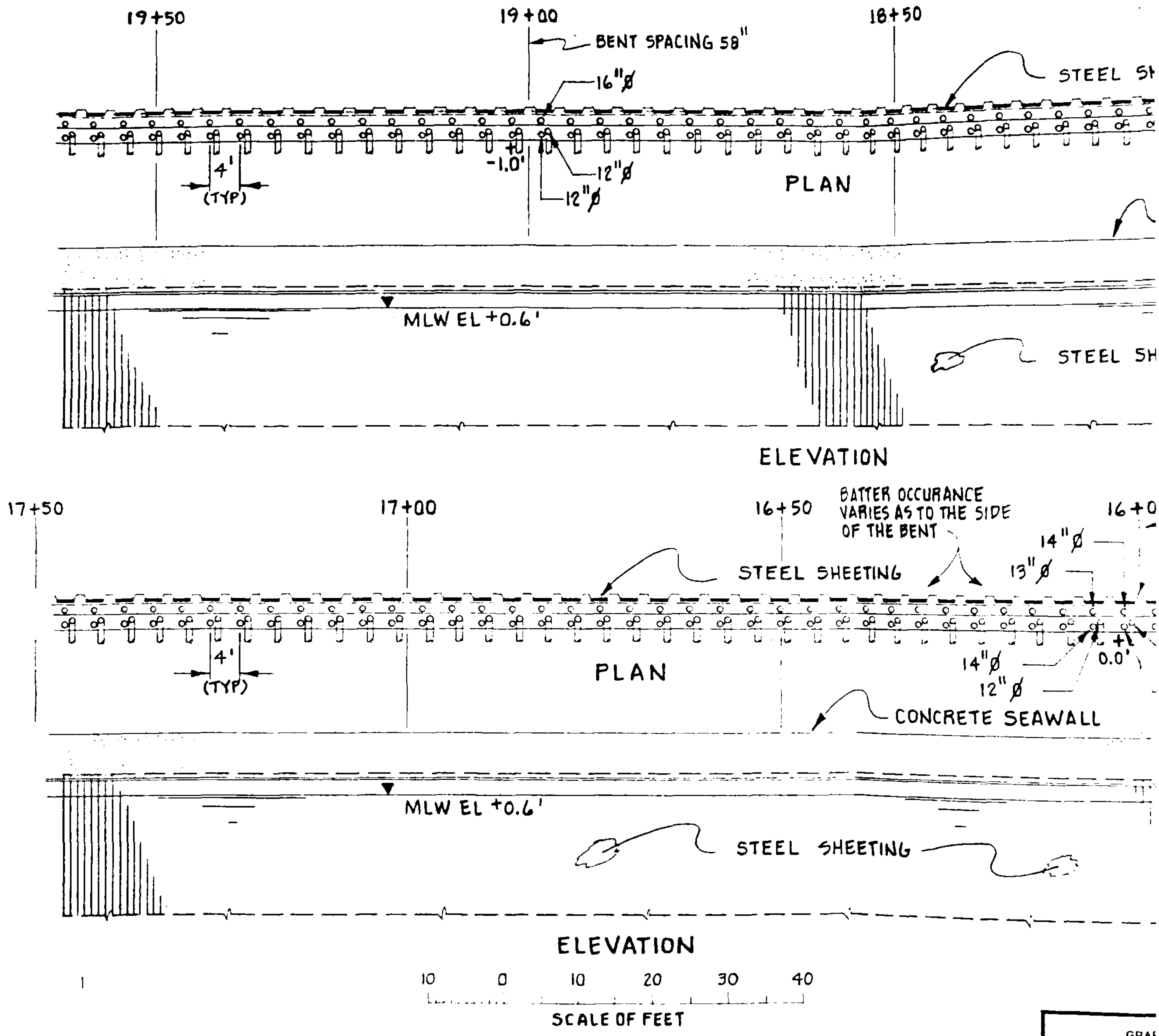
GR

AS



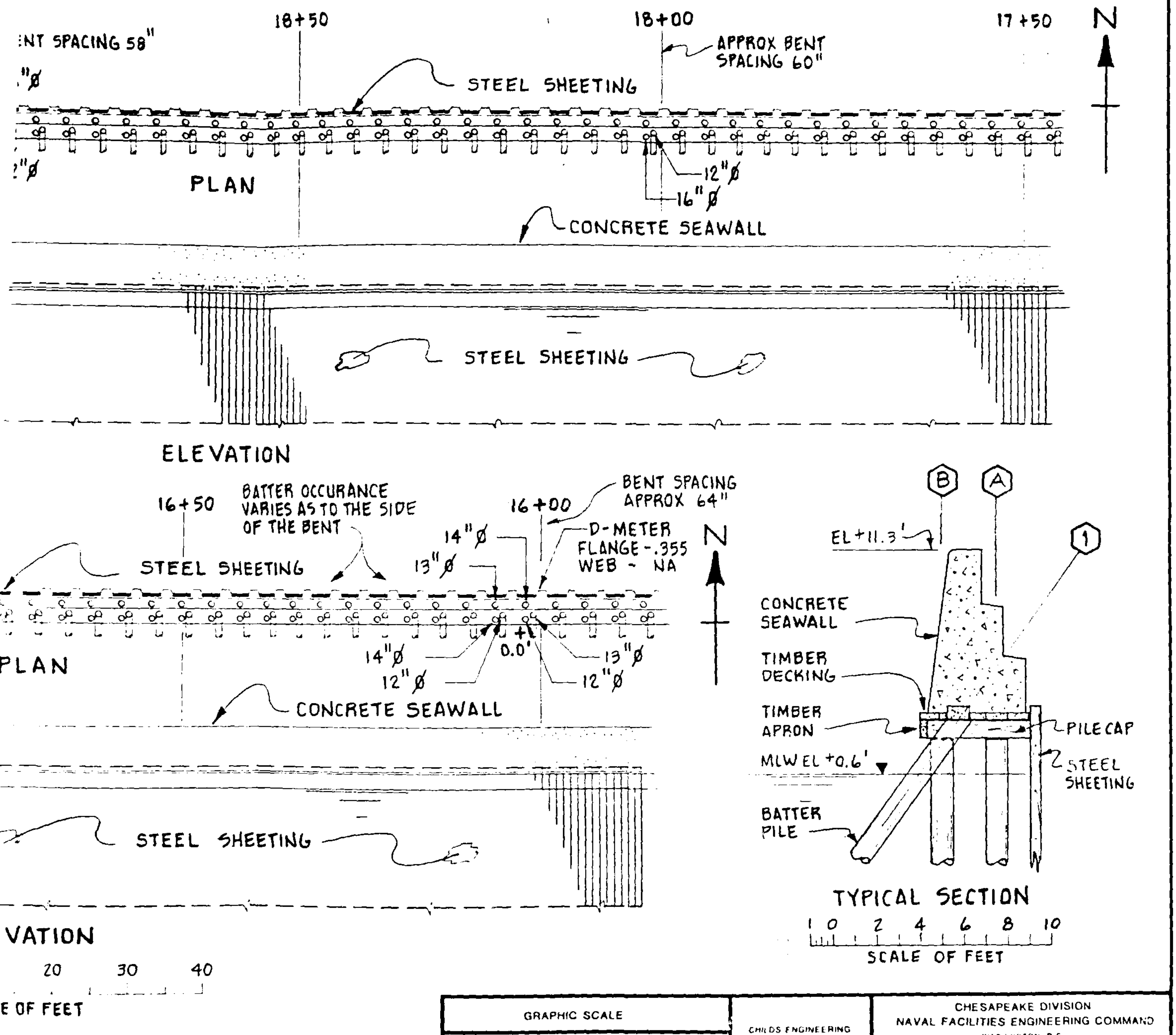
C-14725, C-13046, C-13047 & C-13048.

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D C	
AS SHOWN		PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA PA **EASTERN SEAWALL**	FIG NO 12



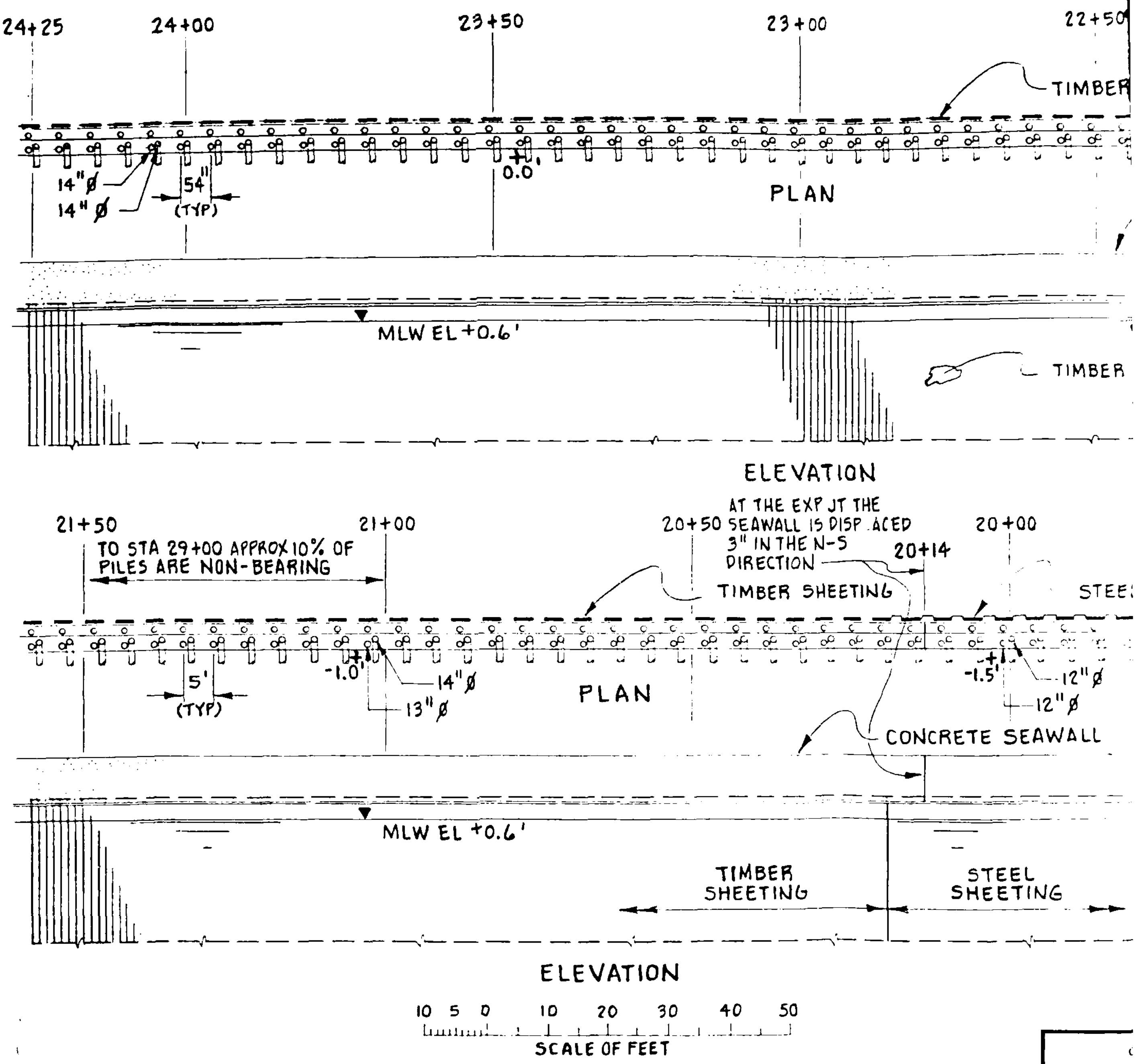
REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWGNOS. C-14724, C-14725, C-13046, C-13047 & C-13048.

AS

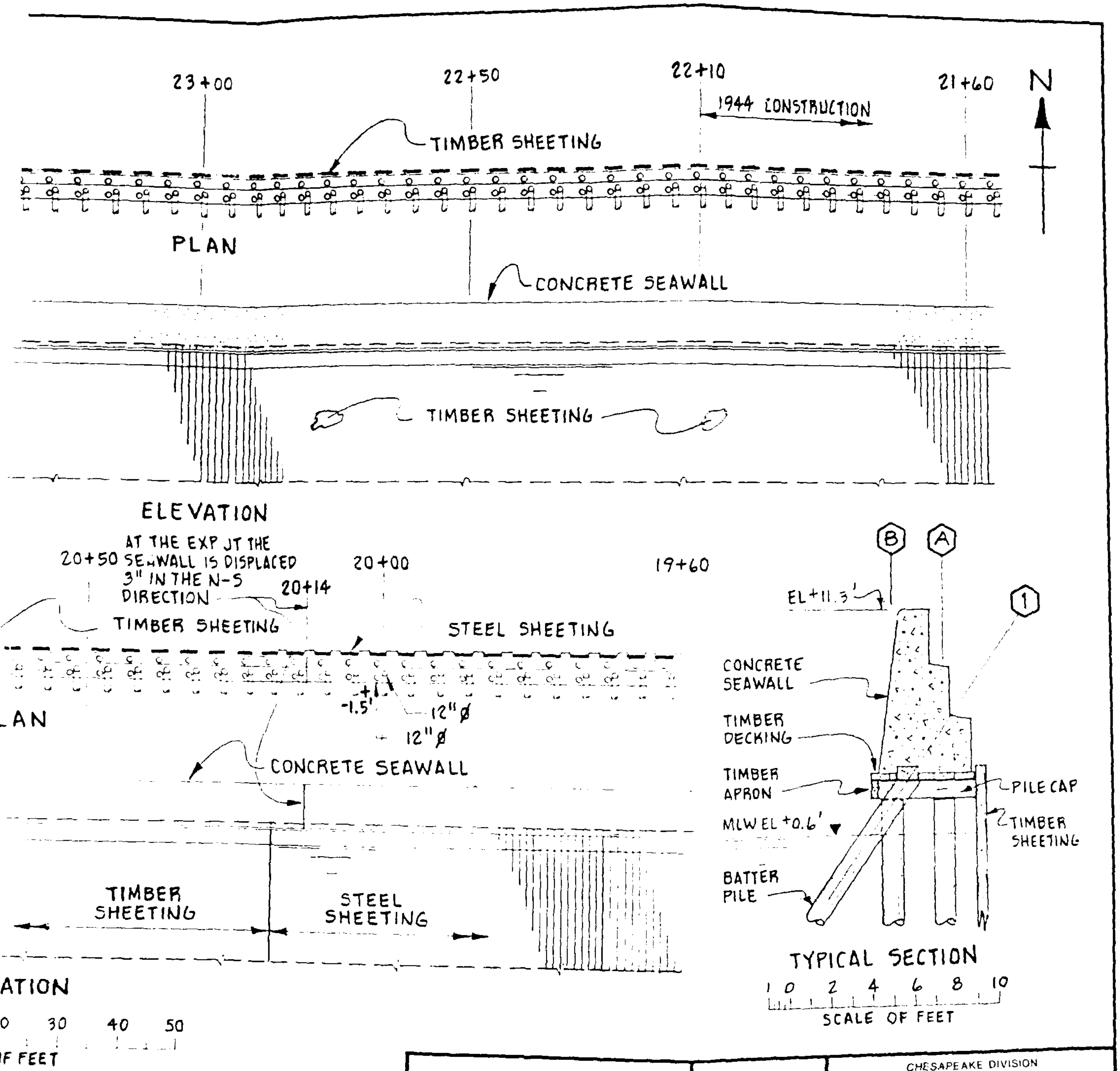


ES
4, C-14725, C-13046, C-13047 & C-13048.

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D.C.	
AS SHOWN		PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA PA **EASTERN SEAWALL**	FIG NO 13



REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWG NO. C-4824, C-4825, C-4826 & C-4827.



C-4825, C-4826 ₤ C-4827.

GRAPHIC SCALE

AS SHOWN

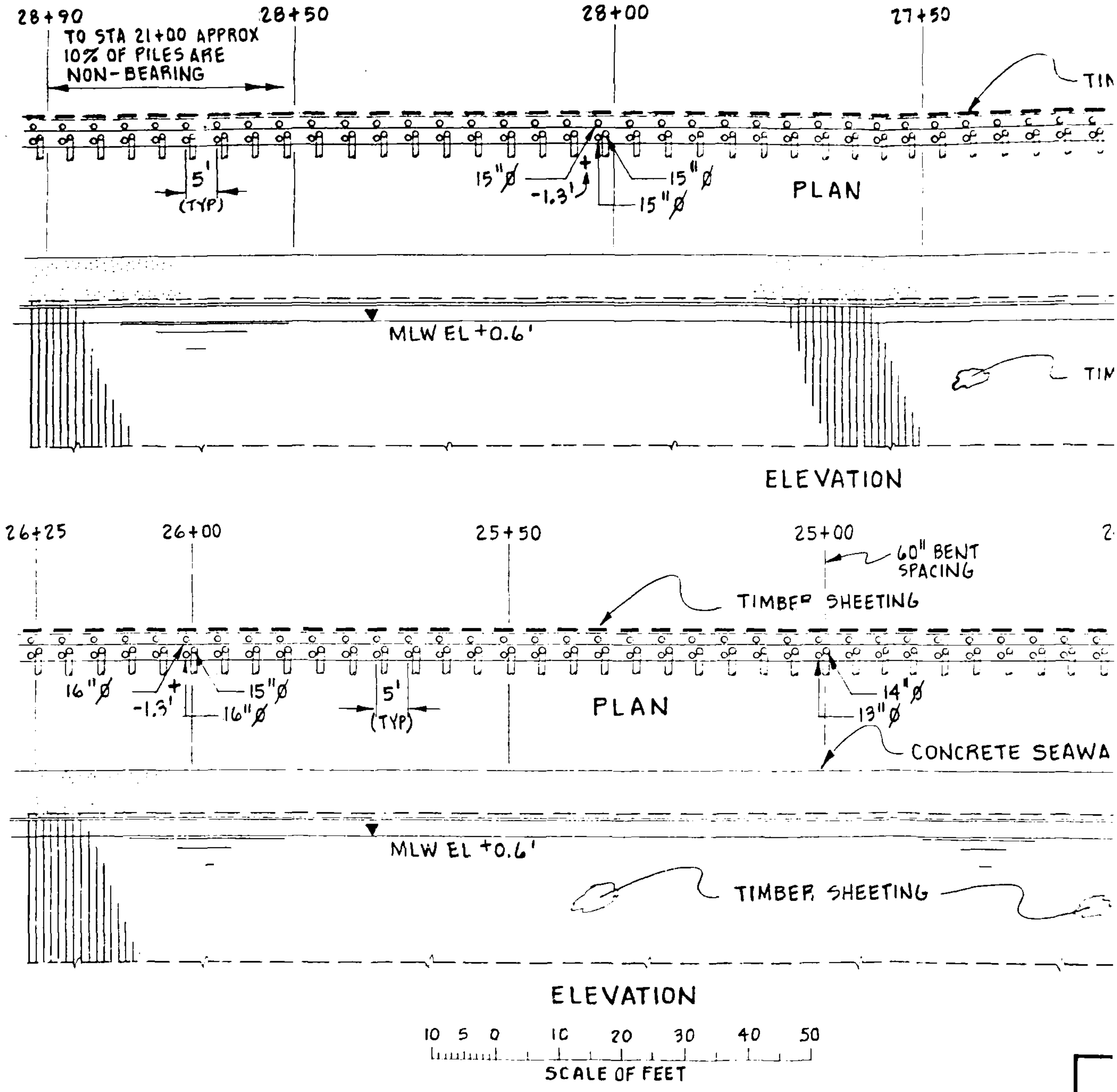
CHILDS ENGINEERING
CORPORATION
BOX 333
MEDFIELD MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D.C.

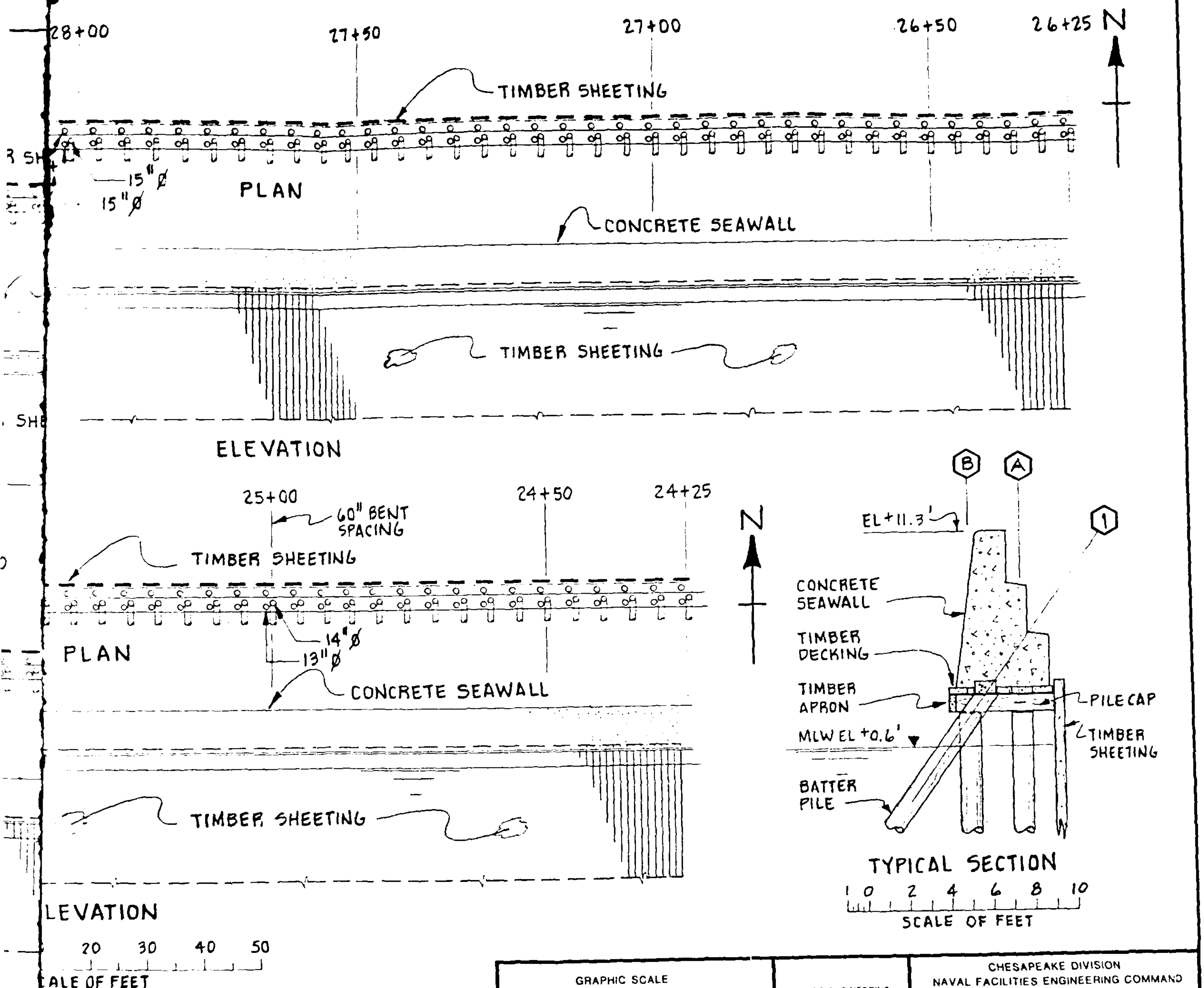
PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA PA

EASTERN SEAWALL

FIG NO
14

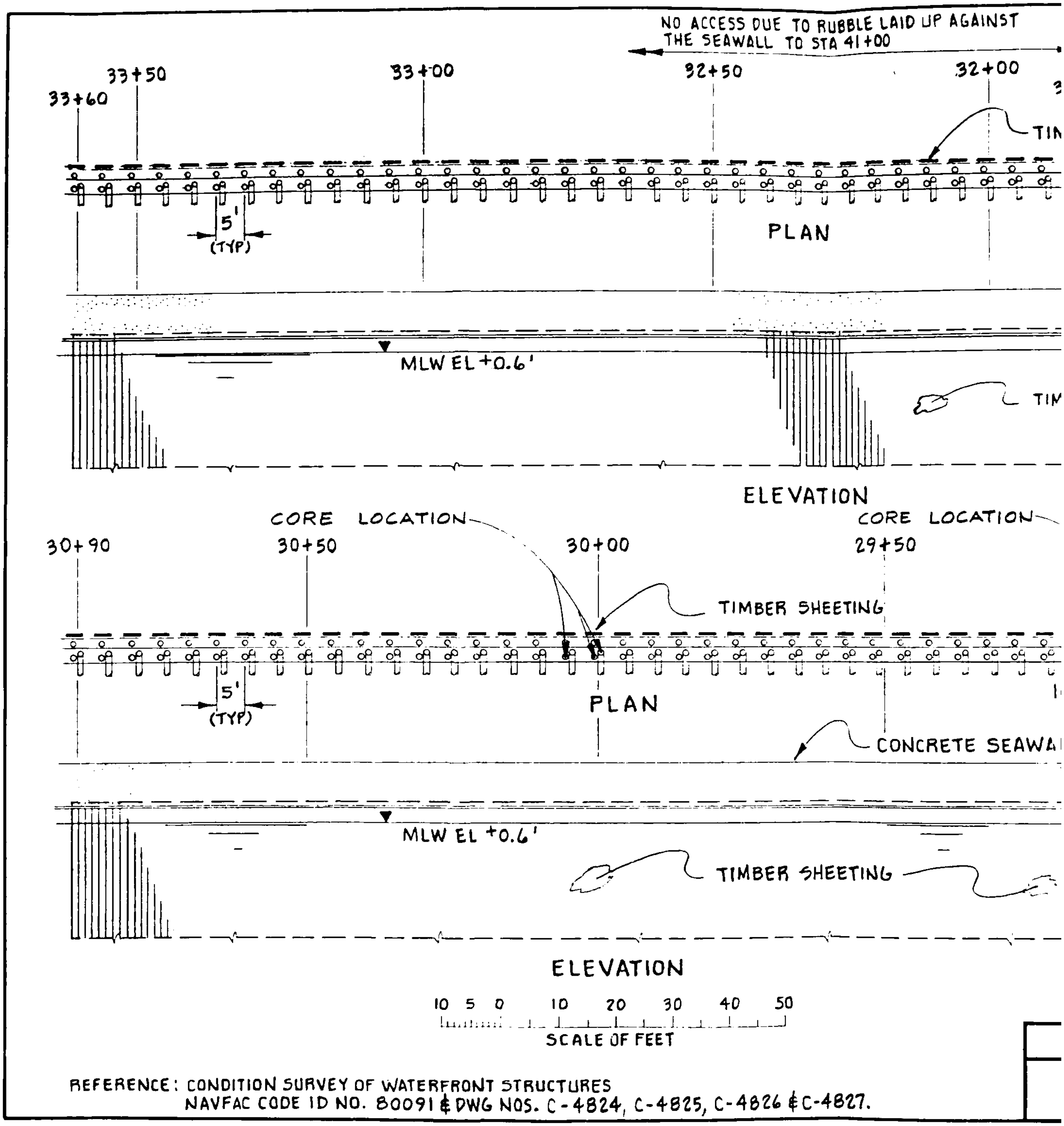


REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWG NOS. C-4824, C-4825, C-4826 & C-4827.

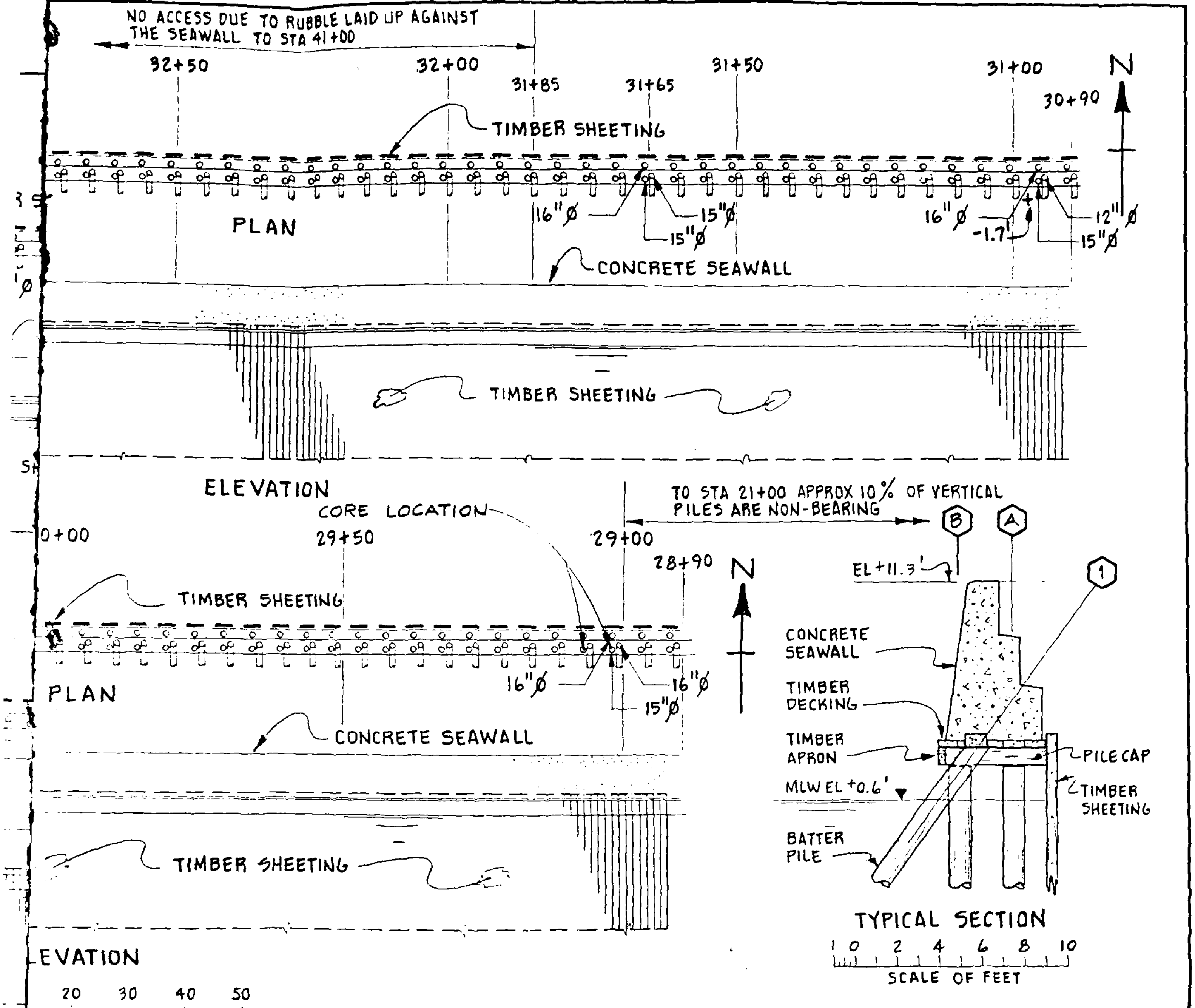


URES
4824, C-4825, C-4826 & C-4827.

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D.C.	
AS SHOWN		PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA PA EASTERN SEAWALL	FIG NO 15

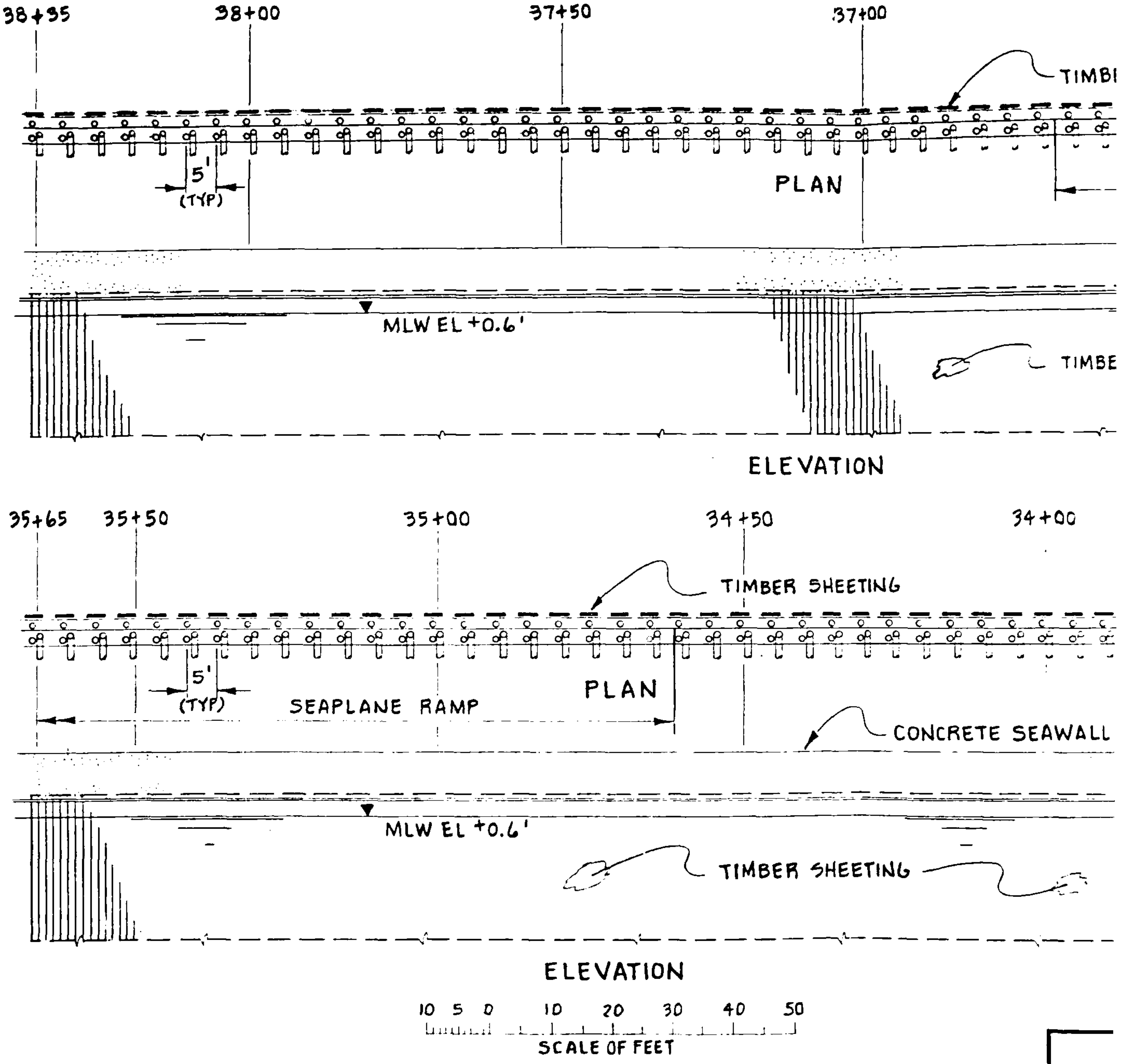


REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWG NOS. C-4824, C-4825, C-4826 & C-4827.

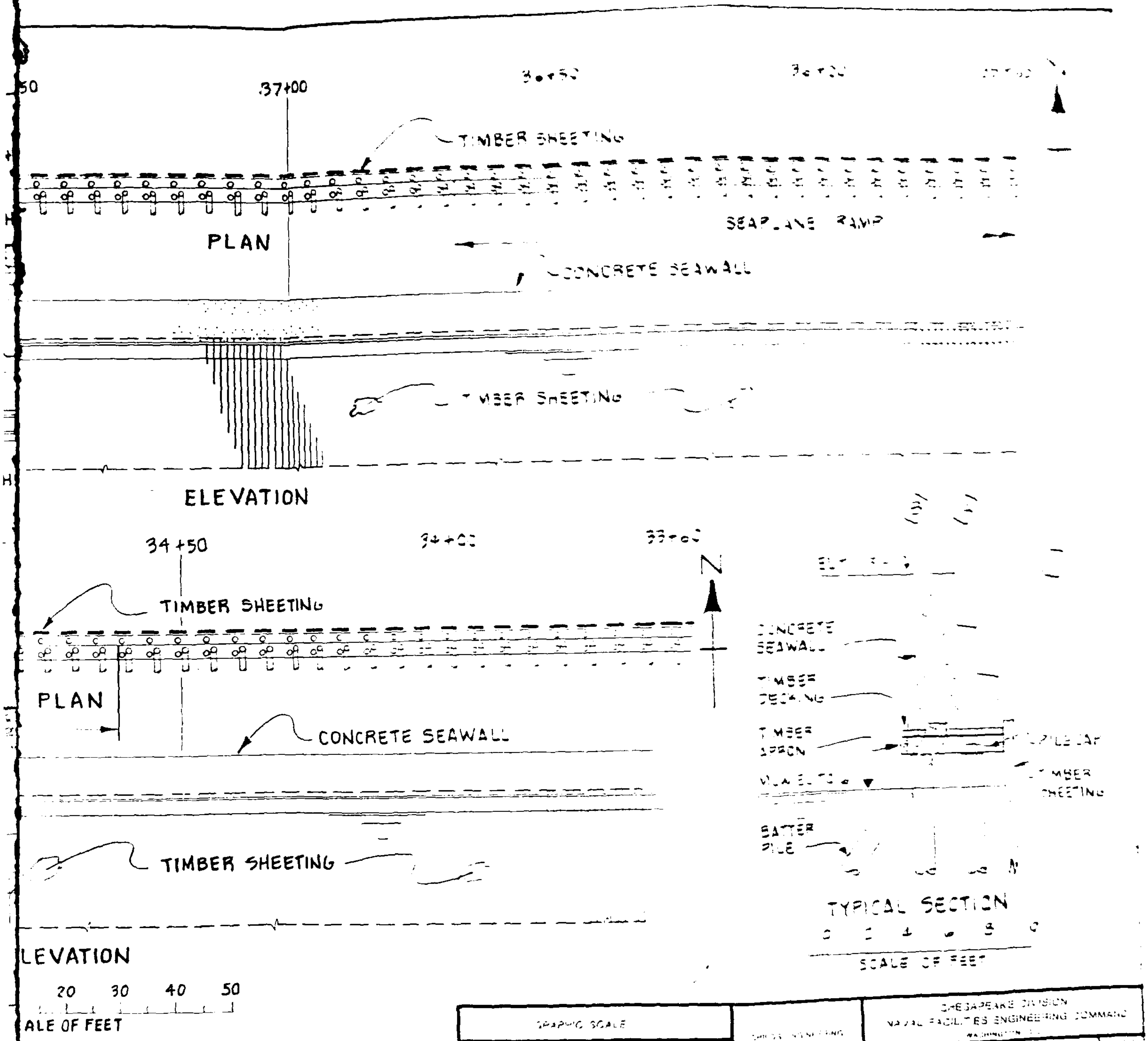


IRES
4824, C-4825, C-4826 & C-4827.

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C.	
AS SHOWN		PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA **EASTERN SEAWALL**	FIG NO. 16



REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWG NOS. C-4824, C-4825, C-4826, C-4827 & C-4606.



URES
4824, C-4825, C-4826, C-4827 & C-4606.

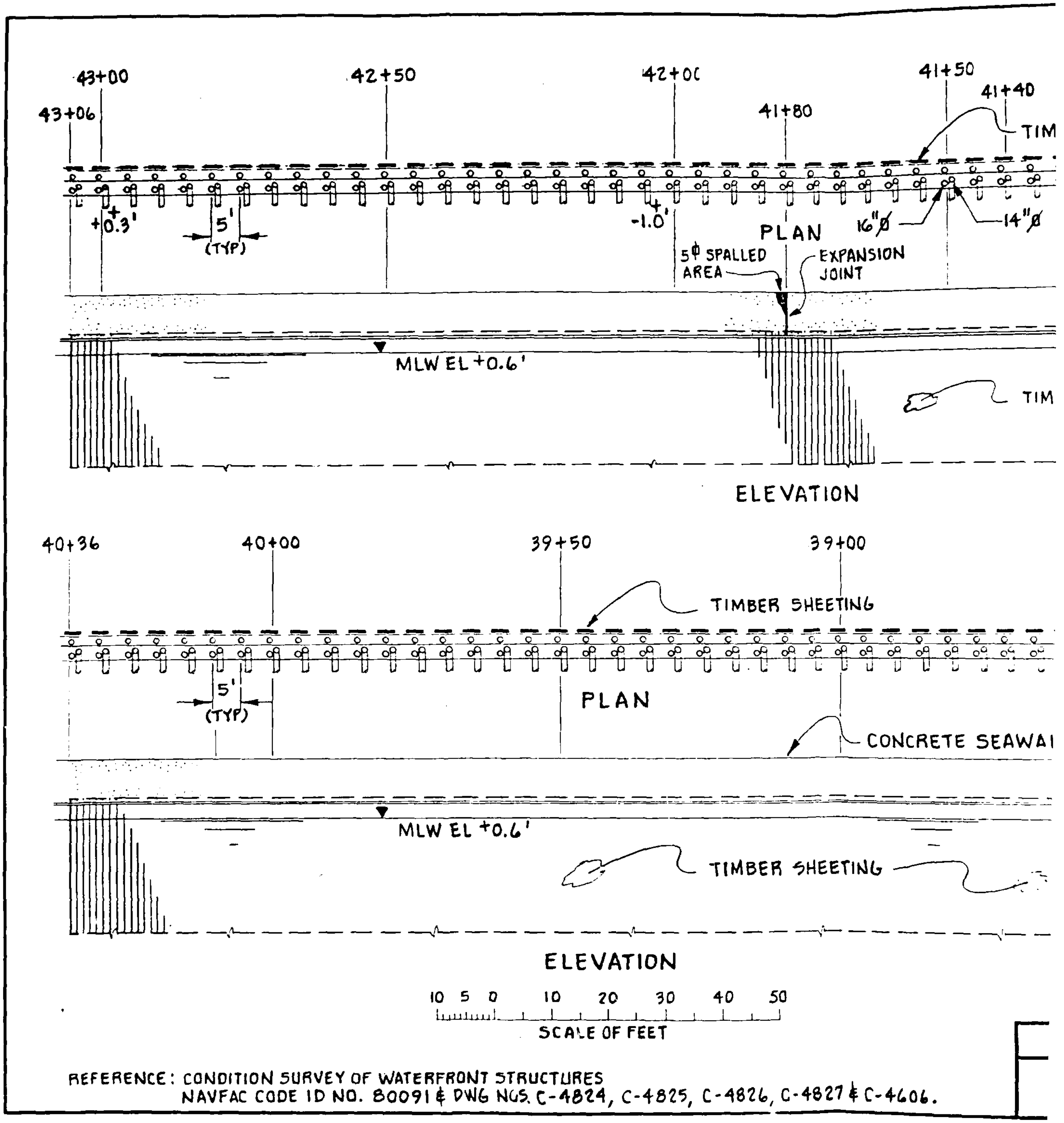
GRAPHIC SCALE

AS SHOWN

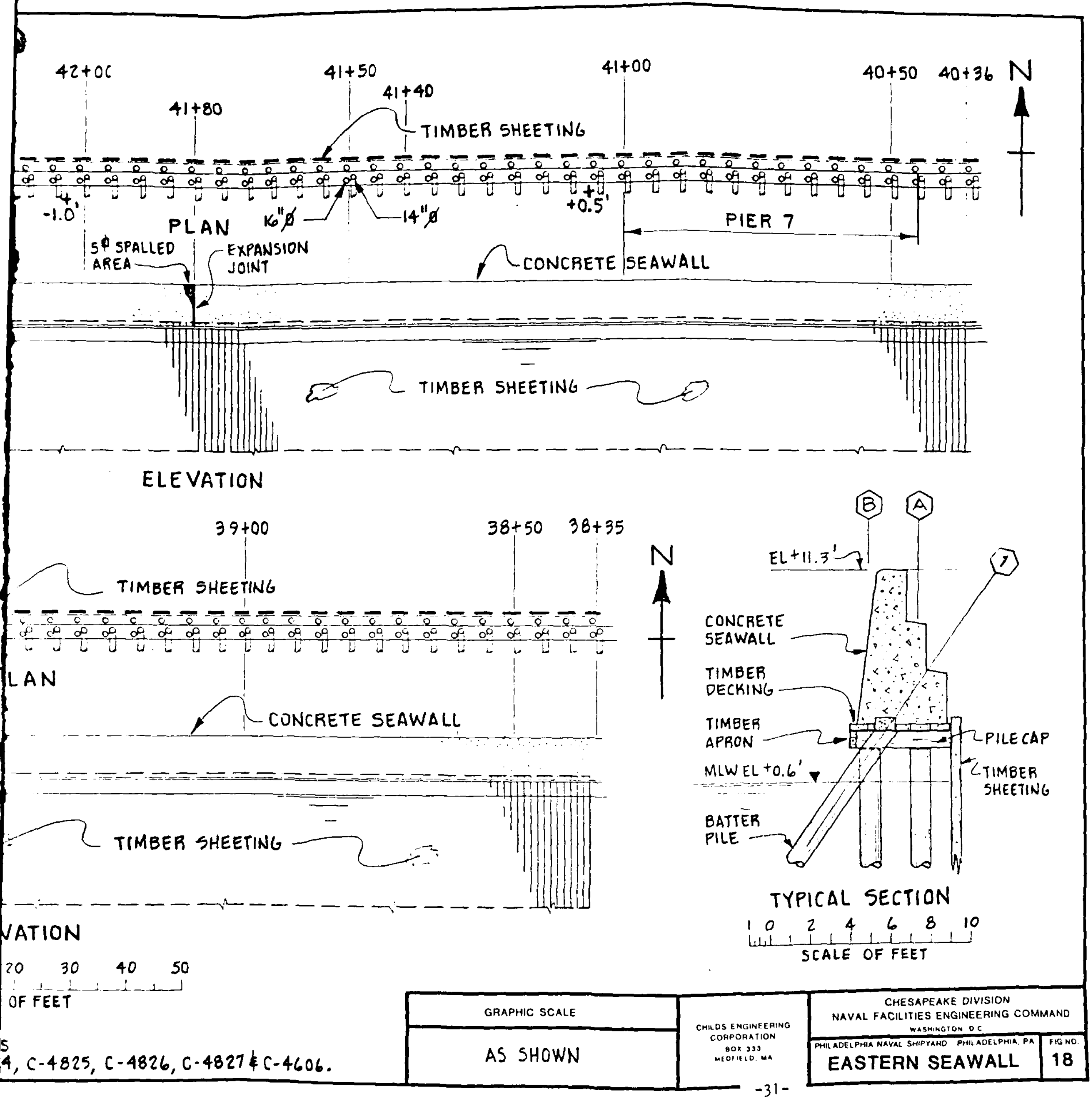
CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D.C.

PHILADELPHIA NAVAL SHIPYARD

EASTERN SEAWALL



REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWG NOS. C-4824, C-4825, C-4826, C-4827 & C-4606.



S
4, C-4825, C-4826, C-4827 & C-4606.

GRAPHIC SCALE

AS SHOWN

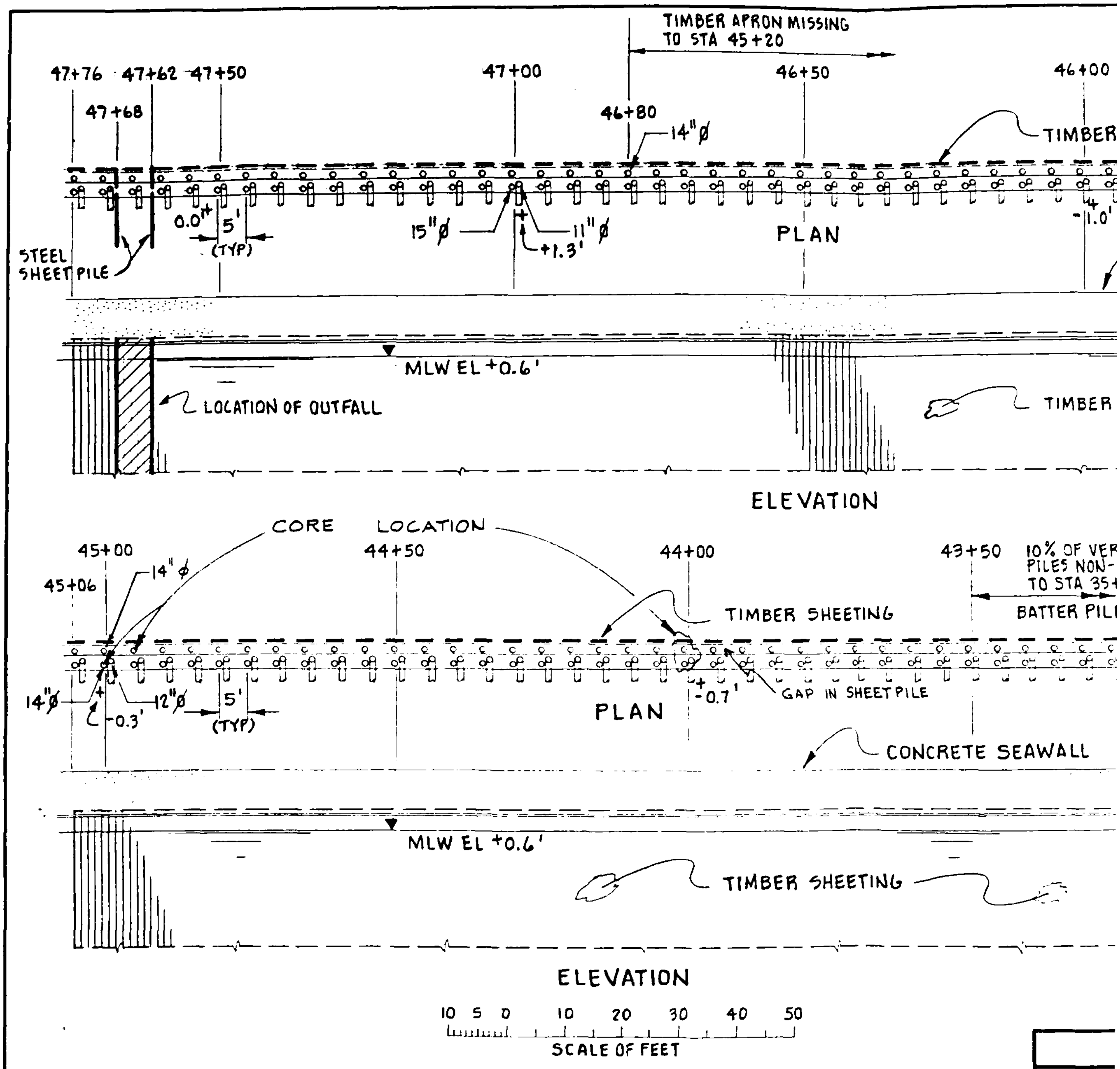
CHILDS ENGINEERING CORPORATION
BOX 333
MEDFIELD, MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D.C.

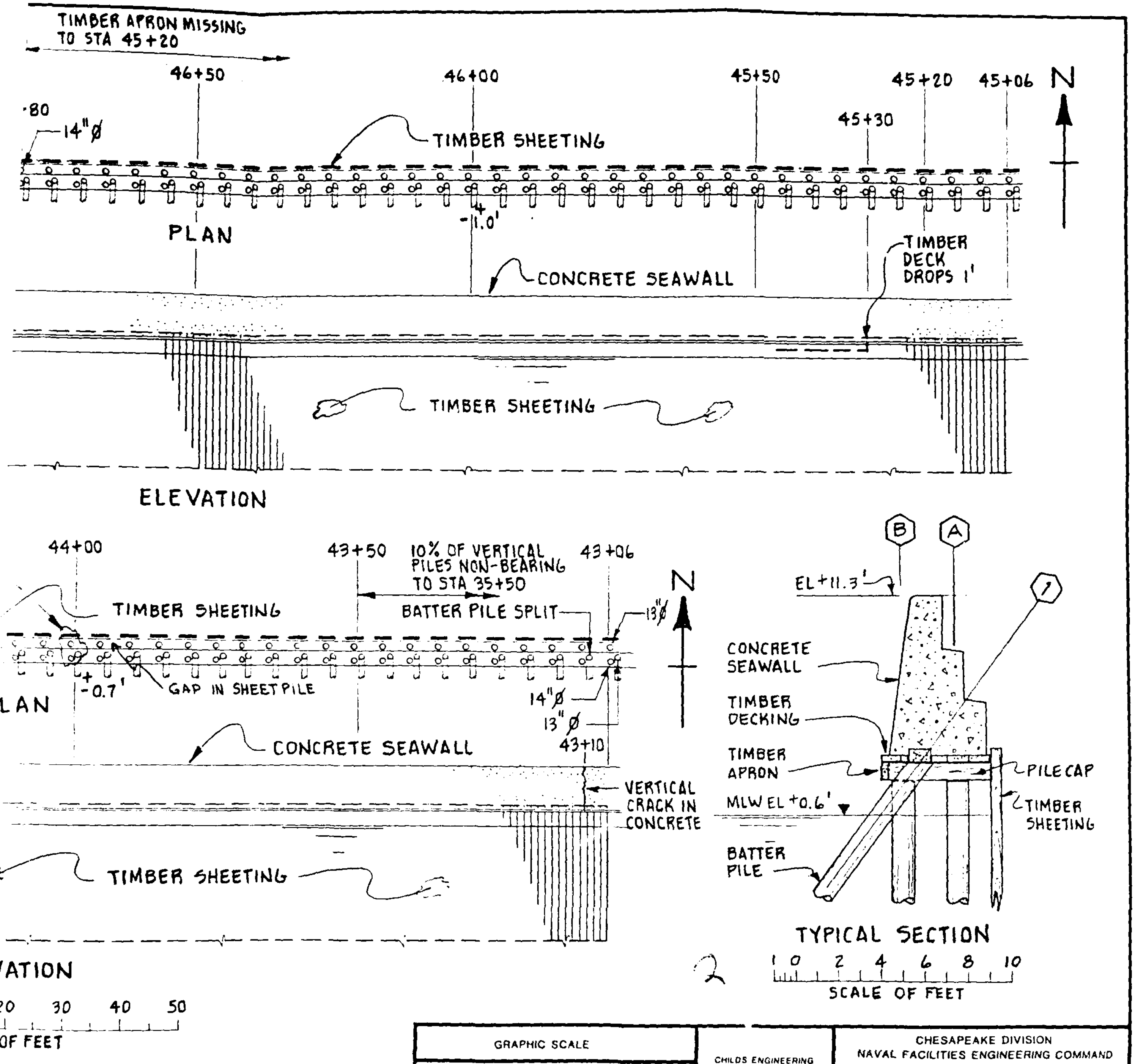
PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA

EASTERN SEAWALL

FIG NO.
18

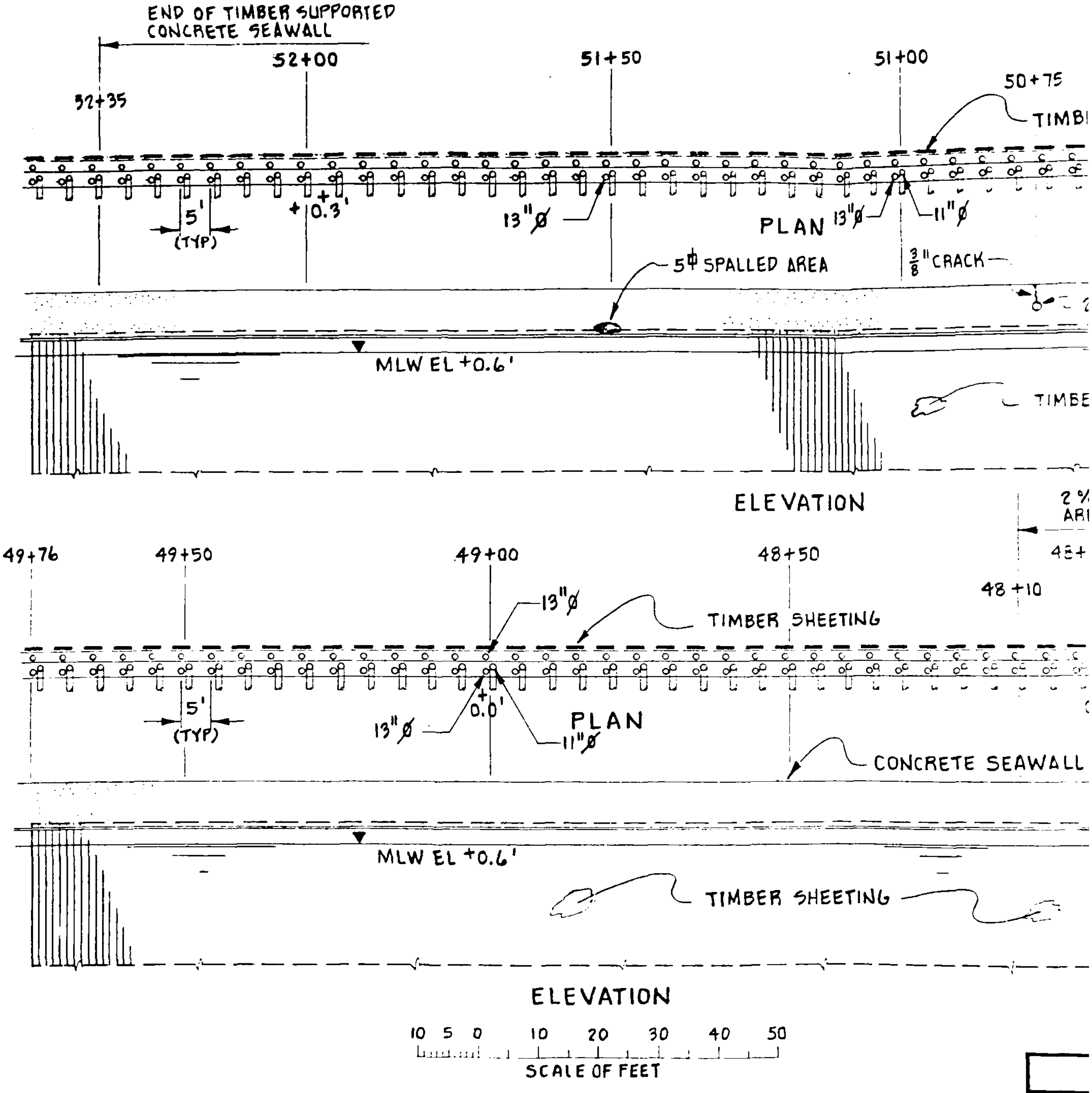


REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWG NOS. C-4824, C-4825, C-4826, C-4827 & C-4606.

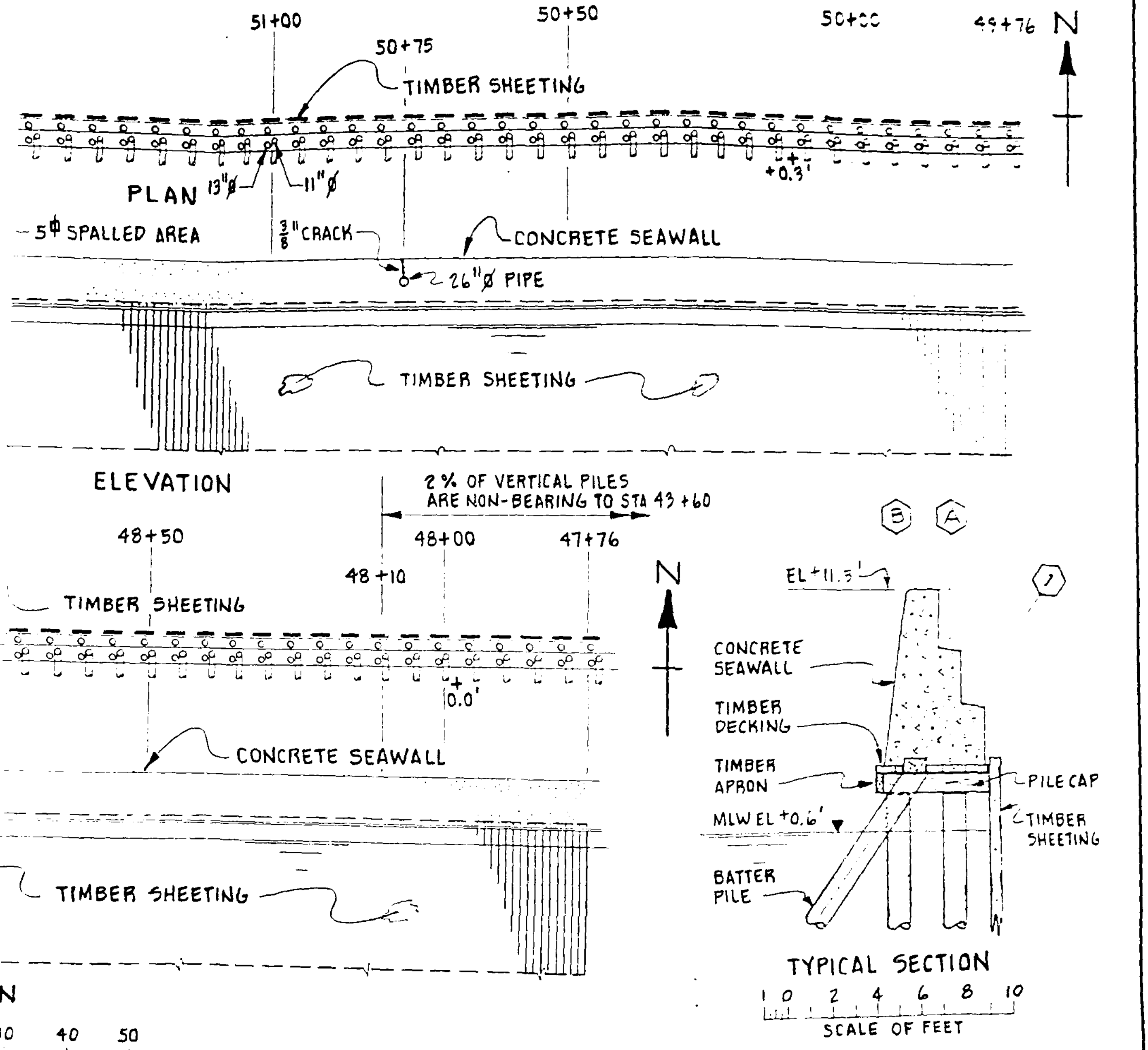


, C-4825, C-4826, C-4827 & C-4606.

GRAPHIC SCALE AS SHOWN	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D C PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA EASTERN SEAWALL FIG NO 19



REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWG NOS. C-4824, C-4825, C-4826, C-4827 & C-4606.



325, C-4826, C-4827 & C-4606.

GRAPHIC SCALE

AS SHOWN

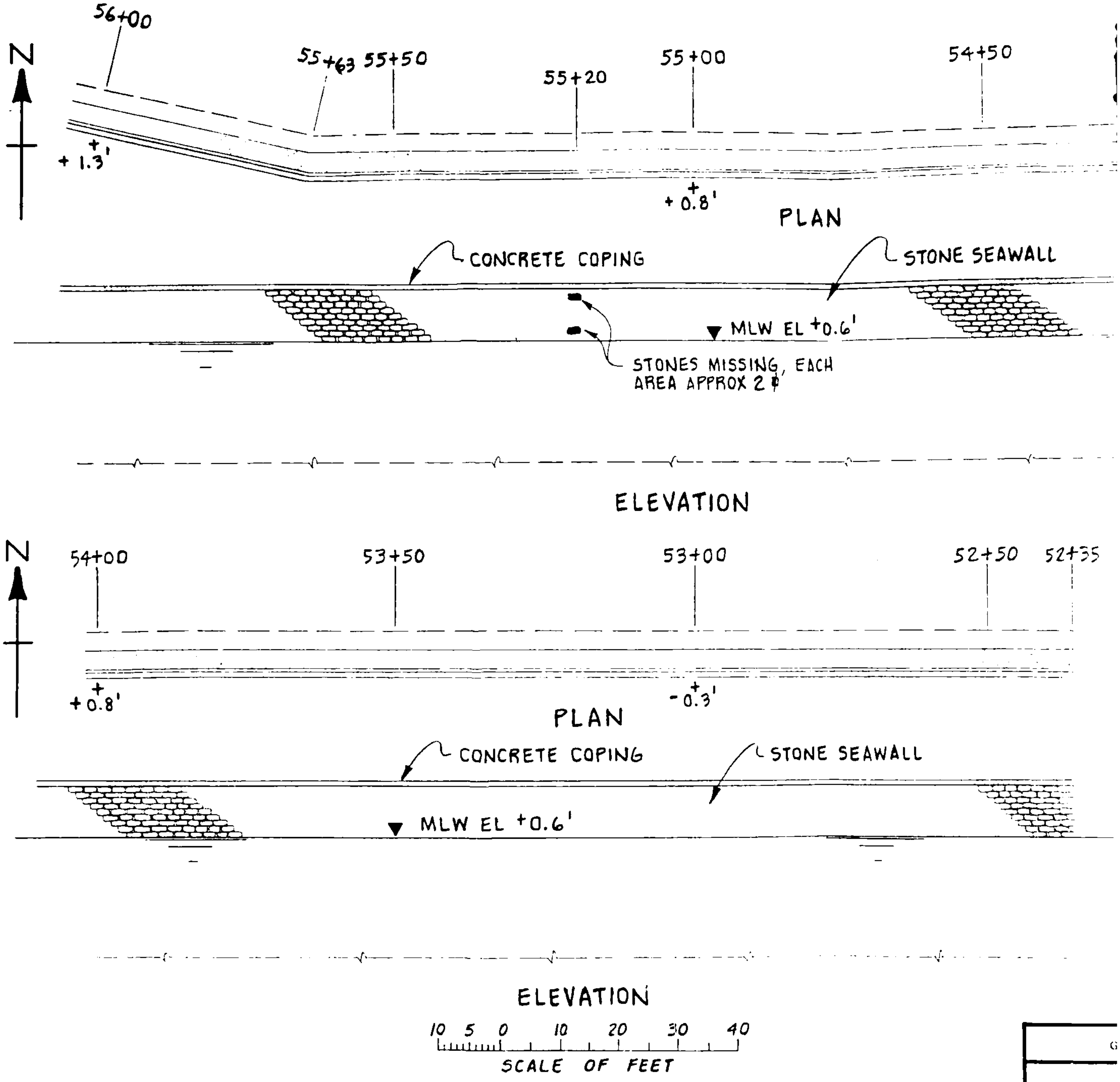
CHILDS ENGINEERING
CORPORATION
BOX 333
MEDFIELD MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D.C.

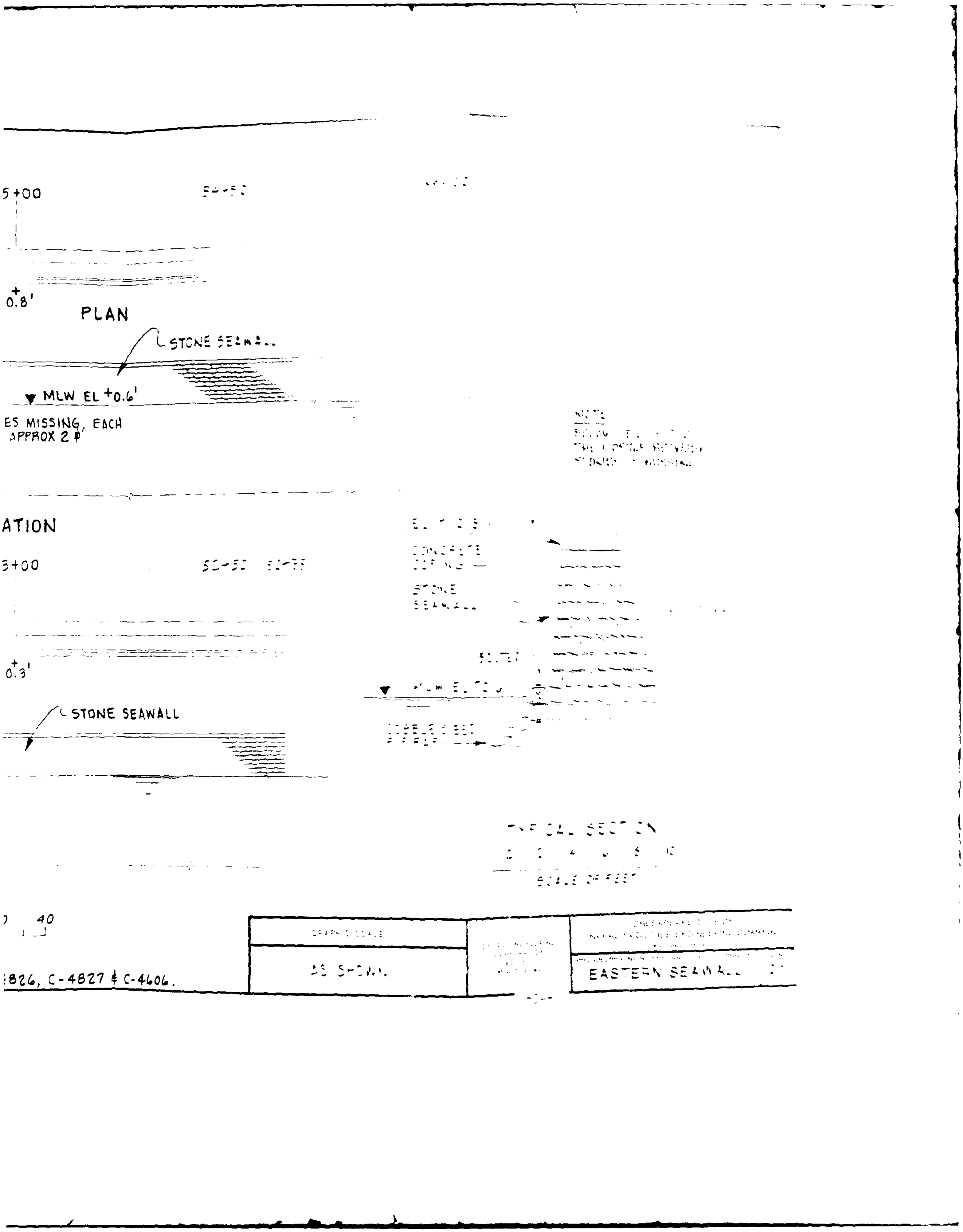
PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA PA

EASTERN SEAWALL

FIG NO
20



REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWG NOS. C-4824, C-4825, C-4826, C-4827 & C-4606.

5+00
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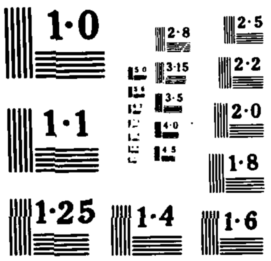
40
1826, C-4827 & C-4606.
GRAPHIC SCALE
AS SHOWN
EASTERN SEAWALL

AD-A168 464 UNDERWATER FACILITIES INSPECTIONS AND ASSESSMENTS AT 2/3
PHILADELPHIA NAVAL S..(U) CHILDS ENGINEERING CORP
MEDFIELD MA OCT 83 CHES/NAVFAC-FPO-1-83(48)-1
UNCLASSIFIED N62477-81-C-0448 F/G 13/2 NL


1·0
2·8
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3·15
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1·1
4·0
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1·25
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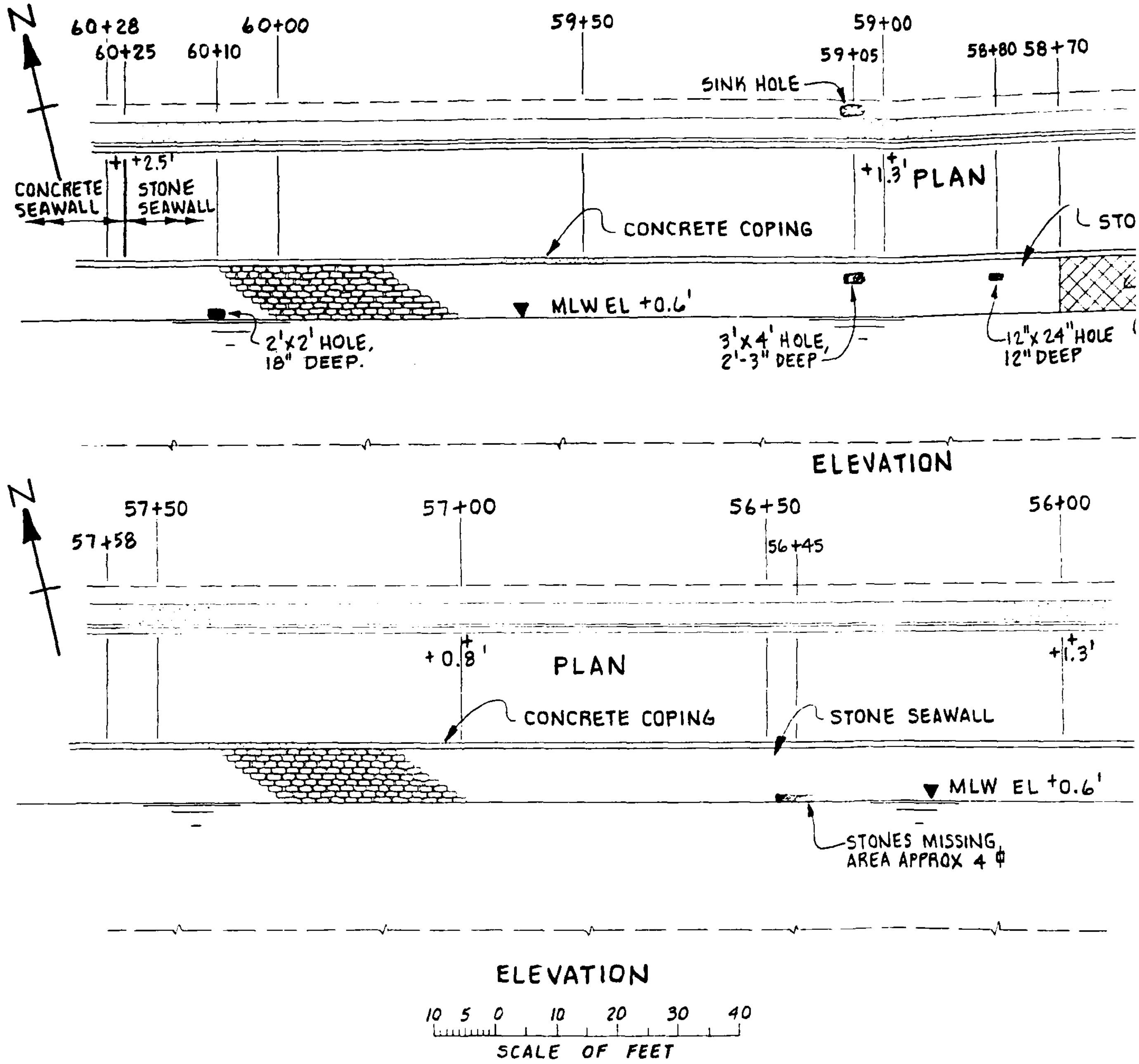
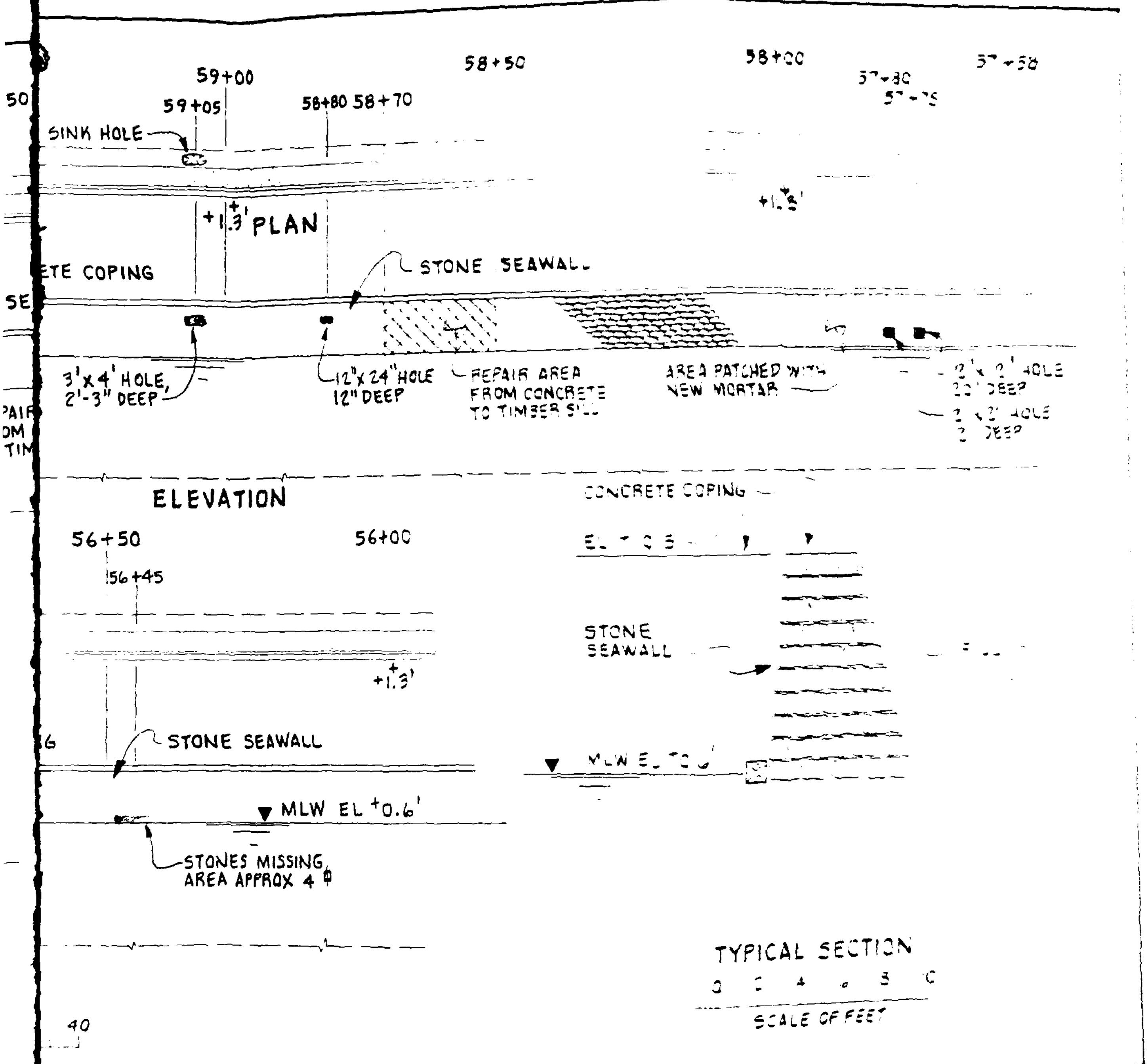


REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWG NOS C-1501, C-1502 & C-1503.



GRAPHIC SCALE

AS SHOWN

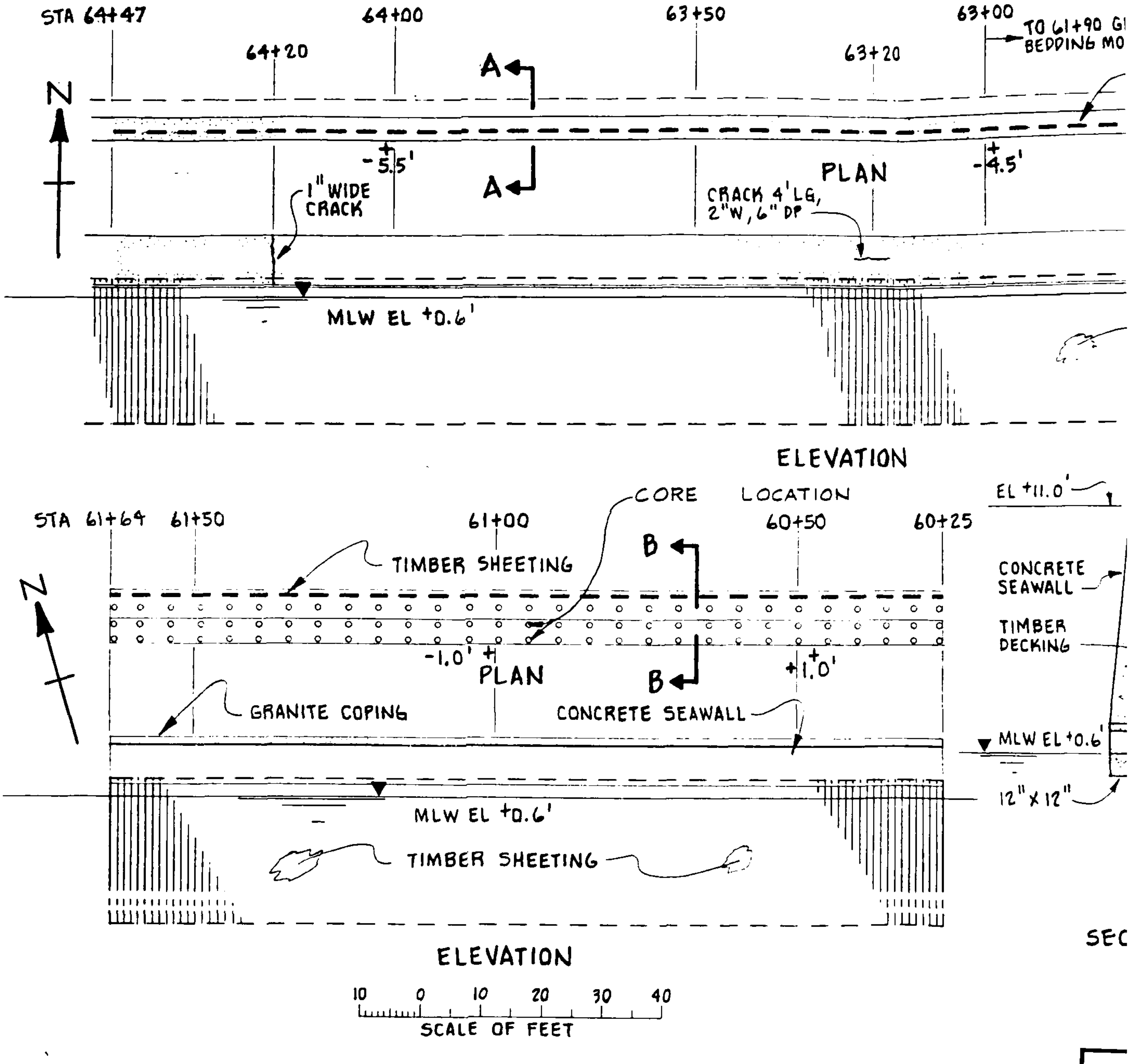
CHILDS ENGINEERING CORPORATION
BOX 333
MEDFIELD, MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON, D.C.

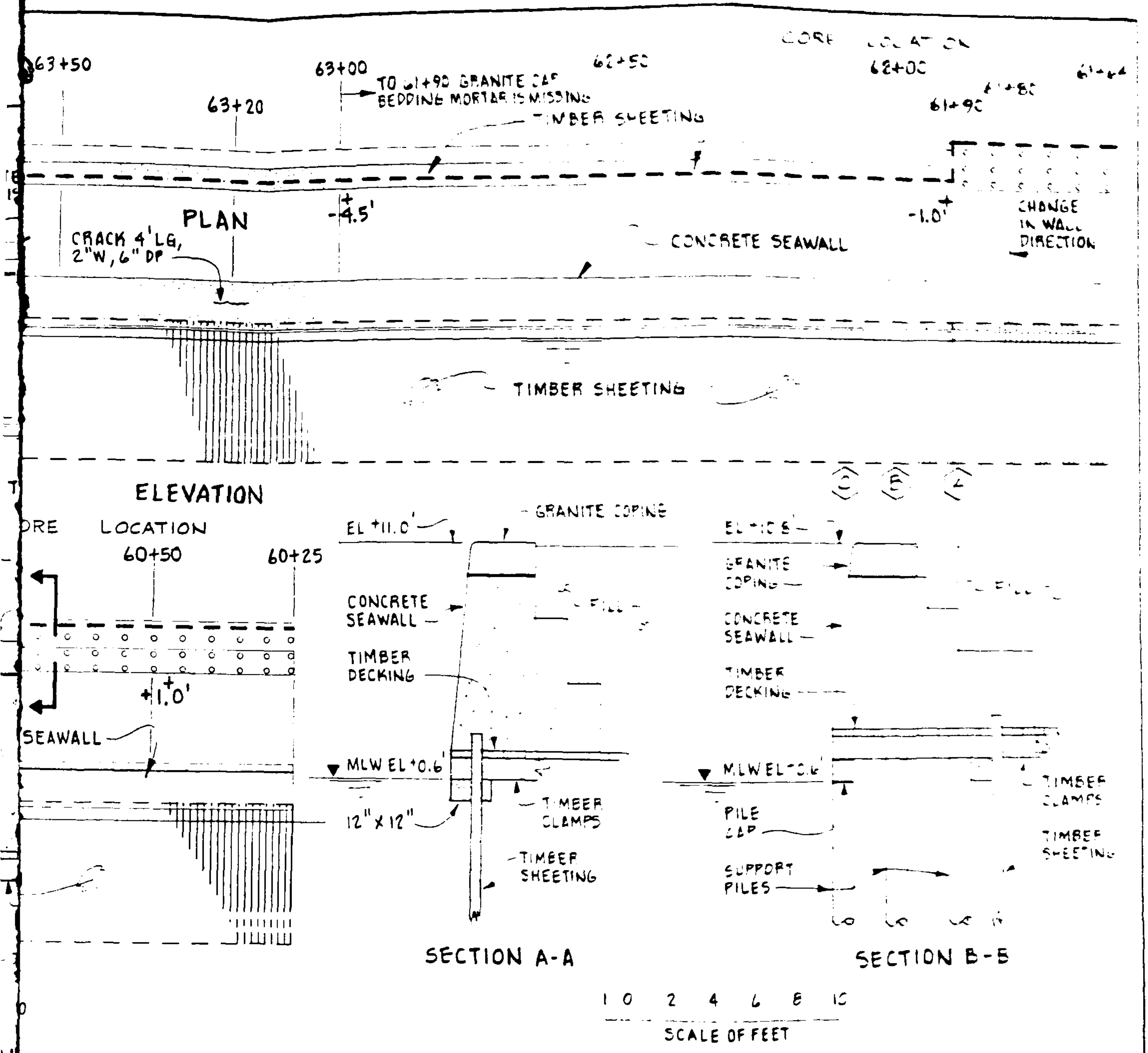
PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA

EASTERN SEAWALL

FIG. NO. 22



REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES
NAVFAC CODE ID NO. 80091 & DWG NOS. C-1501, C-1502 & C-1503.



ES
C-1503.

GRAPHIC SCALE

AS SHOWN

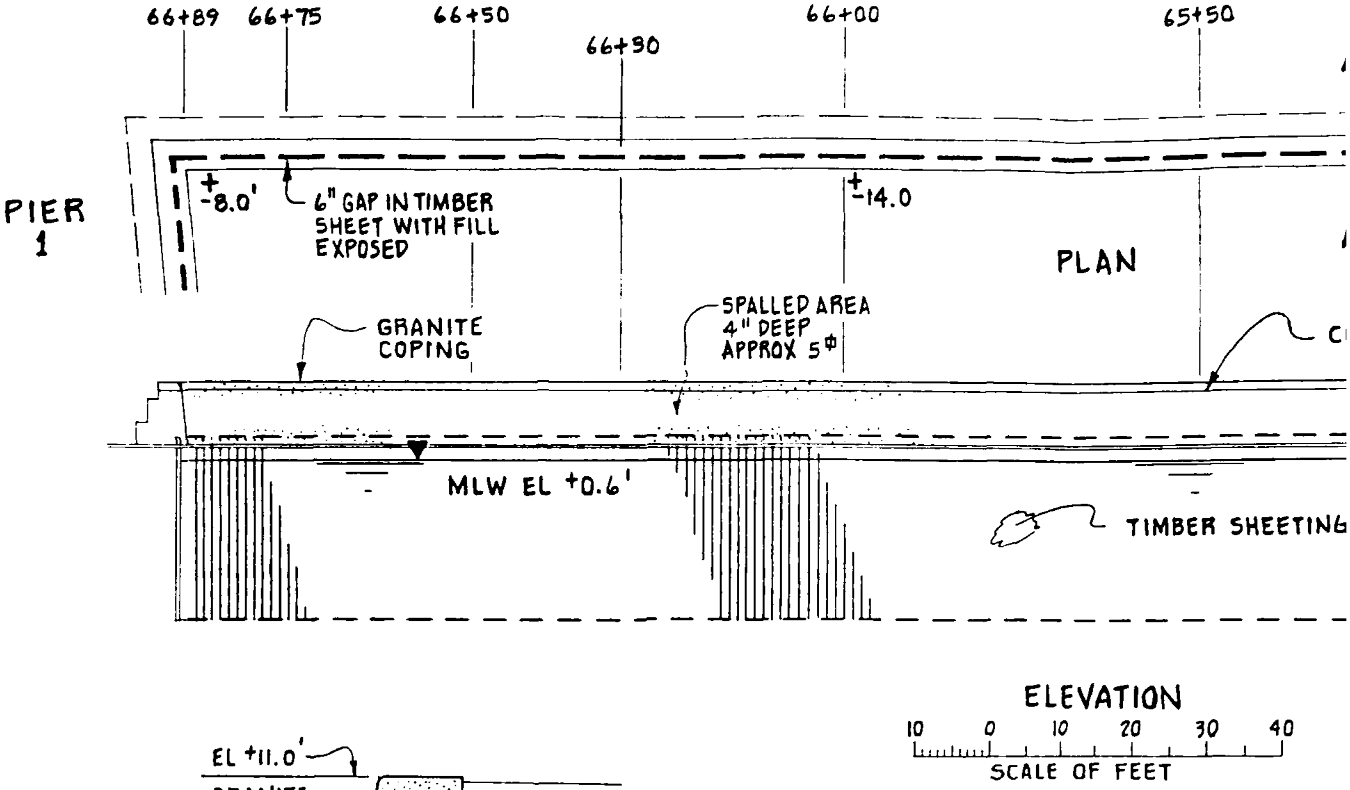
CHILDS ENGINEERING CORPORATION
BOX 333
MEDFIELD, MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON, D.C.

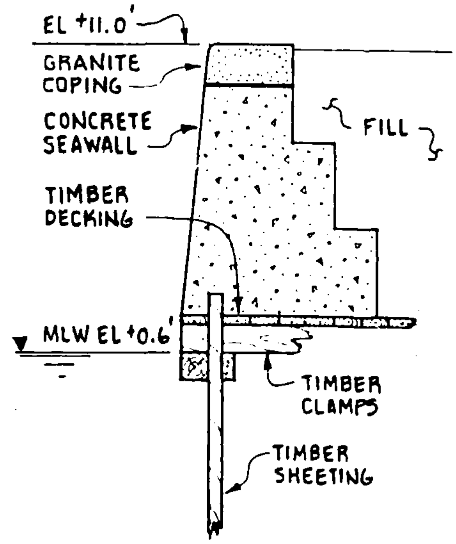
PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA

EASTERN SEAWALL

FIG. NO. 23

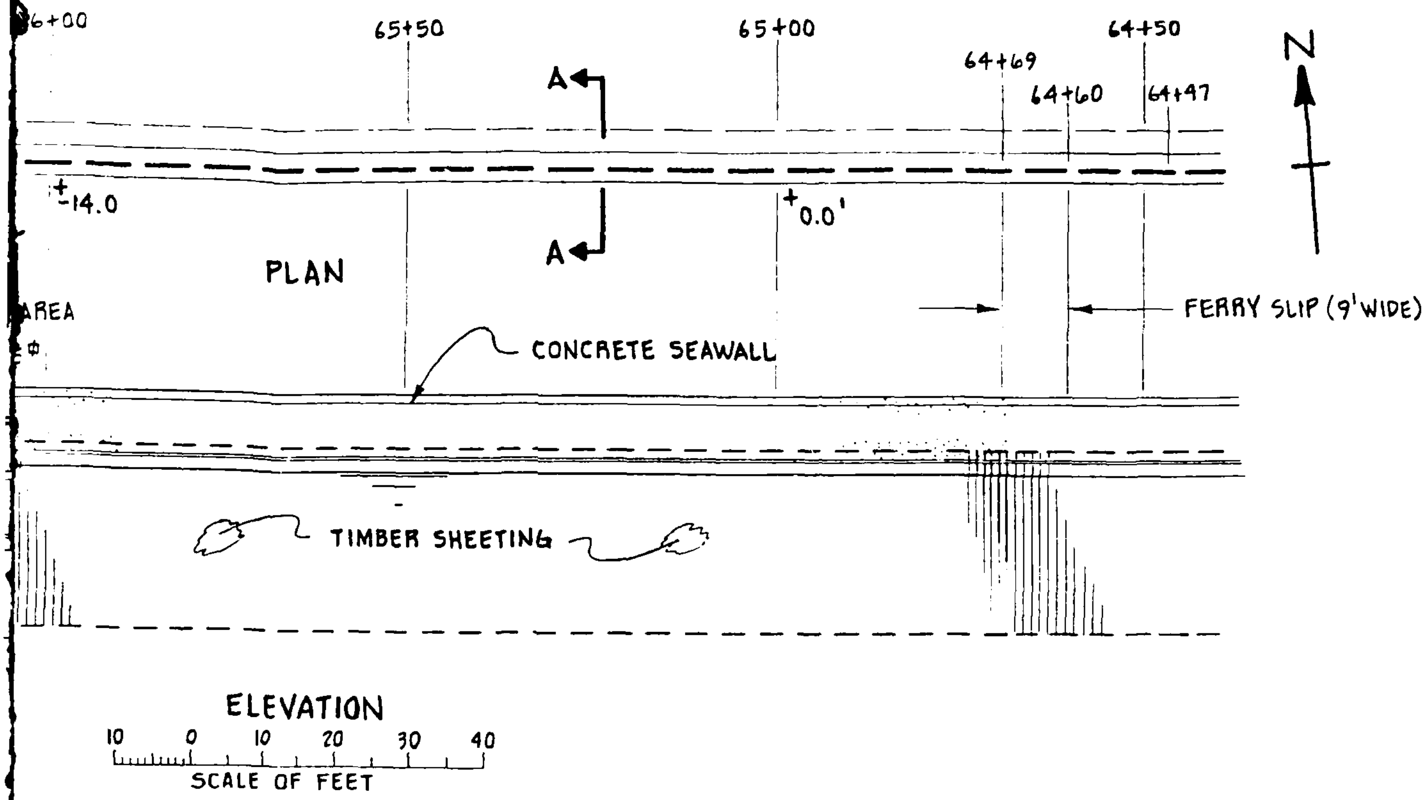


ELEVATION



SECTION A-A

REFERENCE : DWG NOS. P.W. B-1779, P.W. B-1780, P.W. B-1781, P.W. C-5039, P.W. C-13535 & P.W. C-25460.



NG NOS. P.W. B-1779, P.W. B-1780,
C-5039, P.W. C-13535 & P.W. C-25460.

GRAPHIC SCALE

AS SHOWN

CHILDS ENGINEERING
CORPORATION
BOX 333
MEDFIELD, MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D.C.

PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA

EASTERN SEAWALL

FIG NO. 24

4.1.2 OBSERVED INSPECTION CONDITION

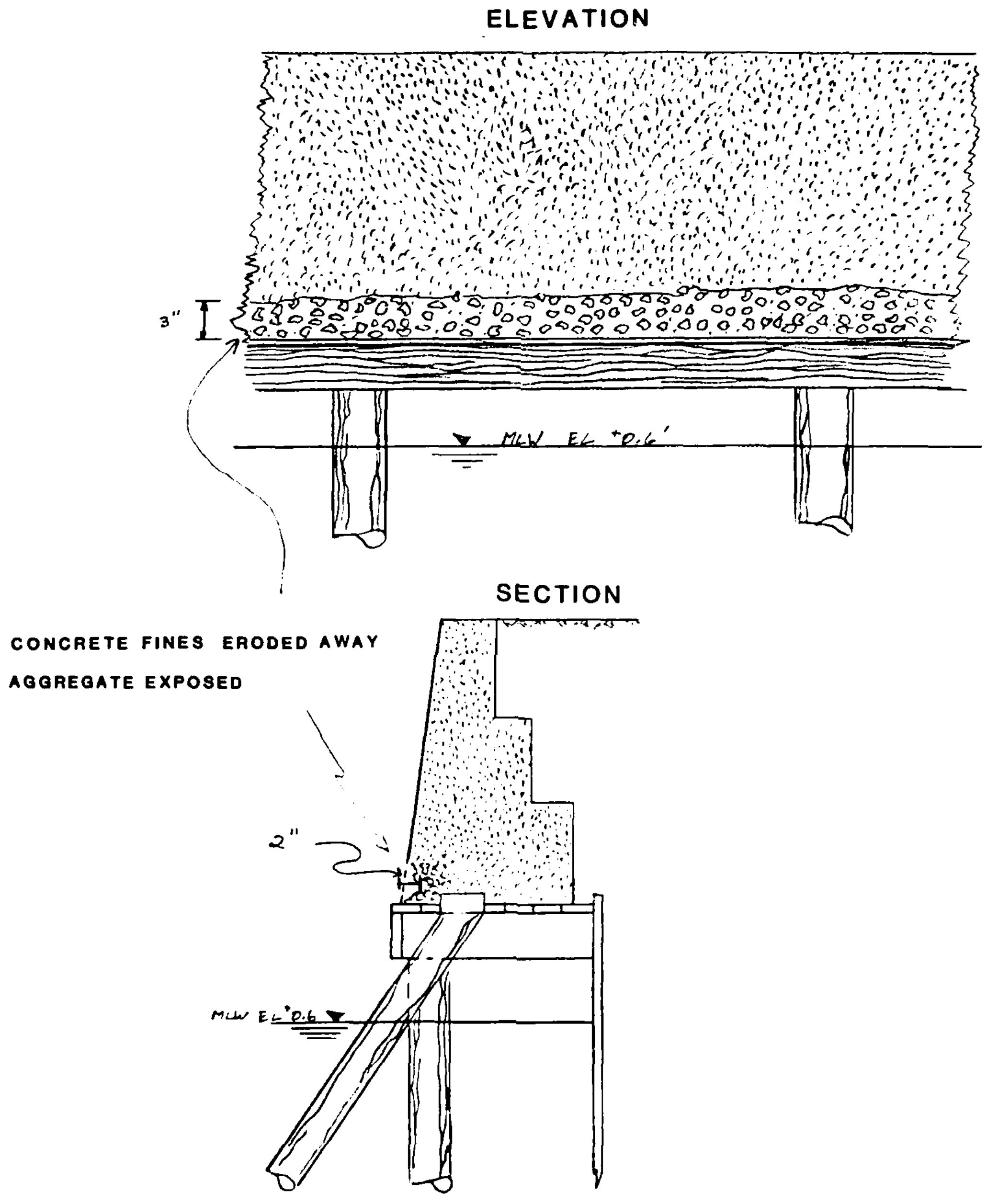
The condition of the timber throughout the length of the seawall was found to be sound. Visual inspection of core samples verifies this. The measurement of minimum pile diameters indicates that there has been no loss of cross-sectional area since original construction. Minimum pile diameters range from 10" to 16".

There is no functional fender system along the full length of the seawall. The concrete seawall showed only minor spalling generally limited to the MLW elevation. This spalling is exemplified by the loss of the fine aggregates and cement while leaving the larger aggregates in place (see Figure 25 and Photo #13). There is some vertical cracking in the seawall at various locations.

The timber and steel sheet pile bulkheads appear to be functional although some slight outward deflection was noticed. There are some locations where large amounts of fill material are leaching out through gaps in the timber sheet pile bulkhead. Corrosion profiles for the steel sheet pile along with steel thickness readings indicate that there is a minimal loss of section of the steel sheet piling. Pile caps and timber decking are in excellent condition.

From Station 8+00 through Station 48+00 there are a considerable number of non-bearing perimeter piles (see Photo #15). Percentages range from 50% of all the piles being non-bearing to 10% of

CONCRETE AGGREGATE EXPOSED



CONCRETE FINES ERODED AWAY AGGREGATE EXPOSED

GRAPHIC SCALE

NO SCALE

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AGGREGATE EXPOSED

FIG NO
25

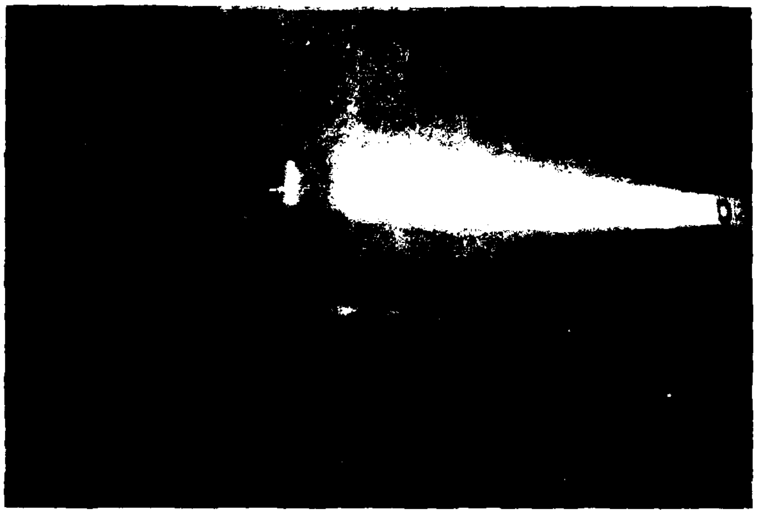


PHOTO NO. 15: Eastern Seawall, Sta. 21+56, perimeter pile; 1" gap between pile and pile cap.

the piles being non-bearing (see Figures 11 through 15 for locations).

Along the stone seawall between Station 52+35 to Station 60+25, there is no mortar remaining between the stones. Also there are areas along the wall (see Figure 22) where stones are missing leaving voids or holes ranging from 6" in depth to 3' in depth. Previously patched areas along the wall are beginning to deteriorate at the lower elevations.

The fasteners used to make the connections for the timber structures were found to be in good condition and functional (see Photo #16).

PHOTO NO. 16: Eastern Seawall, Sta. 21+56, batter pile; illustrates typical condition of pile to pile cap connection. Corrosion has rounded edges of bolt and washer, but in general connections are in good condition.

4.1.3 STRUCTURAL ASSESSMENT

The non-bearing piles occurring at the perimeter of the seawall from Station 8+00 to Station 48+00 appear to be caused by some type of settlement and/or movement (rotation) of the structure. After studying this situation carefully, we conclude that the lateral earth pressure on the sheet pile and seawall exceeded the capacity of the batter piles to resist this force without deflecting past design limits. This, in turn, caused a rotation of the structure in the southerly direction, about the batter pile, therefore causing an uplifting force by the batter pile which, in fact, is lifting the seawall off the vertical pile. It appears that this motion occurred until the sheet pile wall deflected and consequently assisted the batter pile in resisting the lateral earth pressure. It appears that from previously reported conditions (Hudson Engineering 1976) and the present inspection condition, there has not been a significant change in conditions over the past seven years. Apparently the forces involved have reached an equilibrium. The vertical cracking and mis-alignment of the concrete seawall are results of the movement and settlement of the seawall.

Calculations (see Appendix A-9 to A-15) indicate that the seawall structure is capable of supporting only its dead load and can only resist lateral forces imposed by the existing soil.

The previously described condition of the stone seawall (Stations 52+35 to 60+25) is caused by the frequent wetting/drying and freeze/thaw along with the wave and chemical action (sulfate attack) that occurs in the tidal zone.

4.1.4 RECOMMENDATIONS

We recommend that the live-loading behind the Eastern Seawall remain at its present value of 0 pounds per square foot (psf). Apparently this loading capacity does not effect the desired function of the Eastern Seawall. However, if Shipyard personnel decided to upgrade the live-load capacity of the Eastern Seawall, we would recommend the installation of riprap along the southern perimeter from Station 0+00 through Station 48+00 to stabilize the wall. The estimated cost to place rip-rap is $77/lf using the Engineering News Record Construction Cost Index to adjust the Hudson Engineers original cost estimate. The total estimated cost would be approximately $370,000 based on 4800' of rip-rap.

The stone seawall between Station 52+35 and Station 60+25 should be pointed and the loose and missing stones should be replaced. The estimated cost per linear foot of seawall is $40.00. The total estimated cost is approximately $32,000.

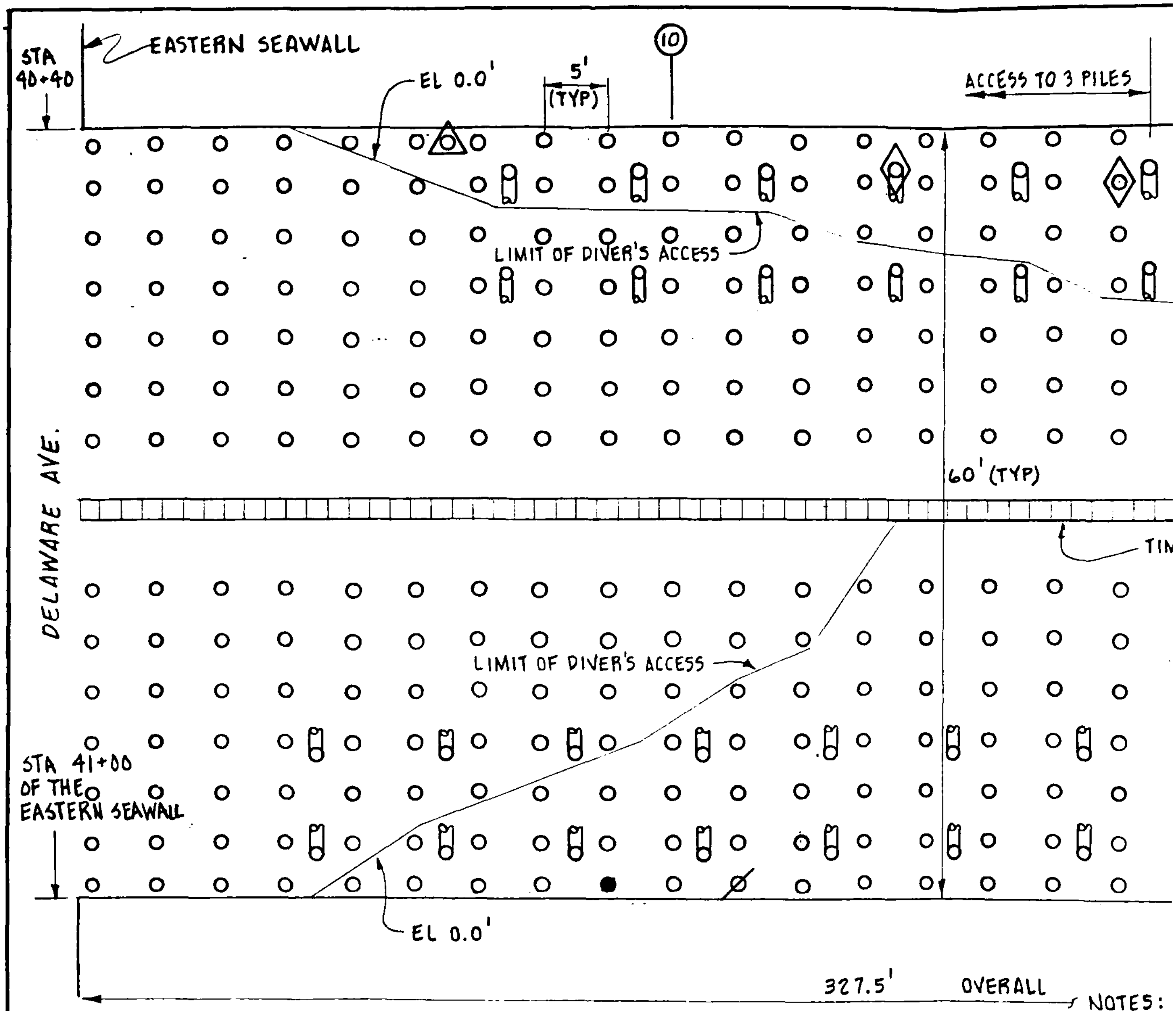
The entire facility should be re-inspected after repairs and in 6 years thereafter. This will enable Shipyard personnel to determine any changes in condition. This report should be used as a baseline for all future inspections.

We estimate the life of this facility as it exists presently to be in excess of 10 years. With the proposed repairs installed, this facility would have a future life in excess of 30 years, providing that the facility is properly maintained and not used beyond its intended purpose, i.e., that to which it was designed.

4.2 PIER 7

4.2.1 Description

Pier 7 is situated at approximately Station 40+70 along the Eastern Seawall (see Figures 4, 26-28). It is adjacent to and to the east of Boathouse 431. The date of construction is sometime prior to 1931. There are approximately 980 vertical piles and 140 batter piles supporting the low deck, earth fill, relieving platform structure. The structure also has a timber sheet pile wall running in the north-south direction through the center of the pier. The overall dimensions of the pier are approximately 328' x 60'. The structural piles are assumed to have a bearing capacity of 15 tons. The original deck elevation is +11.0 above mean low water. During the time of our inspection, there was restricted access to Pier 7 and live-loading was limited to 200 psf.
(Reference 2, see Appendix A-33)



PLAN

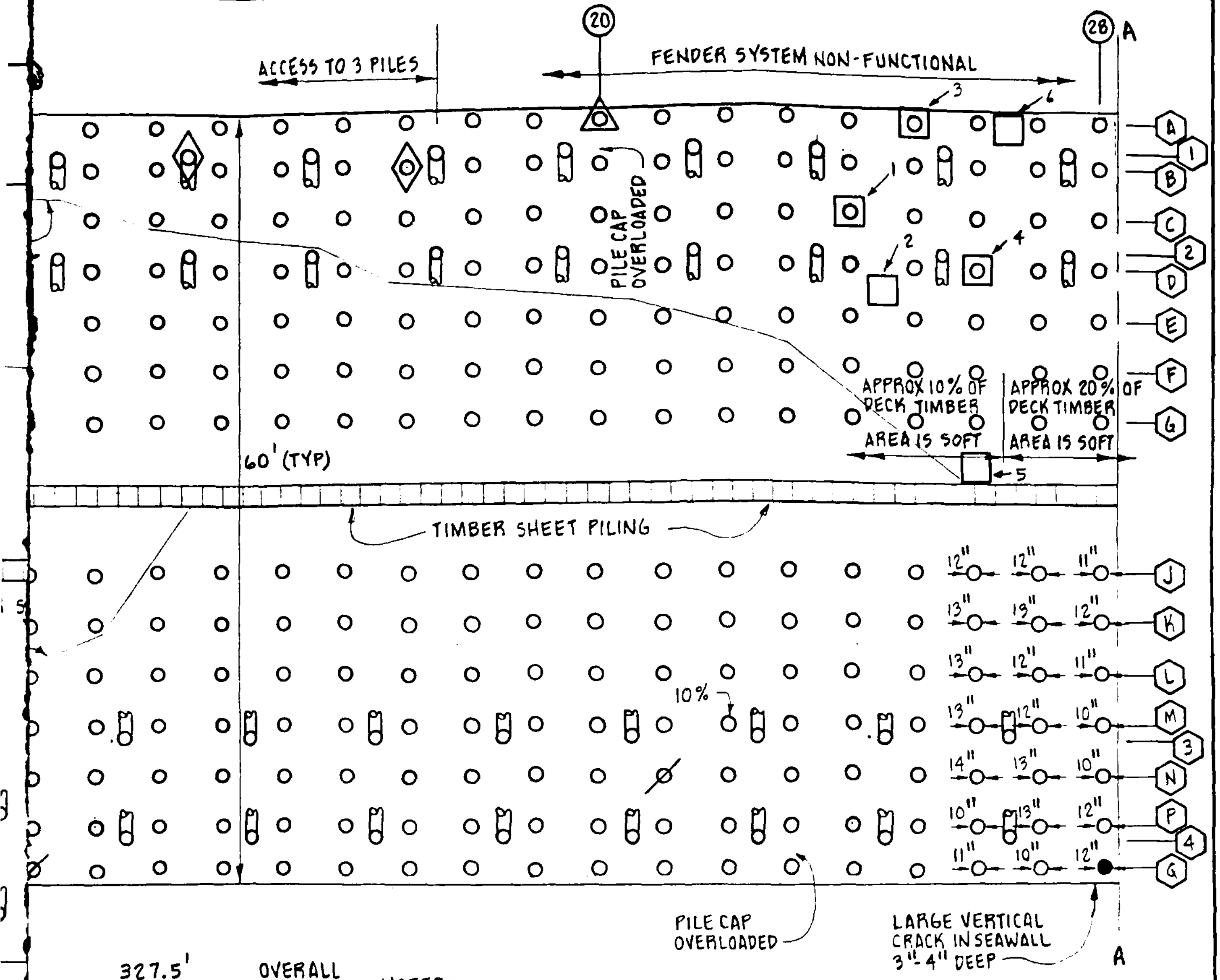
LEGEND

- △ WILD PILE
- ◈ NON-BEARING PILE
- ● BROKEN PILE
- ø DISPLACED- SPLIT PILE
- ⊡^{10} CORE LOCATION (PILE, CAP, DECK) LEVEL 3 INSPECTION
- ☐^{3} CORE TAKEN IN DECK ONLY
- -o- MINIMUM PILE DIAMETER, LEVEL 2 INSPECTION
- -25' SOUNDINGS (FT) BELOW MLW
- 50%→o PERCENT OF PILE BEARING ON PILE CAP

REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES CODE ID NO 80091 & DWG C-2889.

NOTES:

1. APPRO
PROBE
2. CORE S
3. LEVEL



PLAN

NOTES:

1. APPROX 10% OF SURFACE AREA OF TIMBER CAPS & DECK TIMBER CAN BE PROBED WITH A KNIFE TO A DEPTH OF 3"-4".
2. CORE SAMPLES INDICATE 1" OF SOFTNESS IS A TYPICAL CONDITION.
3. LEVEL 1 INSPECTION ON ALL PILES- HIGHER LEVELS OF INSPECTION WHERE NOTED.

Z ←

SPECTION

RENCE: CONDITION SURVEY OF
RFRONT STRUCTURES CODE ID NO
91 & DWG C-2889.

GRAPHIC SCALE

5 0 5 10 15

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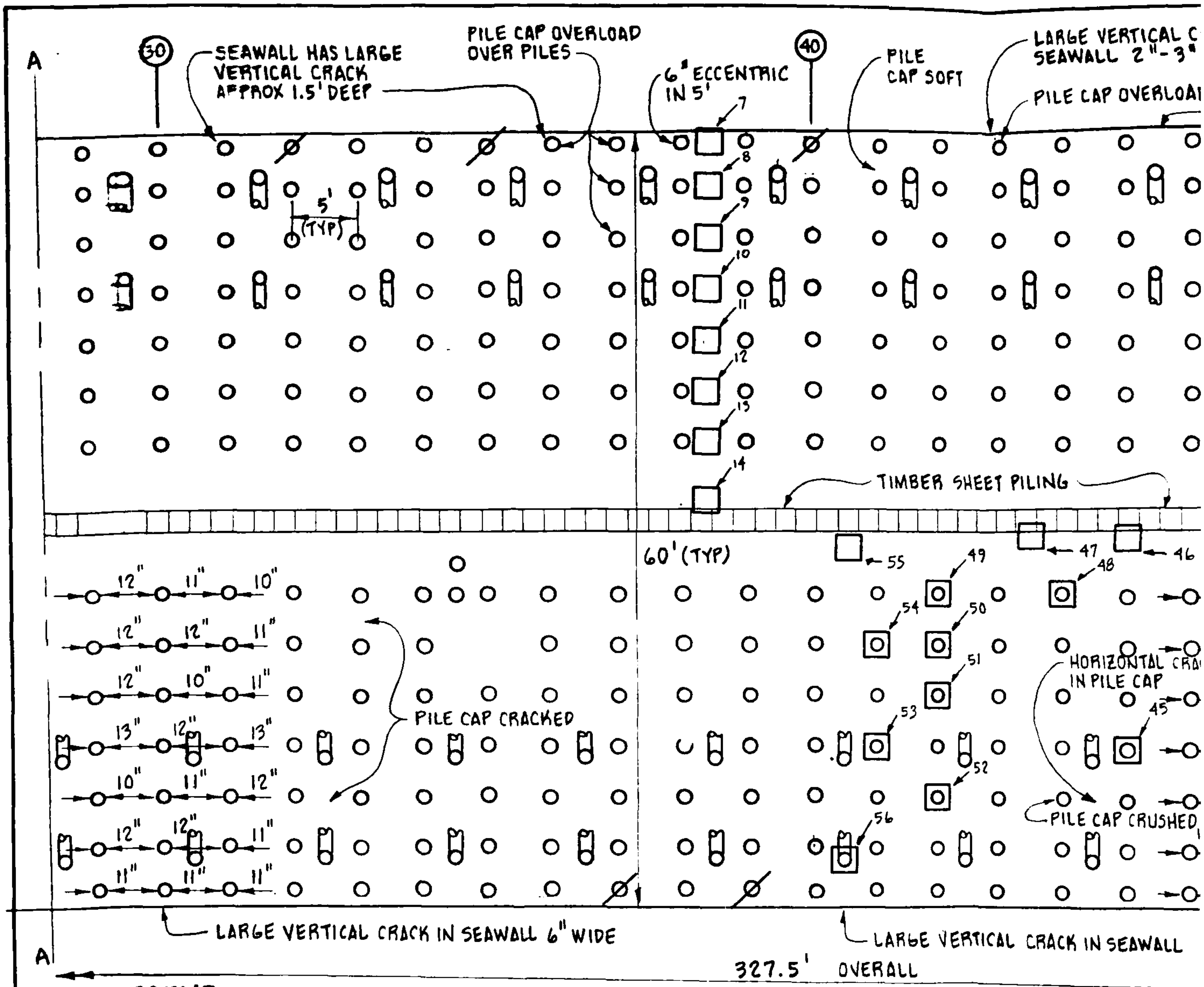
PIER 7

FIG NO. 26

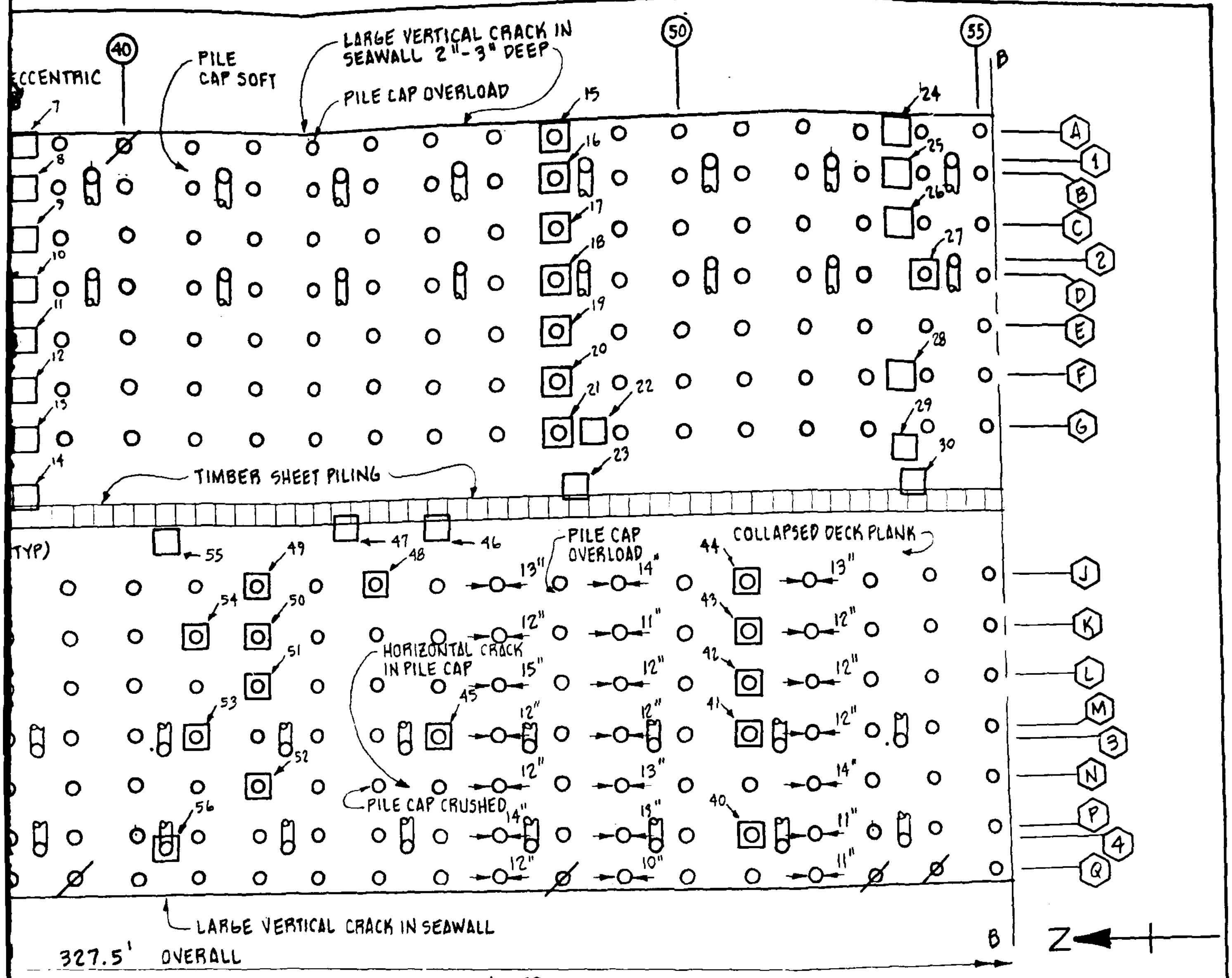


PLAN

LEGEND

●	BROKEN PILE
ø	DISPLACED - SPLIT PILE
⊡ 10	CORE LOCATION (PILE, CAP, DECK) LEVEL 3 INSPECTION
—o—	MINIMUM PILE DIAMETER, LEVEL 2 INSPECTION
☐	CORE TAKEN ON DECK ONLY

REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES CODE ID NO. 80091 & DWG C-2889.



PLAN

NOTES:

1. APPROX 10% OF SURFACE AREA OF TIMBER CAPS & DECK TIMBER CAN BE PROBED WITH A KNIFE TO A DEPTH OF 3"-4".
2. CORE SAMPLES INDICATE 1" OF SOFTNESS IS A TYPICAL CONDITION.
3. LEVEL 1 INSPECTION ON ALL PILES- HIGHER LEVELS OF INSPECTION WHERE NOTED.

REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES CODE ID NO. 80091 & DWG C-2889.

GRAPHIC SCALE
5 0 5 10 15

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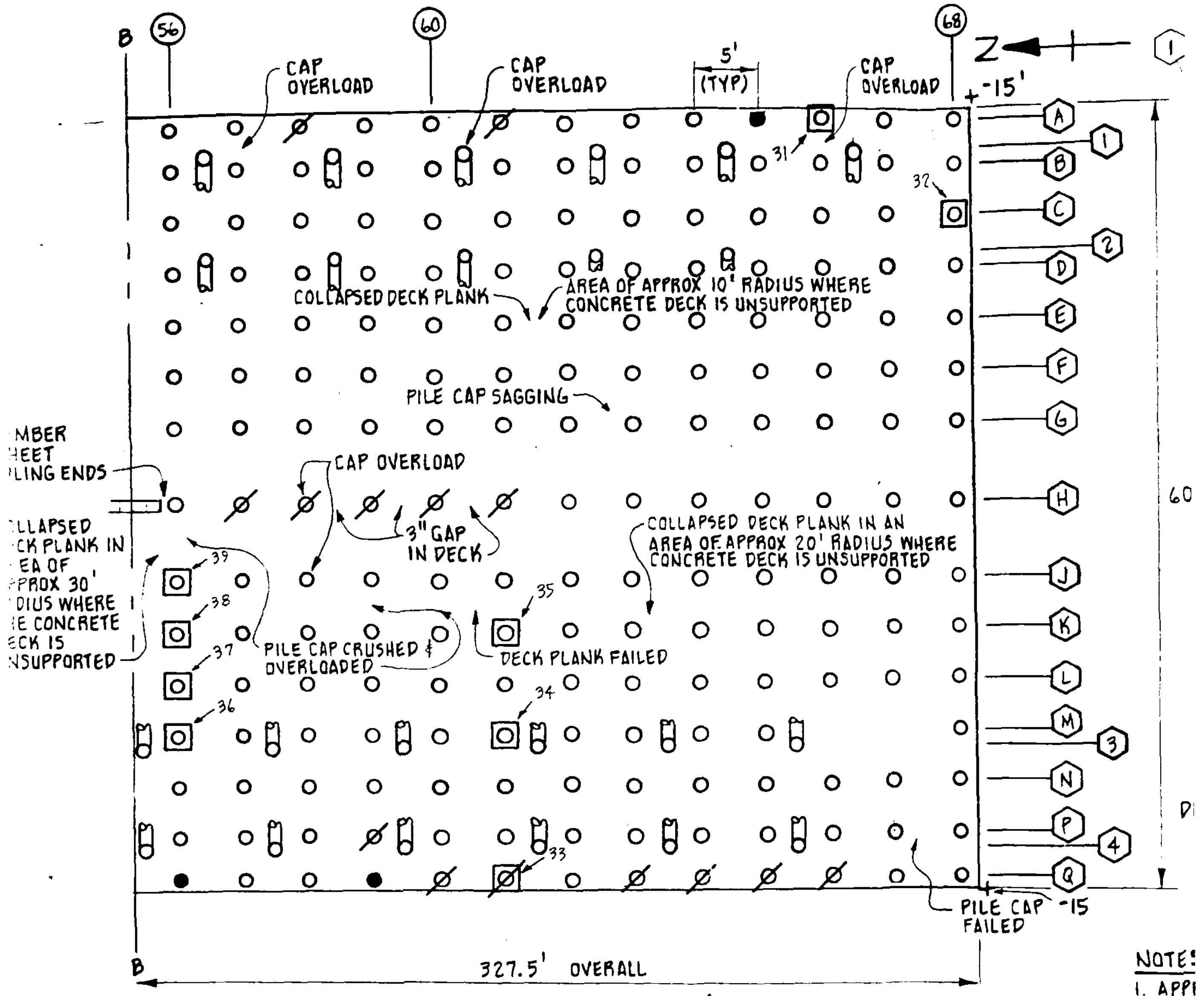
PIER 7

FIG NO.
27



PLAN

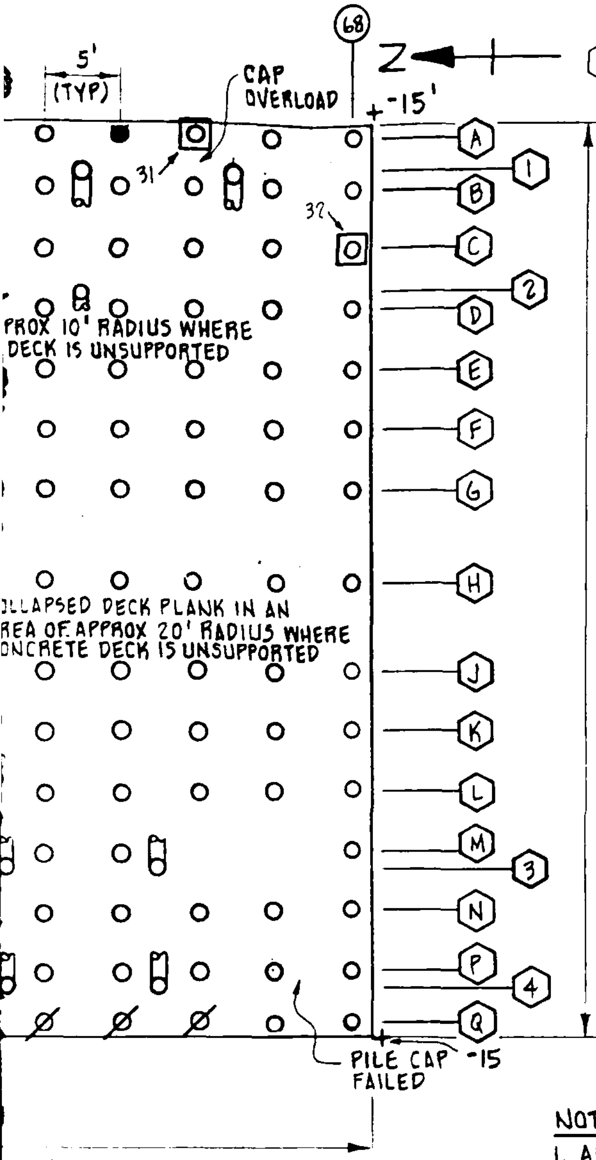
LEGEND

- ● BROKEN PILE
- ⌀ DISPLACED- SPLIT PILE
- ⊡ 10 CORE LOCATION (PILE, CAP, DECK) LEVEL 3 INSPECTION
- -○- MINIMUM PILE DIAMETER, LEVEL 2 INSPECTION
- $\overset{+}{-}25'$ SOUNDINGS (FT) BELOW MLW

REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES CODE ID NO. 80091 & DWG C-2889.

NOTE:
1. APP
PRO
2. COR
3. LEVE

5



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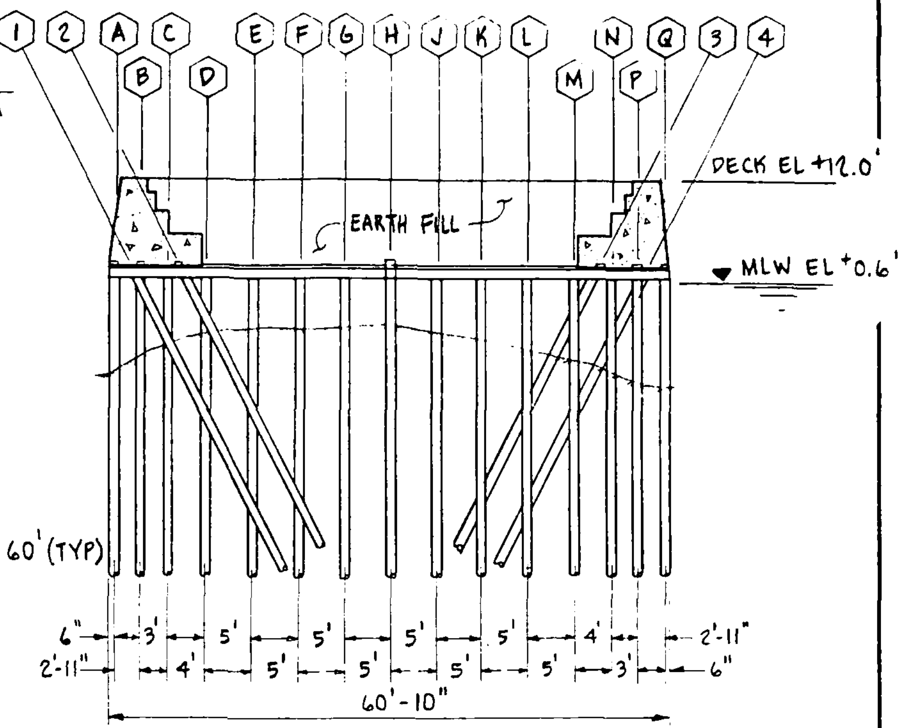

FLOOD

DELAWARE RIVER

EBB

TYPICAL SECTION

NOTES:

1. APPROX 10% OF SURFACE AREA OF TIMBER CAPS & DECK TIMBER CAN BE PROBED WITH A KNIFE TO A DEPTH OF 3"-4".
2. CORE SAMPLES INDICATE 1" OF SOFTNESS IS A TYPICAL CONDITION.
3. LEVEL 1 INSPECTION ON ALL PILES - HIGHER LEVELS OF INSPECTION WHERE NOTED.

REFERENCE: CONDITION SURVEY OF WATERFRONT STRUCTURES CODE ID NO. 80091 & DWG C-2889.

GRAPHIC SCALE
5 0 5 10 15

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PIER 7
FIG NO
28

4.2.2 OBSERVED INSPECTION CONDITION

Specific anomalies detected which relate to the structural piles can be listed as follows:

2	Non-Bearing Piles
24	Split and Displaced Piles
5	Broken Piles
2	Wild Piles

These anomalies as well as other conditions can be found on Figures 26 thru 28, (see Photos #17 and 18). The structural piles have suffered no apparent loss of cross-sectional area, although softness was generally found to be approximately 1" in depth. Minimum pile diameters range from 10" to 14".

The pile caps and deck timber were found to have soft spots where divers could probe with a knife into the wood approximately 2"-6", generally the depth of this softness is 2". There are three specific areas at the south end of Pier 7 (see Figure 28) where deck timbers have failed and earth fill has leached out of the structure. This, in turn, causes the concrete upper deck to be unsupported and therefore very weak. At Bent 44 over Pile N the pile cap has failed (see Photo #19) due to overloading.

The concrete seawall was observed to have extensive cracking and spalling throughout its full length. The concrete and asphalt top deck has undergone settlement and also has extensive cracking and spalling throughout.

The fender system along Pier 7 is non-functional and mostly non-existent. The timber structure fastenings (steel bolts and drift pins) were found to be in good condition.

PHOTO NO. 17: Pier 7, Bent 59, Pile P; pile broken approx. 5' below pile cap due to impact load.

PHOTO NO. 18: Pier 7, Bent 35, Pile A; pile kicked off pile cap and split for a distance of 3' below pile cap due to impact load. Maximum width of split is approx. 6".

PHOTO NO. 19: Pier 7, Bent 62, Pile H; failure of pile cap due to overloading. Pile diameter is approx. 12".

4.2.3 STRUCTURAL ASSESSMENT

The specific anomalies found on the structural piles can be attributed to mechanical damage and generally this damage occurs at the perimeter of the pier. The five (5) split piles occurring at the southern end and in the center of the pier (Bents 56-61, Pile Row H) are the result of a lack of lateral restraint in the pile cap connection (over Pile H). The lateral earth pressure exerted on the peripheral seawall is transferred to the pile cap and then to the connection which is above the pile. When the pile cap separated, the piles split.

The softness associated with the pile caps and deck timber is an advanced state of deterioration. This condition is caused by the biological and chemical erosion of the bonding of the timber fibers and is accelerated in the tidal area where there is frequent wetting and drying. Due to this weakening of the timber, the strength of the member affected is reduced. According to calculations and field observations, we can assume that the ultimate stresses in some of the caps and deck timbers have been reached and in some cases surpassed, causing failure (see Photo #12).

According to calculations (see Appendix A-1 to A-7), the structural piles which are not noted as being mechanically damaged are fully capable of supporting their designed load.

In general, Pier 7 was found to be in poor condition.

4.2.4 RECOMMENDATIONS

Pier 7 is in need of major repairs. At the present time we recommend no live-loading be imposed. Depending upon the intended use of the structure, there are many options. Option A would be to return the structure to full live-loading capacity, (600 psf). We would recommend replacement of the pile-supported relieving platform with a steel sheet pile, solid fill bulkhead. The following is a list of the estimated costs associated with various options:

OPTION A

Demolition of selected portions of Pier 7	$ 150,000
New steel sheet pile bulkhead with fill	1,728,000
Utilities, fender system and miscellaneous	200,000
Sub-total	$2,078,000
Design and Contingencies	422,000
Budget	$2,500,000

OPTION B

Re-use the existing bearing piles to support a new deck surface similar to the existing structure. In re-using the structural piles the maximum live-load capacity that would be acceptable without placement of additional piles or extensive investigation as to the remaining strength of the existing piles would be approximately 50-100 psf. Estimated rebuilding costs breakdown is as follows:

Earthwork and Demolition	$ 200,000
New Timber Caps and Decking	637,000
New Seawall	594,000
Utilities, Fender System and Misc.	200,000
Sub-Total	$1,631,000
Design and Contingencies	422,000
Budget	$2,053,000

The expected life of Option A (50 years) is considerably longer than Option B (15 years). The live-load capacity available to Option A is also greater than Option B. In a cost-benefit evaluation of these options, it appears that when considering replacement in whole, Option A will be more attractive in the long run. However, if a limited use facility is desirable, Option B or a modified Option B may prove to be most economical.

4.3 PIER 1 AND BULKHEAD TO PIER 2

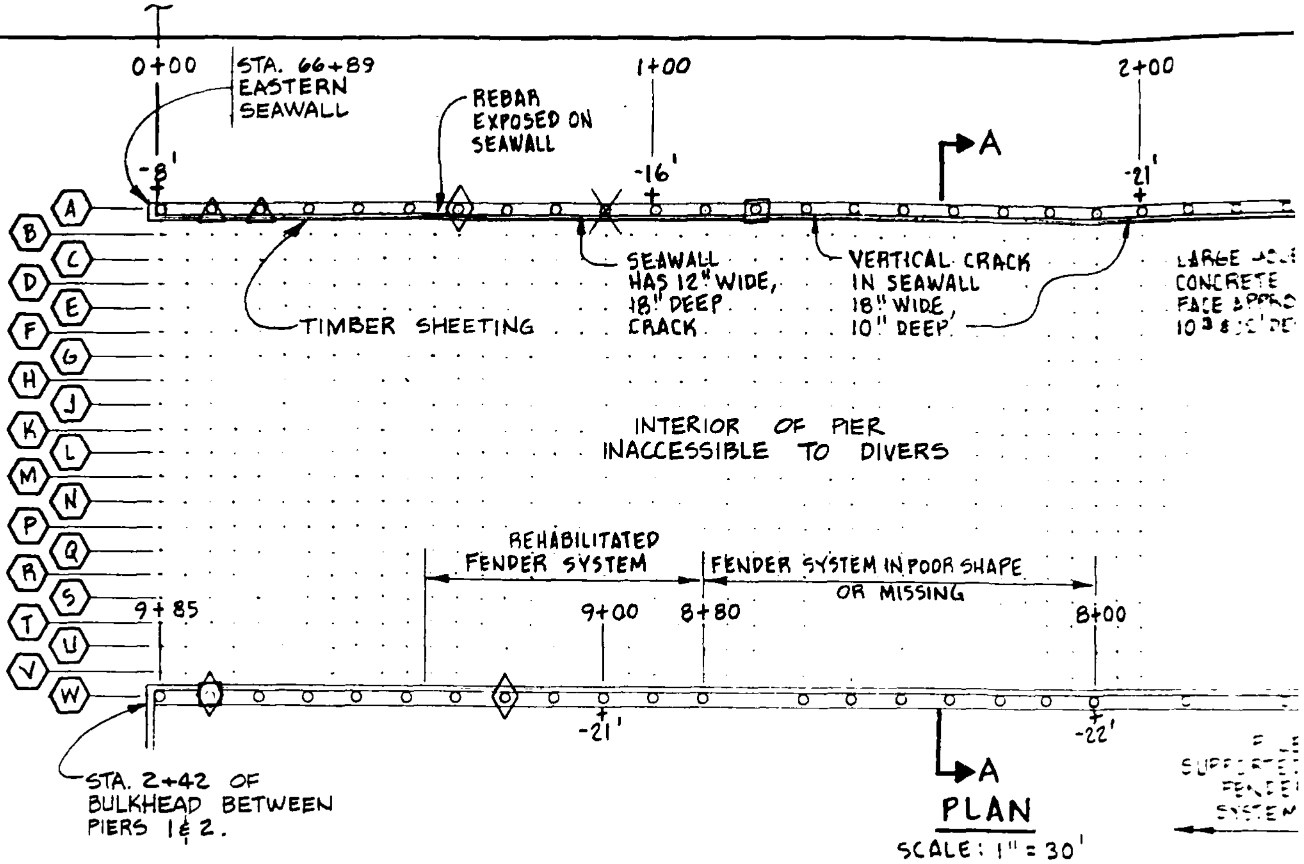
4.3.1 Description

Pier 1 is located on the northern shore of the Delaware River just to the east of Pier 2 and to the west of the ferry slip, (see Figure 4). The northeast corner of Pier 1 is located at Station 66+89 of the Eastern Seawall. Pier 1 was constructed circa 1875 and the original finger pier measured 324'6"x100'. The pier-head measured 150'x69'7". The pierhead was an earth-filled timber crib structure, circa 1890. That portion of the structure which connects the pierhead to the shore was rebuilt as a timber pile-supported, low deck, earth filled relieving platform structure with timber sheet piling surrounding its perimeter. The wood crib pierhead was partially rebuilt in 1890 by replacing the original wood crib from mean low water to El. +11 with a peripheral concrete gravity wall and earth fill.

The overall dimensions of Pier 1 are: finger pier 320'x100'; pierhead 150'x70' (see Figure 29). The assumed pile capacity is 15 tons (driven capacity, see Appendix A-8). The present deck elevation is +11'. During our inspection the live-loading was limited to 300 psf. Pier 1 was functioning as a berthing facility for Navy YTB's.

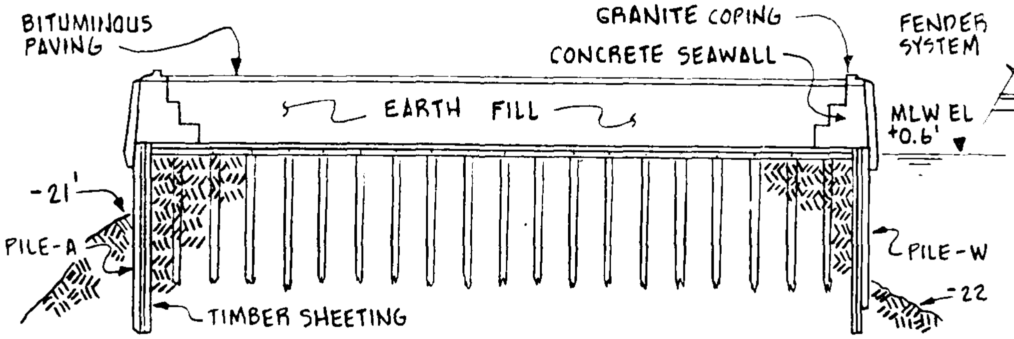
The bulkhead between Pier 1 and Pier 2 was constructed between 1893 and 1904. The structure consists of a low deck relieving platform structure with concrete seawall, earth fill, timber decking, timber pile clamps and timber piles. Timber sheeting runs along the face of the bulkhead, (see Figure 30).

(Reference 2, see Appendix A-33)

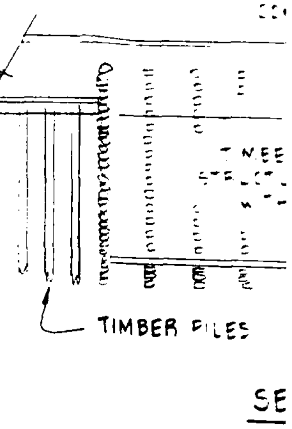


PLAN

SCALE: 1" = 30'



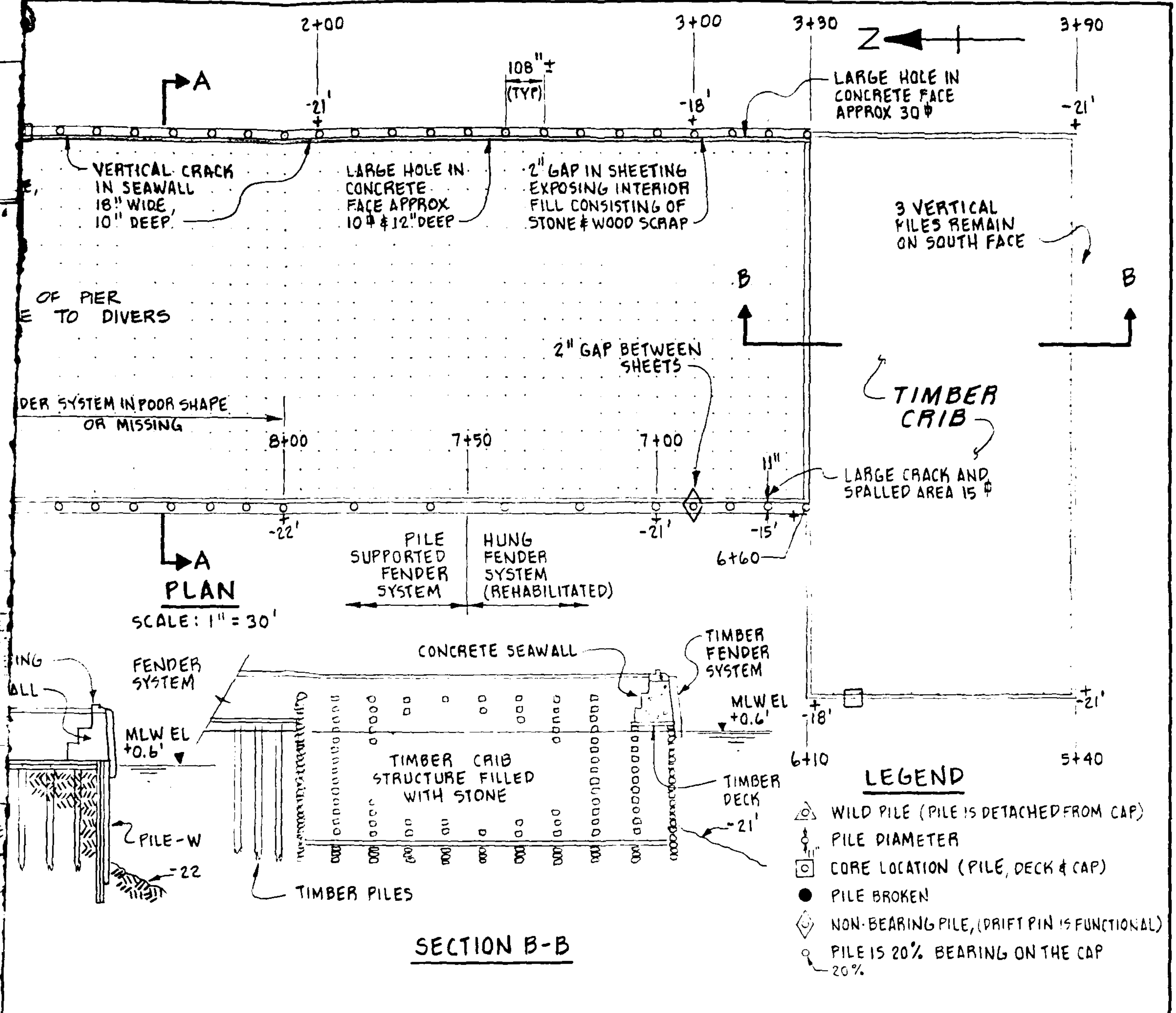
SECTION A-A



NOTE:

LEVEL 1 INSPECTION, UNLESS OTHERWISE NOTED.

REFERENCE: PW B-1779, PW B-1780,
PW B-1781, PW C-5039, PW C-13535
PW C-25460 & HUDSON REPORT
CODE ID NO. 80091.



REFERENCE: PW B-1779, PW B-1780,
PW B-1781, PW C-5039, PW C-13535
PW C-25460 & HUDSON REPORT
CODE ID NO. 80091.

GRAPHIC SCALE

10' 0 10' 20' 30' 40' 50'

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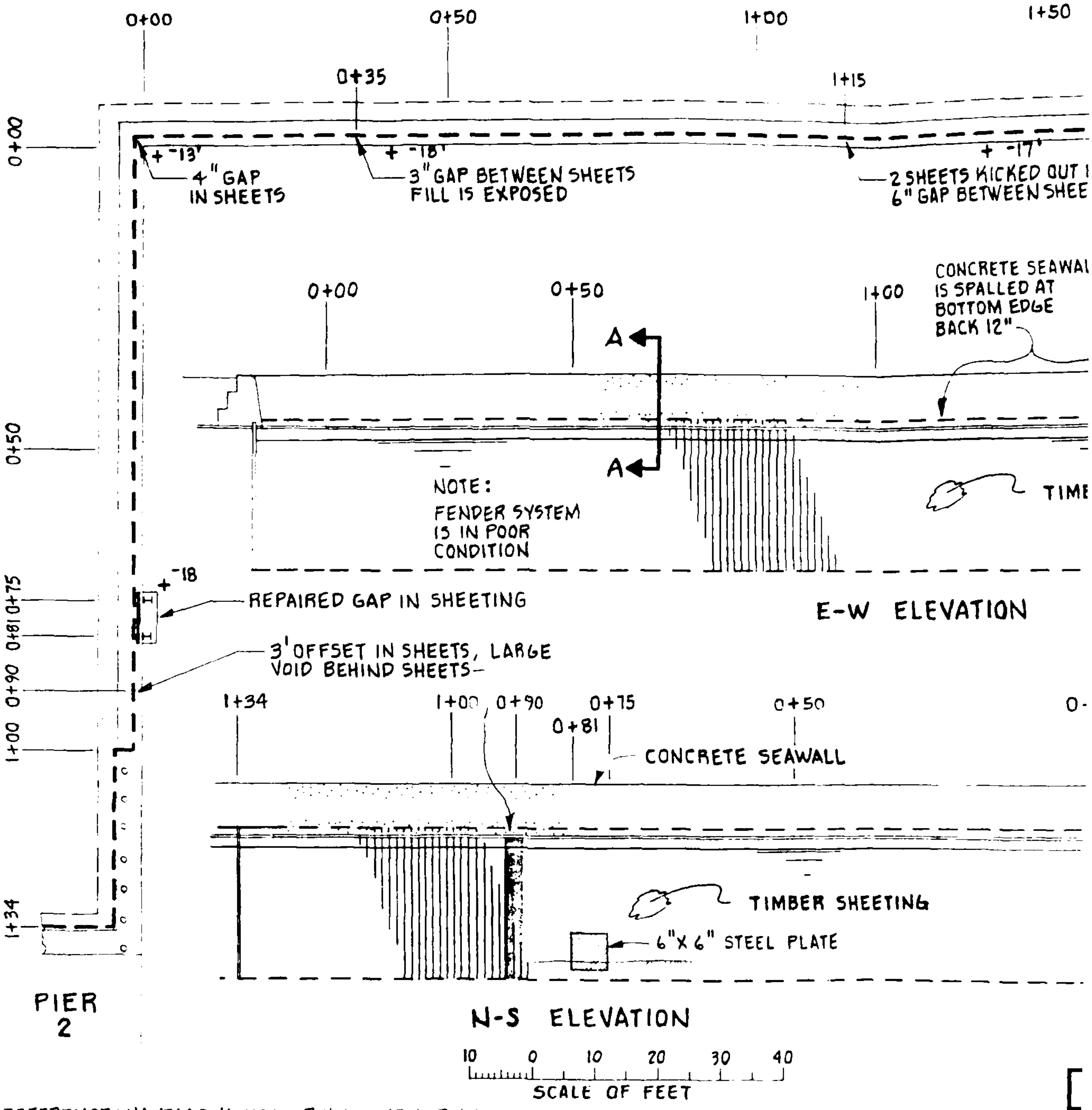
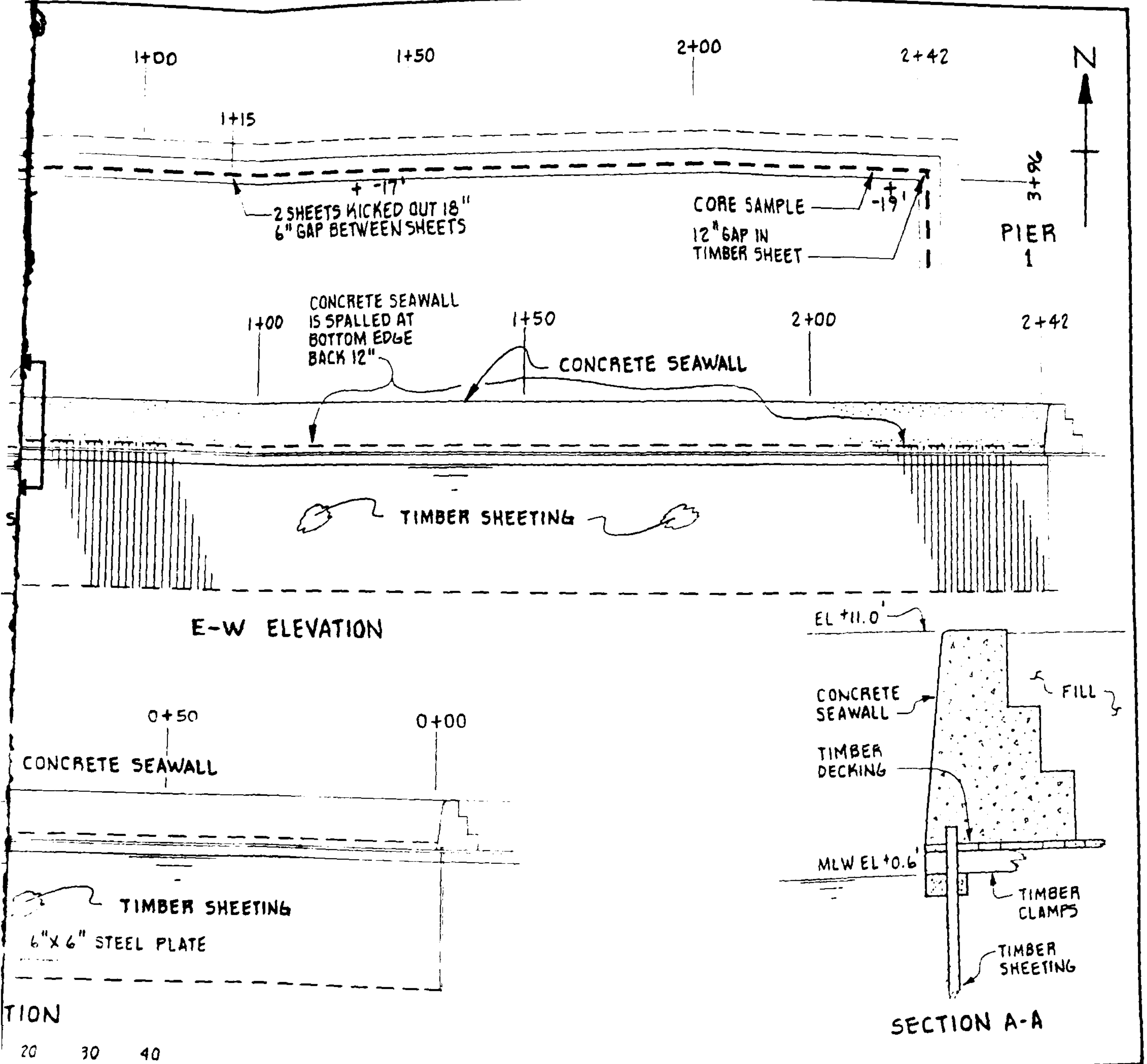
PIER 1

FIG NO. 29



REFERENCE: NAVFAC DWG NOS. PW A-1945-1, PW A-1943-2, PW B-1845, PW C-1616, PW C-3662, PW C-5040, PW C-763, PW C-766, PW C-767, PW C-13427, PW C-3660 & NAVFAC DWG 1240407, CODE ID NO. 80091.



-1945, PW C-1616, PW C-3662, PW C-5040,
W C-3660 & NAVFAC DWG 1246407,

GRAPHIC SCALE

AS SHOWN

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BHD BET. PIERS 1 & 2

FIG NO 30

4.3.2 OBSERVED INSPECTION CONDITION

Access was only available to the perimeter piles and timber sheet piling by divers due to the layout of the pier.

The Pier 1 seawall has undergone severe deterioration over 50% of its surface area. This damage includes spalling and cracking. Some of the cracking and spalling appeared to have been repaired at one time with pneumatically-placed concrete. During the inspection these repairs were in poor condition. Specific locations of the more severe damage can be found on Figure 29. The spalling found on the bulkhead between Piers 1 and 2 is limited to the lower 1' of the seawall with the concrete generally in better condition than on Pier 1.

The fender system is in poor condition over 80% of the structure. On the west side of the pier there is a rehabilitated section as shown on Figure 29. The general condition of the timber on Pier 1 and the bulkhead between Pier 1 and Pier 2 is excellent.

Specific anomalies related to the structural piles are listed as follows:

2 wild piles
4 non-bearing piles
7 gaps in the timber sheet piling exposing fill (see Photo #20)

Locations of these anomalies can be found on Figures 29 and 30. The exact bent and pile spacing could not be verified due to non-accessibility caused by timber sheeting.

PHOTO NO. 20: Pier 1, Sta. 3+00 at the mudline; gap between timber sheet piles exposing fill. Gap width is approx. 2".

At Station 0+90 on the approach bulkhead to Pier 2 there is a large gap in the sheet piling (approximately 3' wide) with fill leaching out and leaving a void below the relieving platform. Other minor gaps in the sheeting were noted. Although there was no observation of recent fill loss, voids are present behind the sheet pile wall.

4.3.3 STRUCTURAL ASSESSMENT

The general condition of Pier 1 and the bulkhead between Pier 1 and Pier 2 is good. The structural piles, except those that are mechanically damaged, are in excellent condition. The crib structure at the outshore end of Pier 1 also appears to be in excellent condition. The seawall on Pier 1 is deteriorated and is in need of repair, although it is functional at this time.

Core samples taken and measurement of the minimum pile diameters along with our calculations (see Appendix A-1 to A-20) indicate that the structure is generally sound.

The gap in the sheet piling on the approach to Pier 2 is creating a fill loss. The cause of this condition should be rectified. The seven (7) other gaps in the timber sheeting appear to be in a stable condition and are not a threat to the integrity of the structure.

4.3.4 RECOMMENDATIONS

We recommend that all mechanically damaged piles be repaired. The estimated cost would be: 7 piles refastened to the pile clamps at $400.00 per pile, total cost of $2,800.00.

We recommend that in the area of the large gap on the approach to Pier 2 live loading be restricted to 0 psf until the timber sheet piling is repaired by patching the gap in a similar manner to that previously used directly adjacent to the large gap along the approach to Pier 2 (see Appendix A-29). The estimated cost for this repair would be approximately $3,000. The spalling associated with the seawall at the perimeter of Pier 1 should be repaired. We recommend that the deteriorated concrete surface be chipped away to sound concrete, wire mesh be installed where needed and provide the wire mesh and deteriorated concrete with a proper cover of pneumatically-placed concrete. The estimated cost per square foot would be $14.16. The estimated total area of cover needed is 4900 sq. ft. The total estimated cost would be $70,000. With the exception of the above-mentioned restriction, current live-loading levels can be maintained (300 psf).

The entire pier should be re-inspected after repairs and in 6 years thereafter. This inspection will enable Shipyard personnel to determine any change in conditions. This report should be used as a baseline for all future inspections.

Upon implementation of the recommended repairs and proper maintenance of the facility, we estimate the future life of this structure to be in excess of 15 years.

4.4 PIER 2

4.4.1 Description

Pier 2 is located to the west of Pier 1 and to the east of Drydock No. 1 along the northern shore of the Delaware River, (see Figures 4, 31-34).

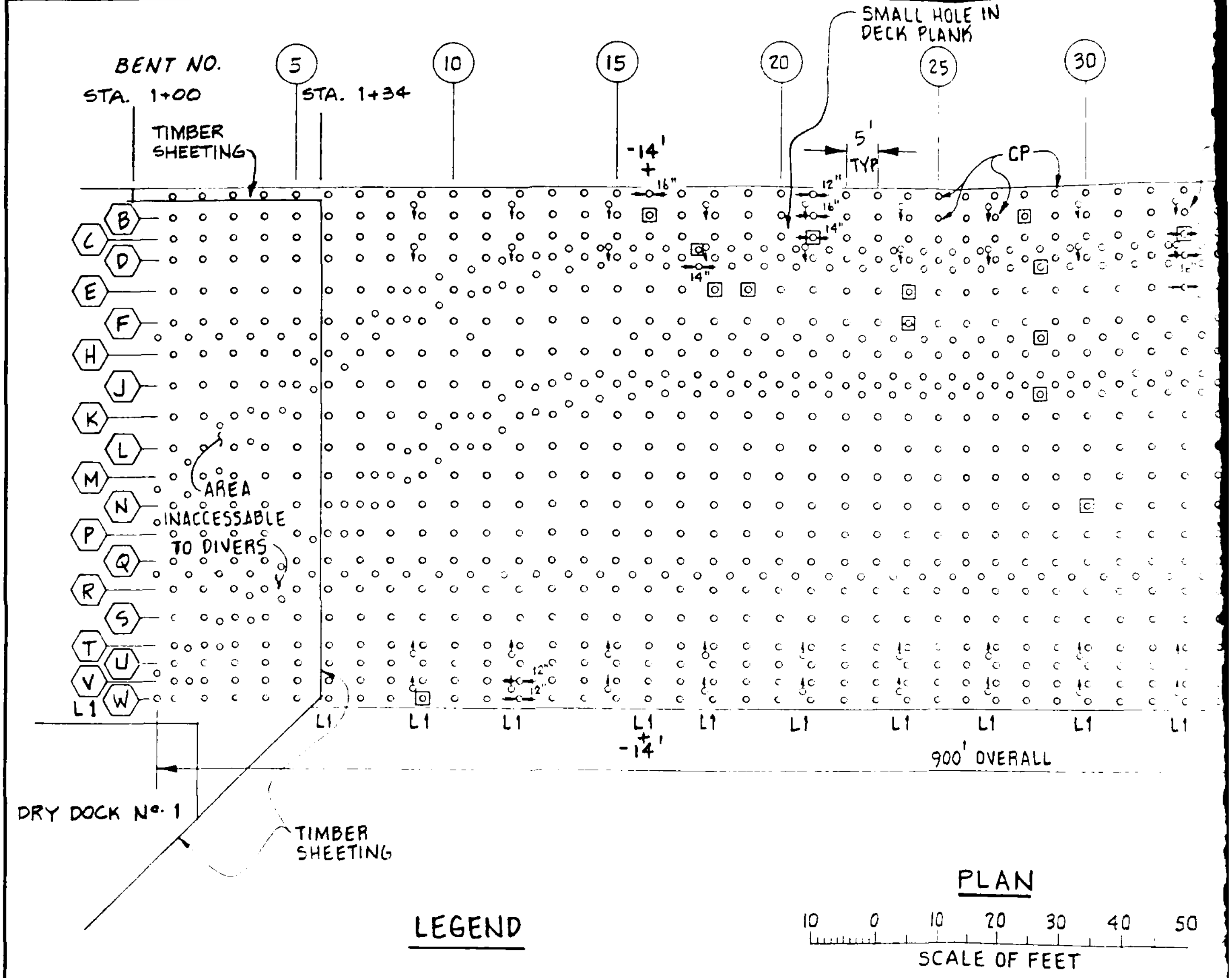
Pier No. 2 was originally constructed circa 1893 measuring approximately 77' in width by 312' in length and consisted of a timber crib with a peripheral concrete seawall. Circa 1903 the pier was extended approximately 240' in the southerly or outshore direction. The extension consisted of 4'6" diameter caissons constructed of 3/8" thick steel plate and filled with concrete. The caissons were installed on a 30' grid pattern and were framed to adjoining caissons with 2-3/8" square tie-rods and turnbuckles. Pile caps consisted of 48" deep steel plate girders running in the transverse direction. Longitudinal framing members consisted of 36" deep steel plate girders framed into the pile cap plate girders. The entire steel superstructure was braced with angle and channel cross-bracing. The paving consisted of a sub-base course of concrete and a top course of brick.

Circa 1929 the original 312' of crib structure failed. Public Works Drawing No. B-3662 dated August 30, 1929 indicated the bottom of the crib structure settled approximately 2' and the peripheral concrete seawall settled and moved in the outward direction approximately 2'. Circa 1930 the entire failed section of Pier No. 2 was removed and a new low deck, earth-filled, pier structure was installed consisting of piles, pile caps, decking, peripheral concrete seawall, earth fill and concrete paving.

Circa 1940 the steel structure installed circa 1903 was removed and a new low deck pier structure, with concrete crane and railroad track foundations, was installed. In addition, the pier was extended approximately 250 feet in the northerly direction.

Presently, Pier 2 measures approximately 900'x80'. The timber structural piles are arranged in bents spaced on 5' centers. There are approximately 28 vertical piles and 4 batter piles per bent (see Figure 34), totaling 6300 for the structure. At every other bay, there is a tie-rod system attached to adjacent pile caps. The solid round tie-rod is set parallel to the pile caps in the center of the bay and attached to the pile cap ends at opposite sides of the pier by means of a wide flange section (see Appendix A-30). This system helps restrain lateral pressures exerted on the seawall.

The top of the concrete seawall elevation is +12'. The design pile capacity is 15 tons (driven capacity, see Appendix A-1 to A-11). The present allowable deck live-load is 600 psf. At the time of the inspection, Pier 2 was functioning as a berthing facility for Navy YTB's and a YFNB.
(Reference 2, Appendix A-33)



PLAN

LEGEND

▣ CORE LOCATION (PILE, CAP, DECK) LEVEL 3 INSPECTION

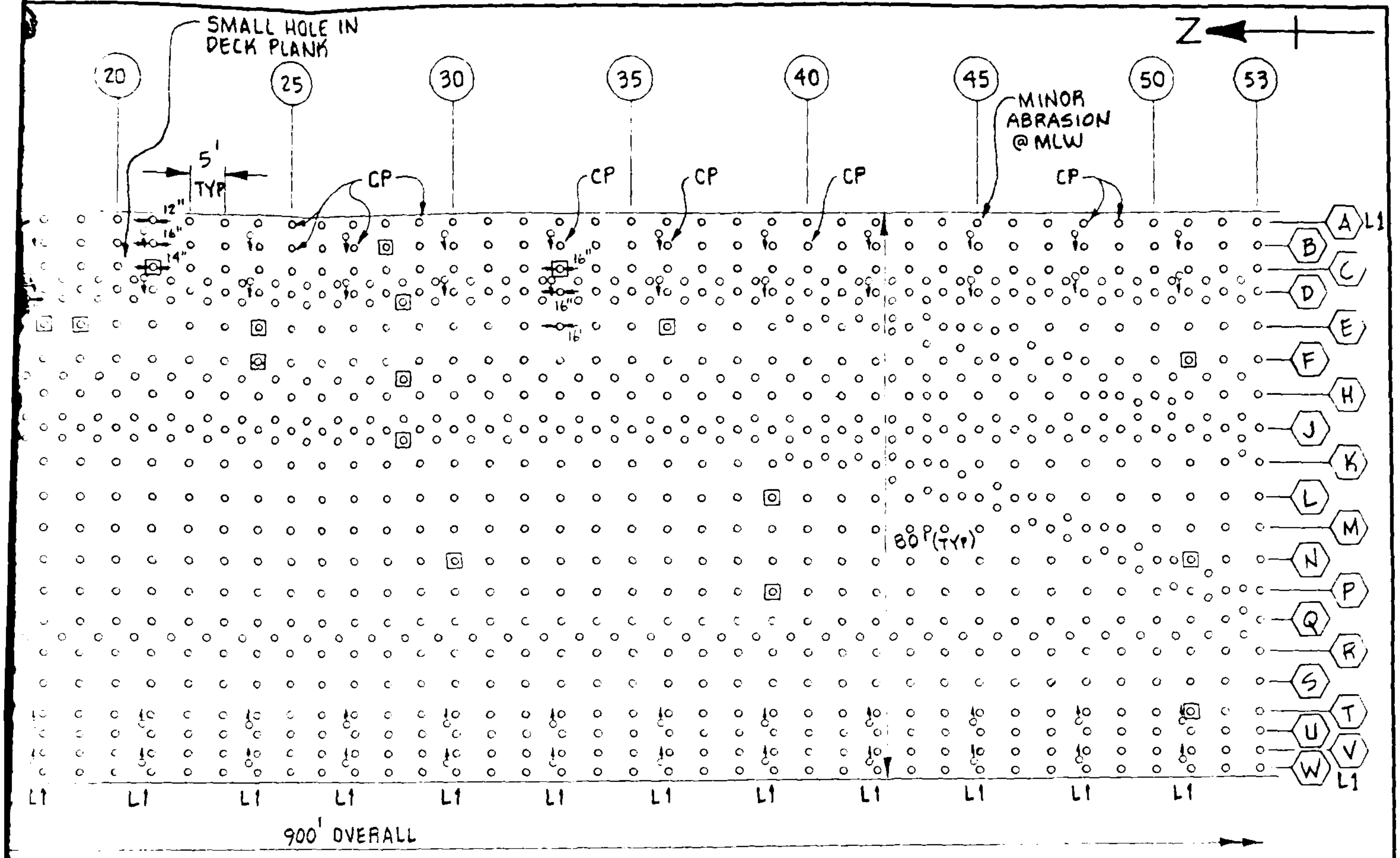
—o— 12" MINIMUM PILE DIAMETER, LEVEL 2 INSPECTION

$\overset{+}{-25'}$ SOUNDINGS (FT) BELOW MLW

L1 LEVEL 1 INSPECTION – PERFORMED ON ALL PILES WITHIN BENT OR ROW

CP CLAMP REPAIR

REFERENCE: NAVFAC DWG NOS. PW A-1945-1, PW A-1943-2, PW B-1845, PW C-1616, PW C-3662, PW C-5040, PW C-763, PW C-766, PW C-767, PW C-13427, PW C-3660, NAVFAC DWG NO. 1240407 & CODE ID NO. 80091.

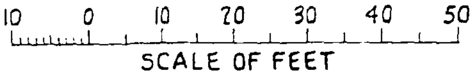


NOTES :

1.) ALL PILES INSPECTED MODIFIED LEVEL 1 UNLESS OTHERWISE NOTED.

2.) TIE RODS OCCUR AT EVERY OTHER BAY.

PLAN



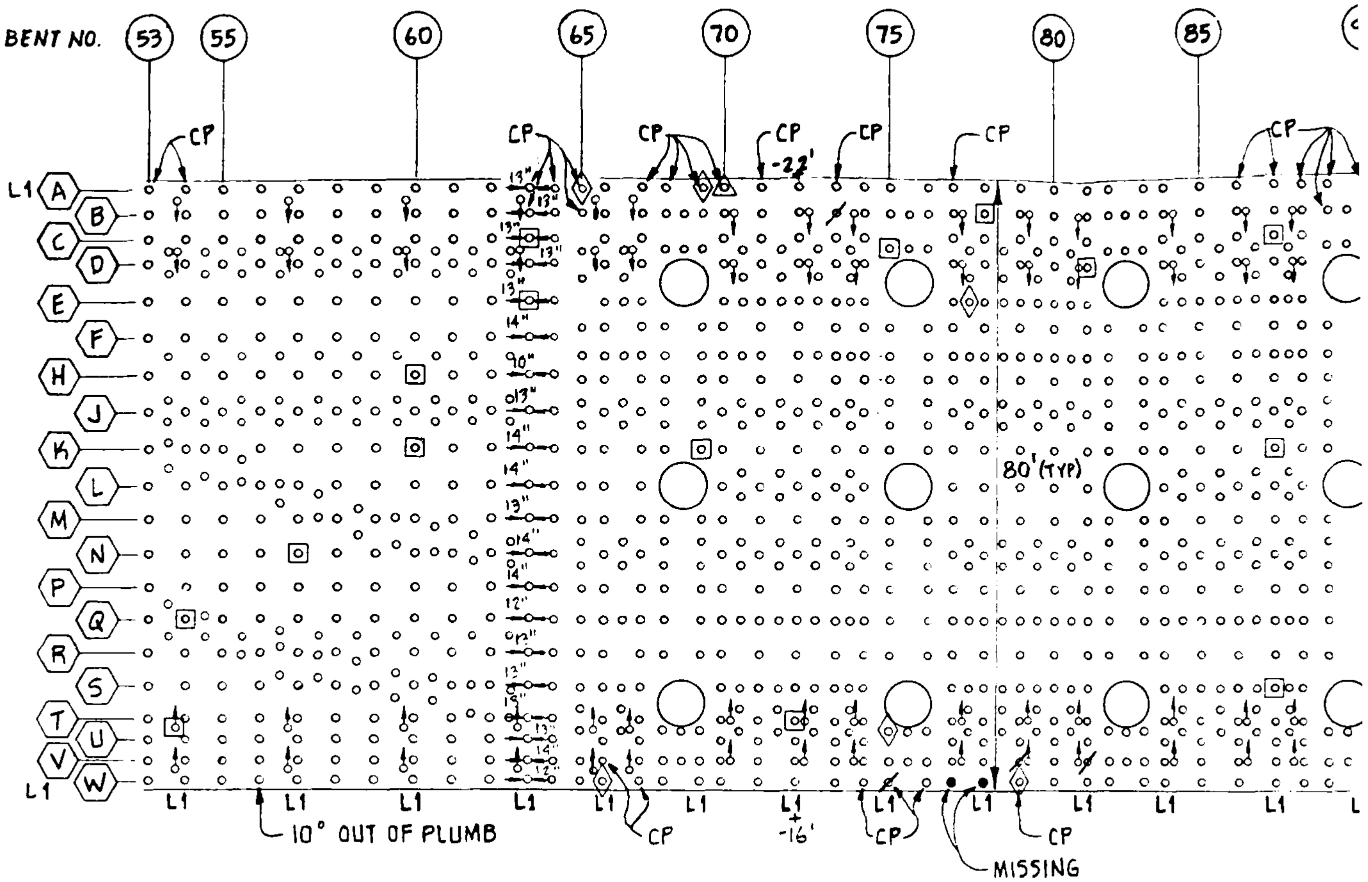
CAP, DECK) LEVEL 3 INSPECTION
ER, LEVEL 2 INSPECTION
V MLW
- PERFORMED ON ALL PILES
WITHIN BENT OR ROW

1-2, PW B-1845, PW C-1616, PW C-3662,
767, PW C-13427, PW C-3660,
591.

GRAPHIC SCALE
AS SHOWN

CHILDS ENGINEERING
CORPORATION
BOX 333
MEDFIELD MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D.C.
PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA
PIER 2
FIG. NO.
31

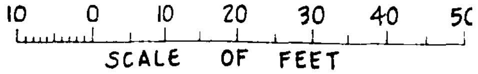


LEGEND

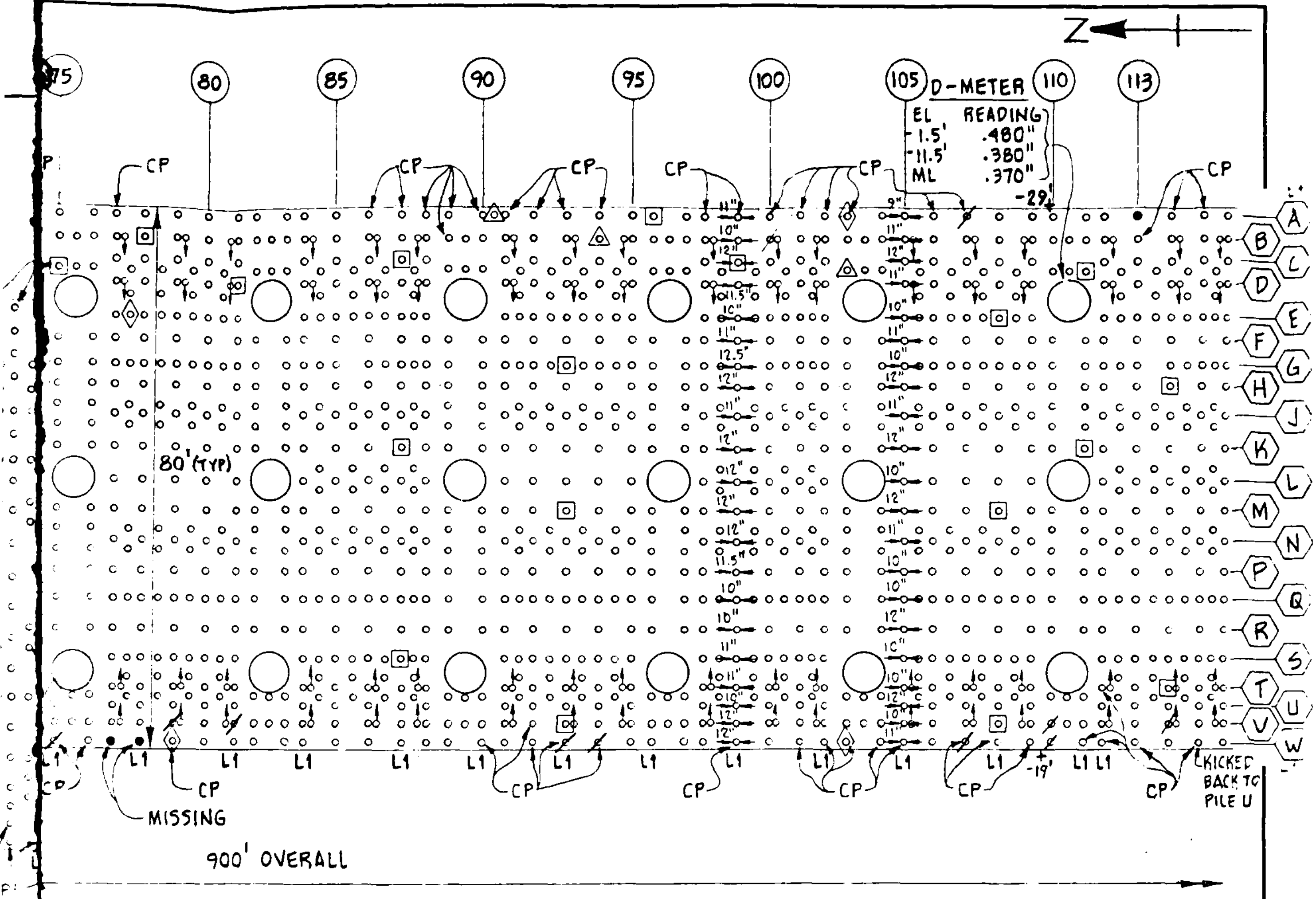
△	WILD PILE
◈	NON-BEARING PILE
●	BROKEN PILE
ø	DISPLACED-SPLIT PILE
▣	CORE LOCATION (PILE, CAP, DECK) LEVEL 3 INSPECTION
—o—12"	MINIMUM PILE DIAMETER, LEVEL 2 INSPECTION
-25'	SOUNDINGS (FT) BELOW MLW
L1	LEVEL 1 INSPECTION - PERFORMED ON ALL PILES WITHIN BENT OR ROW
CP	CLAMP REPAIR

900' OVERALL

PLAN



REFERENCE: NAVFAC DWG NOS. PW A-1945-1, PW A-1943-2, PW B-1845, PW C-1616, PW C-3662,
PW C-5040, PW C-763, PW C-766, PW C-767, PW C-13427, PW C-3660,
NAVFAC DWG NO. 1240407 & CODE ID NO. 80091.



PLAN

NOTES:

1.) ALL PILES INSPECTED MODIFIED LEVEL 1 UNLESS OTHERWISE NOTED.

2.) TIE RODS OCCUR AT EVERY OTHER BAY.

3 INSPECTION

PECTION

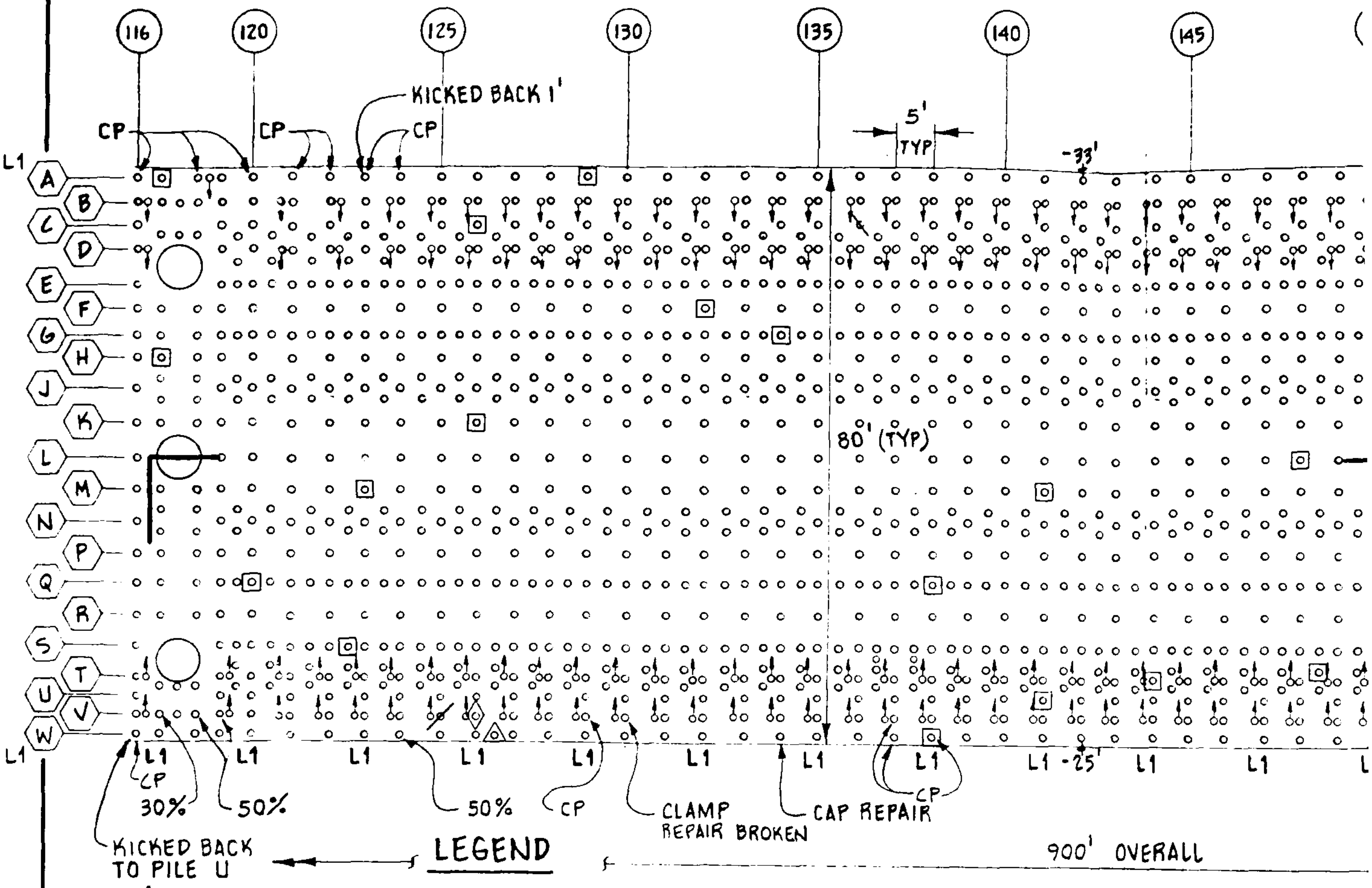
LL PILES
ROW

, PW C-1616, PW C-3662,
, PW C-3660,

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D.C.	
AS SHOWN		PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA PIER 2	FIG NO 32

GRAP

S



LEGEND

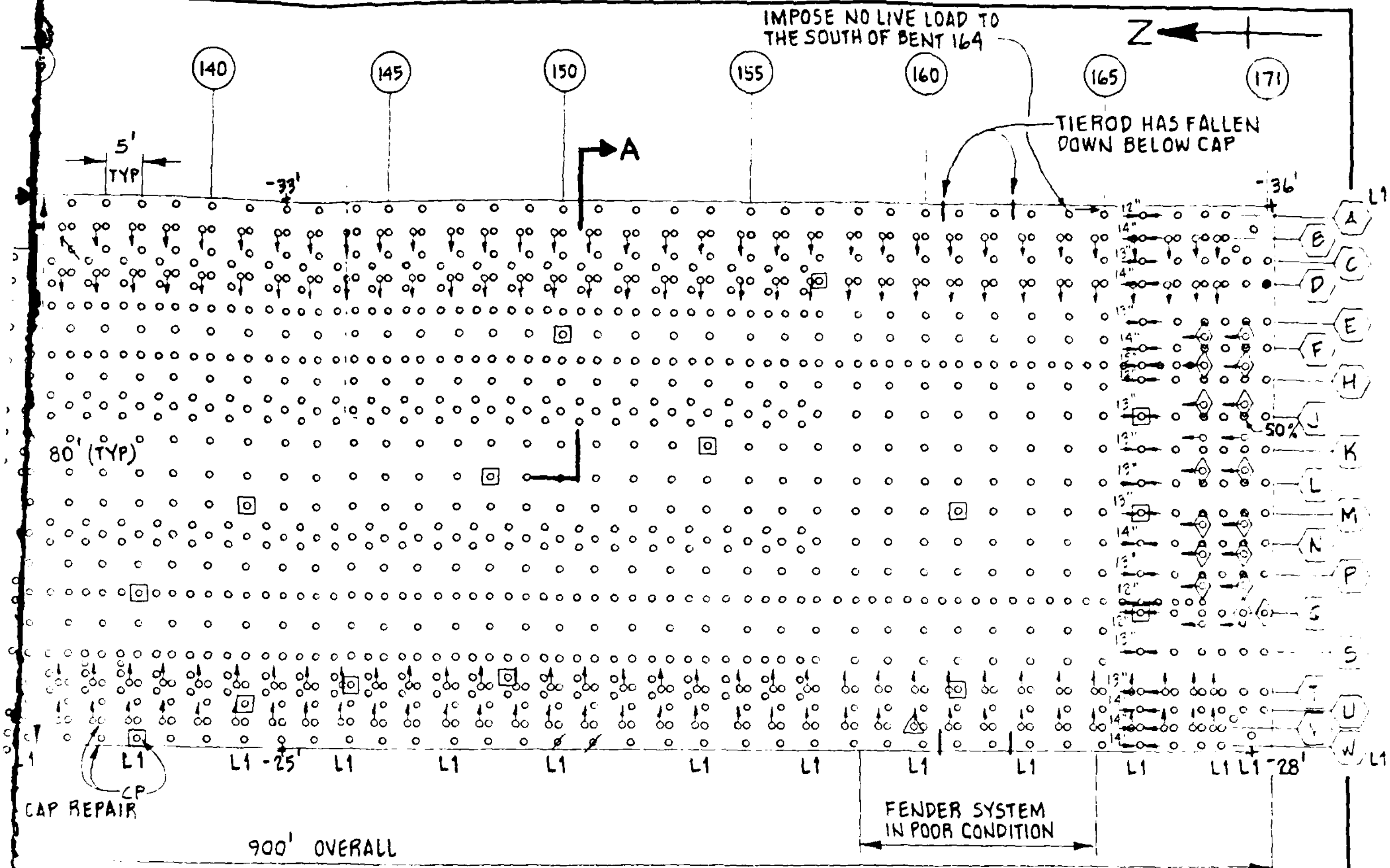
- △ WILD PILE
- ◇ NON-BEARING PILE
- • BROKEN PILE
- ø DISPLACED-SPLIT PILE
- ⊡ CORE LOCATION (PILE, CAP, DECK) LEVEL 3 INSPECTION
- —o— 12" MINIMUM PILE DIAMETER, LEVEL 2 INSPECTION
- -25' SOUNDINGS (FT) BELOW MLW
- 50% 50 PERCENT OF PILE BEARING ON PILE CAP
- L1 LEVEL 1 INSPECTION - PERFORMED ON ALL PILES WITHIN BENT OR ROW
- CP CLAMP REPAIR

PLAN

10 0 10 20 30 40 50

SCALE OF FEET

REFERENCE: NAVFAC DWG NOS. PW A-1945-1, PW A-1943-2, PW B-1845, PW C-1616, PW C-3662,
PW C-5040, PW C-763, PW C-766, PW C-767, PW C-13427, PW C-3660,
NAVFAC DWG NO. 1240407 & CODE ID NO. 80091.



NOTE:

1.) ALL BATTER PILES AT OUTSHORE END OF PIER ARE NON-BEARING.

2.) ALL PILES INSPECTED MODIFIED LEVEL 1 UNLESS OTHERWISE NOTED.

3.) TIE RODS OCCUR AT EVERY OTHER BAY.

PLAN

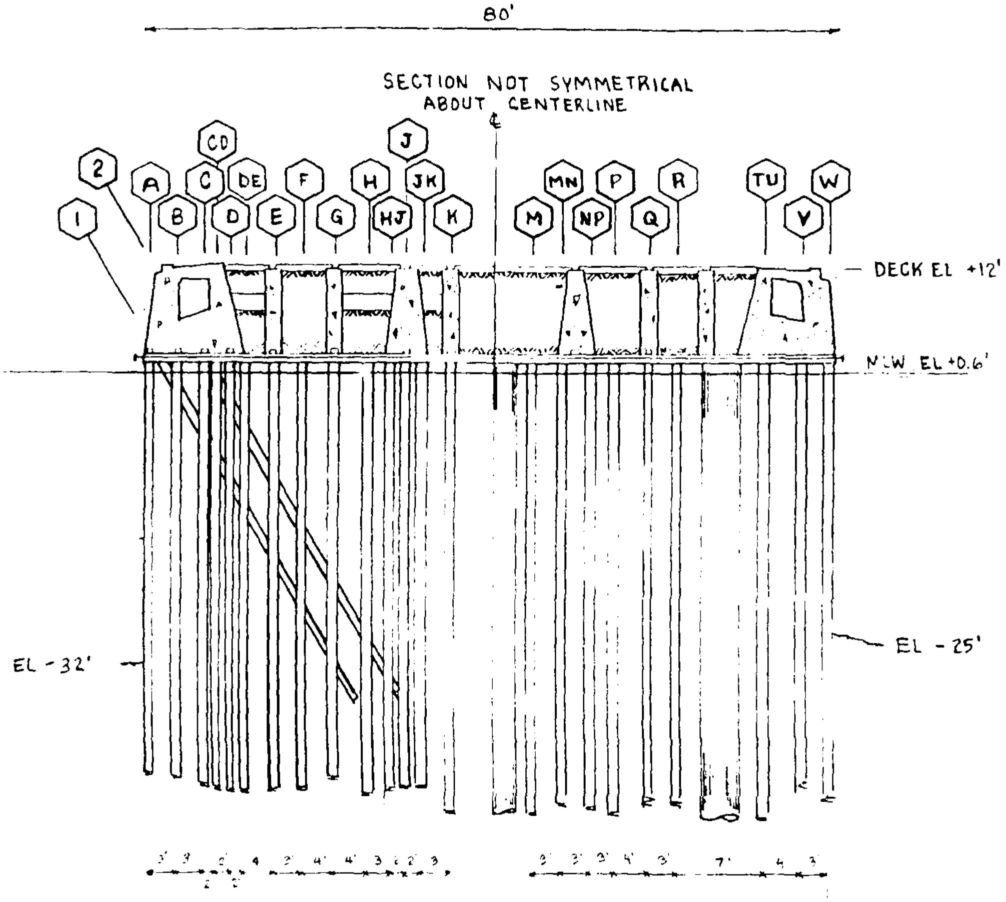
10 0 10 20 30 40 50
SCALE OF FEET

K) LEVEL 3 INSPECTION
VEL 2 INSPECTION

N PILE CAP
RMED ON ALL PILES
N BENT OR ROW

N B-1845, PW C-1616, PW C-3662,
N C-13427, PW C-3660,

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D.C.	
AS SHOWN		PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA PIER 2	FIG. NO 33



SECTION A-A

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D C	
NO SCALE		PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA PA **PIER 2**	FIG NO **34**

4.4.2 OBSERVED INSPECTION CONDITION

Quantities of specific structural pile anomalies are listed as follows:

5 broken piles
6 wild piles
23 non-bearing piles
16 split and displaced piles

Locations of these anomalies along with other abnormalities have been noted on Figures 31 through 33. Also there are approximately 83 piles which have been previously refastened to the pile cap with a clamp arrangement (see Photo #21). At the southern end of Pier 2 there is a high concentration of non-bearing batter piles as shown on Figure 33.

Core samples taken indicate that there is some softness (approximately 2" maximum) associated with about 5% of the bearing piles sampled. Minimum pile diameters measured indicate that there has been no loss of cross-sectional area since construction. Minimum pile diameters range from 10" to 14".

Steel thickness measurements taken on the steel caissons indicate that there is minimal metal loss, although divers reported that the surface of the steel was heavily pitted (see Photo #22). Measurements were also taken on the steel tie rod and WF beam used to retain the pier. These measurements also indicate that there is a minimal loss to the cross-sectional area of the sections. There is pitting similar to that found on the caissons, also found on the tie-rods (see Photo #23).

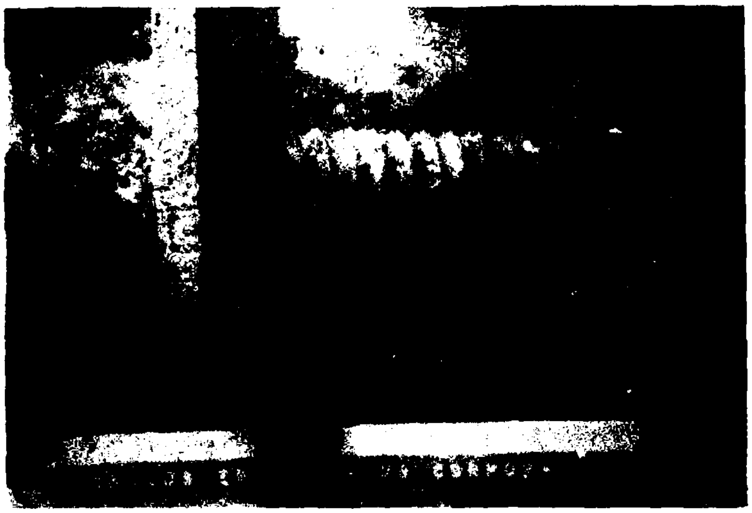


PHOTO NO. 21: Pier 2, Bent 56, Pile A; typical pile repair. Clamp fastens pile to pile cap.

PHOTO NO. 22: Pier 2, caisson between Bents 96-97 and Piles C-E; riveted lap joint on steel caisson. Illustration of typical corrosion conditions.

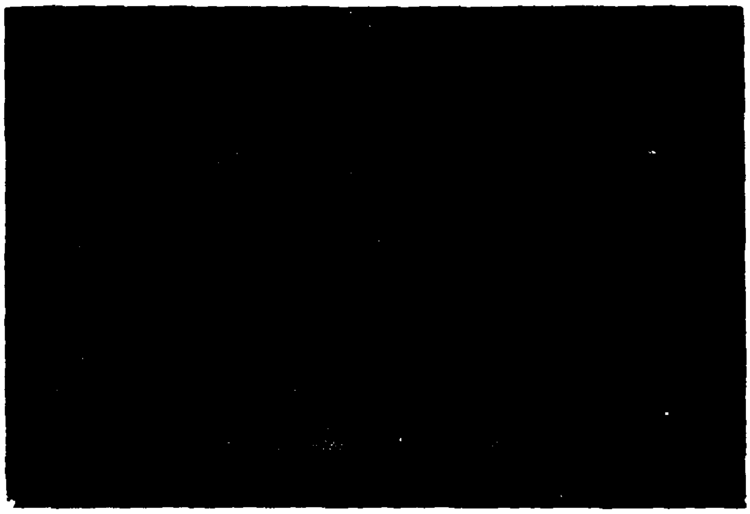


PHOTO NO. 23: Pier 2, between Bents 55-56, Piles A-B; typical tie-rod with orange corrosion nodes.

The timber sheet pile at the north end of the pier is in excellent condition as are the concrete seawall, pile caps and deck planking. The fastenings and fender system were in good condition along the pier, except for the fender system in the southern 50' of the pier which is in poor condition. Between Bents 160 and 161 and Bents 162 and 163, the tie-rod system has been dislodged from the pile caps and has fallen down.

4.4.3 STRUCTURAL ASSESSMENT

In studying the situation at the south end of Pier 2 where there are many non-bearing batter piles, it appears as if the south end of the pier is beginning to translate to the south, therefore, separating the batter piles from the pile cap. The cause of this condition is the lateral earth pressure acting on the seawall. The force exerted on the seawall is transferred to the pile cap and eventually to the pile cap connections. When high live-loads are imposed on the top deck of the pier and relatively close to the concrete seawall for an extended period of time, a resultant component horizontal (lateral) force acts on the seawall.

There is a change in the arrangement of the pile caps at Bent 167 on Pier 2. From Bent 167 to the southern end of the pier the pile caps run north and south (see Appendix A-31). The higher lateral earth pressure has caused the connection of the north-south pile cap to the east-west caps to fail (Bent 167), therefore allowing the end of the pier to begin to translate. Apparently the translation stopped when the surcharge was removed or in the process of translating the pressure was reduced. The end result is a lateral movement of approximately 4" to 6". Apparently translation of the seawall had occurred or was realized elsewhere on Pier 2 and has been rectified by the installment of the tie-rod system running across (east and west) the pier (see Appendix A-30).

The general condition of Pier 2 is good. The core samples taken indicate some softness in the timber, but it does not appear to be serious at this time. According to calculations (see Appendix A-1 to A-15 and A-21 and A-22), the reduced area of timber caused by softness has not reduced the capacity of the piles. The driven capacity of the piles rather than the column capacity is the limiting factor for this facility. Calculations indicate that Pier 2 is fully capable of supporting its present designated live-loading (600 psf), except at the south end between Bents 163 and 171, due to the potential increase of lateral pressure on the seawall resulting in the translation of that section of the relieving platform to the outshore direction.

The structural damage to perimeter piles is apparently caused by berthing and mooring forces transmitted through the use of camels.

The loss of steel, caused by corrosion on the tie-rods and WF beams is not serious and does not effect the structural integrity of the tie-back system.

4.4.4 RECOMMENDATIONS

The 5 broken piles as shown in Figures 31, 32 and 33 should be replaced at an estimated cost of $1,000 per pile. The total estimated cost would be $5,000.

The 6 wild piles, 16 split and displaced piles and 23 non-bearing piles as located in Figures 31 - 33 should be reconditioned (posted or clamped) where needed and refastened to the pile cap. The estimated cost per pile is $400.00. The total estimated cost is approximately $18,000.

The tie-rods which are unfastened from the pile cap should be returned to their original position and refastened to the pile cap. The total estimated cost is $3,000.

Until repairs are made we recommend that no live-loading occur to the south of Bent 163. A permanent solution to the problem in this area would be to install a tie-back system to the concrete seawall. This repair would cost an estimated $5,000/tie-back with eight (8) tie-backs needed, the estimated cost is $40,000 (see Appendix A-5).

Live-loading in deck areas directly associated with damaged (broken, split and wild piles) should be restricted to 25% of the current recommended live-load capacity until those piles are repaired. Following the implementation of the recommended repairs, live-loading can be maintained at current levels (600 psf).

The entire pier should be re-inspected after repairs and in 6 years thereafter with particular attention being paid to the timber softness. This will enable Shipyard personnel to determine any change in conditions. This report should be used as a baseline for all future inspections.

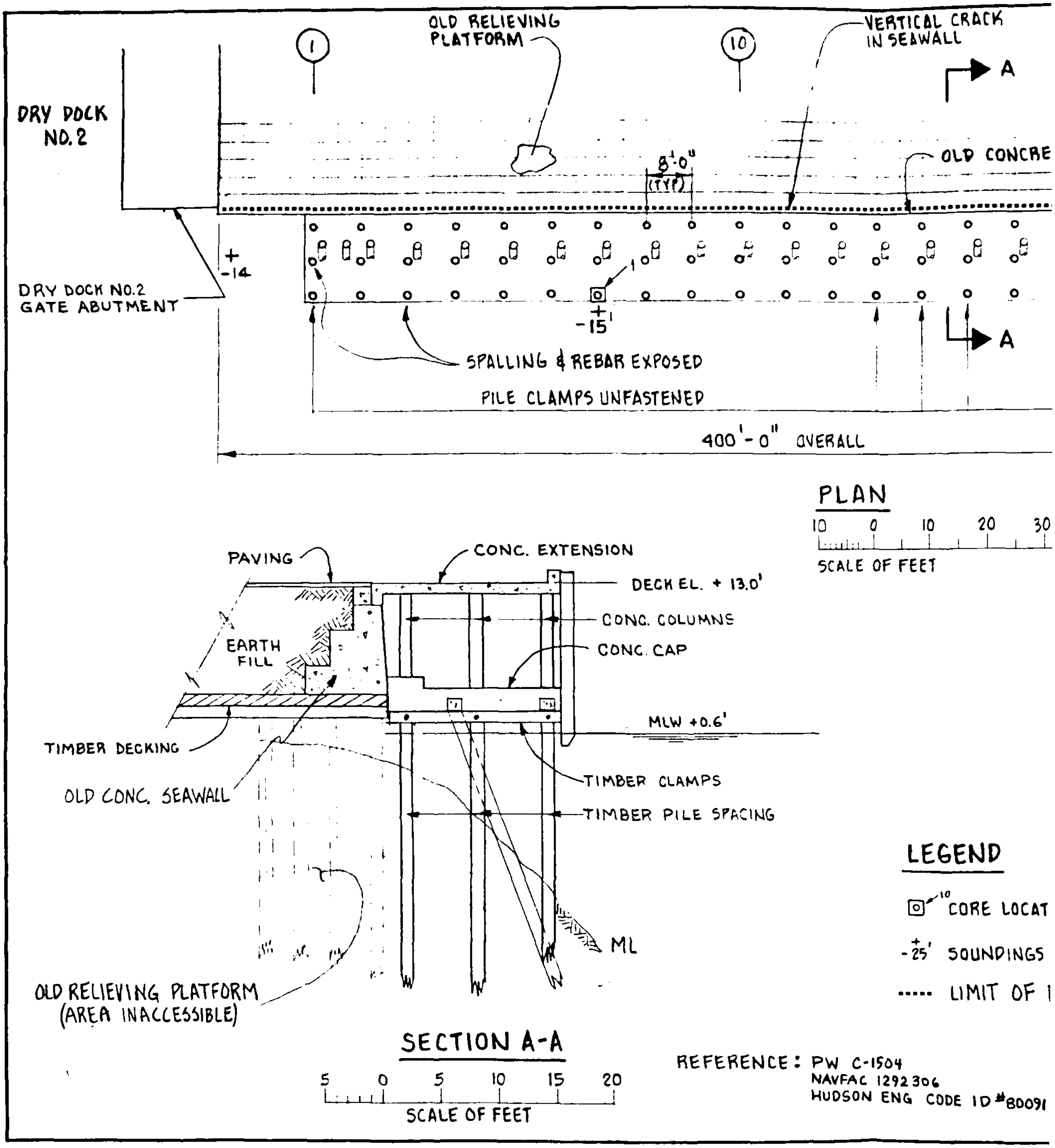
Upon implementation of the recommended repairs, we estimate the future life of this facility to be in excess of 15 years.

4.5 WHARVES 4A AND 4B

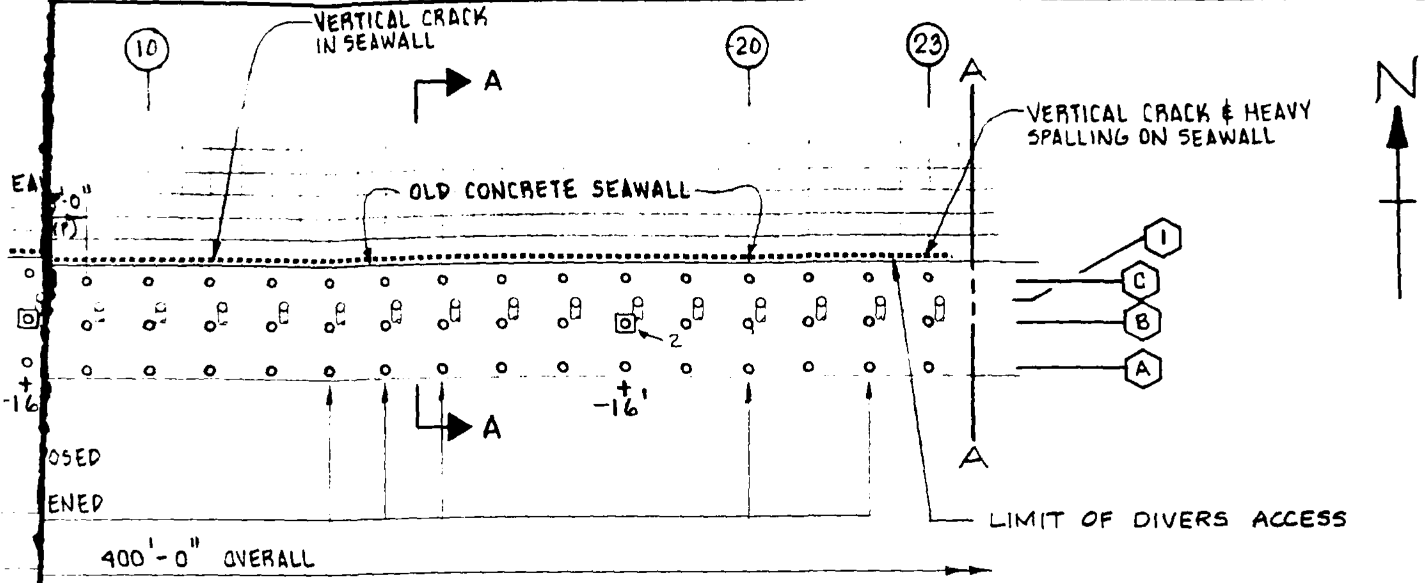
4.5.1 Description

Wharf 4A is located to the west of Pier 4 and to the east of Drydock No. 2 on the northern shore of the Delaware River, (see Figures 4, 35 and 36). Dates of construction are 1906 and 1942. The original structure consists of a timber pile-supported low deck relieving platform structure. In 1942 the original structure was expanded outshore approximately 15'. The new construction consists of 200 timber piles supporting a high deck, concrete superstructure (see Figure 35). The design pile capacity is 15 tons. The deck elevation is +13. The designated live-load capacity at the time of our inspection was 300 psf. During the inspection this wharf was being utilized as a parking location for automobiles.

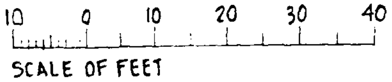
Wharf 4B is located to the east of Pier 4 and to the west of Drydock No. 1 on the northern shore of the Delaware River. Dates of construction are 1893 and 1969. The structure consists of timber pile bents inshore of the timber or steel sheet piling depending on location (see Figures 37 and 38). The deck elevation is approximately +11.5'. The designated live-load capacity is 300 psf. During our inspection Wharf 4B was being used as a storage location for piping and also as a berthing facility for the local harbor police.
(Reference No. 2, Appendix A-33)



REFERENCE: PW C-1504
NAVFAC 1292306
HUDSON ENG CODE ID #80091



PLAN



40

CH EL. + 13.0'

COLUMNS

CAP

MLW +0.6'

CLAMPS

PILE SPACING

PIL

EL

S A

NOTE:

MANY CONCRETE COLUMNS HAVE BEEN REPAIRED BY GUNNITE.

LEVEL 1 INSPECTION ON ALL PILES - HIGHER LEVELS OF INSPECTION WHERE NOTED.

LEGEND

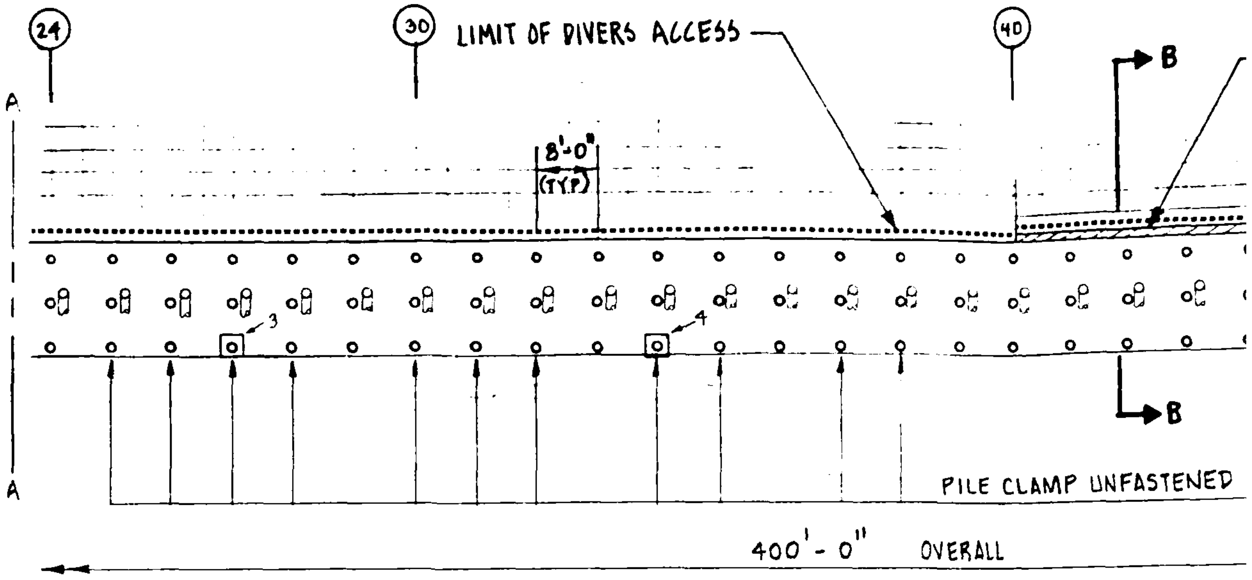
▣ 10 CORE LOCATION (PILE, CAP, DECK) LEVEL 3 INSPECTION

$\overset{+}{-25}$' SOUNDINGS (FT) BELOW MLW

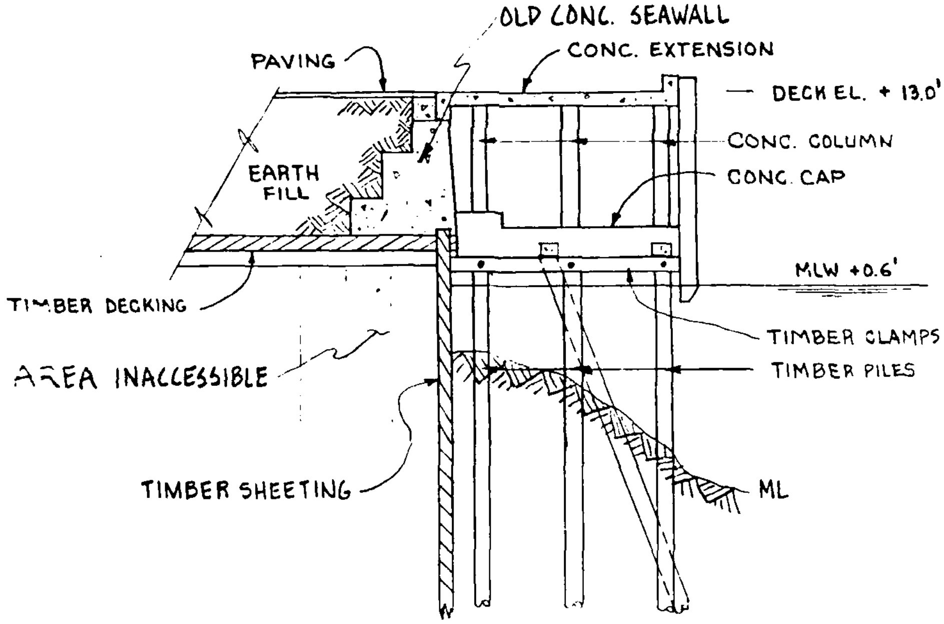
..... LIMIT OF DIVERS ACCESS

REFERENCE: PW C-1504
NAVFAC 1292306
HUDSON ENG CODE ID #80091

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C.	
AS SHOWN		PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA WHARF 4A	FIG NO 35

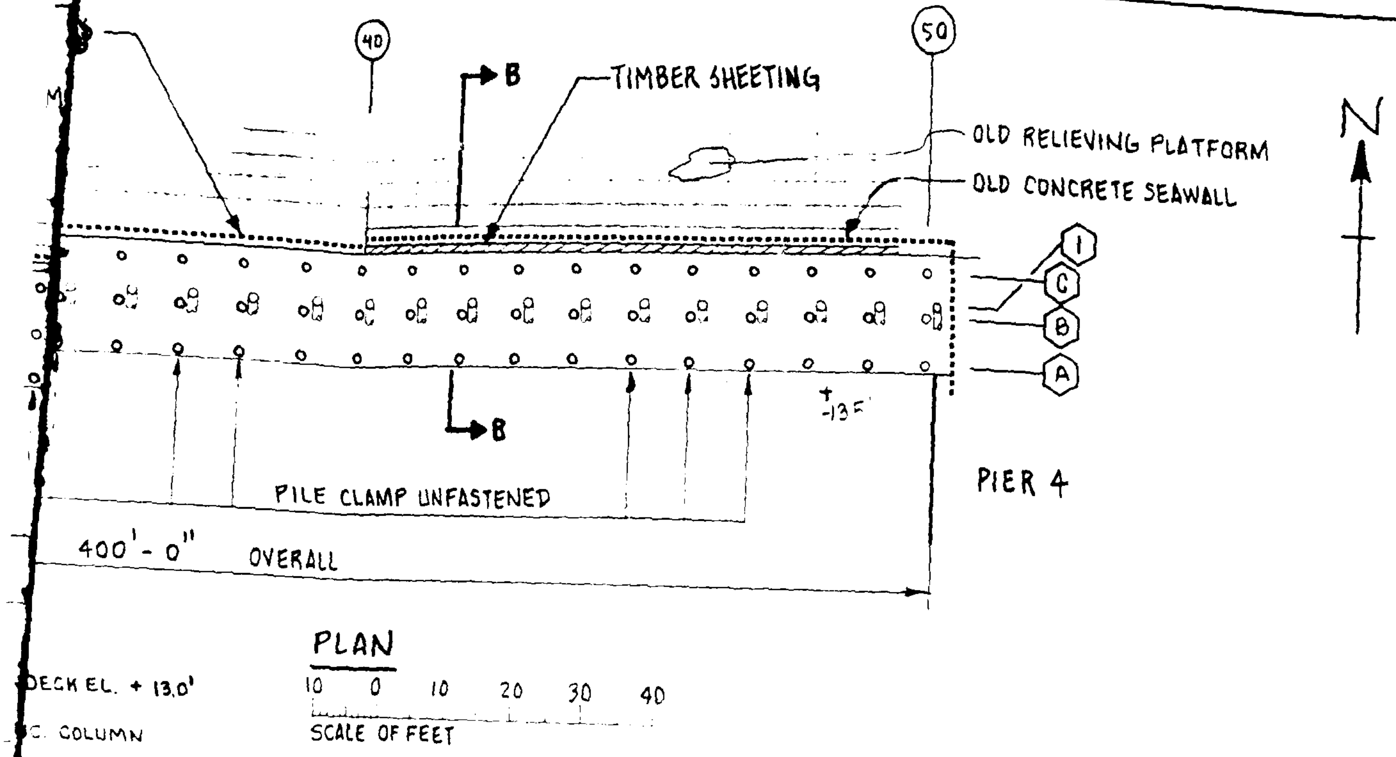


PLAN

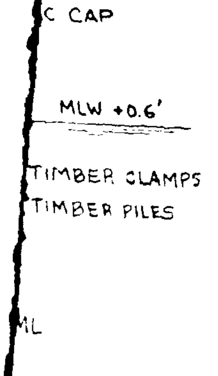


SECTION B-B

REFERENCE: PW C-1504
NAVFAC 1292306
HUDSON ENG CODE ID #80091



PLAN



LEGEND

Ⓒ

Ⓒ ^{10} CORE LOCATION (PILE, CAP, DECK) LEVEL 3 INSPECTION

$\overset{+}{-25'}$ SOUNDINGS (FT) BELOW MLW

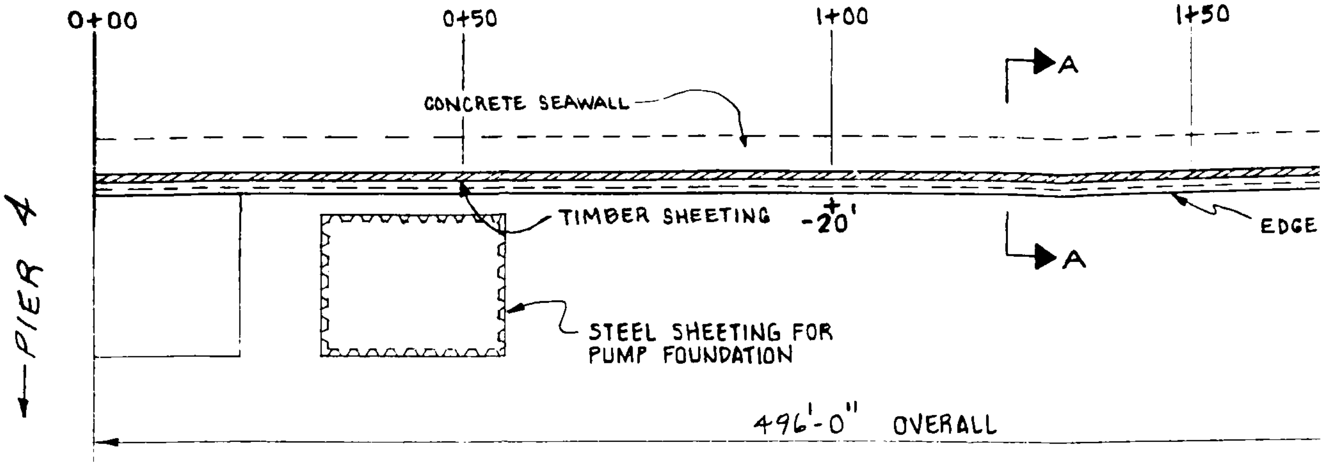
······· LIMIT OF DIVERS ACCESS

NOTES:

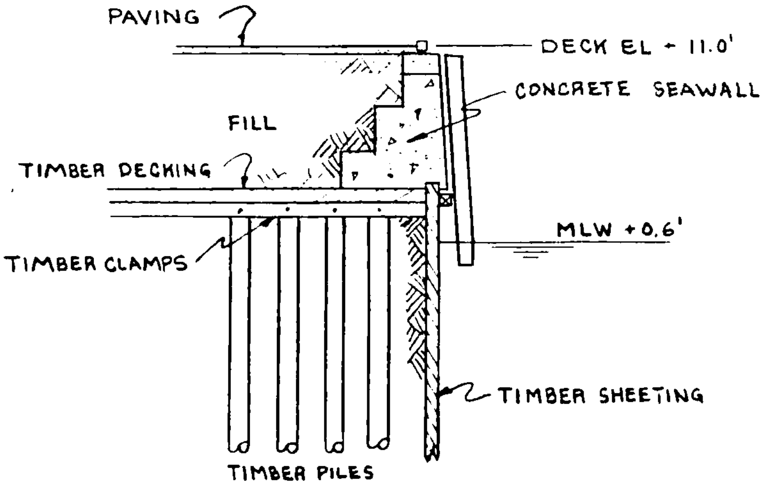
LEVEL 1 INSPECTION ON ALL PILES - HIGHER LEVELS OF INSPECTION WHERE NOTED.

REFERENCE: PW C-1504
NAVFAC 1292306
HUDSON ENG CODE 10 #80091

GRAPHIC SCALE AS SHOWN	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D C PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA WHARF 4A	FIG NO 36

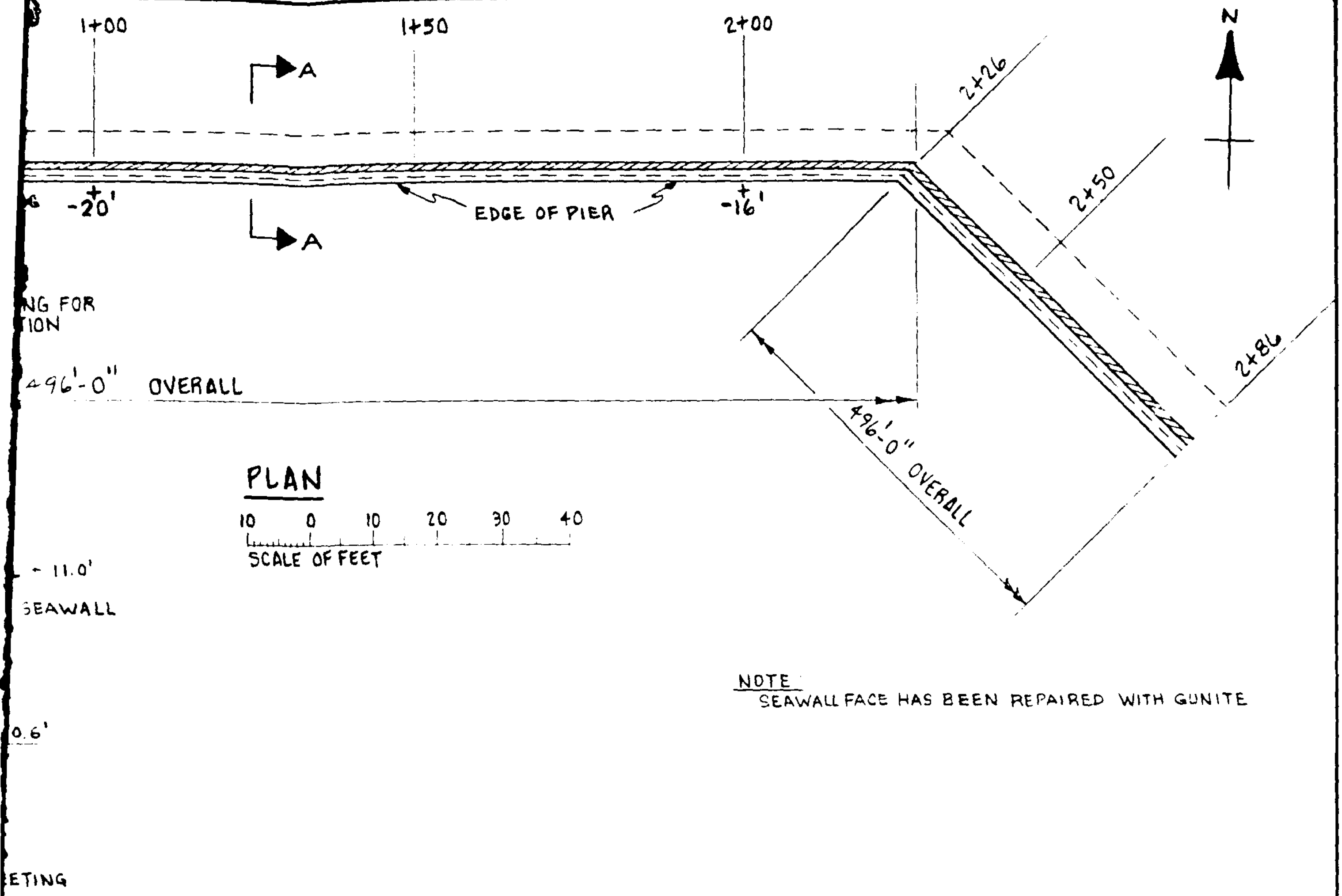


PLAN



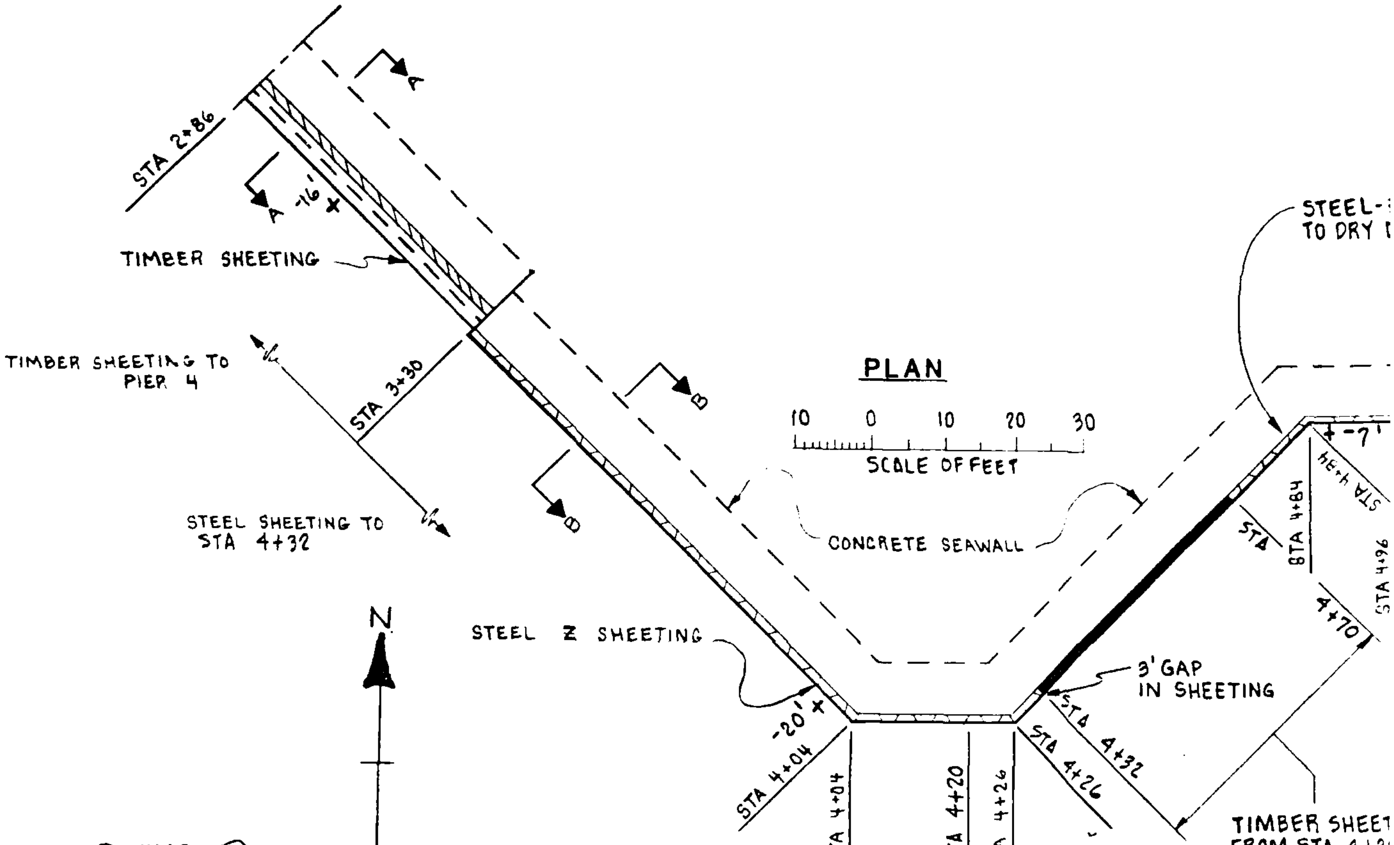
SECTION A-A

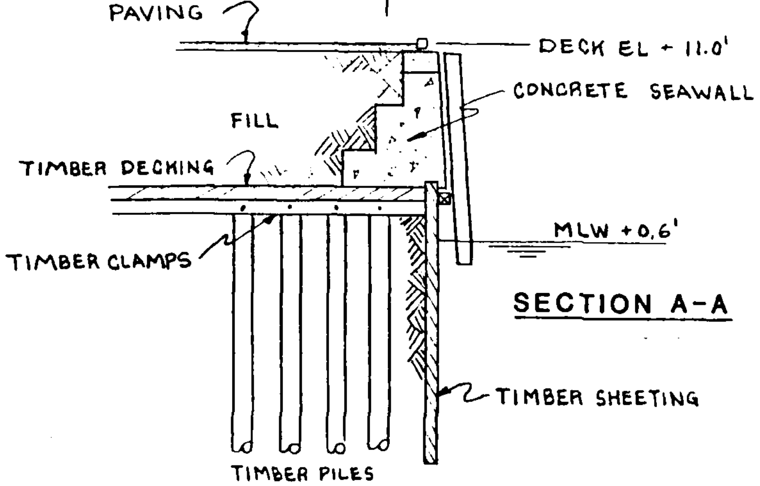
REFERENCE: PW C-1504
NAVFAC 1292306
HUDSON ENG CODE ID #80091



NOTE
SEAWALL FACE HAS BEEN REPAIRED WITH GUNITE

GRAPHIC SCALE AS SHOWN	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D.C. PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA WHARF 4B FIG NO 37





SECTION A-A

STEEL THICKNESS READING

STA 4+20

EL (FT)	WEB (IN)	FLANGE (IN)
+2.0	.425	.410
0.0	.430	.425
-4.0	.435	.425
-6.0	.435	.420
ML	.450	.385
THEORETICAL VALUES	.375	.500

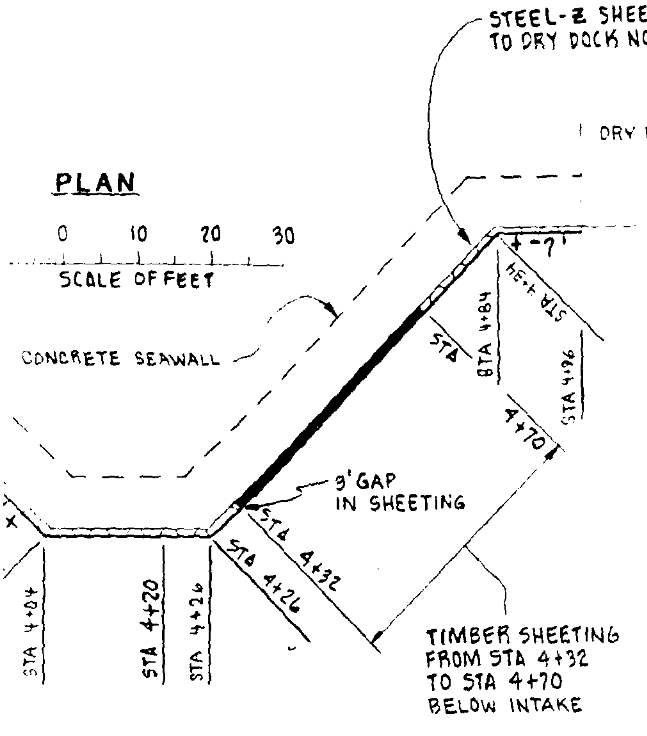
ASSUME PZ SECTION

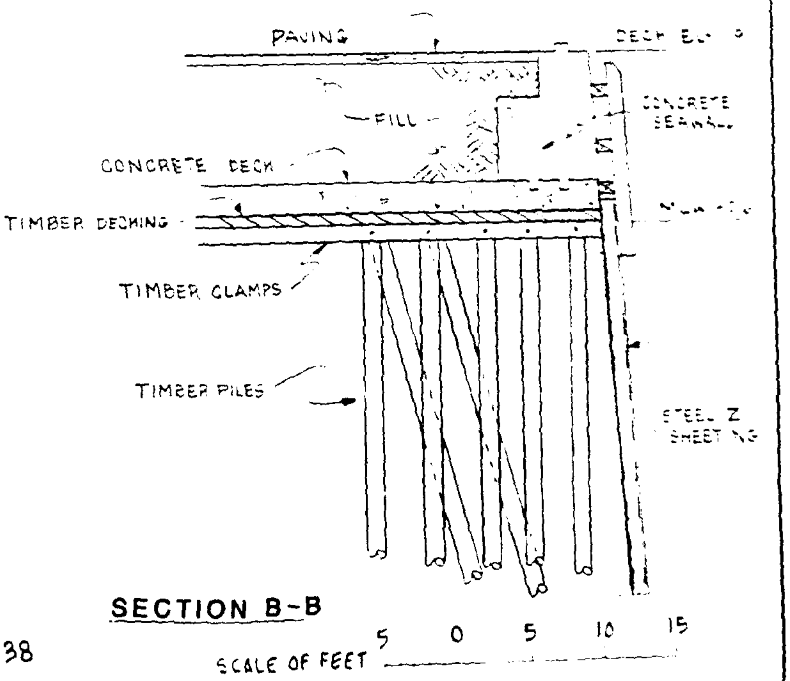
REFERENCE : PW C-1504
NAVFAC 1292306
HUDSON ENG CODE ID #80091

NOTE:

BETWEEN STA 4+70 & 4+84 STEEL SHEET PILE IS ANCHORED BY FOUR (4) TIE-BACKS.

PLAN





SECTION B-B

STEEL THICKNESS READING

STA 4+20

EL (FT)	WEB (IN)	FLANGE (IN)
+2.0	.425	.410
0.0	.430	.425
-4.0	.435	.425
-6.0	.435	.420
ML	.450	.385
THEORETICAL VALUES	.375	.500

ASSUME PZ 32 OR 38 SECTION

REFERENCE: PW C-1504
NAVFAC 1292306
HUDSON ENG CODE ID #80091

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D.C.	
AS SHOWN		PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA PA WHARF 4B	FIG NO 38

4.5.2 OBSERVED INSPECTION CONDITIONS

WHARF 4A

On Wharf 4A there were no specific anomalies concerning the structural piles. General observations of the timber piles indicate that there has been no loss of cross-sectional area, the structural piles are in excellent condition. Inspection of the core samples taken reveals that the average timber softness is less than 1", and confirms other favorable visual observations.

The timber clamps (non-structural) used to align the pile bents are becoming unfastened and falling away from the piles (see Figures 35 and 36 and Photo #24). Otherwise the fasteners connecting the batter piles to the longitudinal beam are in excellent condition (see Photo #25).

The concrete superstructure appears to be in good condition. There have been some repairs made on the concrete columns (see Section B-B, Figure 36) and beams using pneumatically-placed concrete. These repairs are in excellent condition. On Bents 1 to 4 there are approximately 200 square feet of spalled surface area with some reinforcing bar exposed. Generally, this damaged area is located near mean low water (see Photo #26).

PHOTO NO. 24: Wharf 4A, Bent 18; shows timber pile clamp connection. Note broken timber clamp and pitting on washer. Bolt is approx. 1" in diameter.

PHOTO NO. 25: Wharf 4A, Bent 18, batter pile; typical batter pile to pile cap connection. Washer is 3" in diameter.

PHOTO NO. 26: Wharf 4A; example of typically good repair of a concrete column with pneumatically-placed concrete.

WHARF 4B

The timber sheet piling along Wharf 4B was found to have approximately 1/2" of softness and was generally in sound condition. There are various locations along the bulkhead where the timber sheet piling was mis-driven, resulting in the piling being kicked away from the face of the sheet pile wall at the ML. Also there was a gap between the timber sheet piling and steel sheet piling (see photos #27 and 28) exposing fill material. Just below the intake pipe adjacent to Dry Dock No. 1, there is a large gap (3') in the sheet piling (Sta. 4+32) where the steel and timber meet. This gap is exposing fill material and allowing the fill material to wash out.

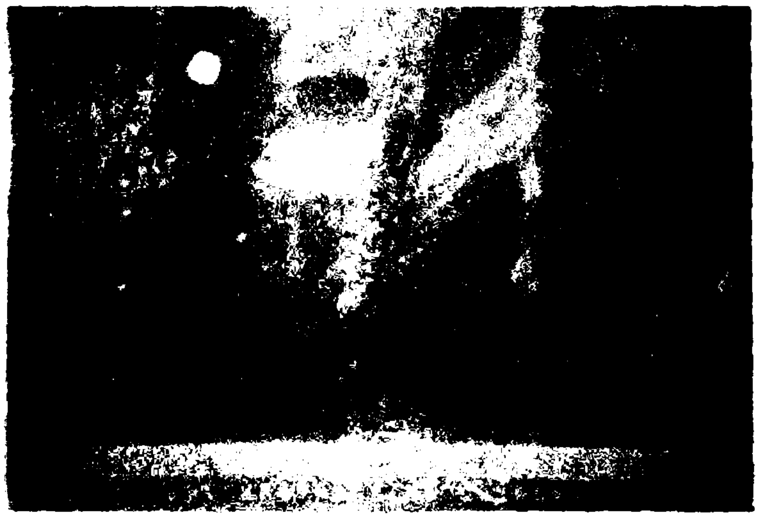
The surface of the steel sheet piling is very rough and pitted. The outer layer of corrosion is a soft orange corrosion by-product with pockets of trapped gas. Closer to the surface of the steel is a harder black layer of corrosion by-product. Pits are as deep as 1/16". Steel thickness readings (see Figure 38) show that there is minimal loss of steel due to corrosion (see Photo #29).

The concrete seawall along Wharf 4B is in fair condition. The surface of the seawall has been repaired using pneumatically-placed concrete, these repairs are beginning to deteriorate.

The fender system along Wharf 4B is in good condition. There are areas of localized impact damage.

PHOTO NO. 27: Wharf 4B, Sta. 4+32; 1" gap between timber sheet pile and steel sheet pile walls at approx. El. -10'. Fill exposed.

PHOTO NO. 28: Wharf 4B, Sta. 4+32; 5" gap between timber sheet pile and steel pile walls at El. -15'. Fill exposed. Dimensions of the triangular gap are 1' wide at ML x 15' high.



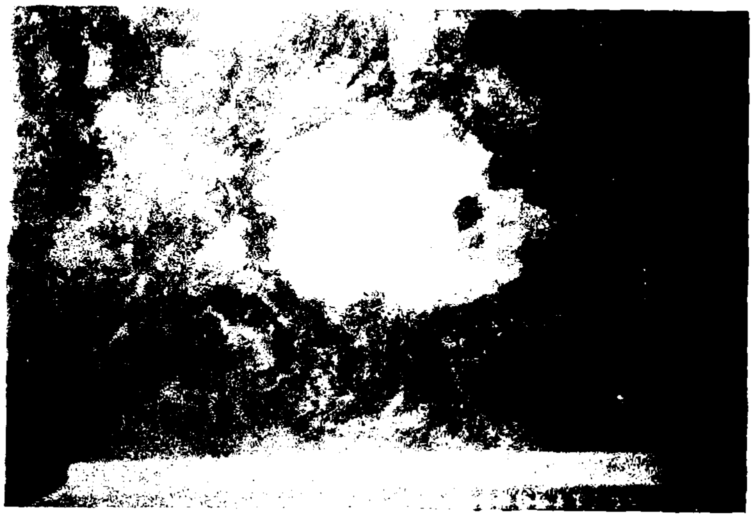


PHOTO NO. 29: Wharf 4B, Sta. 4+20, El. -6.0'; typical pitting of steel sheet pile wall. Pits approx. 1/16" deep. Orange corrosion nodes are also visible.

4.5.3 STRUCTURAL ASSESSMENT

WHARF 4A

Our analysis of typical structural piles on Wharf 4A (see Appendix A-1 to A-8) shows that the piles are fully capable of supporting the imposed live-loading (300 psf).

The unfastened timber clamps are not an integral part of the structure and do not effect the structural integrity of the wharf. Similarly, spalled areas of the superstructure do not present a condition of immediate concern, although if repairs are not made, there will be structural problems in the future.

WHARF 4B

Our analysis of the typical sheet piling (see Appendix A-18 to A-20) shows that the bulkhead is fully capable of functioning as it was designed. We could not inspect the structural piles supporting the relieving platform because access was blocked by the sheet pile. Therefore, we can only assume that the structural piles are in sound condition.

The large gap found at Sta. 4+32 is an anomaly which has been present since construction. This gap is allowing fill material to leach out from behind the wall leaving void spaces behind the wall. This condition should be repaired.

4.5.4 RECOMMENDATIONS

On Wharf 4A the spalled areas of the concrete superstructure should be repaired using pneumatically-placed concrete to provide the proper cover over the reinforcing steel. The estimated cost to cover one square foot of area and prepare the concrete surface is $14.16. The total estimated cost would be 200 sq.ft. @ $14.16/sf = $2,832.

The large gap near the intake pipe in the sheet pile wall of Wharf 4B (Station 4+32) should be fixed by a similar method to that employed on Pier 2 (see Appendix A-29) at an estimated cost of $3,000.

We recommend no reduction in the live-loading imposed presently on Wharves 4A and 4B (300 psf). Upon implementation of these repairs we estimate the life of the inspected portions of this facility to be in excess of 20 years.

The entire pier should be re-inspected after repairs and in 6 years thereafter. This inspection will enable Shipyard personnel to determine any change in conditions. This report should be used as a baseline for all future inspections.

4.6 Pier 4

4.6.1 Description

Pier 4 is located to the east of Wharf 4A and to the west of Wharf 4B on the northern shore of the Delaware River (see Figure 4).

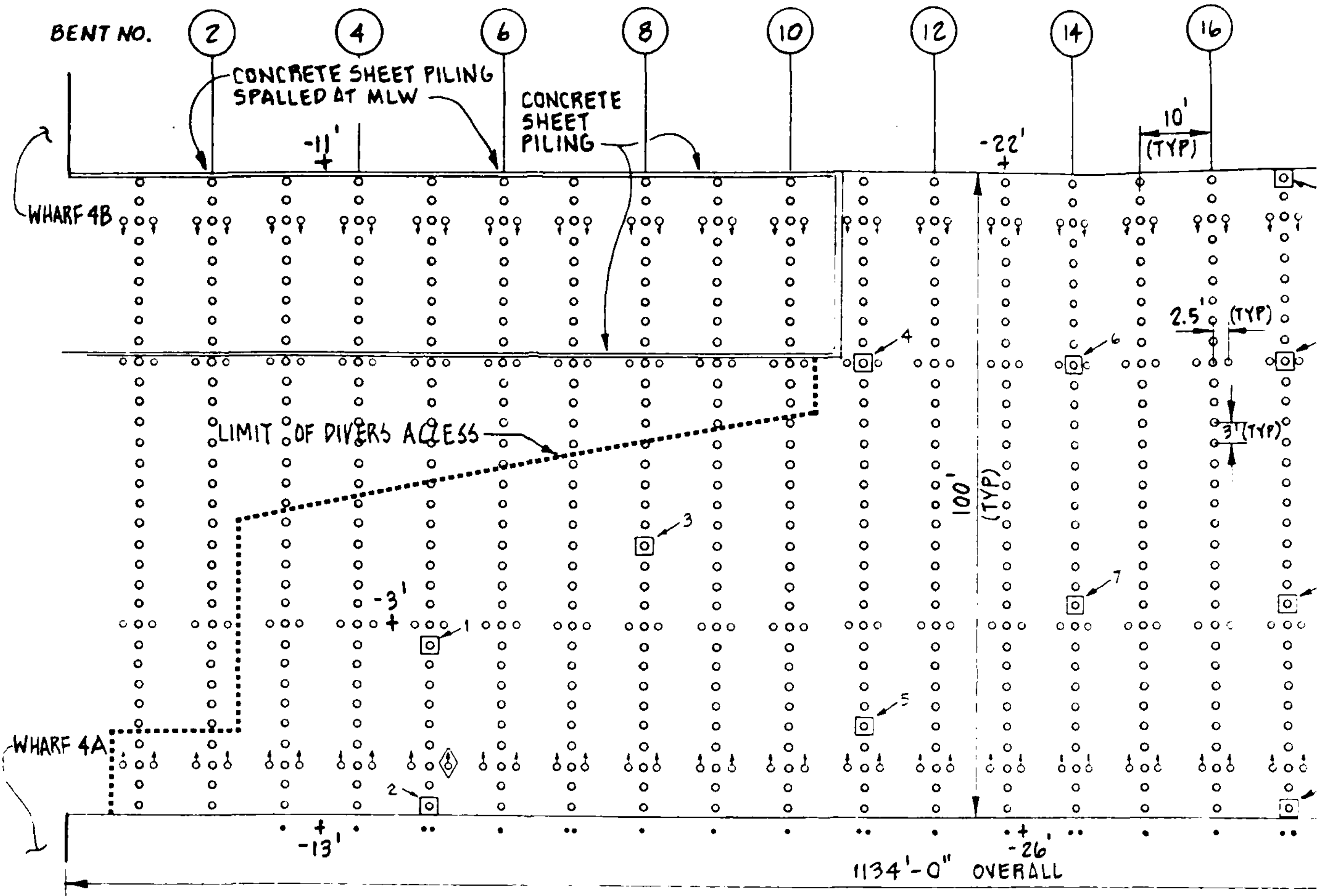
The inshore 1000 lineal feet of the timber pile-supported, transverse concrete cap wall, longitudinal concrete crane rail beams and concrete deck pier structure were constructed circa 1917.

Circa 1946, a 150 lineal foot extension was added to the outshore end consisting of two (2) simple span steel bar joist walkways, each approximately 8 feet wide and approximately 50 feet long, an intermediate steel H-pile supported concrete dolphin and a 25-foot by 48-foot wide outshore mooring-turning dolphin.

Circa 1969, the steel walkways and inner mooring dolphin were removed. A new steel H-pile supported, high deck, concrete, pier structure was constructed, measuring approximately 155 feet in length by approximately 100 feet in width. The existing outshore mooring-turning dolphin was incorporated into the extension.

The existing structure is approximately 1134 feet long by 100 feet wide. It consists of approximately 4000 timber piles and approximately 400 steel H-piles. The piles are arranged in bents of 36 piles with 10-foot bent spacing. The crane foundation perimeter is surrounded by concrete sheet piling and the inshore foundation is also surrounded by concrete sheet piles (see Figures 39 - 43). The design timber pile capacity is assumed to be 20 tons (driven capacity). The live-load capacity presently allowed on Pier 4 is 1200 psf. The deck elevation is +12.5 according to Shipyard datum. During our inspection, Pier 4 was being utilized as a permanent mooring for an aircraft carrier and a temporary mooring for a YRDM.

Reference 2, (see Appendix A-33)



PLAN

10 0 10 20 30 40 50

SCALE OF FEET

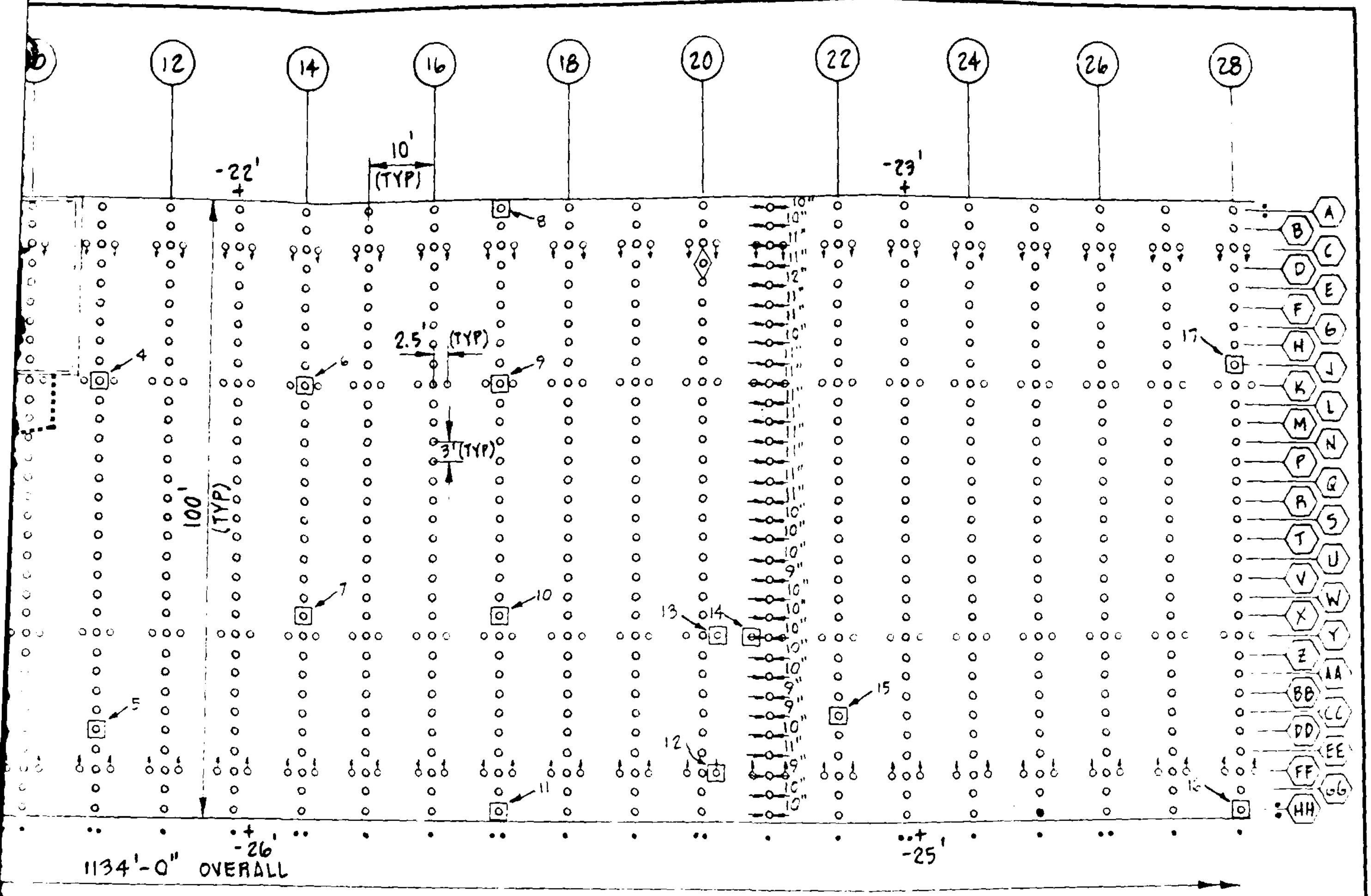
LEGEND

- ◈ NON-BEARING PILE
- ● BROKEN PILE
- ▣ CORE LOCATION (PILE, CAP, DECK)
- —o— MINIMUM PILE DIAMETER
- $\overset{+}{-25'}$ SOUNDINGS (FT) BELOW MLW
- • MODIFIED LEVEL 1 INSPECTION } PERFORMED ON ALL PILES WITHIN BENT OR ROW
- •• LEVEL 1 INSPECTION } PERFORMED ON ALL PILES WITHIN BENT OR ROW
- ······ LIMIT OF DIVERS ACCESS

REFERENCE: NAVFAC DWG NOS

1135461	1240405	1308529	PW C-2248
1135462	1308525	1308530	PW C-2249
1135463	1308526	1308531	PW C-2250
1135464	1308527	1308532	PW C-2251
1135465	1308528	1308533	PW C-13535

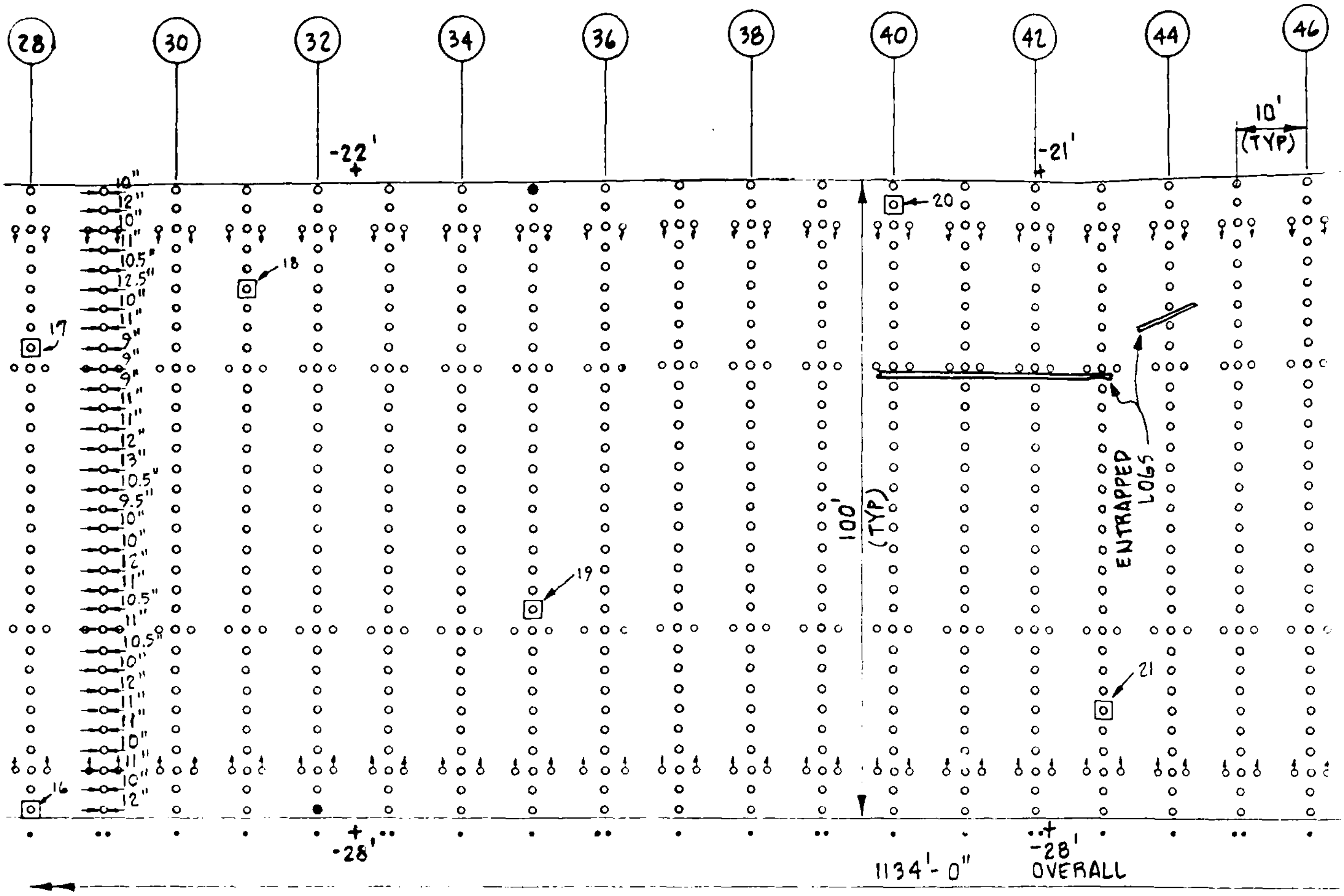
CONDITION SURVEY CODE ID NO. 80091



PLAN

REFERENCE: NAVFAC DWG NOS
1135461 1240405 1308529 PW C-2248
1135462 1308525 1308530 PW C-2249
1135463 1308526 1308531 PW C-2250
1135464 1308527 1308532 PW C-2251
1135465 1308528 1308533 PW C-13535
CONDITION SURVEY CODE ID NO. 80091

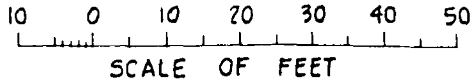
GRAPHIC SCALE AS SHOWN	CHILDS ENGINEERING CORPORATION BOX 313 MEDFIELD MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D.C. PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA PIER 4 FIG NO 39



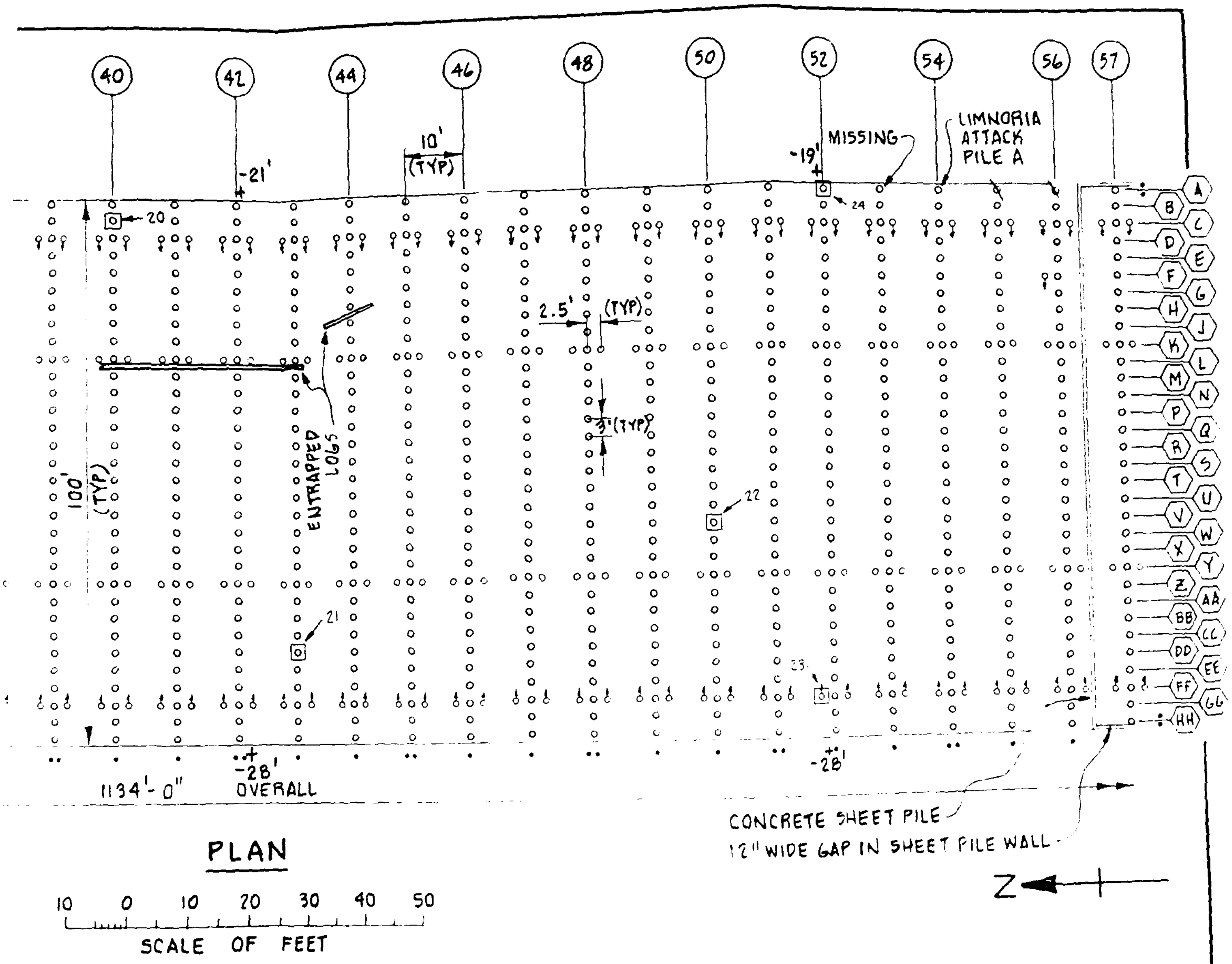
LEGEND

- · MODIFIED LEVEL 1 INSPECTION } PERFORMED ON ALL PILES WITHIN BENT OR ROW
- ·· LEVEL 1 INSPECTION
- ● BROKEN PILE
- ↘ DISPLACED- SPLIT PILE
- ⊡ 10 CORE LOCATION (PILE, CAP, DECK)
- —o— 12" MINIMUM PILE DIAMETER
- -+25' SOUNDINGS (FT) BELOW MLW

PLAN



REFERENCE: NAVFAC DWG NOS
1135461 1240405 1308529 PW C-2248
1135462 1308525 1308530 PW C-2249
1135463 1308526 1308531 PW C-2250
1135464 1308527 1308532 PW C-2251
1135465 1308528 1308533 PW C-13535
CONDITION SURVEY CODE ID NO. 80091



PLAN

REFERENCE: NAVFAC DWG NOS
1135461 1240405 1308529 PW C-2248
1135462 1308525 1308530 PW C-2249
1135463 1308526 1908531 PW C-2250
1135464 1308527 1308532 PW C-2251
1135465 1308528 1308593 PW C-13535
CONDITION SURVEY CODE ID NO. 80091

GRAPHIC SCALE
AS SHOWN

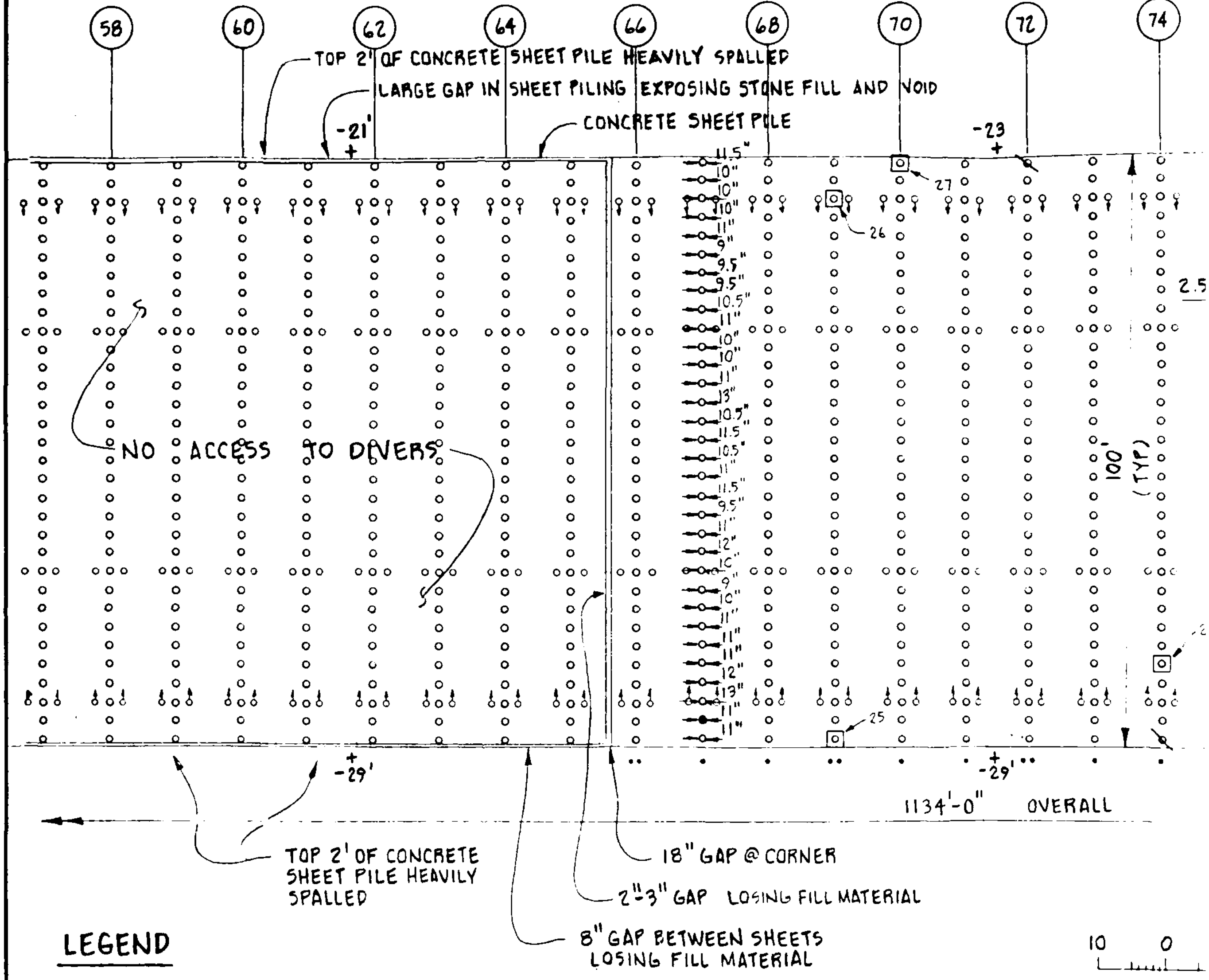
CHILDS ENGINEERING
CORPORATION
BOX 333
MEDFIELD MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D.C.

PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA PA

PIER 4

FIG NO
40



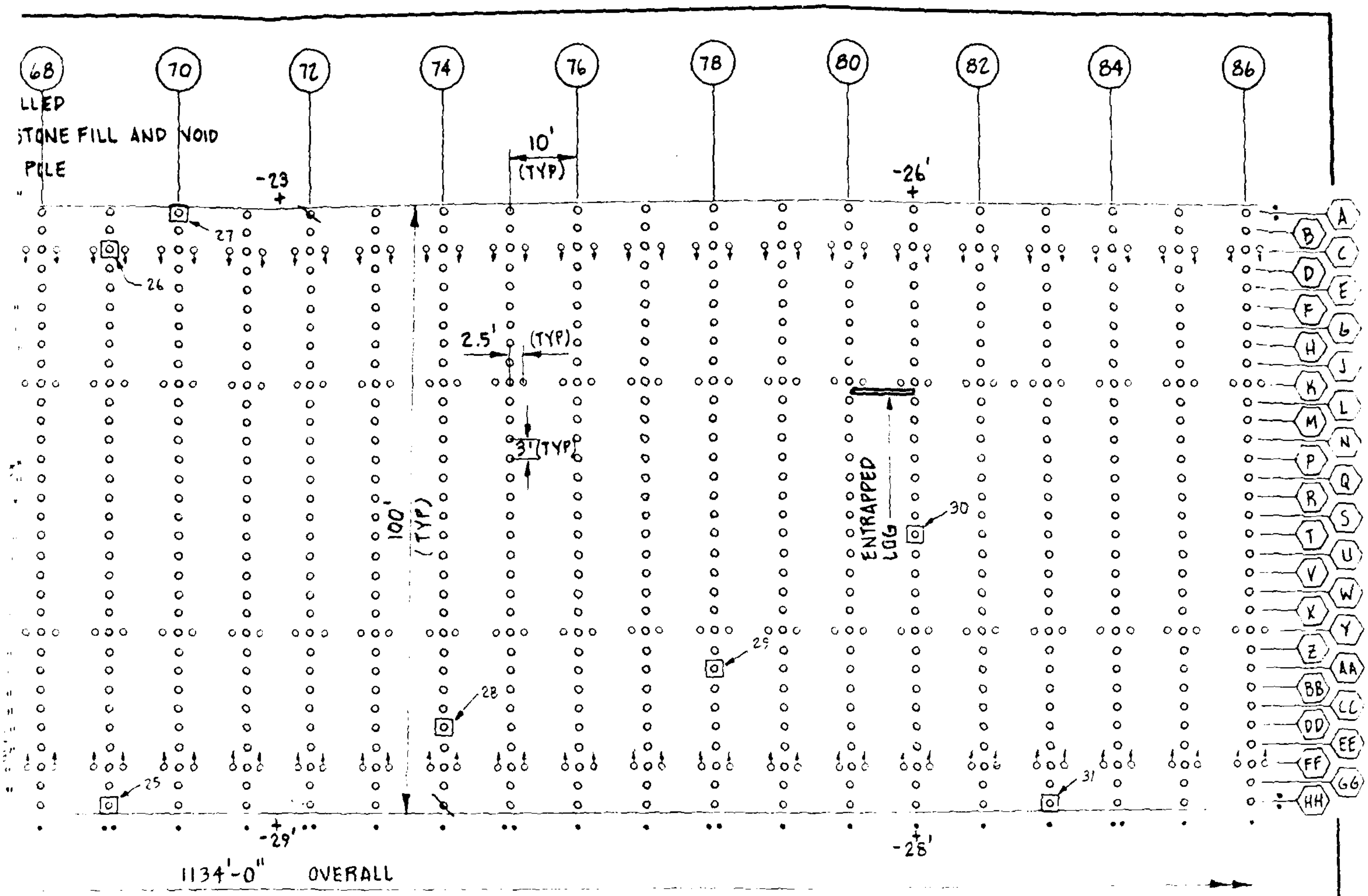
LEGEND

- ● BROKEN PILE
- ↘ DISPLACED- SPLIT PILE
- ⊡ CORE LOCATION (PILE, CAP, DECK)
- —○— MINIMUM PILE DIAMETER
- $\overset{+}{-25'}$ SOUNDINGS (FT) BELOW MLW
- • MODIFIED LEVEL 1 INSPECTION } PERFORMED ON ALL PILES WITHIN BENT OR ROW
- •• LEVEL 1 INSPECTION

REFERENCE: NAVFAC DWG NOS

1135461	1240405	1308529	PW C-2248
1135462	1308525	1308530	PW C-2249
1135463	1308526	1308531	PW C-2250
1135464	1308527	1308532	PW C-2251
1135465	1308528	1308533	PW C-13535

CONDITION SURVEY CODE ID NO. 80091



@ CORNER

LOHING FILL MATERIAL

N SHEETS
ATERIAL

PLAN

10 0 10 20 30 40 50
SCALE OF FEET

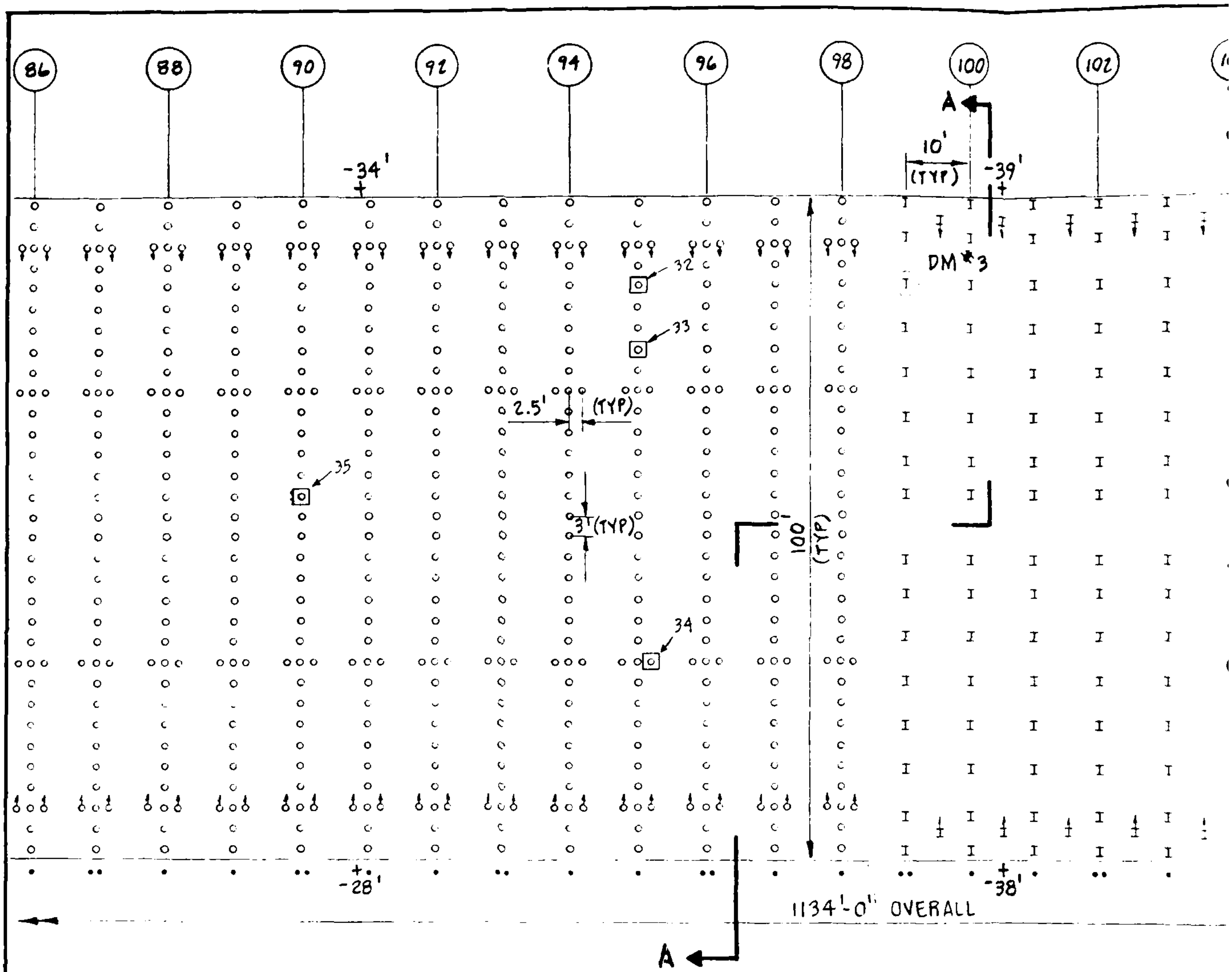
Z

EFERENCE: NAVFAC DWG NOS

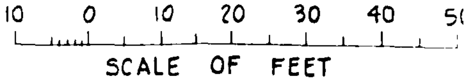
35461	1240405	1308529	PW C-2248
35462	1308525	1308530	PW C-2249
35463	1308526	1308531	PW C-2250
35464	1308527	1308532	PW C-2251
35465	1308528	1308533	PW C-13535

ONDITION SURVEY CODE ID NO. 80091

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D.C.	
AS SHOWN		PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA PIER 4	FIG NO. 41



PLAN



LEGEND

⊡ 10 CORE LOCATION (PILE, CAP, DECK)

-o- MINIMUM PILE DIAMETER

+ -25' SOUNDINGS (FT) BELOW MLW

DM*1 D-METER LOCATION

• MODIFIED LEVEL 1 INSPECTION

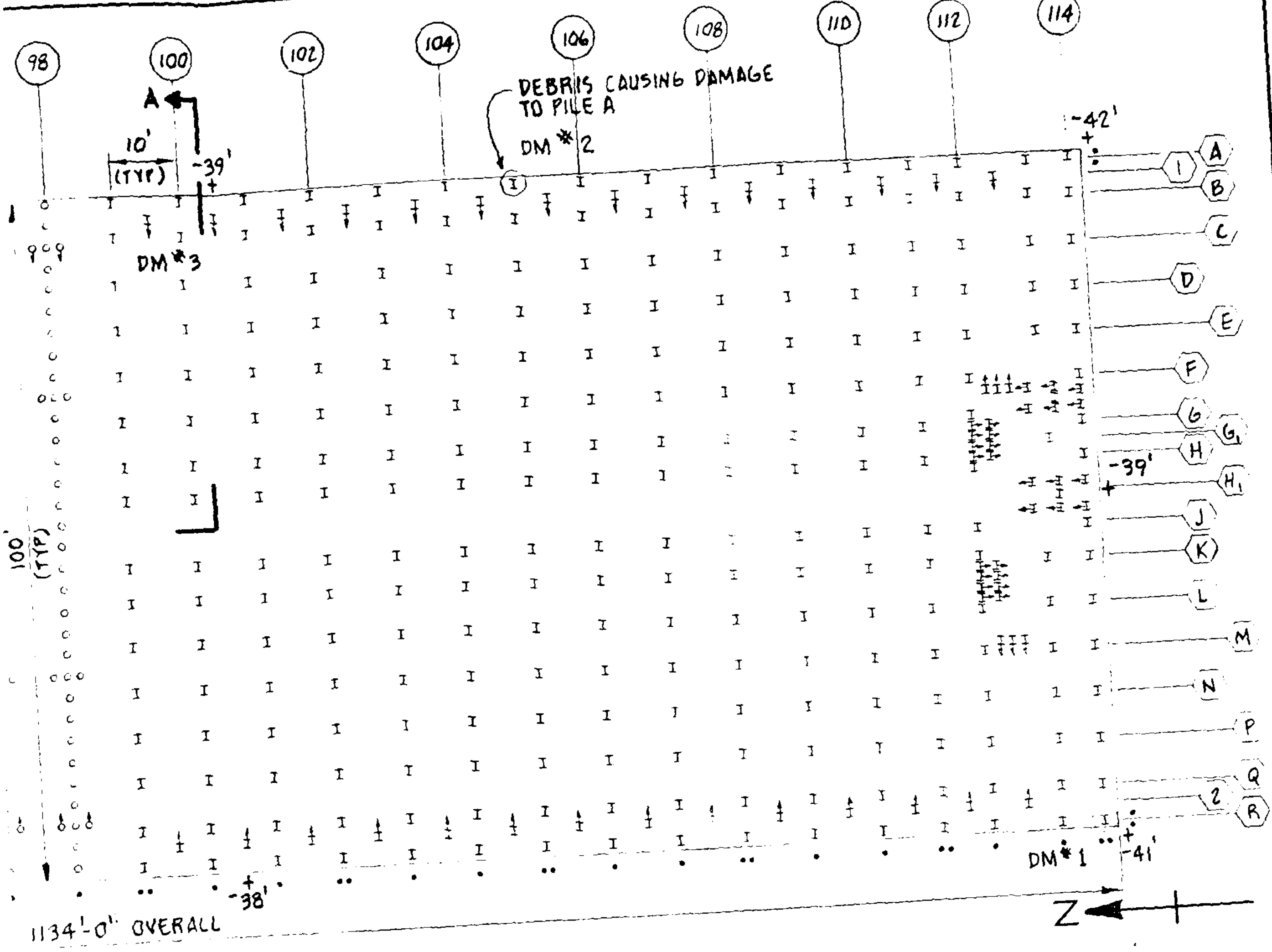
•• LEVEL 1 INSPECTION

PERFORMED ON ALL PILES WITHIN BENT OR ROW

REFERENCE: NAVFAC DWG NOS

1135461	1240405	1308529	PW C-2248
1135462	1308525	1308530	PW C-2249
1135463	1308526	1308531	PW C-2250
1135464	1308527	1308532	PW C-2251
1135465	1308528	1308533	PW C-13535

CONDITION SURVEY CODE ID NO. 80091



PLAN

10 0 10 20 30 40 50

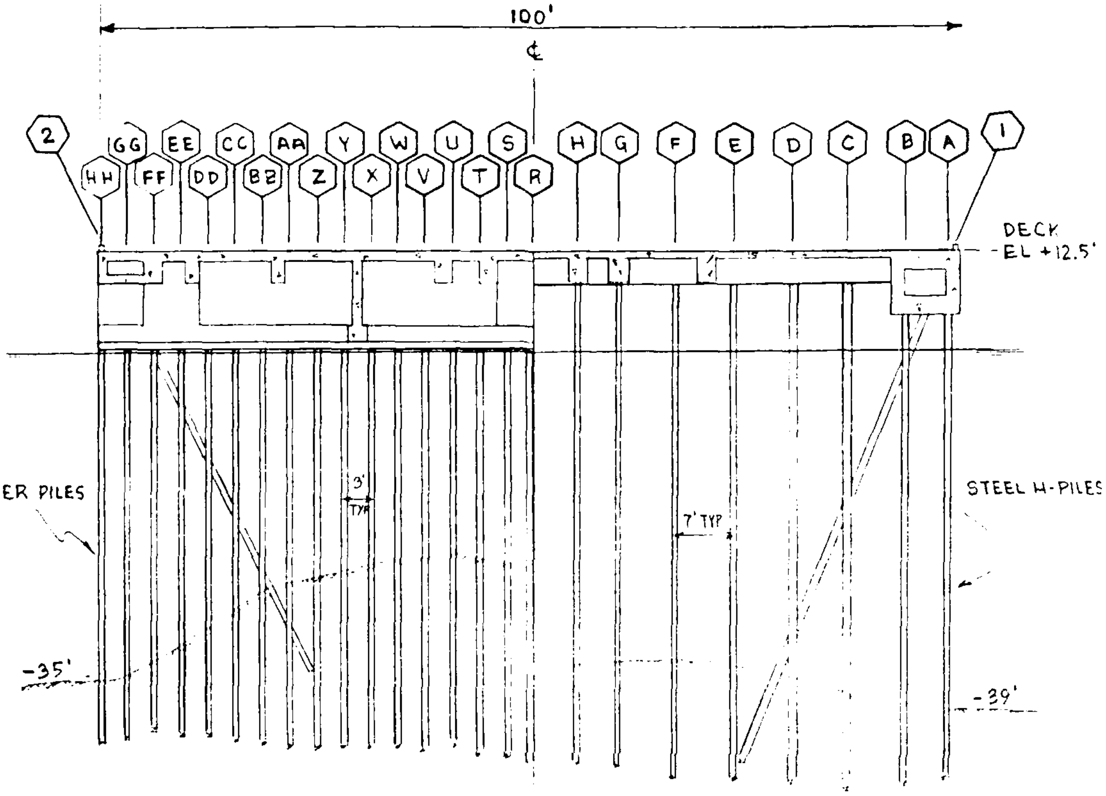
SCALE OF FEET

RENCE: NAVFAC DWG NOS
61 1240405 1308529 PW C-2248
62 1308525 1308530 PW C-2249
63 1308526 1308531 PW C-2250
64 1308527 1308532 PW C-2251
65 1308528 1308533 PW C-13535
TION SURVEY CODE ID NO. 80091

GRAPHIC SCALE
AS SHOWN

CHILDS ENGINEERING
CORPORATION
BOX 333
MEDFIELD MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D.C.
PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA
PIER 4
FIG NO
42



SECTION A-A

GRAPHIC SCALE

NO SCALE

CHILDS ENGINEERING
CORPORATION
BOX 333
MEDFIELD MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D.C.

PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA PA

PIER 4

FIG NO
43

4.6.2 OBSERVED INSPECTION CONDITIONS

Specific anomalies detected which relate to the structural piles are tabulated as follows:

2 non-bearing piles
4 broken piles
4 split and displaced piles

The anomalies can be located on Figures 39 through 42.

Visual inspection of core samples of the timber piles and timber clamps indicate that there is generally 1/2" of softness in the timber. This condition can be found throughout the facility.

Minimum pile diameters measured ranged from 9" to 13". Active Limnoria were found sporadically throughout the facility, although they were generally not highly active (see Photo #30). From our observations we conclude that there has not been a loss of cross-sectional area associated with the timber piles due to Limnoria or any other environmental interaction since their placement.

Fastenings used to connect the timber clamps and batter piles were found to be in excellent condition. The fender system was also found to be in good condition.

Inspection of the steel H-piles located at the south end of the pier reveals that there is minimal loss of steel due to corrosion. Typically, the most severe corrosion occurs at elevations near the mudline (see Figure 44).

Repairs were made to the concrete pile caps at the ends of most bents. These repairs are in excellent condition. Across 60% of

STEEL THICKNESS READINGS

PIER 4

BENT 99		PILE 3E
EL	WEB	FLANGE
0.0	.610	.650
-10.0	.605	.610
-20.0	.630	.605
ML	.560	.605

BENT 105		PILE 8E
EL	WEB	FLANGE
0.0	.645	.620
-10.0	N/A	N/A
-20.0	N/A	.575
ML	N/A	N/A

BENT 114		PILE 1W
EL	WEB	FLANGE
+1.0	.590	.665
-9.0	N/A	.600
-19.0	.600	.615
-42.0	N/A	N/A

Pile Type: HP 12x74
Original Thickness: Web .605"
Flange .610"

THICKNESS READINGS

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D C	
NO SCALE		PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA PIER 4	FIG NO 44

<u>PHOTO NO. 30:</u> Pier 4, Bent 54, Pile A; *Limnoria* tracks at the mudline. Timber core plug is 3/4" in diameter. Algal growth is approx. 1/4" deep.

the spans made for the concrete crane rails, there are large tension cracks approximately 1" to 2" wide. These cracks and also areas of the underside of the concrete deck which were spalling are exposing reinforcing steel to the environment, resulting in the corrosion of the steel reinforcing. Some repairs were made to this type of spalling with pneumatically-placed concrete, but they are deteriorating and essentially non-functional.

There are two locations on the pier (see Figures 39 and 41) that have a concrete sheet pile enclosure. The concrete sheet piling is deteriorating at El. -0.0. There is spalling of the corners of the piles and reinforcing steel is exposed. Generally, for the length of the concrete sheet piling the concrete is approximately 1/4" soft. In four locations below the stationary crane (see Figure 41), there are large gaps in the sheet pile wall. An attempt was made at one time to plug these gaps. The method of repair used concrete bags placed in the gap. These repairs are ineffective and fill is still leaching out from behind the wall, leaving large void spaces behind the wall.

From Bent 40 to Bent 43 between Piles K and L there is a large floating log entrapped inside the pier. Due to tidal action and wave action the log is abrading the adjacent structural piles resulting in a 30% loss of their original diameter. There is a similar condition at Bent 43 between Piles G and H and between Bents 80 and 81 between Piles K and L.

4.6.3 STRUCTURAL ASSESSMENT

The specific anomalies listed in the previous section can be attributed to camel overloading.

Through observations and analysis of the structural piles (see Appendix for typical timber pile and H-pile analysis, Page A-2, A-23),we can assume that no reduction in live-loading is necessary due to deficiencies in the pile foundation (pending implementation of our recommendations).

The tension cracks in the crane rail beams could present a problem if the reinforcing steel is corroded to a point where there is a significant loss of steel. Also these cracks are an indication that the beam has been overstressed.

During our inspection the large stationary crane permanently placed on Pier 4 was non-functional. The concrete sheet pile wall surrounding the piles directly below the crane is assumed to be placed to retain fill material surrounding the structural piles. The fill material will theoretically shorten the unsupported length of the pile and therefore increase the pile's column capacity. Since the crane is not functional at this time, the increased capacity is not fully utilized. Hence, the concrete sheet pile wall which retains the fill material is a redundant portion of the structure as a whole. Deterioration of the concrete sheet pile wall is noted although repairs to the deteriorated portions of the wall would serve no purpose at this time.

4.6.4 RECOMMENDATIONS

The four broken piles should be replaced. At an estimated cost of $1,000/pile, the total estimated cost is $4,000 plus mobilization/ demobilization. The four split and displaced piles and the two non-bearing piles should be reconditioned (posted or clamped) where needed and refastened to the pile cap at an estimated cost per pile of $400.00. The total estimated cost is $2,400.

We recommend a more detailed inspection of the pier superstructure be made as the above water superstructure was not within the scope of this inspection. Particular attention should be directed to the cracking and corrosion of reinforcement steel in the lower cord of the crane rail beams.

At the three locations where there is abrasion being caused (see Figures 40 and 41), the source of this abrasion should be removed. This involves removing the free floating logs. The total estimated cost is $3,000.

Live-loading capacity of enclosed areas or areas where access to all piles was restricted, are assumed to be adequate structurally since direct access could not be obtained. The capacity of the piles can only be assumed unless excavated.

Live-loading in deck areas directly associated with damaged (broken, split and wild piles) should be restricted to 25% of the current recommended live-load capacity until those piles are repaired. Following the implementation of the recommended repairs, live-loading can be maintained at current levels (1200 psf).

The entire pier should be re-inspected after repairs and in 6 years thereafter. This inspection will enable Shipyard personnel to determine any changes of conditions, using this report as a baseline for all future inspections.

APPENDIX

Conceptual Design

JOB PNSY
SHEET NO ______ OF ______
CALCULATED BY CDS DATE ______
CHECKED BY PMR DATE 11-21-83
SCALE ______

THE MAJORITY OF THE FACILITIES AT THE PNSY CONSIST OF THE LOW DECK, EARTH FILL, TIMBER PILE SUPPORTED, RELIEVING PLATFORM STRUCTURE. GENERALLY THE BENT SPACING IS 4' ON CENTER AND THE PILE SPACING IS 4' ON CENTER. DUE TO LOOSE QUALITY CONTROL DURING THE CONSTRUCTION OF SOME FACILITIES, ON A REGULAR BASIS THE BENT SPACING IS AS MUCH AS 5' AND THE PILE SPACING IS ALSO 5'. THESE MAXIMUM SPACINGS ARE NOT TYPICAL AND ARE NOT CONTROLLING FACTORS. THE FOLLOWING CALCULATIONS HAVE TAKEN THE AVERAGE EXISTING CONDITION AND DETERMINED THE LIMITING COMPONANTS WITH RESPECT TO THE TOP DECK LIVE LOAD CAPACITY. ALSO IN THE APPENDIX ARE ANALYSES OF SPECIFIC ANOMALIES AND NON-TYPICAL CONDITIONS.

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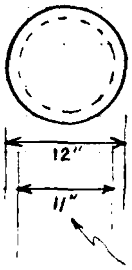
Box 333
MEDFIELD, MA 02052

JOB PNSY
SHEET NO 1 OF 6
CALCULATED BY CDS DATE 11-° 83
CHECKED BY FAR DATE 11-21-83
SCALE

DETERMINE TIMBER PILE COLUMN CAPACITY



REDUCED PILE DIAMETER DUE TO TIMBER SOFTNESS

ASSUME :

$E = 1.6 \times 10^6$ #/in^2
$\ell = 480''$ = 40'
$K = .7$
$r = 2.75$ in

FROM THE TIMBER CONSTRUCTION MANUAL

USE : $$F_{c'} = \frac{3.619\ E}{\left(\frac{K\ell}{r}\right)^2}$$ AITC

$F_{c'} = 387$ #/in^2

$F_c = 387$ #/in^2 (.9) DURATION OF LOAD FACTOR

$F_c = 348$ #/in^2

$P = F_c\ A$

$P = (348$ #/in^2) 95 in^2

$P = 33$ K = 16.5 TONS

16.5T COLUMN CAPACITY > 15T DRIVEN CAPACITY

THEREFOR THE DRIVEN CAPACITY IS LIMITING

JOB PNSY
SHEET NO 2 OF 6
CALCULATED BY CDS DATE 11-9-83
CHECKED BY FAR DATE 11-21-83
SCALE

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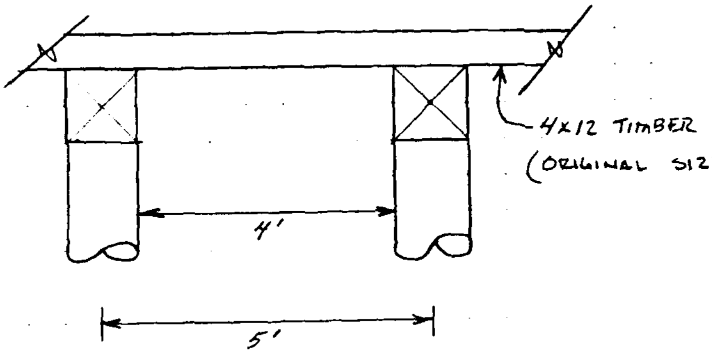
DETERMINE DECK TIMBER CAPACITY



ASSUME :

$F_b = 1650 \#/in^2$

$M_{max} = \frac{w\ell^2}{12}$

$F_v = 120 \#/in^2$

$F_{c\perp} = 315 \#/in^2$

$\ell = 48''$

REDUCED SECTION = 3½ x 11¼

BENDING

$S = 23 \ in^3$

$M_{max} = S \ F_b \ .9$ ← DURATION OF LOAD FACTOR

$M_{max} = 34$ in K

$w = \frac{12 \ M_{max}}{\ell^2}$

$w = .17$ K/in = 2.13 K/Ft

HORIZ. SHEAR

FOR RECTANGULAR BEAMS $F_v = \frac{3V}{2A}$

$w = \frac{2 \ F_v \ A \ .9}{3 \ \ell'}(2)$ $\ell' = 40''$

$w = .14$ K/in = $\underline{\underline{1.68 \ K/Ft}}$ LIMITING

CRUSHING

AREA OF BEARING 144 in²

$w = \frac{(315\#/in^2)(144 \ in^2) \ .9 \ (.67)}{60 \ in}$ DURATION OF LOAD; MOISTURE CONTENT FACTOR TCM P.270 TABLE 2.14 g

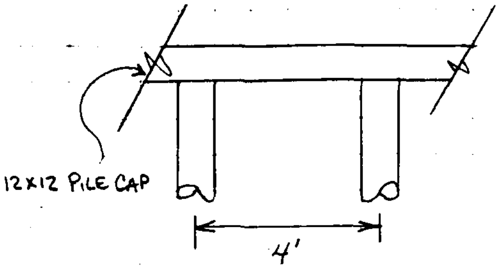
$w = 455\#/in = 5.46$ K/Ft

A-3

PRODUCT 204-1 NEBS Inc. Groton Mass. 01471

JOB PNSY
SHEET NO 3 OF 6
CALCULATED BY CDS DATE 11-9-83
CHECKED BY TAR DATE 11-21-83
SCALE

DETERMINE TIMBER PILE CAP CAPACITY



ASSUME: $F_b = 1650 \#/IN^2$

$M_{MAX} = \frac{Wl^2}{12}$

$F_v = 120 \#/IN^2$

$F_{c\perp} = 315 \#/IN^2$

USE REDUCED SECTION DUE TO SOFTNESS
11x11

BENDING

$S = 221 \ IN^3$

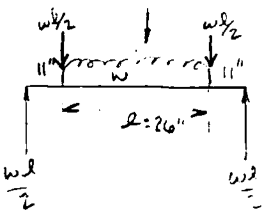
$M_{MAX} = S \ F_b \ .9$ ← DURATION OF LOAD FACTOR

$M_{MAX} = 328{,}185 \ IN-\#$

$W = \frac{12 \ M_{MAX}}{l^2}$

$W = 1.7 \ K/IN = 20 \ K/FT$

HORIZ. SHEAR



FOR RECTANGULAR BEAMS $F_v = \frac{3V}{2A}$

$W = \frac{2 \ F_v A \ .9}{3l} \ (2)$ ← DURATION OF LOAD

$W = 670 \#/IN = \underline{\underline{8 \ K/FT}}$ LIMITING

CRUSHING

AREA OF BEARING, $A_B = 180 \ IN^2$

$W = \frac{(315 \#/IN^2)(180 \ IN^2) \ .9 \ .67}{48 \ IN}$

DURATION OF LOAD

MOISTURE CONTENT FACTOR

$W = 8.54 K/FT$

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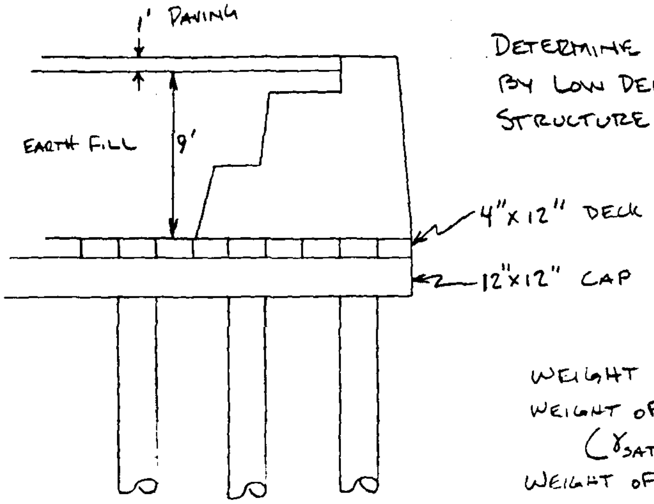
JOB PNSY
SHEET NO 4 OF 6
CALCULATED BY CDS DATE 11-9-83
CHECKED BY FATZ DATE 11-21-83
SCALE



DETERMINE UNIT DEAD LOAD IMPOSED BY LOW DECK, EARTH FILL, RELIEVING PLATFORM STRUCTURE.

WEIGHT OF PAVING - $150\ ^{\#}/\mathrm{Ft}^2 = 150^{\#}$
WEIGHT OF EARTH FILL - $125^{\#}/\mathrm{Ft}^3 @ 9\mathrm{Ft}^3 = 1125^{\#}$
$(\gamma_{SAT} = 125^{\#}/\mathrm{Ft}^3)$
WEIGHT OF 4" DECK $= 64^{\#}/\mathrm{Ft}^3 @ .33\mathrm{Ft}^3 = 21^{\#}$
$1296^{\#}/\mathrm{Ft}^2$

WEIGHT OF PILE CAP $= 64^{\#}/\mathrm{Ft}^3 @ 4\ \mathrm{Ft}^3 = 256^{\#}$

- IN CONSIDERING THE DL ON TIMBER PILES AND TIMBER PILE CAPS USE A UNIT LOAD OF $1.5\ \mathrm{K}/\mathrm{Ft}^2$
- IN CONSIDERING THE DL ON THE TIMBER DECKING USE A UNIT LOAD OF $1.3\ \mathrm{K}/\mathrm{Ft}^2$

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JOB PNS'Y
SHEET NO 5 OF [illegible]
CALCULATED BY CDS DATE
CHECKED BY PATC DATE 11-21-8[illegible]
SCALE

IN ASSUMING MAXIMUM BENT SPACING OF 5' AND A PILE SPACING OF 4' WE CAN DETERMINE THE ALLOWABLE LIVE LOAD CAPACITY

LIMITING FACTORS -

PILECAP -	8 K/Ft
DECK PLANK -	1.68 K/Ft²
TIMBER PILE -	30 K

- THE DL ON THE TIMBER PILE IS 1.5 K/Ft² ; ASSUME AN AREA OF 16 Ft² IS SUPPORTED BY 1 PILE DUE TO THE TYPICAL LOAD DISTRIBUTION.

TOTAL DL = 24 K
ALLOWABLE LOAD = 30 K /PILE

LL = 30K - 24K = 6 K
LL = 375 PSF

IF THE ALLOWABLE LOAD IS 40 K /PILE
THEN LL = 1000 PSF

- THE DL ON THE PILE CAP IS 1.5 K/Ft² ; ASSUME AN AREA OF 16 Ft² IS SUPPORTED.

TOTAL DL = 24 K
ALLOWABLE LOAD FOR 4' SPAN = 32 K

LL = 32K - 24K = 8 K
= 500 #/Ft²

AD-A168 464 UNDERWATER FACILITIES INSPECTIONS AND ASSESSMENTS AT 3/3
PHILADELPHIA NAVAL S..(U) CHILDS ENGINEERING CORP
MEDFIELD MA OCT 83 CHES/NAVFAC-FPO-1-83(48)-1
UNCLASSIFIED N62477-81-C-0448 F/G 13/2 NL

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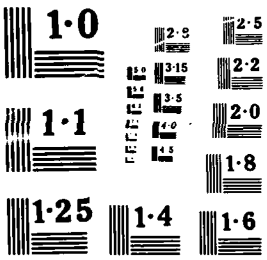
1·0
2·8
2·5
3·15
2·2
3·5
2·0
1·1
4·0
4·5
1·8
1·25
1·4
1·6

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- THE DL ON THE DECK PLANKING IS 1.3 K/Ft^2 WITH A 5' BENT SPACING; THE LENGTH OF DECK PLANK UNDER CONSIDERATION IS 3.33'

TOTAL DL = 4.3 K
TOTAL ALLOWABEE LOAD = 5.5 K

LL = 5.5K − 4.3 K = 1.29 K
= 388 #/Ft^2

IN THE TYPICAL RELIEVING PLATFORM STRUCTURE THE TIMBER DECKING IS THE CAPACITY WHICH LIMITS THE LIVE LOADING. THE CALCULATIONS SHOW THAT 388 #/Ft^2 IS THE MAXIMUM LL THAT THE TYPICAL RELIEVING PLATFORM CAN HANDLE. ALTHOUGH, IF THE PILE SPACING AND BENT SPACING ARE LESS THAN 4' AND 5' RESPECTIVLY, THE CAPACITY OF THE RELIEVING PLATFOM IS MUCH GREATER.

TIMBER PILE DATA SUMMARY

FACILITY	**RANGE OF STRUCTURAL TIMBER SOFTNESS DETECTED	RANGE OF PILE DIAMETERS OBSERVED	TIMBER PILE DRIVEN CAPACITY***
Eastern Seawall	3/4" ave.	11" - 15"	3 - 20
Pier 7	2" - 6"	10" - 15"	15
Pier 1 & Bulkhead	1" ave.	11"	15
Pier 2	1" - 2"	10" - 14"	15
Wharves 4A & 4B	3/4" ave.	9" - 14"	15
Pier 4	1/2" ave.	9" - 13"	15 - 20
Pier 5	1/4" - 1/2"	10" - 17"	20
Barge Basin & Bkhd	1/2" - 1"	9" - 14"	15
Pier 6	1/4" - 1"	10" - 14"	15
Pier 6A-Bulkhead	1" - 4"	10" - 13"	15
DD Wharves	1/2" ave.	11" - 18"	15
Wharves K,J,I,H,G	1/2" ave.	10" - 16"	15
Wharf F/Pier F	1/2" - 1"	11" - 15"	15
Wharf E	1/2" - 1½"	9" - 14"	15
Rowan Ave.	NA*	NA*	NA*
2nd Street	1½" - 3"	9" - 12"	15
Preble Ave.	1½" - 2"	8" - 10"	15
Broad Street	1½" - 3"	11" - 14"	15
Wharf L	1/2" - 1½"	9" - 10"	15
Wharf N	1" - 3"	9" - 14"	15

* NA = Not Applicable

** For detailed account of timber softness, i.e., variations between piles, caps, decking; see individual facility's Observed Inspection Condition.

*** Timber pile driven capacities have been extrapolated from GFI such as the Hudson Engineers Report or actual NAVFAC or PW drawings.

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TYPICAL SEAWALL CALCULATIONS

EASTERN SEAWALL

THE PURPOSE OF THE FOLLOWING CALCULATIONS IS TO A) DETERMINE THE SURCHARGE W THAT WOULD RESULT IN SLIDING AND OVERTURNING OF THE CONCRETE SEAWALL, AND B) DETERMINE THE LATERAL STABILITY OF THE STRUCTURE GIVEN THAT BOTH VERTICAL PILES ARE NON-BEARING.

USING NAVFAC DRAWING NO. 2044129 ENTITLED BORING LOCATIONS AND BORING LOGS, AND INFORMATION OBTAINED FROM THE HUDSON ENGINEERING REPORT OF 1976, A TYPICAL SOIL PROFILE IS CONSTRUCTED. ASSUMED VALUES OF ϕ ARE USED TO CALCULATE LATERAL EARTH PRESSURE COEFFICIENTS K_A AND K_P. UNIFORM DEAD LOADS ARE CALCULATED USING AN ASSUMED VALUE OF γ, THE UNIT WEIGHT OF SOIL.

LATERAL EARTH FORCES DUE TO THE SOIL AND AN UNKNOWN SURCHARGE ARE CALCULATED AND SET EQUAL TO THE KNOWN FORCES RESISTING SLIDING OF THE CONCRETE SEAWALL. THE SURCHARGE THAT WOULD RESULT IN IMPENDING SLIDING CAN THEN BE SOLVED FOR. SIMILARLY, THE OVERTURNING MOMENT DUE TO THE SAME LATERAL EARTH FORCES ARE SET EQUAL TO THE KNOWN RESISTING MOMENT, AND THE SURCHARGE THAT WOULD RESULT IN IMPENDING ROTATION OF THE SEAWALL IS SOLVED FOR. THE RESULTS INDICATE THAT THE SURCHARGE THAT CAN BE PLACED BEHIND THE SEAWALL IS LIMITED TO 216 PSF BY THE OVERTURNING FAILURE MODE.

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TYPICAL SEAWALL CALCS.

EASTERN SEAWALL

CALCULATE K_A FOR MISC FILL (SAND, CLAY, SOME GRAVEL)

ASSUME $\phi = 25°$ p30 TABLE 2-2 BOWLES
$\delta = 16°$ p343 TABLE 11-6 BOWELS
$\beta = 0$ } GEOMETRY OF WALL
$\alpha = 90°$ }

$$K_A = \frac{\sin^2(\alpha+\phi)}{\sin^2\alpha \sin(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2}$$

$= .362$

CALCULATE K_A, K_P FOR GRAY SILT, SEAMS FINE SAND

ASSUME $\phi = 20°$ p30 TABLE 2-2 BOWLES
$\delta = 16°$ p343 TABLE 11-6 BOWLES
$\beta = 0$ } GEOMETRY OF WALL
$\alpha = 90°$ }

$$K_A = \frac{\sin^2(\alpha+\phi)}{\sin^2\alpha \sin(\alpha-\delta)\left[1+\sqrt{\frac{\sin(\phi+\delta)\sin(\phi-\beta)}{\sin(\alpha-\delta)\sin(\alpha+\beta)}}\right]^2}$$

$= .433$

$$K_P = \frac{\sin^2(\alpha-\phi)}{\sin^2\alpha \sin(\alpha+\delta)\left[1-\sqrt{\frac{\sin(\phi+\delta)\sin(\phi+\beta)}{\sin(\alpha+\delta)\sin(\alpha+\beta)}}\right]^2}$$

$= 3.12$

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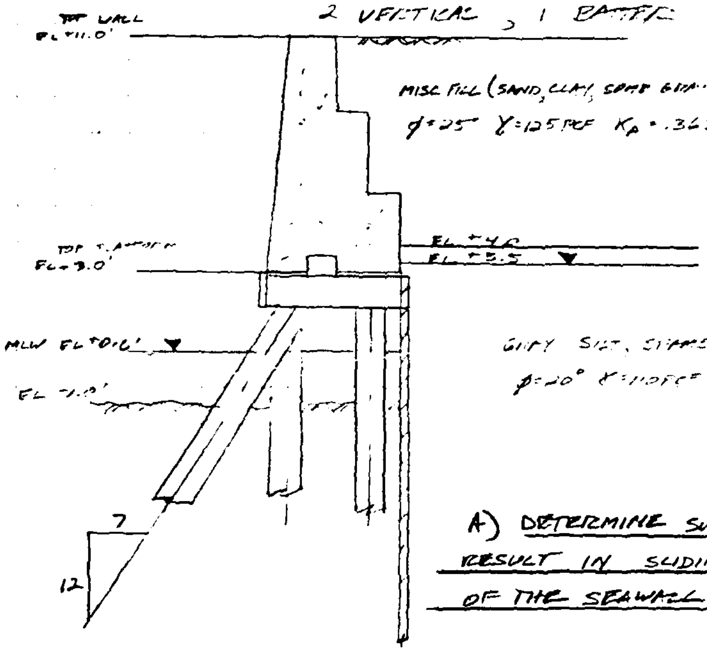
TYPICAL SEAWALL CALCS.

EASTERN SEAWALL

PILE BENTS @ 5'-0 CTRS.
3 PILES PER BENT
2 VERTICAL , 1 BATTER



MISC FILL (SAND, CLAY, SOME GRAVEL)
$\phi = 25°$ $\gamma = 125PCF$ $K_A = .362$

SILTY SILT, SOME FINE SAND
$\phi = 20°$ $\gamma = 110PCF$ $\gamma_{SAT} = 60PCF$ $K_A = .433$ $K_P = 3.12$

A) DETERMINE SURCHARGE W THAT WOULD RESULT IN SLIDING AND OVERTURNING OF THE SEAWALL

DETERMINE DEAD LOADS

WEIGHT OF EARTH $[(2.[illegible])([illegible]) - (1.17)(2.67)](125PCF)$
$= 1,115 \#/LF$

WEIGHT CONC $= \left[\left(\frac{3.83+4.5}{2}\right)(8) - 8.[illegible]\right] 150\#/LF$

$= 3,660 \#/LF$
$\Sigma(\text{EARTH + CONC}) = 4775\#/LF$

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JOB PNSY
SHEET NO. 4 OF 7
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SCALE

TYPICAL SEAWALL CALCS.

EASTERN SEAWALL

TIMBER (WET)

DECK $\left[\left(\frac{3"}{12"/'} \times 4.83'\right) + \left(\frac{8"}{12"/'} \times 1'\right)\right](45^{\#}/cf)$

= 84 PLF

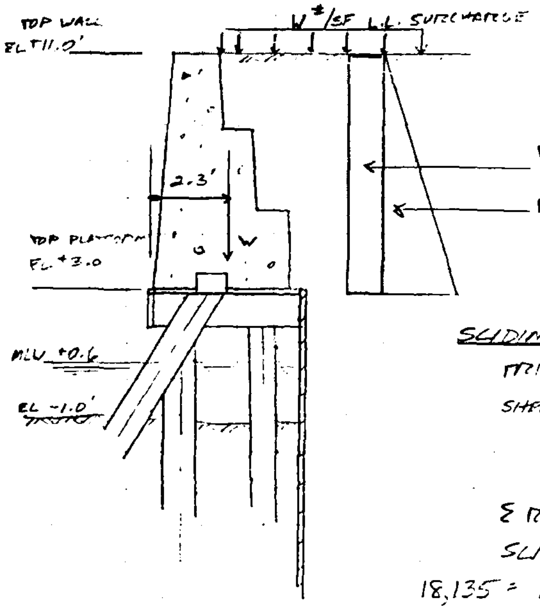
PILE CAP

$\frac{(1' \times 1')(4.58)}{5'\ o.c.}\ (45^{\#}/cf) = 41\ PLF$

TOTAL = 4900 PLF

D.L. = 4900/4.5 = 1089 PSF

USE 1090 PSF UNIFORM D.L.



$P_1 = (.362)(w)(8) = 2.90w$ PLF @ EL +7.0

$P_2 = \frac{(.362)(125^{\#}/cf)(8)^2}{2} = 1,448$ @ EL +5.67

SLIDING

FRICTION F = μW = (.33)(4775) = 1575 PLF

SHEAR IN 8"x12" KEY

V = H(11.5)(12"/ft)

= 120 PSI (11.5 IN)(12 IN/FT) = 16,560#/1

Σ RESISTING FORCE = 18,135 PLF

SLIDING FORCE = 1448 + 2.90w

18,135 = 1448 + 2.90w

w = 5754 PSF L.L.

OVERTURNING

OVERTURNING MOMENT = (2.90w)(4') + (1448)(2.67) = $11.6w_0$ + 3866 FT-# /LF

RESISTING MOMENT = (4900 PLF)(1.3) = 6,370'#/LF

$11.6w_0$ + 3866 = 6370

w_0 = 216 PSF LIVE LOAD

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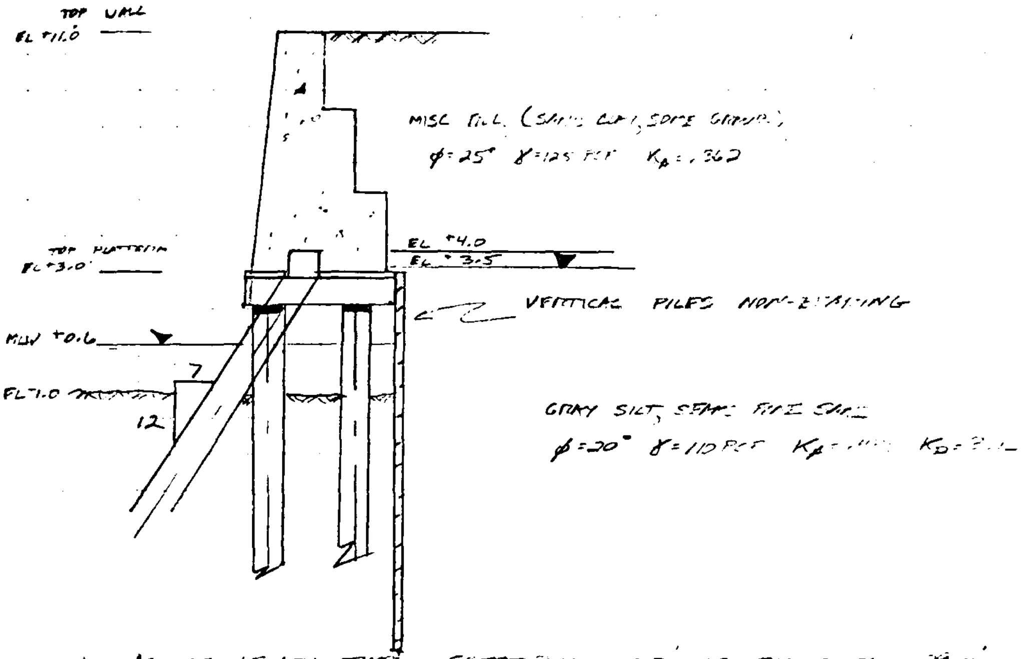
JOB: [illegible]
SHEET NO. 5 OF 7
CALCULATED BY: PAR DATE:
CHECKED BY: CDS DATE:
SCALE: NONE

EASTERN SEAWALL

B) DETERMINE LATERAL STABILITY OF STRUCTURE GIVEN THAT BOTH VERTICAL PILES ARE NON-BEARING.



1) ASSUME LENGTH TIMBER SHEET PILES = 32', DRIVEN TO EL -29.0'
2) ASSUME TIMBER SHEET PILE, CONCRETE SEAWALL ACT TOGETHER AS RETAINING WALL.
3) ASSUME WORST TIDAL CONDITION @ MLW, WATER ELEVATION ON BACKSIDE
IS $+0.6' + \frac{1}{2}(5.8') = +3.5'$
4) ASSUME FIXED-EARTH SUPPORT

DETERMINE RESULTANT FORCE R_A @ EL +3.0 DUE TO LATERAL EARTH PRESSURE.*

* FOUNDATION ENGINEERING - PECK ; WINTERKORN AND FANG - 428 FOR DERIVATION OF PRESSURE DIAGRAM

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JOB PNSY
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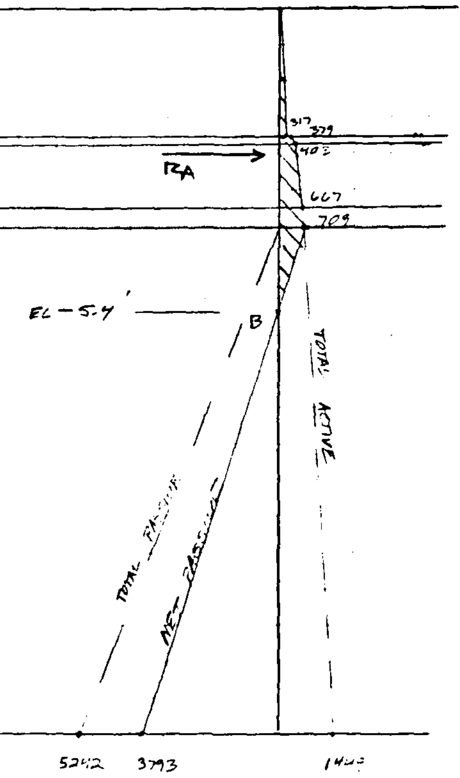
EASTERN SEAWALL

		COL 1	COL 2	COL 3	COL 4	COL 5
		γh	γh	$\gamma_w h$	P_A	P_P
+11.0'	7.0'					
+4.0'	0.5'	875	—	0	317/379	—
+3.5'	2.9'	930	—	0	403	—
+0.6'	1.6'	1104	—	189	478	—
−1.0	28.0'	1200	0	189	520	0
−29.0		2880	1680	189	1247	5242

PRESSURE DIAGRAM



LOCATE PT B $\frac{709}{x} = \frac{4502}{28}$ $x = 4.41$

PT B @ EL −5.4

JOB PNSY
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EASTERN SEAWALL

$\Sigma M_B^{G} = 0$

$$\frac{(709)(4.4)}{2}(2.93) + (667)(1.6)(5.2) + \frac{(42)(1.6)(4.93)}{2} + (403)(2.9)(7.45)$$

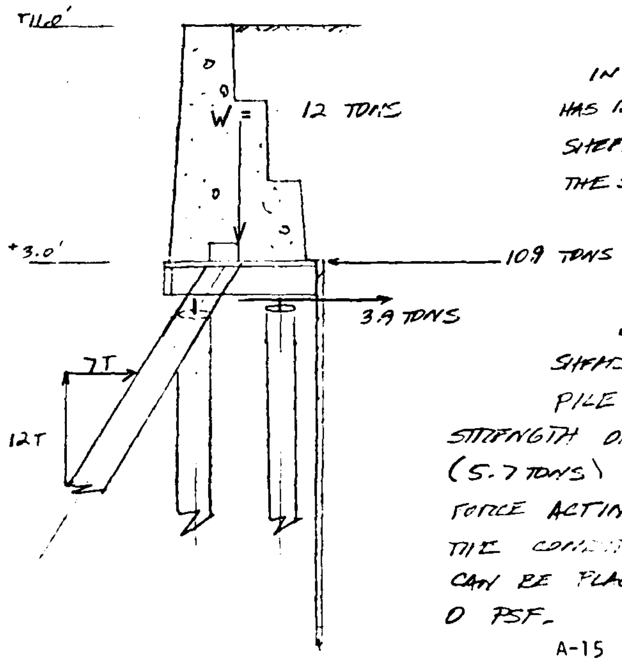
$$+ \frac{(264)(2.9)}{2}(6.97) + (379)(.5)(9.2) + \frac{(24)(.5)}{2}(9.1) + \frac{(312)(7)}{2}(11.7) = 8.4\ R_A$$

$R_A = 4{,}338$ # / FT × 5' BENT SPACING = 21,690 # = 10.9 TONS

SINCE BOTH VERTICAL PILES ARE NON-BEARING, DEAD LOAD OF 24,500# (12 TONS) MUST BE CARRIED BY VERTICAL COMPONENT OF BATTER PILE. BY GEOMETRY, THIS RESULTS IN A HORIZONTAL COMPONENT OF 7 TONS AVAILABLE TO RESIST LATERAL EARTH FORCE OF 10.9 TONS. ADDITIONAL HORIZONTAL RESISTANCE IS PROVIDED BY SHEAR STRENGTH OF TIMBER SHEET PILE WALL WHICH IS MOBILIZED WHEN THE WALL DEFLECTS OUTWARD. THE MAXIMUM SHEAR STRENGTH OF THE TIMBER SHEET PILE WALL IS

$$H = \frac{3}{2}\frac{V}{bd} \qquad H = 110\ PSI \qquad b = 12'' \quad d = 12''$$

$$V = 5.7\ TONS$$



IN CONCLUSION, ALTHOUGH THE SEAWALL HAS ROTATED CLOCKWISE AND THE TIMBER SHEET PILE WALL DEFLECTED OUTWARD, THE SYSTEM IS IN EQUILIBRIUM. THE LATERAL EARTH FORCE OF 10.9 TONS IS RESISTED BY THE HORIZONTAL COMPONENT OF THE BATTER PILE PLUS THE SHEAR STRENGTH OF THE TIMBER SHEET PILE WALL. NOTE THAT THE SHEAR STRENGTH OF THE TIMBER SHEET PILE WALL (5.7 TONS) IS NOT EXCEEDED BY THE SHEAR FORCE ACTING ON IT (3.9 TONS). HOWEVER THE CONDITION LIMITS THE SURCHARGE THAT CAN BE PLACED BEHIND THE SEAWALL TO 0 PSF.

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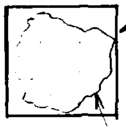
SCALE

TYPICAL TIMBER SOFTNESS

PIER 7

DETERMINE REDUCED CAPACITY OF TIMBER PILE CAP DUE TO TIMBER SOFTNESS.

$F_b = 1650\ \#/in^2$
$F_v = 120\ \#/in^2$
$F_{c\perp} = 315\ \#/in^2$
$\ell = 48''$



ORIGINAL 12X12 CAP
A = 144 IN²

AREA OF TIMBER REMAINING AFTER A REDUCTION IS CONSIDERED.
ASSUME $A' \approx 95\ IN^2$

BENDING

$S = 130\ in^3$ ← DURATION OF LOAD FACTOR
$M_{max} = S\ F_b\ .9$
$M_{max} = 193\ in\text{-}k$

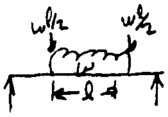
$$W = \frac{12\ M_{max}}{\ell^2} = 12.1\ k/ft$$

HORIZ. SHEAR

$$W = \frac{2 F_v A'\ .9}{3 \ell} (2) = \underline{\underline{6.3\ k/ft}}$$

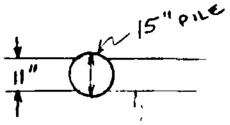
USE $F_v = \frac{3V}{2A}$

WHERE $V = \frac{w\ell}{2}$



$\ell = 26$

CRUSHING



PILE CAP

ASSUME AREA OF BEARING $A' \approx 165''$

MOISTURE CONTENT REDUCTION FACTOR

$$w = \frac{(315\ \#/in^2)(165\ in^2)\ .9\ .67}{48\ in} = 7.8\ k/ft$$

$$w = \frac{F_{c\perp} A}{2}$$

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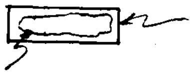
TYPICAL TIMBER SOFTNESS

PIER 7

DETERMINE REDUCED CAPACITY OF TIMBERDECKING DUE TO SOFTNESS.

$F_b = 1650\ \#/in^2$
$F_V = 120\ \#/in^2$
$F_{c\perp} = 315\ \#/in^2$



ORIGINAL 4X12 DECKING

AREA OF TIMBER REMAINING AFTER A REDUCTION IS CONSIDERED

ASSUME $A' \approx 30\ in^2$

BENDING

$S = 15\ in^3$

$M_{max} : S\ F_b\ .9$ — DURATION OF LOAD FACTOR

$M_{max} = 22.2\ in\text{-}k$

$$W = \frac{12\ M_{max}}{\ell^2} = 1.37\ k/ft$$ where $\ell = 45''$

HORIZ. SHEAR

$$W = \frac{2\ F_V\ A'\ .9}{3\ \ell}(2) = \underline{\underline{1.23\ k/ft}}$$ where $\ell = 42''$

CRUSHING

AREA OF BEARING $120\ in^2$

$$W = \frac{(315\#/in^2)(120\ in^2)\ .9\ \ .68}{60\ in} = 4.6\ k/ft$$

.68 — MOISTURE CONTENT REDUCTION FACTOR

WE CONCLUDE THAT THE LIMITING FACTOR ON PIER 7 IS THE HORIZONTAL SHEAR CAPACITY OF THE REDUCED SECTION OF THE DECK PLANK

- WHEN LOADING ON THE DECK PLANK IS ANALYZED THE $DL = 1.3\ k/ft^2 >$ ALLOWABLE LOAD $= 1.23\ k/ft^2$

— THIS IS A CONDITION OF IMPENDING FAILURE —

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PIER 1

GIVEN THAT THE WORST CASE MUDLINE ELEVATION IS -20 FEET, CALCULATE THE MAXIMUM MOMENT AND HORIZONTAL SHEAR ACTING ON THE TIMBER SHEET PILE WALL DUE TO LATERAL EARTH FORCES. DERIVE THE PRESSURE DIAGRAM USING SOIL PROPERTIES ASSUMED BEFORE AND THE METHOD DESCRIBED IN <u>FOUNDATION ENGINEERING HANDBOOK</u>; WINTERKORN AND FANG p428.

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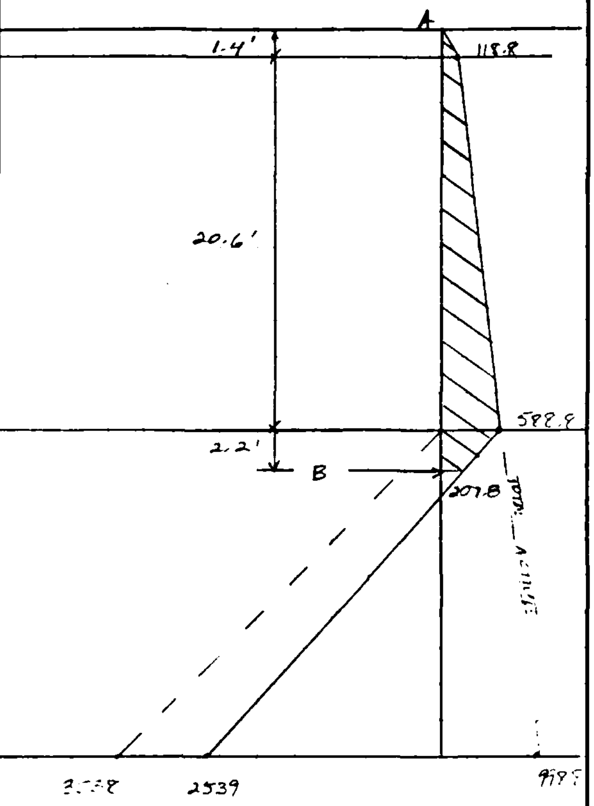
PIER 1

	COL 1	COL 2	COL 3	COL 4	COL 5
	γh	γh	$\gamma_w h$ 0	P_A	P_P
DECK EL +2.0' MLW +0.6'	88.2		86.8	32	
ML EL -20'	1386		86.8	502	
EL -38'	2520	1,134	86.8	912	3535

PRESSURE DIAGRAM



$K_A = .362$
$K_P = 3.12$
$\gamma_W = 62$ PSF
$\gamma_{SUB} = 63$ PSF

$W = (1.4)(118.8)/2 + [(588.8 + 118.8)/2](20.6) + [(588.8 + 207.8)/2](2.2$
$= 8247$ LB / FT RUN WALL

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PIER 1

BENDING

$M_{MAX} \simeq wL/8$

$= (8.25)(24.2)/8$

$= 25.0$ FT-K

USING 11.5 x 11.5 CROSS-SECTION

$S = 253.5 \text{ in}^3$ $F_b = 1650$ PSI

$(S)(F_b)(.9) = 31.4$ FT-K > 25.0 FT-K ✓

HORIZONTAL SHEAR

MAXIMUM SHEAR OCCURRS AT APPROXIMATELY PT B, THE POINT OF CONTRAFLEXURE.

$\Sigma M_A = 0$

$$\frac{(1.4)(118.8)}{2}(.93) + (20.6)(118.8)(11.7) + \frac{(470)(20.6)}{2}(15.1)$$

$$+ (2.2)(207.8)(23.1) + \frac{(381)(2.2)}{2}(22.7) = 24.2\,B$$

B = 5.04 K = MAX V

$$V_{ALL} = F_v \frac{2A}{3} = (120 \text{ PSI})\left(\frac{2}{3}\right)(11.5^2)$$

$= 10.6$ K > 5.04 K ✓

IN CONCLUSION, MAXIMUM MOMENT AND HORIZONTAL SHEAR DO NOT EXCEED ALLOWABLE VALUES

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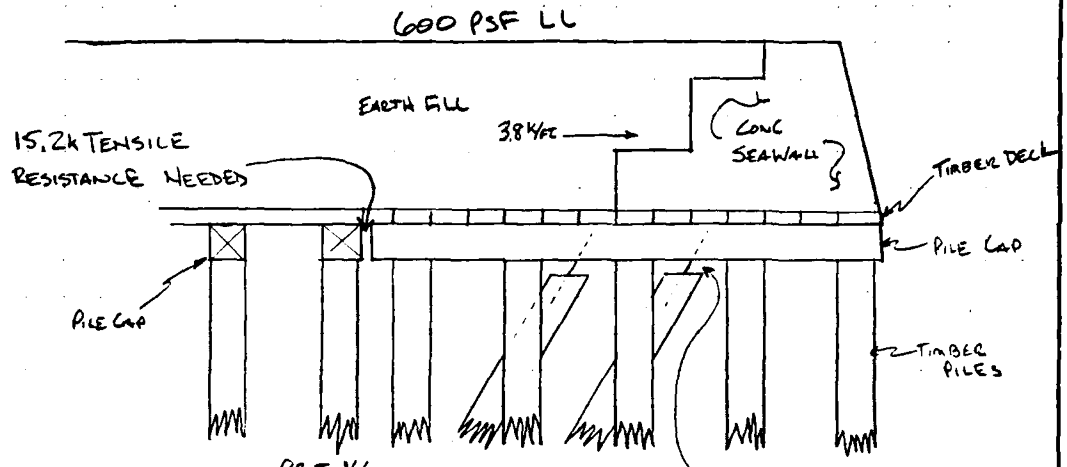
PIER 2

DETERMINE THE FORCES ACTING ON THE SOUTHERN END OF PIER 2.



SECTION THROUGH THE SOUTH END OF PIER 2

FIND LATERAL EARTH PRESSURE INCLUDING LL -

ASSUME : $K = .362$ - FROM APENDIX PG A-10

$\gamma = 125$ PCF

$h = 9'$

$Q = 600$ PSF

$$\text{LATERAL EARTH PRESSURE} = \tfrac{1}{2} K h^2 \gamma + K Q h$$

$$= \tfrac{1}{2}(.362)(9)^2 125 + (.362)(600)(9)$$

$$= 3787 \text{ \#/Ft} = 3.8 K/Ft$$

FOR A 4 FOOT BENT SPACING THE FORCE/PILE CAP IS 15.2 K.

THIS FORCE MUST BE RESISTED BY A CONNECTION AT THE LAST FULL PILE BENT (BENT 166).

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SCALE

Pier 2

OUR OBSERVATIONS INDICATE THAT THERE HAS BEEN A LATERAL MOVEMENT OF APROX 6". THE REASON FOR THIS MOVEMENT IS THE OVERLOADED CONNECTION OR RESTRAINING FORCES.

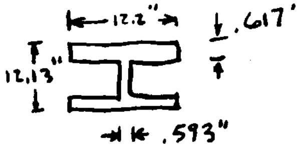
TO STOP FURTHER MOVEMENT TO THE SOUTH, THE RESTRAINT MUST BE UP GRADED.

A POSSIBLE SOLUTION IS ILLUSTRATED ON PAGE A-31 THE INSTALLATION OF A DEADMAN AND TIE-BACK SYSTEM ACCORDING TO THE SOIL CONDITIONS FOUND IN THE FIELD WOULD RELIEVE THE PRESSURE BEING TRANSLATED TO THE VERTICLE BEARING PILES.

JOB
SHEET NO OF
CALCULATED BY DATE
CHECKED BY DATE
SCALE

PIER 4

STEEL H-PILE COLUMN CAPACITY



$I = 187 \text{ IN}^4$
$A = 21.5 \text{ IN}^2$
$r = 2.95 \text{ IN}$
$\ell = 564 \text{ IN}$
$\frac{k\ell}{r} = 152.9 > 126.1 = C_c$

USE:

$$F_a = \frac{12\pi^2 E}{23\left(\frac{k\ell}{r}\right)^2}$$

$F_a = 6.38 \text{ KSI}$

$P = F_a(A) = 137.17 \text{ K}$

ESTIMATE $DL + LL = 100 K/_{PILE} < \underline{137.17}$ OK

TIMBER PILE LOADING + CAPACITY

LIVE LOADING/BENT = 10'x100' x 1200 PSF = 1200K/BENT

DEAD LOAD/BENT = 40 K (ASSUMED)

OF PILES/BENT = 40

TOTAL LOAD/PILE = 31 K/PILE

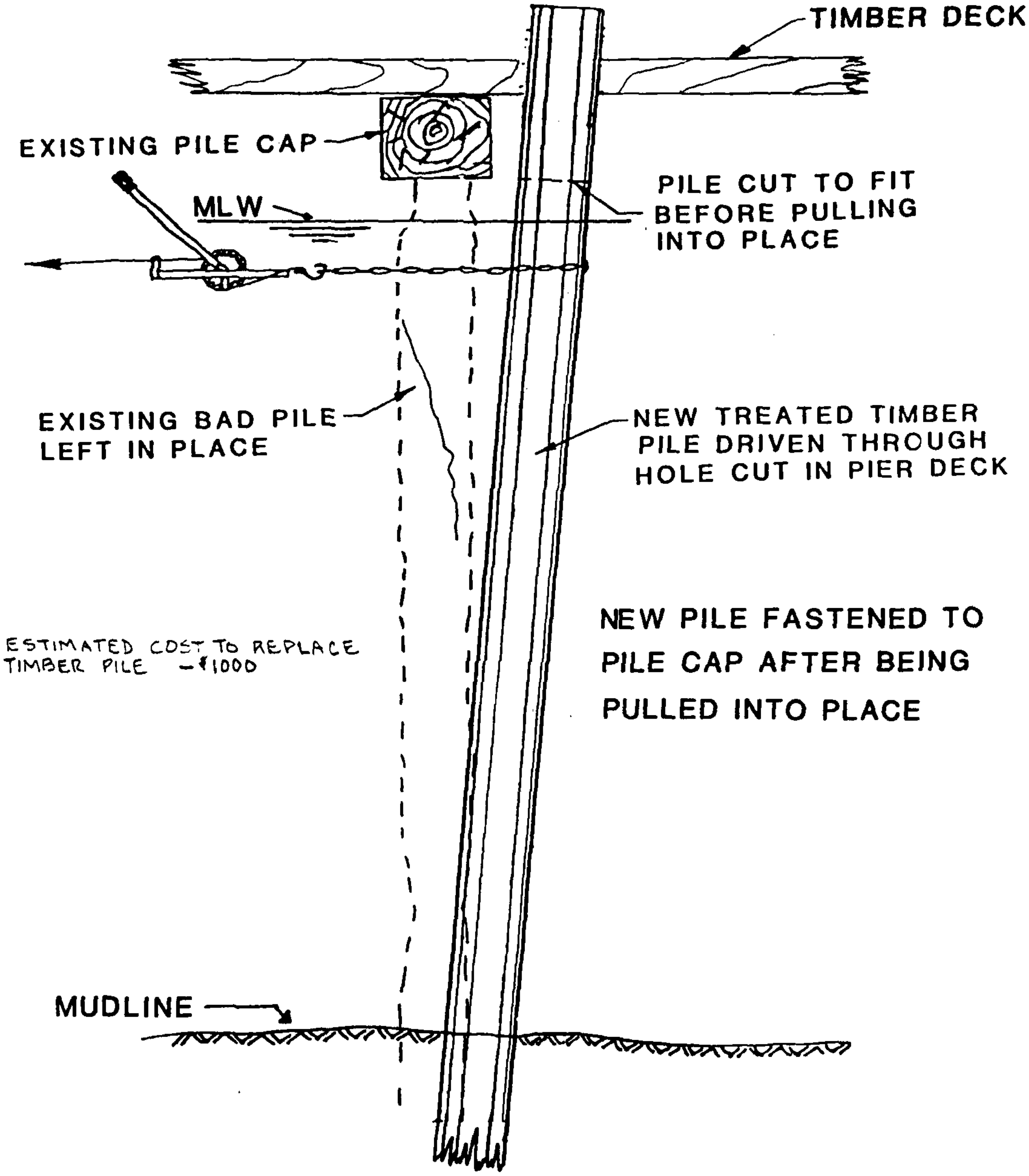
ASSUMED PILE DRIVEN CAPACITY 15-20T OK

CALCULATED PILE COLUMN CAPACITY 16T OK
(SEE 1 + A2)

THE STRUCTURAL TIMBER PILES WERE FOUND TO BE IN GOOD TO EXCELLANT CONDITION WITH NO APPARENT LOSS OF STRENGTH (EXCEPTING LOCAL CONDITIONS)

REPLACEMENT TIMBER PILE

CONCEPTUAL DESIGN



NEW PILE FASTENED TO PILE CAP AFTER BEING PULLED INTO PLACE

ESTIMATED COST TO REPLACE TIMBER PILE —$1000

GRAPHIC SCALE

NO SCALE

CHILDS ENGINEERING
CORPORATION
BOX 333
MEDFIELD MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D.C.

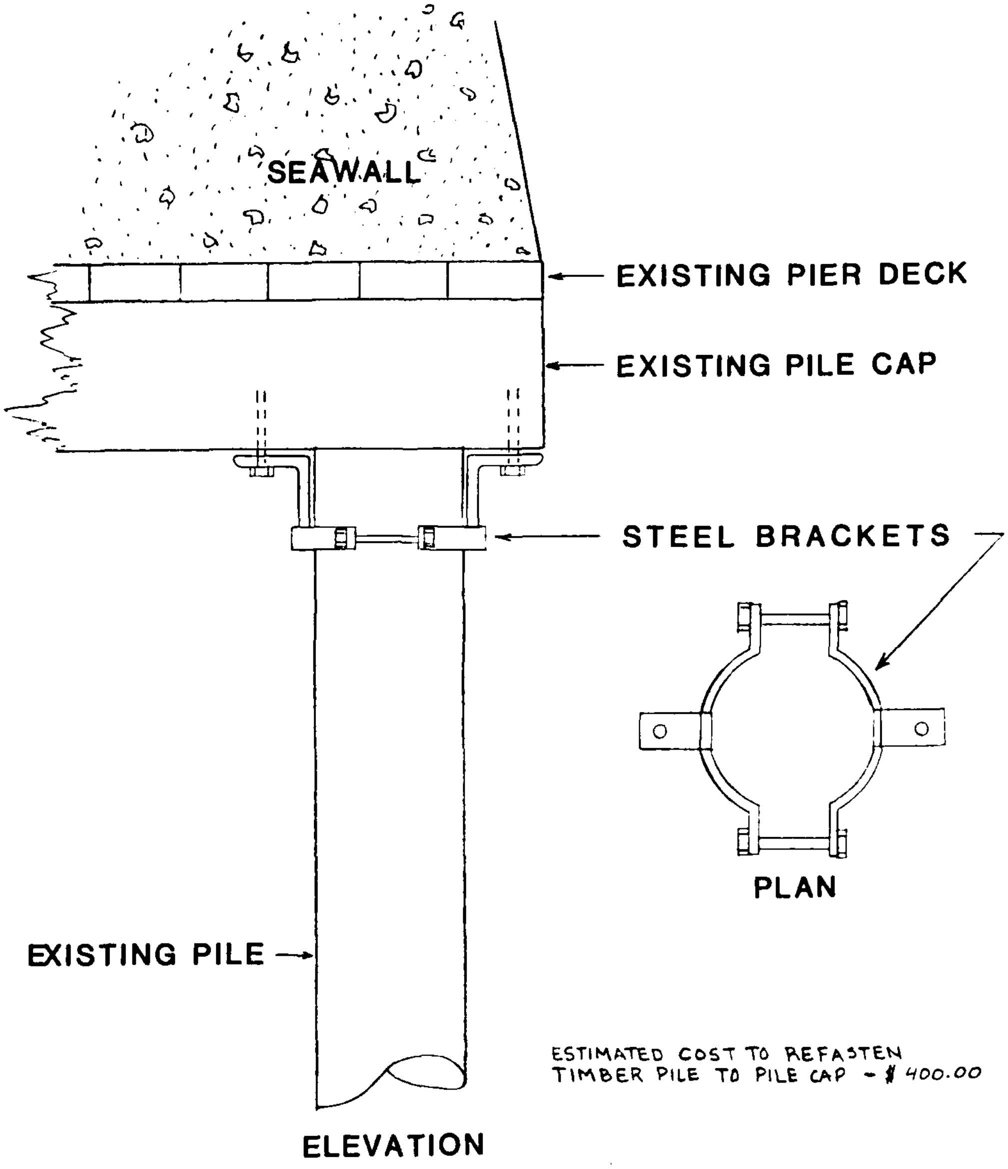
PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA

REPLACEMENT TIMBER PILE

FIG NO
A1

CONCEPTUAL DESIGN

REFASTEN TIMBER PILE TO PILE CAP



ESTIMATED COST TO REFASTEN
TIMBER PILE TO PILE CAP - $ 400.00

GRAPHIC SCALE

NO SCALE

CHILDS ENGINEERING
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MEDFIELD, MA

A-25

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D C

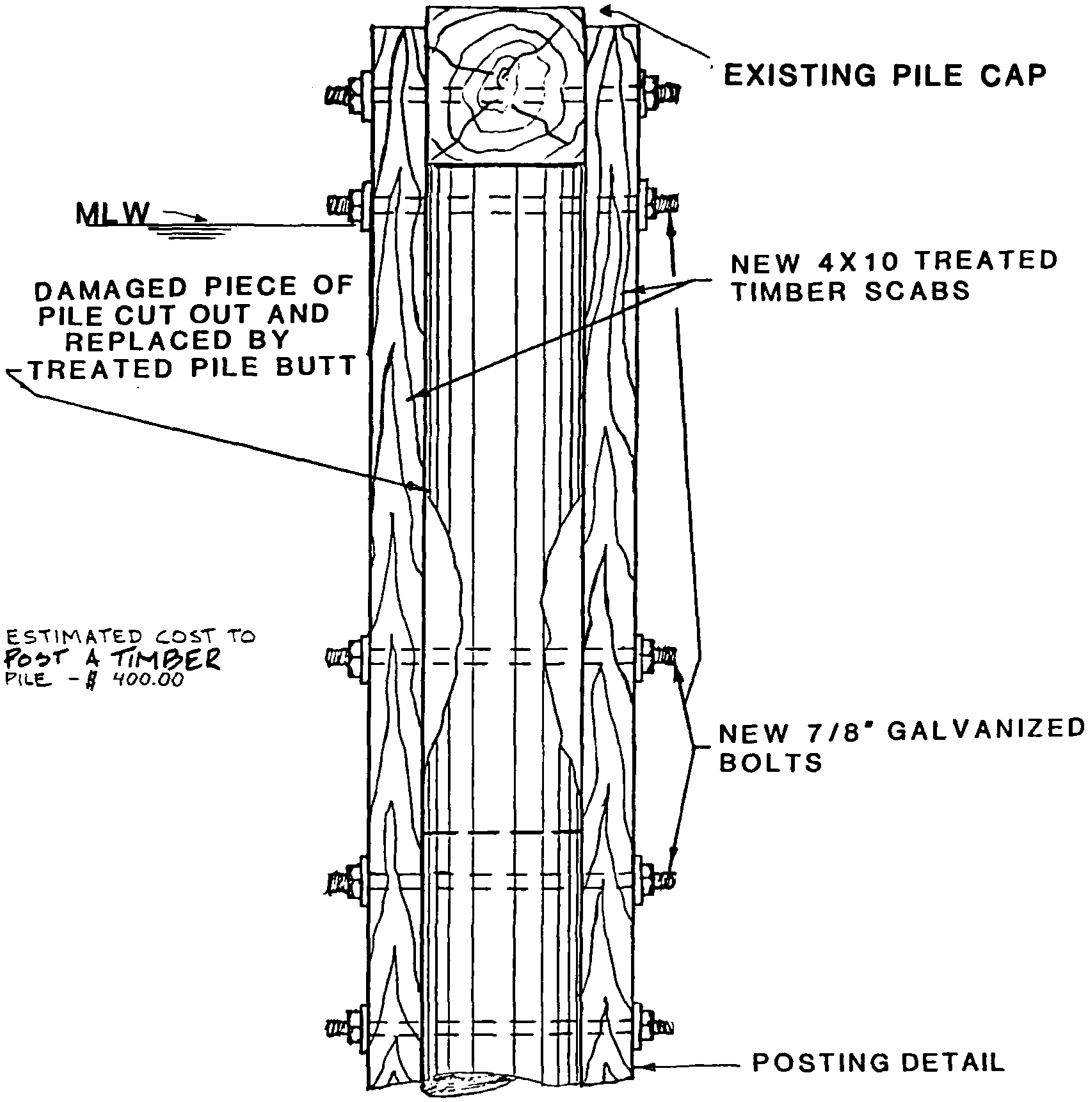
PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA

CLAMP TIMBER PILE

FIG NO
A2

POSTED TIMBER PILE

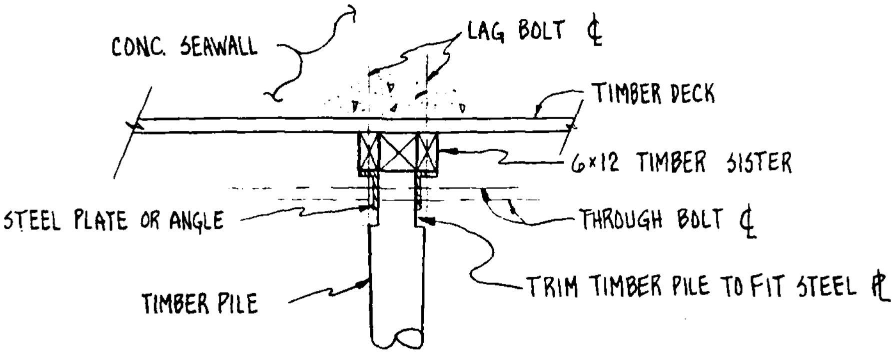
CONCEPTUAL DESIGN



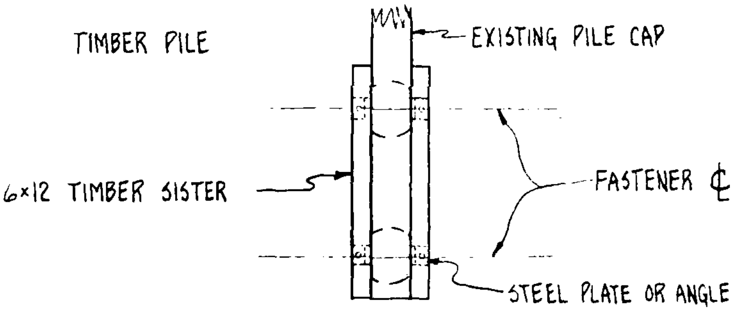
ESTIMATED COST TO
POST A TIMBER
PILE - $ 400.00

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD, MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D C	
NO SCALE		PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA POSTED TIMBER PILE	FIG.NO A3

CONCEPTUAL DESIGN



SECTION



PLAN

GRAPHIC SCALE

NOT TO SCALE

CHILDS ENGINEERING
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BOX 333
MEDFIELD, MA

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D C

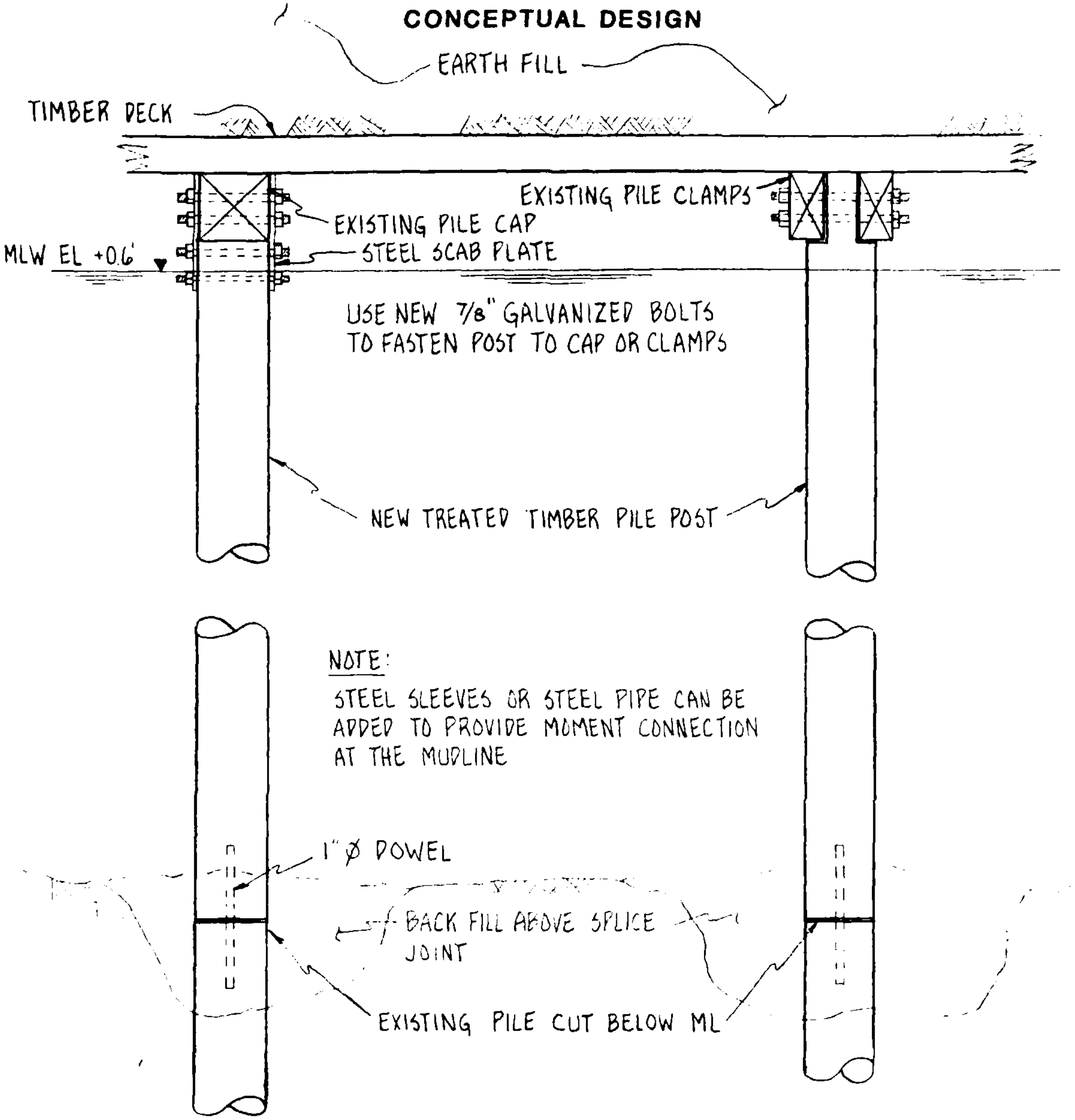
PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA

PILE CAP SISTER

FIG NO

TIMBER PILE LONG POST

CONCEPTUAL DESIGN



USE NEW 7/8" GALVANIZED BOLTS TO FASTEN POST TO CAP OR CLAMPS

NOTE:
STEEL SLEEVES OR STEEL PIPE CAN BE ADDED TO PROVIDE MOMENT CONNECTION AT THE MUDLINE

NOTE: 1. CROSS BRACING MUST BE USED BETWEEN ADJACENT LONG POSTS TO INSURE STABILITY IF THERE IS NO MOMENT CONN. AT THE ML
2. ESTIMATED COST OF REPAIR IS $1000 PER PILE

GRAPHIC SCALE
NO SCALE

CHILDS ENGINEERING
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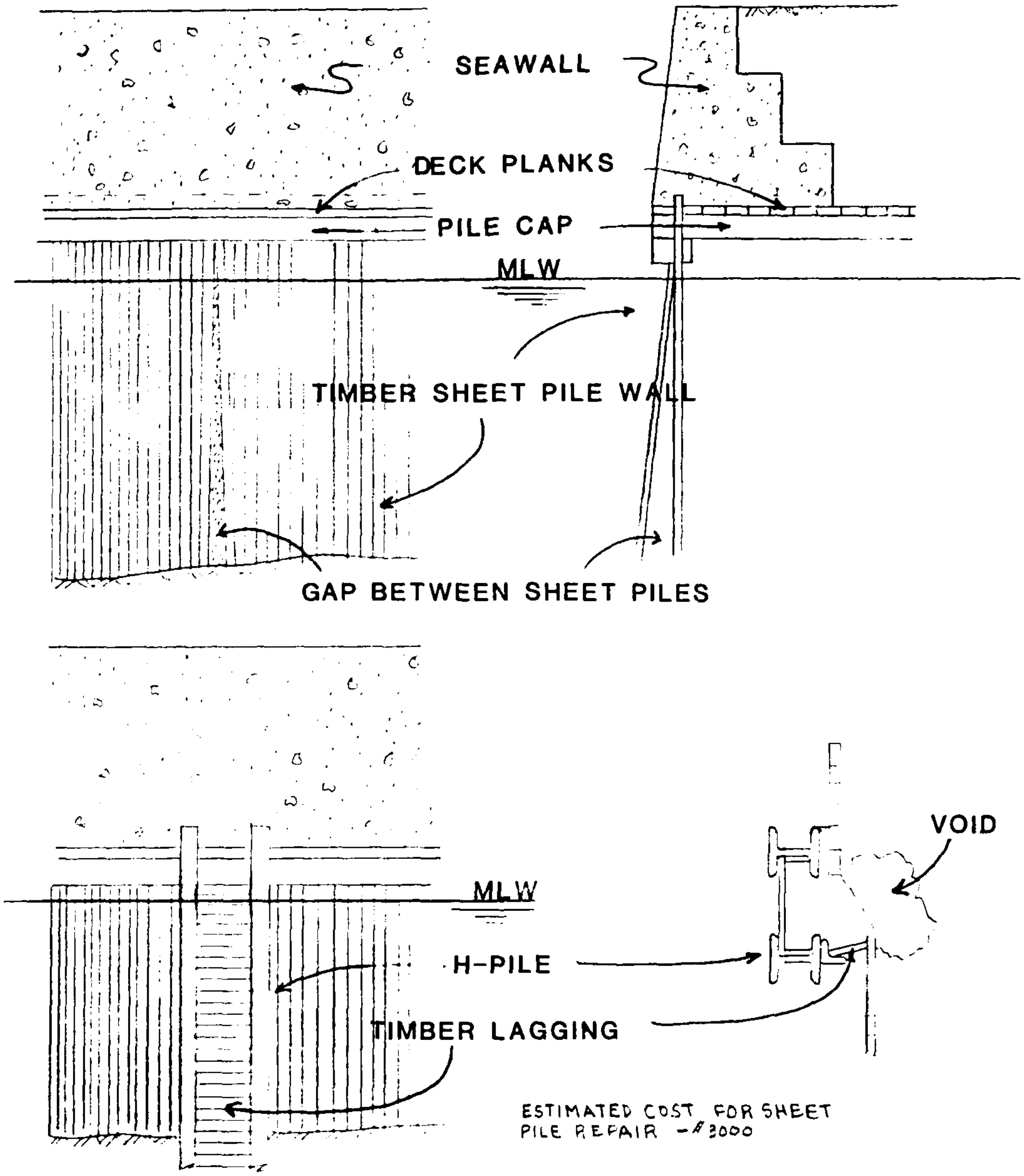
MEDFIELD, MA
A-28

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D C
PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA PA
TIMBER PILE LONG POST
FIG NO

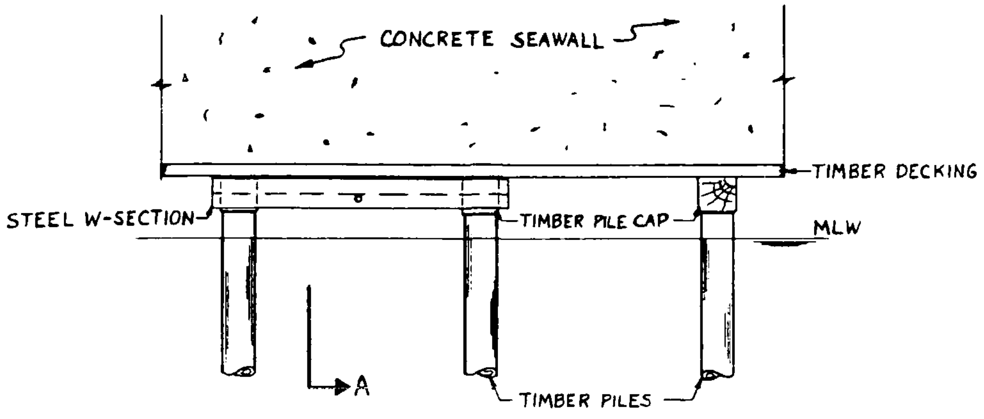
SHEET PILE REPAIR
CONCEPTUAL DESIGN



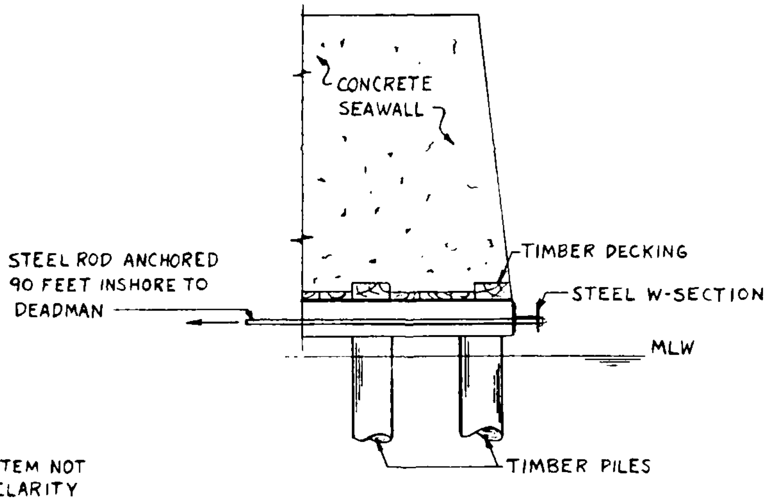
ESTIMATED COST FOR SHEET
PILE REPAIR -#3000

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D C	
NO SCALE	A-29	PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA SHEET PILE REPAIR	FIG NO. A4

CONCEPTUAL DESIGN



ELEVATION



SECTION - A

NOTE:
FENDER SYSTEM NOT
SHOWN FOR CLARITY

GRAPHIC SCALE

NOT TO SCALE

CHILDS ENGINEERING
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MEDFIELD, MA

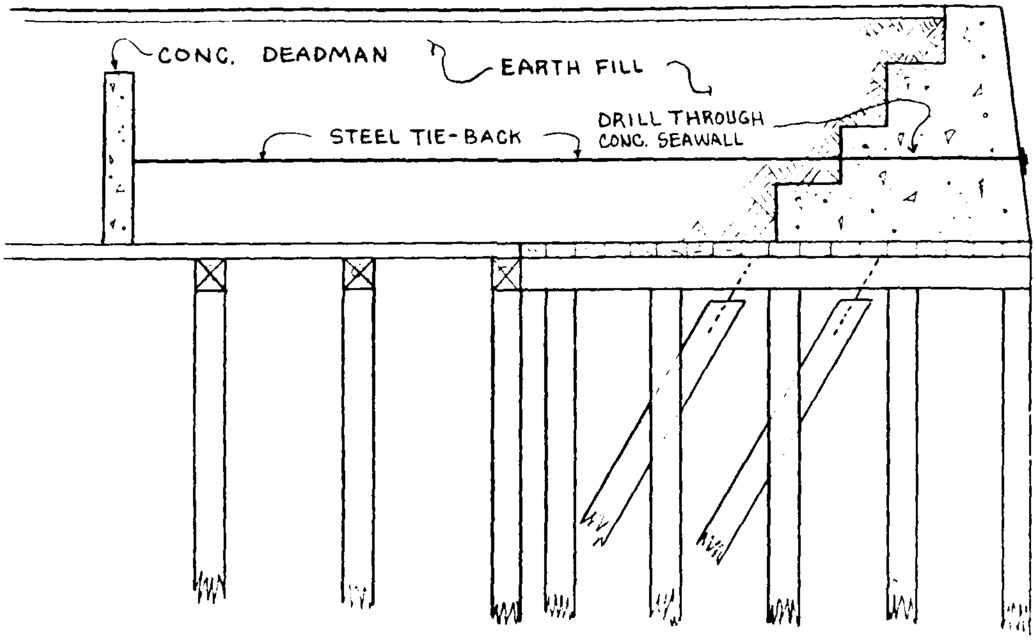
A-30

CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON D.C.

PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA

TIE-BACK DETAIL

FIG NO
A8



PIER 2 TIE-BACK SYSTEM

CONCEPTUAL DESIGN

ESTIMATED COST PER TIE-BACK. -$5000

GRAPHIC SCALE	CHILDS ENGINEERING CORPORATION BOX 333 MEDFIELD MA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON D.C.	
0 1' 2' 3' 4' 5'	A-31	PHILADELPHIA NAVAL SHIPYARD PHILADELPHIA, PA PIER 2 TIE-BACK	FIG NO A5

CHILDS ENGINEERING CORPORATION
Box 333
MEDFIELD, MA 02052

JOB YNSY
SHEET NO 1 OF 1
CALCULATED BY CDS DATE
CHECKED BY DATE
SCALE

1) REPLACEMENT PILE - UNIT COST $1000 (IN PLACE)

2) PILE TOP REPAIR i.e. REFASTEN, SHORT POST, PILE CAP SISTER

ASSUME : CREW

1	FOREMAN	$1100 / DAY
2	DOCK BUILDERS	
1	LABORER	
1	DIVER	

AVERAGE LABOR COST PER REPAIR	- 275
MATERAILS COST PER REPAIR	- 125
AVE. COST/REPAIR	$400

3) LONG POST REPAIR

CREW COST/DAY = $1100 / DAY

CREW COST / REPAIR	$750
MATERALS COST/REPAIR	$250
AVE COST / REPAIR	$1000

4) TIMBER SHEET PILE REPAIR

2 STEEL H-PILES IN PLACE (UNIT COST)	$2000
COST OF MISC. MATIERALS	500
COST OF LABOR	500
EST. TOTAL	$3000

NOTE: ① COSTS ARE BASED ON 1983 U.S. EAST COAST PRICES.
② COSTS DO NOT INCLUDE MOBILIZATION / DEMOBILIZATION

REFERENCES

1. Master Plan for Naval Base, Philadelphia, PA August 1975

2. Divers Inspection, Engineering Evaluation and Preliminary Recommendations for Piers and Bulkheads at the Philadlephia Naval Shipyard, Philadelphia, Pennsylvania; prepared by Hudson Engineers, Inc., Philadelphia, PA September 1976

END

DATE
FILMED

7-86