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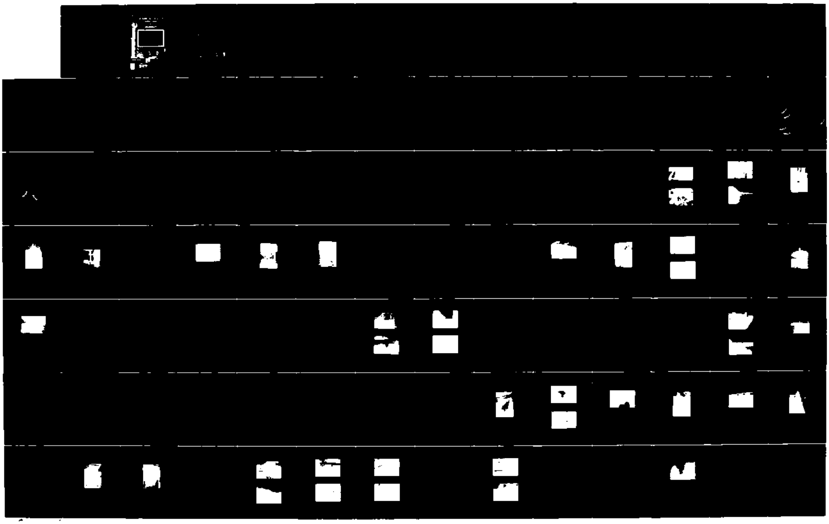

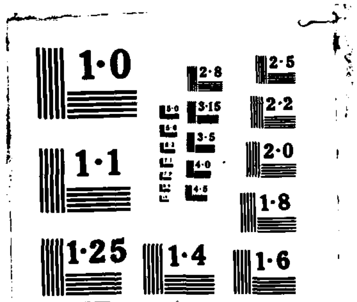
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UNDERWATER FACILITIES INSPECTIONS AND ASSESSMENTS AT

NAVAL AIR STATION

PENSACOLA, FLA.

FPO-1-83 (1) JANUARY 1983

Performed for:

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

Under:

CONTRACT N62477-81-C-0286
TASK 2

By:

ARTHUR V. STROCK & ASSOCIATES, INC.
DEERFIELD BEACH, FLORIDA 33441

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ADA 167358

Unclassified

SECURITY CLASSIFICATION OF THIS PAGE

REPORT DOCUMENTATION PAGE

1a. REPORT SECURITY CLASSIFICATION Unclassified	1b. RESTRICTIVE MARKINGS
2a. SECURITY CLASSIFICATION AUTHORITY	3. DISTRIBUTION AVAILABILITY OF REP. Approved for public release; distribution is unlimited
2b. DECLASSIFICATION/DOWNGRADING SCHEDULE	
4. PERFORMING ORGANIZATION REPORT NUMBER	5. MONITORING ORGANIZATION REPORT # FPO-1-83[redacted](1)
6a. NAME OF PERFORM. ORG. 6b. OFFICE SYM Arthur V. Strock & Associates, Inc.	7a. NAME OF MONITORING ORGANIZATION Ocean Engineering & Construction Project Office CHESNAVFACENGCOM
6c. ADDRESS (City, State, and Zip Code) Deerfield Beach, FL 33441	7b. ADDRESS (City, State, and Zip) BLDG. 212, Washington Navy Yard Washington, D.C. 20374-2121
8a. NAME OF FUNDING ORG. 8b. OFFICE SYM	9. PROCUREMENT INSTRUMENT INDENT # N62477-81-C-0286, Task 2
8c. ADDRESS (City, State & Zip)	10. SOURCE OF FUNDING NUMBERS PROGRAM ELEMENT # / PROJECT # / TASK # / WORK UNIT ACCESS #

11. TITLE (Including Security Classification)
Underwater Facilities Inspections & Assessments at Naval Air Station Pensacola, FLA

12. PERSONAL AUTHOR(S)

13a. TYPE OF REPORT	13b. TIME COVERED FROM	TO	14. DATE OF REP. (YYMMDD)	15. PAGES
			83-01	174

16. SUPPLEMENTARY NOTATION

17. COSATI CODES			18. SUBJECT TERMS (Continue on reverse if nec.)
FIELD	GROUP	SUB-GROUP	Underwater inspection, Naval Air Station Pensacola, FLA

19. ABSTRACT (Continue on reverse if necessary & identify by block number)
The objective of the underwater facility assessments conducted at the U.S. Naval Air Station in Pensacola, Florida was to provide an evaluation of the structural condition of four (4) selected piers, three (3) bulkheads and two (2) seawalls. Inspection of the facilities was limited by the scope of (Con't)

20. DISTRIBUTION/AVAILABILITY OF ABSTRACT SAME AS RPT.	21. ABSTRACT SECURITY CLASSIFICATION
22a. NAME OF RESPONSIBLE INDIVIDUAL Jacqueline B. Riley	22b. TELEPHONE 202-433-3881 / 22c. OFFICE SYMBOL

DD FORM 1473, 84MAR

SECURITY CLASSIFICATION OF THIS PAGE

BLOCK 19 (Con't)

the project to a visual/tactile evaluation by diving engineers. Significant representative defects were photo-documented. The facilities were evaluated to be in fair to good condition.

The steel H-piles and concrete piles supporting Finger Pier 303, Pier 3238 and the platform of Pier 303 appear in good condition. The steel sheet piles of Pier 303 and Pier 302 generally appear in good condition; only two small holes were observed as an apparent result of corrosion. The granite blocks of Bulkhead 177 and 178 appear in excellent condition while some missing blocks (voids) were noted and the joints between blocks lack a seal in many areas. Seawall 384, Seawall 1824 and Bulkhead 708-B appear in fair condition; tongue-and groove joints in these structures appear in poor condition where the seal has deteriorated.

EXECUTIVE SUMMARY

The objective of the underwater facility assessments conducted at the U. S. Naval Air Station in Pensacola, Florida was to provide an evaluation of the structural condition of four (4) selected piers, three (3) bulkheads and two (2) seawalls. Inspection of the facilities was limited by the scope of the project to a visual/tactile evaluation by diving engineers. Significant representative defects were photo-documented. The facilities were evaluated to be in fair to good condition.

The steel H-piles and concrete piles supporting Finger Pier 303, Pier 3238 and the platform of Pier 303 appear in good condition. The steel sheet piles of Pier 303 and Pier 302 generally appear in good condition; only two small holes were observed as an apparent result of corrosion. The granite blocks of Bulkhead 177 and 178 appear in excellent condition while some missing blocks (voids) were noted and the joints between blocks lack a seal in many areas. Seawall 384, Seawall 1824 and Bulkhead 708-B appear in fair condition; tongue-and-groove joints in these structures appear in poor condition where the seal has deteriorated.

The following page is an Executive Summary table which summarizes the assessed condition of the facilities and repair recommendations.

U.S. NAVAL AIR STATION
PENSACOLA, FLORIDA
EXECUTIVE SUMMARY TABLE

FACILITY	YEAR BUILT	NUMBER OF VERTICAL BEARING PILES	NUMBER OF BATTER PILES	FACILITY SIZE	STRUCTURAL TYPE	R
Seawall 384	1931	---	---	3403 L.F. of waterfront	Cast in place reinforced concrete on wood piles	S j
Seawall 1824	1924	---	---	2409 L.F. of waterfront	Cast in place reinforced concrete on wood piles with a concrete slab beach	S j
Pier 3238	1965	42	4	360' long 6' wide	12" and 16" square prestressed concrete piles	R
Bulkheads 177,178	1852	---	---	1781 L.F. of waterfront	Granite block gravity structure	F e d
Pier 303	1940	140	66	1695 L.F. of waterfront	18" square prestressed concrete piles supporting platform; concrete capped steel sheet pile bulkhead, (MZ32 and fabricated sections)	S p u
Finger Pier 303	1940	48	26	280' long 6' wide	Concrete jacketed steel H-piles, 14BP73 and 10BP42	
Pier 302	1940	---	---	581' long 67' wide 1230 L.F. of waterfront	Concrete capped steel sheet pile bulkhead	
Bulkhead 708-B	1943	---	---	1875 L.F. of waterfront	Cast in place reinforced concrete on wood piles	

I.S. NAVAL AIR STATION
PENSACOLA, FLORIDA
:XECUTIVE SUMMARY TABLE

LITY	STRUCTURAL TYPE	RECOMMENDATIONS	ESTIMATED REPAIR COST
: L.F. waterfront	Cast in place reinforced concrete on wood piles	Seal tongue and groove joints	$10,000
) L.F. waterfront	Cast in place reinforced concrete on wood piles with a concrete slab beach	Seal tongue and groove joints	$ 7,500
' long wide	12" and 16" square prestressed concrete piles	Repair pile caps	$15,500
1 L.F. waterfront	Granite block gravity structure	Fill voids and seal joints; evaluate and modify upland drainage	$90,000
5 L.F. waterfront	18" square prestressed concrete piles supporting platform; concrete capped steel sheet pile bulkhead, (MZ32 and fabricated sections)	Seal holes in steel sheet piles; evaluate and modify upland drainage	$ 3,500
' long wide	Concrete jacketed steel H-piles, 14BP73 and 10BP42	Replace damaged pile; inspect cathodic protection system	$20,000
' long ' wide 0 L.F. waterfront	Concrete capped steel sheet pile bulkhead	Seal joints in cap; evaluate and modify upland drainage	$11,000
5 L.F. waterfront	Cast in place reinforced concrete on wood piles	Seal tongue and groove joints; evaluate and modify upland drainage	$10,000

2

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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.

Supported by Contract No. N62477-81-C-0286, this program provides task oriented engineering services in support of the inspection, analysis and design, and monitoring of repairs for the submerged portions of selected Navy waterfront facilities. All services required to produce this report were provided by Arthur V. Strock & Associates, Inc. of Deerfield Beach, Florida under Task No. II of this contract.

The labor and expenses required to carry out underwater facility inspections is dependent on a significant number of factors. The size and number of structural members to be inspected is important. However, the structural condition of such structural members has as much influence on the total degree of accuracy and efficiency with which the task can be performed. As the size and number of structural members increases so does the required effort to carry out an inspection. Furthermore, the poorer the structural condition, the greater the number of irregularities which must be documented for a thorough inspection. To effectively assess the condition of any structure, the engineer must consider possible failure conditions and how failure can be evidenced in the structure. To provide a comprehensive inspection of a structure underwater, water clarity, ambient light levels, and degree of biofouling must be considered in advance of the inspection. Mechanical aids such as underwater lights, clearwater bags, wide-angle lenses, and scrapers can be used to aid in visual assessment of underwater structures. Other factors such as water temperature, pollutants, harmful sealife, in addition to waves, currents and tidal action must all be considered.

The inspections at Naval Station, Pensacola were performed following consideration of all these factors using SCUBA life-support systems. The ability to utilize SCUBA at this facility greatly increased the efficiency with which this specific inspection was performed. Details of field procedures followed under this task are given in Section 3 of this report.

1.1 Task Description

This task entails engineering services necessary to perform an underwater inspection, analysis and recommendation of repairs of the structural members supporting waterfront facilities at the Naval Air Station, Pensacola, Florida.

1.2 Report Content

In this report inspection procedures, results of inspection and analysis of the findings are provided. Each facility inspected at the Naval Air Station is described as to its location, function, construction, inspection condition and condition assessment. Recommendations for each facility are also provided. Structural assessment calculations and cost estimate breakdowns are given in the Appendix. Also, as supplementary information, a brief description of the Naval Air Station is included giving a description of the Mainside Complex and hydrographic features. This supplementary information was obtained from the Master Plan for the Naval Air Station.

SECTION 2.0 - ACTIVITY DESCRIPTION

The purpose of this section is to provide a general description of the Naval Air Station in Pensacola, Florida. The section includes brief descriptions of the Naval Air Station's mainside complex and pertinent hydrographic data. The information is provided to aid in identification of the facility and to support all considerations necessary to accurately assess the condition of the facilities inspected under this task.

2.1 - Location

The Naval Air Station addressed in this report is located in Pensacola, Florida. Figure 1 illustrates the regional location of the Naval Air Station. Figure 2 illustrates the vicinity of the Naval Air Station to the City of Pensacola and surrounding areas.

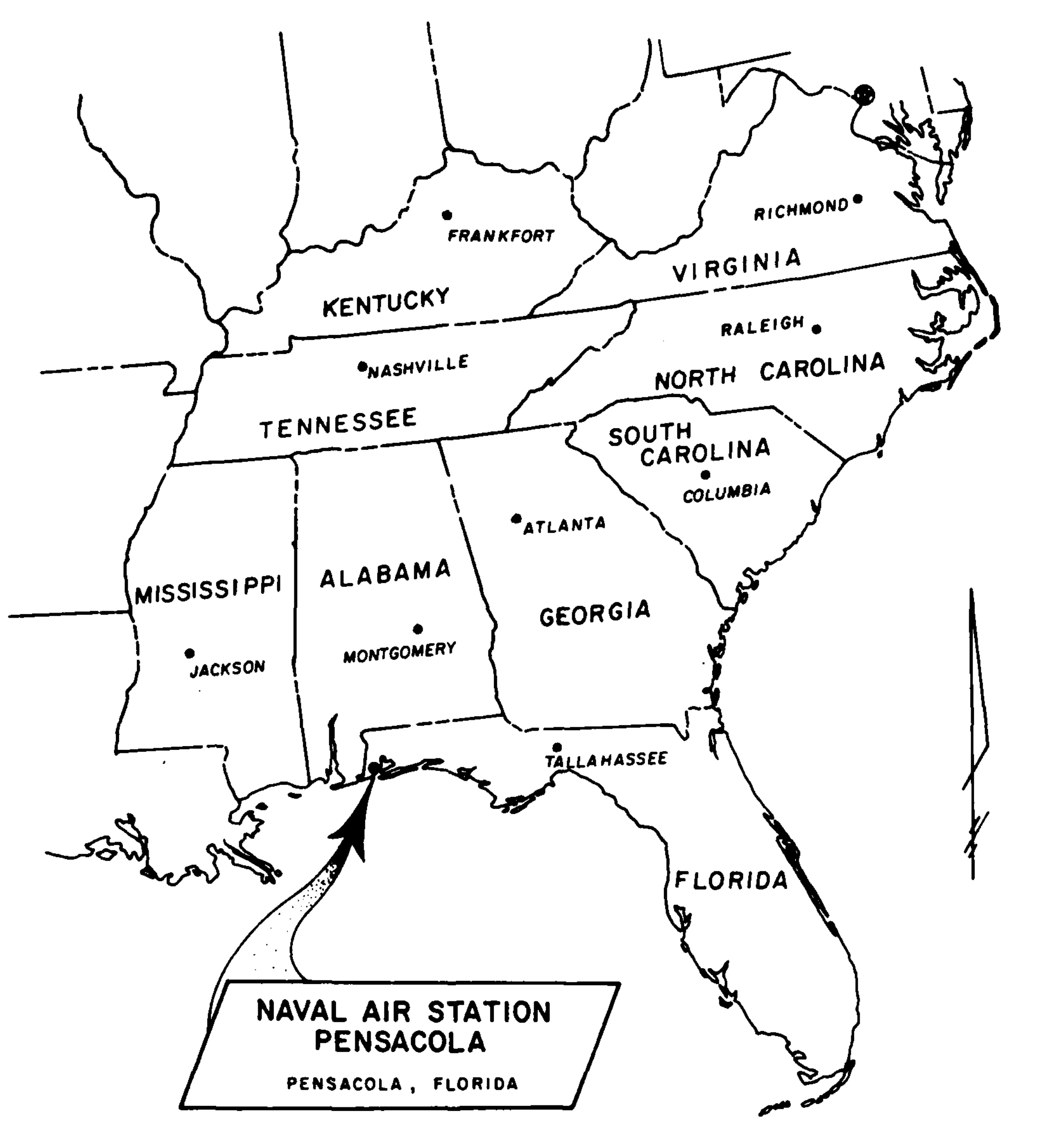
2.2 Mainside Complex

The mainside complex consists of approximately 5,589 acres, including easements and supports 44 tenant command organizations and activities. The topography of the complex is basically flat ranging from sea level to approximately 40 feet above mean sea level. The southern and eastern edges of the complex have coastal soils bordering saltwater and consisting of coastal dune sand and beach tidal marsh. Bayou Grande (bordering the northern edge of the complex) contains gray sandy soils on low lands, somewhat poorly drained with a moderately high water table. Soil composition in two smaller areas, jutting into the western side of the complex consist of gray or very dark gray fine sands with poor drainage bordering a fresh water swamp. In the interior of the complex are sandy subsoils and light gray sands which are generally well-drained. Based on information furnished by the Corps of Engineers, Mobile District, the 100 year frequency tidal flood elevation at the NAS is 9 feet above mean sea level (approximate).

2.3 Port of Pensacola

The Port of Pensacola is the primary port facility for West Florida. The 3 channels leading into the Port are the entrance channel (37 feet deep), the bay channel (33 feet deep) and the inner channel (33 feet deep). The U. S. Army Corps of Engineers maintains the depth of the commercial channel and the Navy funds the Navy channel (wider and deeper than commercial channel).

There are 19 berthing spaces ranging in length from 400 to 500 feet with various capacities in the Port System. The



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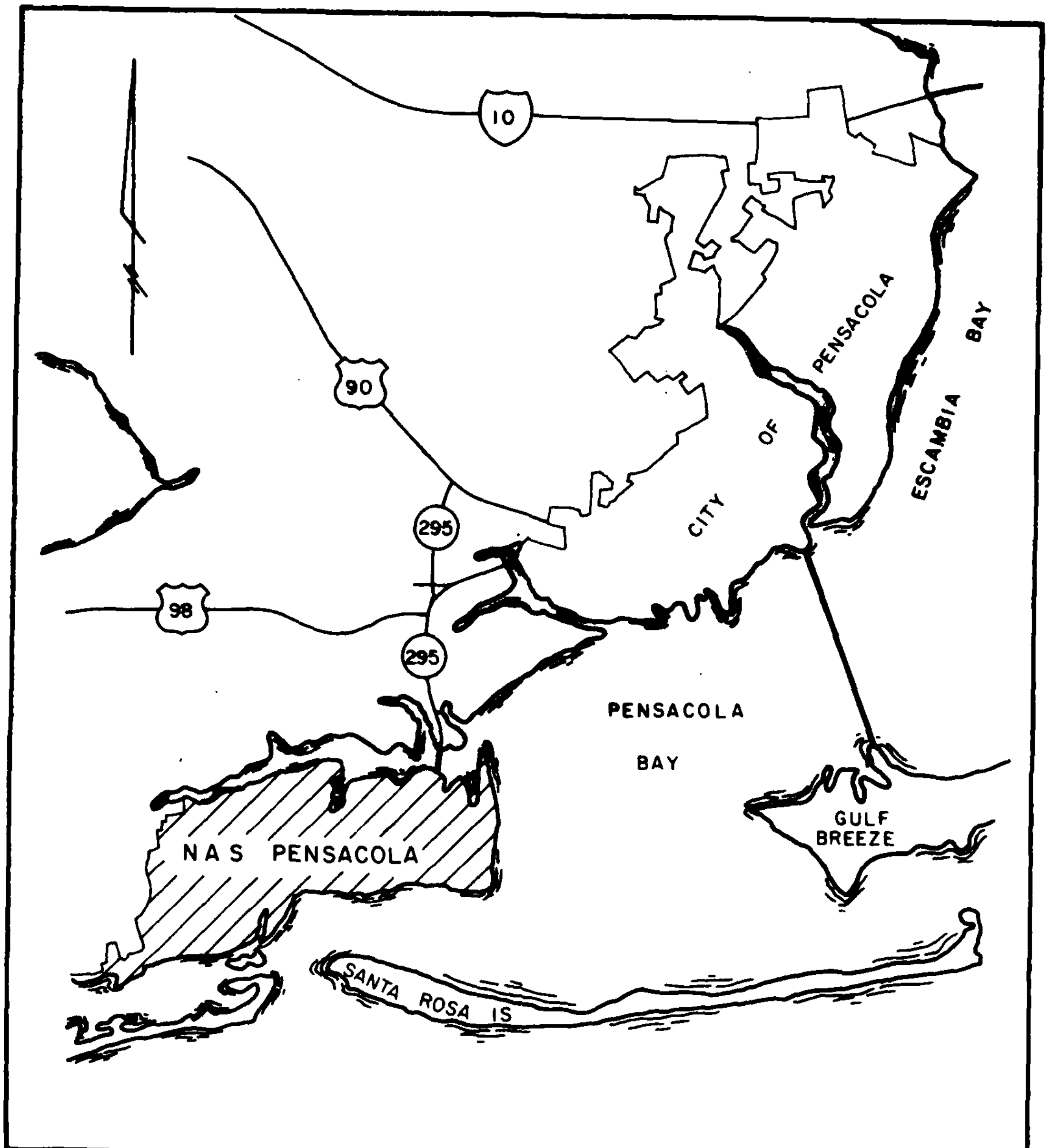
REGIONAL MAP

FIGURE 1



PREPARED FROM THE NAS PENSACOLA BASE MASTER PLAN	PREPARED BY ARTHUR V. STROCK & ASSOCIATES, INC. 629 S.E. 9th ST. DEERFIELD BEACH, FLORIDA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C. VICINITY MAP	FIGURE 2

Port consists of 3 general cargo warehouses and 6 steel storage tanks that have a storage capacity of 2 million gallons.

2.4 Reference

GFI - Pensacola Base Master Plan

SECTION 3.0 - INSPECTION PROCEDURE

3.1 Level of Inspection

During the period January 17 - January 28, 1983, a team of diving engineers performed Level I, Level II and Level III underwater inspections of selected piers, bulkheads and seawalls at the Naval Air Station in Pensacola, Florida. Level I "swim-by" inspections were made on all of the structural members of the designated piers, bulkheads and seawalls. The more thorough Level II inspection entailed the divers cleaning an area of the structural member while making note of the condition of the cleaned area. Photographs were taken to illustrate specific and typical conditions. Level III inspections were generally limited to key structural areas that had possible hidden or interior damage, loss in cross-sectional area and material homogeneity. Visual documentation and a sampling of section measurements were taken via an underwater camera and an underwater ultrasonics thickness tester. Soundings were taken and referenced to the top of the waterfront facility.

3.2 Inspection Procedure

The combination of water conditions, pier construction and required levels of inspection allowed the diving engineers to perform the pier, bulkhead and seawall inspections utilizing SCUBA diving equipment. The scope of work included inspection of 12,225 feet of seawalls and bulkheads and 325 piles supporting nine different facilities. In addition, thirteen ramps were inspected.

Level I, Level II and Level III inspections were performed independently by a team of two divers and two surface support personnel. Each member of the team spent half of a working day in the water performing inspections and half of a working day as a surface support personnel. Level I inspection was performed on all piles supporting the pier structures and along the entire length of bulkheaded structures. The divers directly recorded structural conditions with a pencil and mylar form prepared for each structure, or the divers related their observations to the surface support personnel for recording. Level I inspection was typically made in two passes. The first pass was made at the surface, whereas one diver inspected the splash zone and another diver inspected the intertidal region. The second pass of Level I inspection was made by two divers swimming just above the mudline. Level II inspection entailed a detailed inspection of steel sheet pile structures at every 200 linear feet and concrete and granite structures at every 300 linear feet on the average. Level II inspection was performed on approximately 15% of the square concrete piles supporting the platform of Pier 303. A Level II (detailed with cleaning) inspection was performed on 12 piles of Finger Pier 303 and 4 piles of Finger 3238. Under Level II inspection structural members were cleaned over a small area (a foot square typically) as necessary to evaluate their structural condition; square concrete piles were cleaned on three sides. Level III inspection encompassed the use of an ultrasonic thickness measuring device on all steel sheet pile and steel H pile structures on the areas cleaned under Level II. These areas cleaned under Level II were at either the splash zone or mudline area for the steel structures. The ultrasonic device was calibrated prior to its use at each facility and at the beginning of each day to insure its precision and accuracy. The ultrasonic testing was also performed independently of the Level I and Level II inspection.

Figure 3 is a copy of the data recording slate used for seawalls, bulkhead and ramp inspection. Figure 4 is a copy of the recording slate used for pile supported pier inspections. The data recording sheets allowed the diving engineer or surface support personnel to easily record the inspectors observations. Such observations included:

1. The amount of marine growth on the piles.
2. The condition of the concrete and/or steel, i.e. whether it was cracked, spalled, failed or corroded.
3. The amount of visible rust or exposed steel
4. Any other comments the engineer felt should be recorded concerning his observations.

Figure 5 illustrates the diver inspection paths typically followed along seawalls, bulkheads, and pile supported structures. Level I and Level II inspections were performed in one pass where water depths were less than four feet. Divers swam up and down the piles of piers supported by steel H-bearing piles or square concrete piles.

Prior to inspection, the bents on each pier were numbered, starting at the bulkhead and proceeding to the outboard end of the pier. Similarly, the waterfront was stationed to identify the location of observations.

The diving engineers performed a concentrated inspection of piles at the cap, at the mudline and at the midsection of the piles. This concentrated effort was made in these areas as they are expected to demonstrate damage due to overloading of the piles. Figure 6 shows the expected failure modes for typical pier piles in terms of the physical condition, structural model and moment diagram. The most probable cause of pile failure is direct impact and a forced displacement at the cap due to a mooring force impact at the cap. In both cases, the maximum moment exists near the cap or near the mudline. For both modes of probable pile failure, exterior

STATION:

TOP OF WALL

M.H.W.

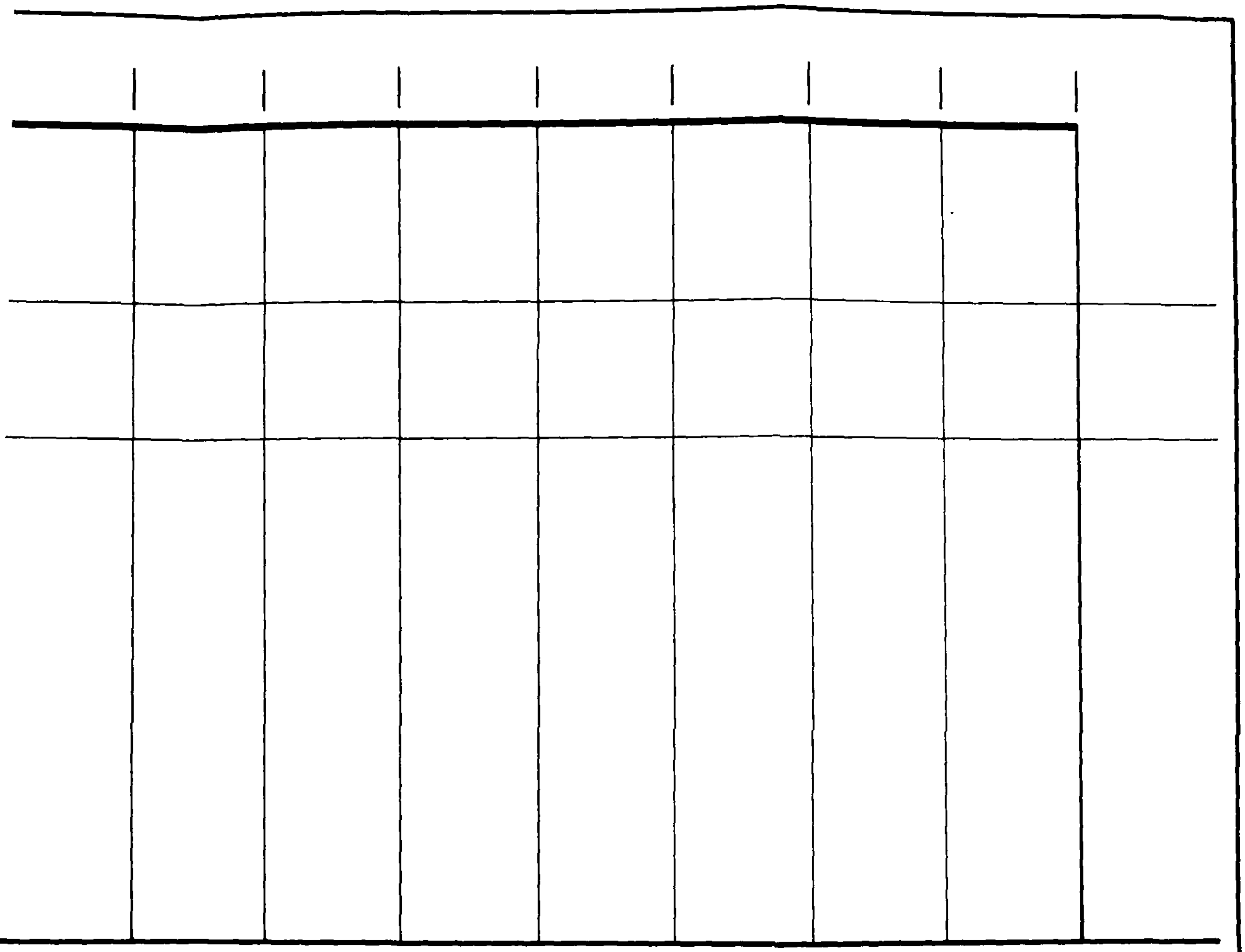
M.L.W.

MUDLINE

ELEVATION VIEW OF WET FACE

N.T.S.

NOTES:



ION VIEW OF WET FACE

N.T.S.

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NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON, D.C.

DATA RECORDING SLATE
SEAWALLS, BULKHEADS
& RAMPS

FIGURE
3

2

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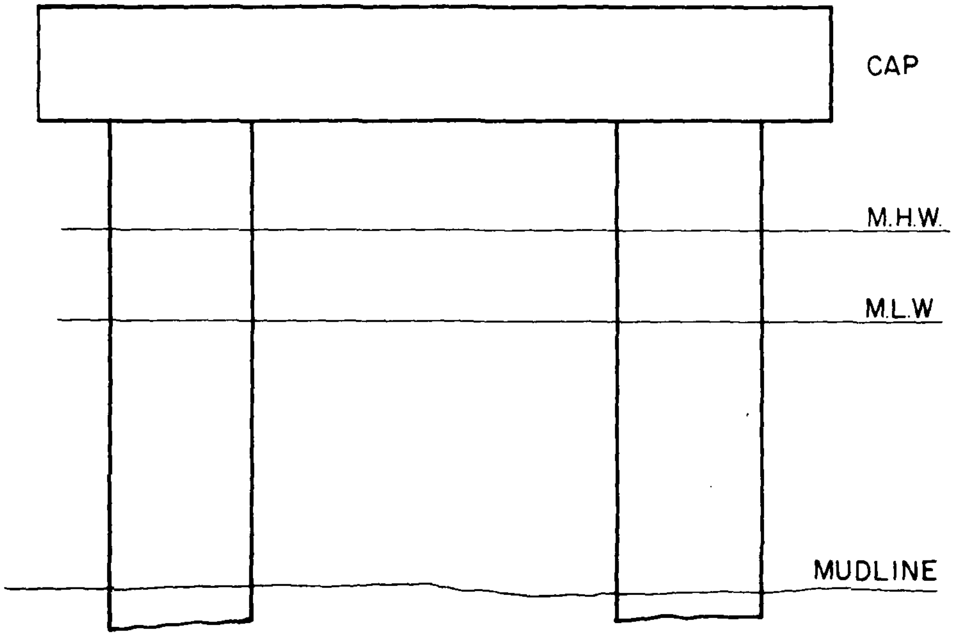

NOTES:

ELEVATION VIEW
N.T.S.

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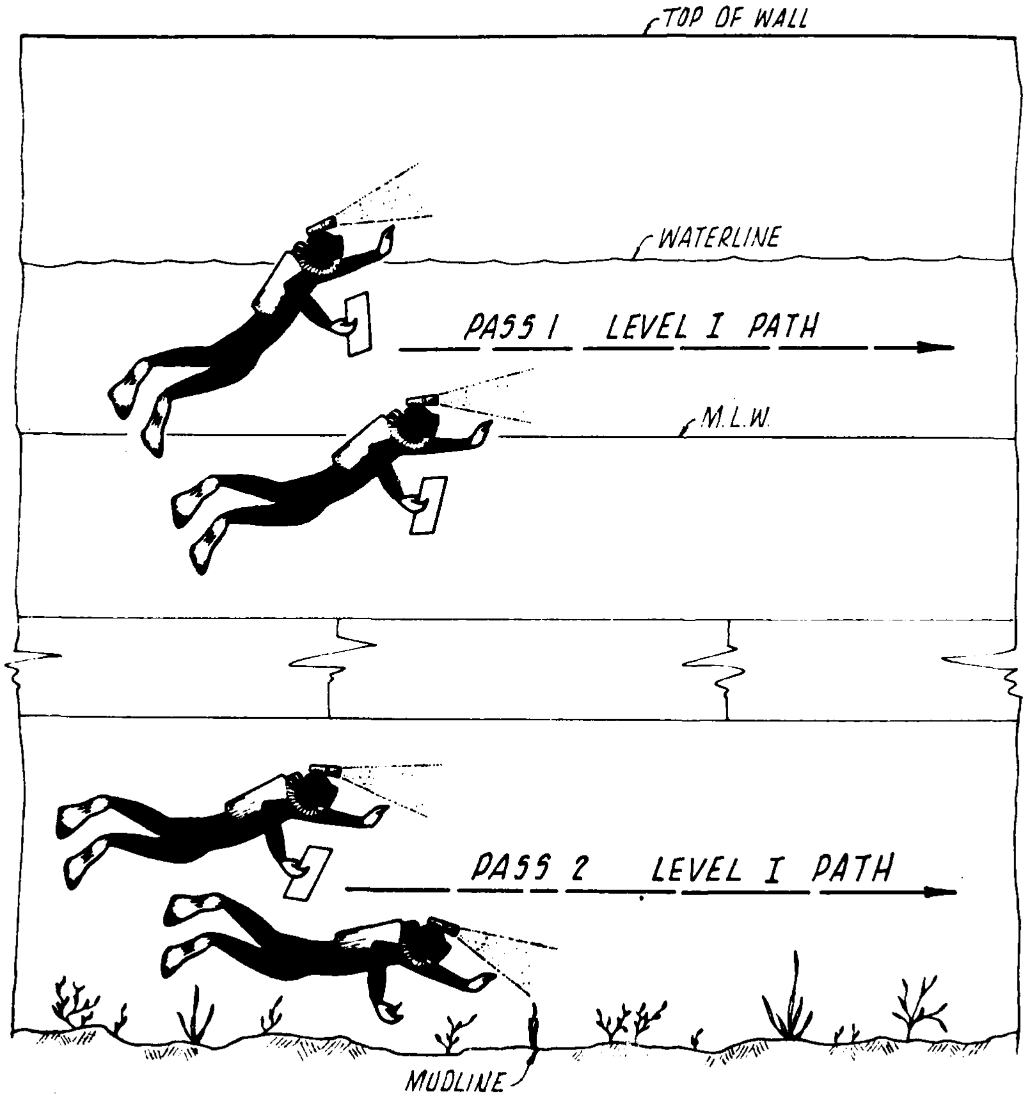
		DATA RECORDING SLATE PILE SUPPORTED PIERS	FIGURE 4

DIVER INSPECTION PATH
BULKHEADS & SEAWALLS



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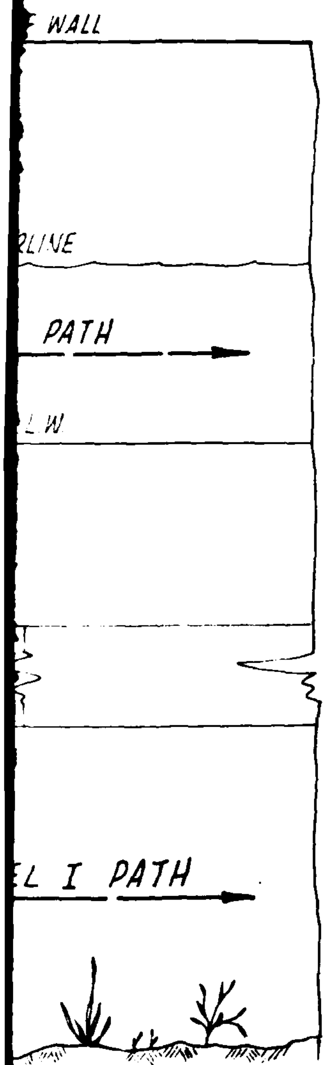
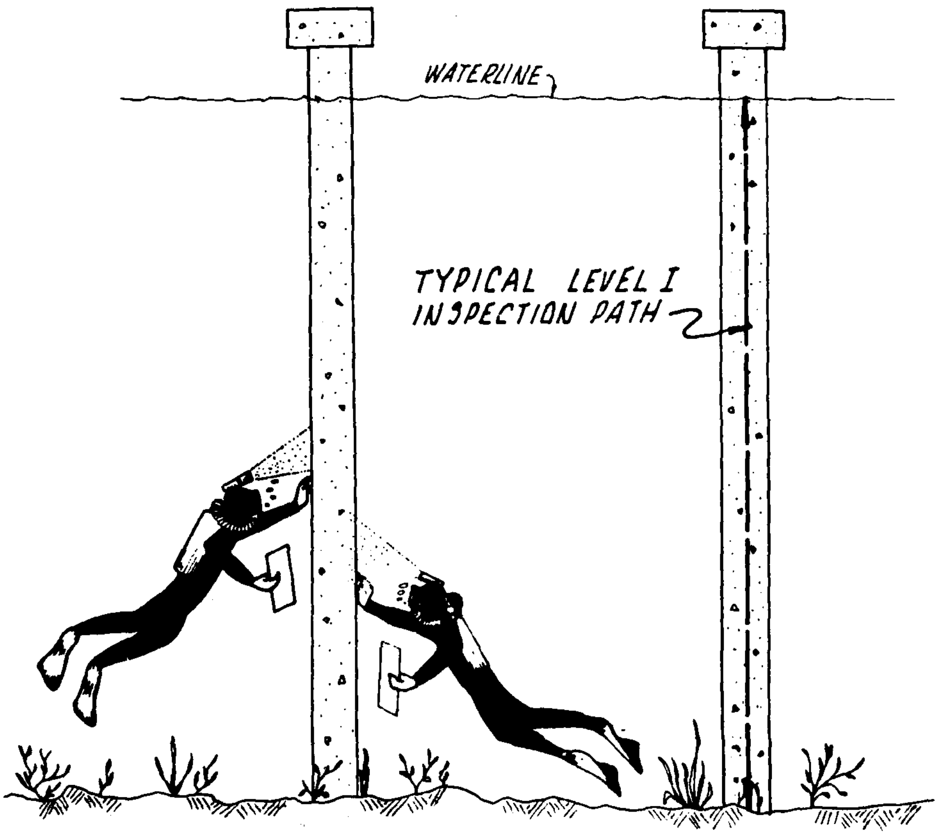


DIVER INSPECTION PATH

PILE SUPPORTED STRUCTURES



2

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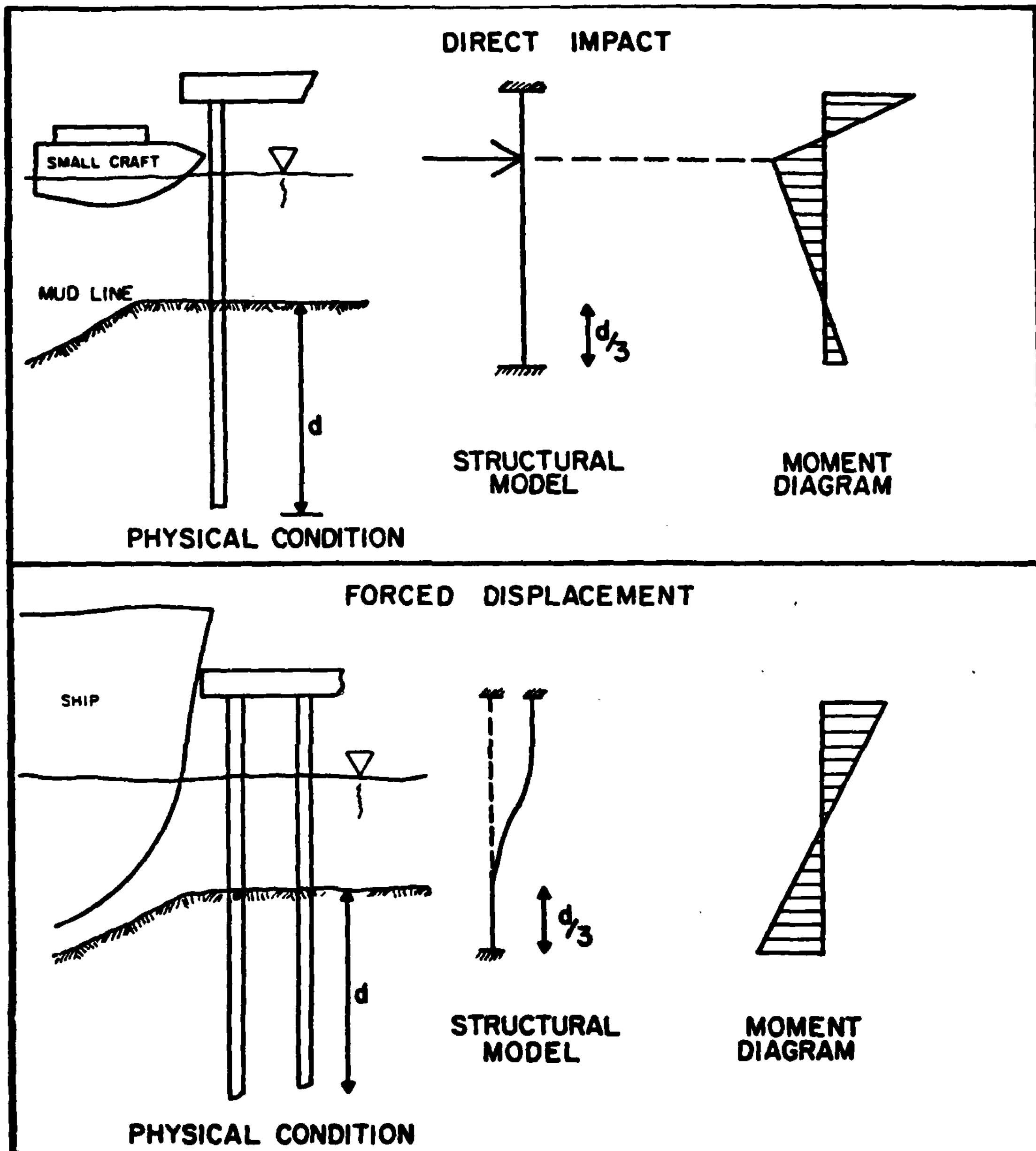
DIVER INSPECTION PATH - LEVEL I

FIGURE 5



REFERENCE
CHELLIS, R.D.
"PILE FOUNDATIONS"
2nd EDITION
NEW YORK
McGRAW-HILL

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PIER PILE
FAILURE MODES

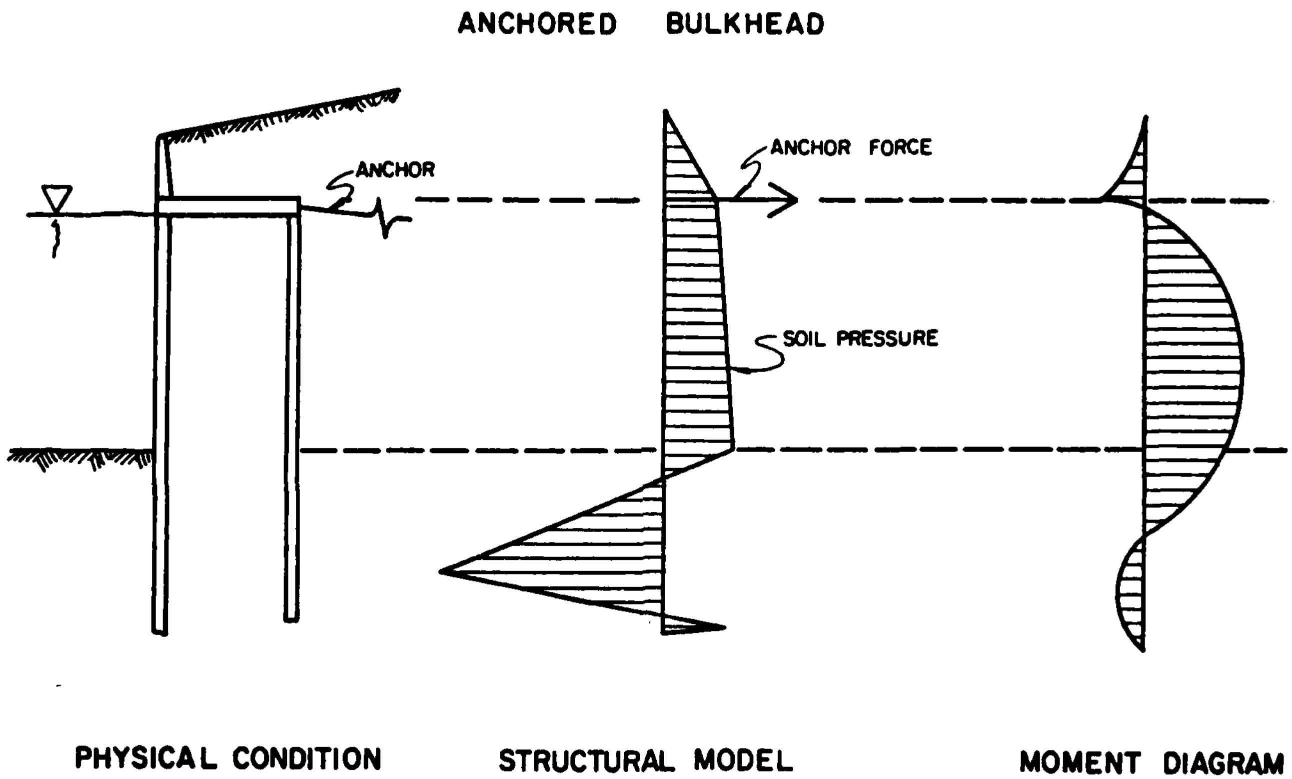
FIGURE 6

piles are most likely to fail due to their proximity to applied loads. Figure 7 shows the typical physical conditions, structural model and moment diagram for an anchored bulkhead with a fixed earth support. Pile failure is most likely to occur as a result of a moment failure. Furthermore, the splash zone and tidal zone were closely inspected as these areas are most susceptible to reinforcement steel corrosion and concrete deterioration due to exposure to the elements.

3.3 Inspection Equipment

The following equipment was employed by diving engineers during pier, bulkhead and seawall inspections:

1. Standard scuba diving equipment;
2. Nikonos-II underwater camera with Toshiba strobe;
3. Subawider II wide-angle underwater lens attachment (used for taking close-up photographs in turbid water);
4. Assorted scrapers, chipping hammers, calipers, measuring tapes;
5. Data recording slates and;
6. Ultrasonics thickness tester (Panametrics - Model 5222UG).



REFERENCE RICHART, F.E. JR. "ANCHORED BULKHEAD DESIGN BY NUMERICAL METHOD" TECHNICAL PAPER 189 UNIVERSITY OF FLORIDA AUG., 1960	PREPARED BY ARTHUR V. STROCK & ASSOCIATES, INC. 829 S.E. 9TH ST. DEERFIELD BEACH, FLORIDA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C. ANCHORED BULKHEAD & FIXED EARTH SUPPORTS — FIGURE 7

SECTION 4.0 - FACILITIES INSPECTED

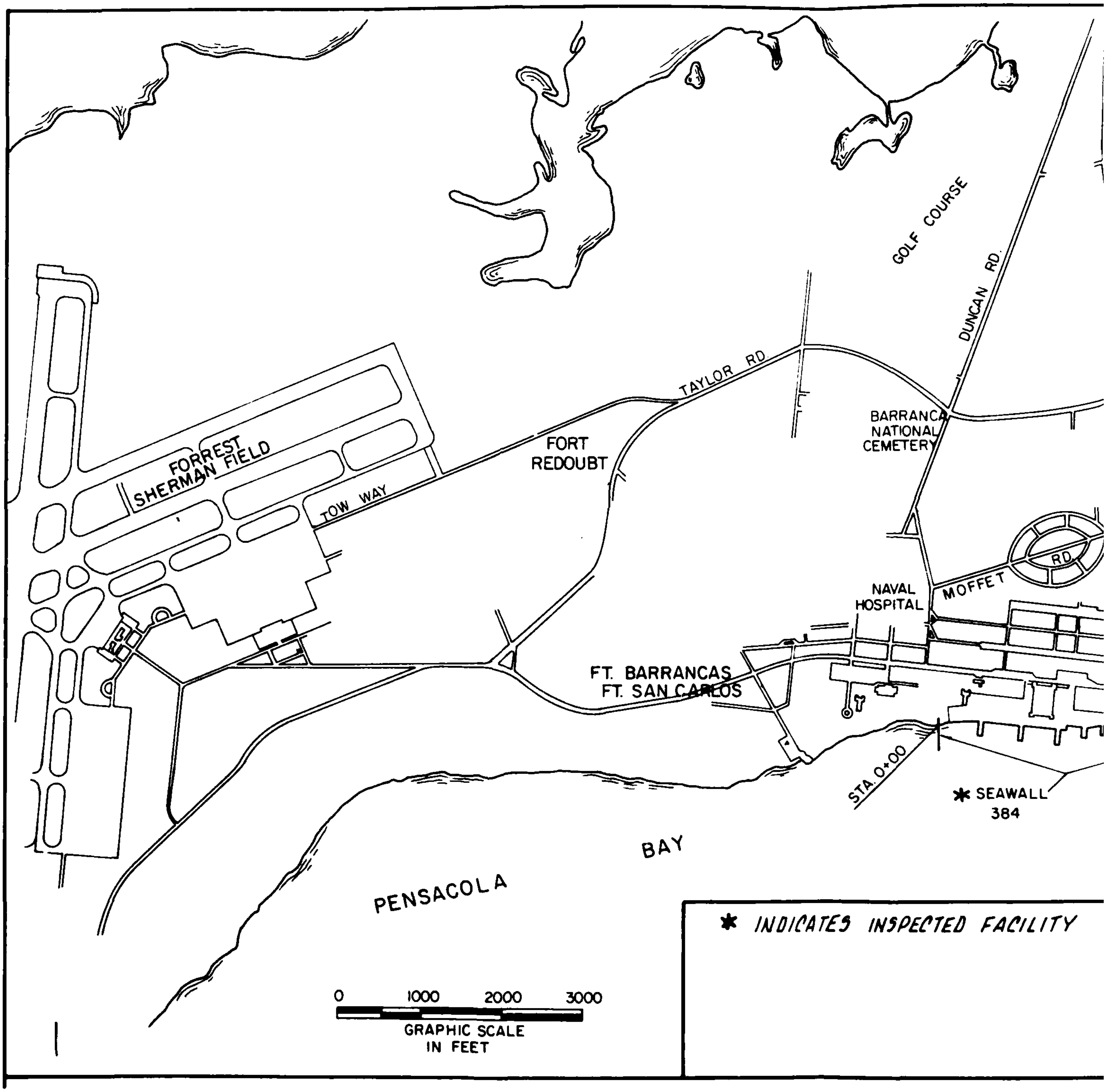
4.1 Designated Facilities Inspected

The facilities inspected at the Pensacola Naval Air Station are listed in Table 1 which also lists the stations assigned to each facility. The location on base of these facilities is illustrated in Figure 8. Appendix A contains drawings illustrating the stationing used to identify the location of observations made in the field; these drawings also illustrate physical features of the facilities.

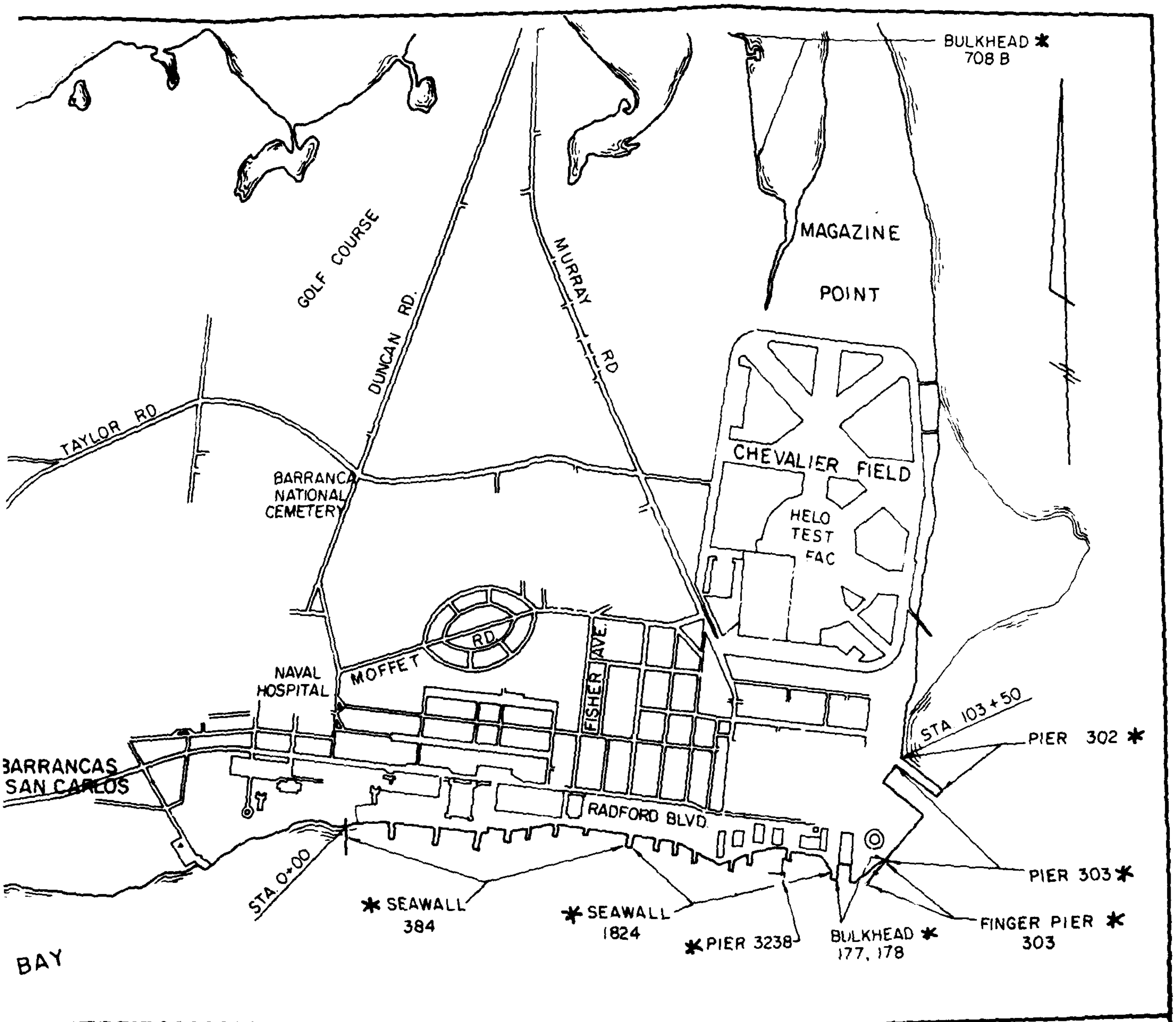
In the remaining portion of this section, each facility inspected at Naval Air Station, Pensacola is referenced separately. A description of its construction, specific observed conditions, assessment of these conditions and recommendations for repairs are included for each facility inspected. Appendix B contains the transcripts of field observations identified by facility and station. Appendix C contains details of the structural evaluation of the facilities. Appendix D contains a breakdown of cost estimates for repair of each facility.

TABLE 1
FACILITIES INSPECTED
AND
ASSIGNED STATIONING

FACILITY	STATIONS
Seawall 384	0+00 - 34+03
Seawall 1824	34+03 - 58+12
Bulkhead 177,178	58+83 - 76+60
Pier 303	75+00 - 91+28
Pier 302	91+28 - 103+50
Pier 3238	at 40+88
Finger Pier 303	at 74+85
Bulkhead 708B	Separate Stationing (see Section 4.9)



FORREST SHERMAN FIELD
TOW WAY
TAYLOR RD
FORT REDOUBT
GOLF COURSE
DUNCAN RD.
BARRANCAS NATIONAL CEMETERY
NAVAL HOSPITAL
MOFFET RD
FT. BARRANCAS
FT. SAN CARLOS
STA. 0+00
* SEAWALL 384
PENSACOLA BAY
0 1000 2000 3000
GRAPHIC SCALE IN FEET
* INDICATES INSPECTED FACILITY



* INDICATES INSPECTED FACILITY	PREPARED BY ARTHUR V. STROCK & ASSOCIATES, INC. 829 S.E. 9th ST. DEERFIELD BEACH, FLORIDA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C. PROJECT SITE — FIGURE 8

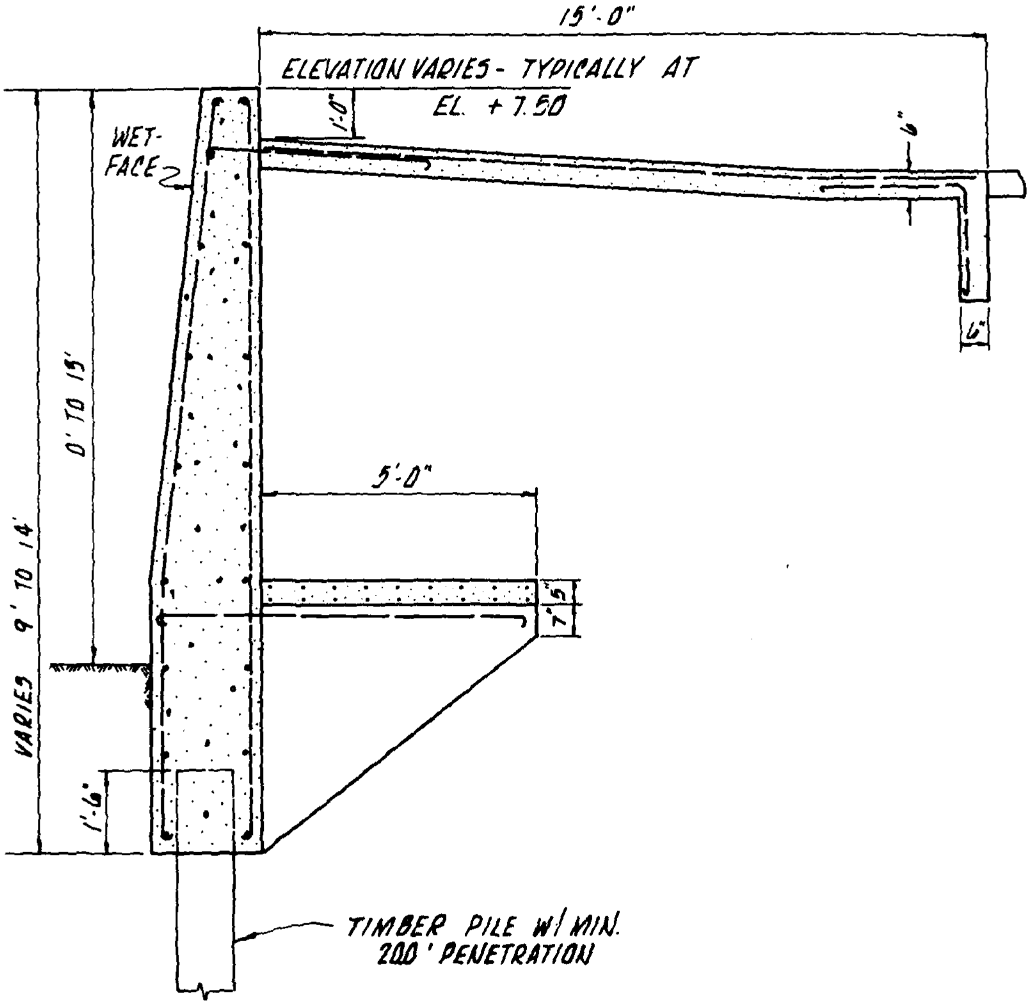
4.2 SEAWALL 384

4.2.1 Description

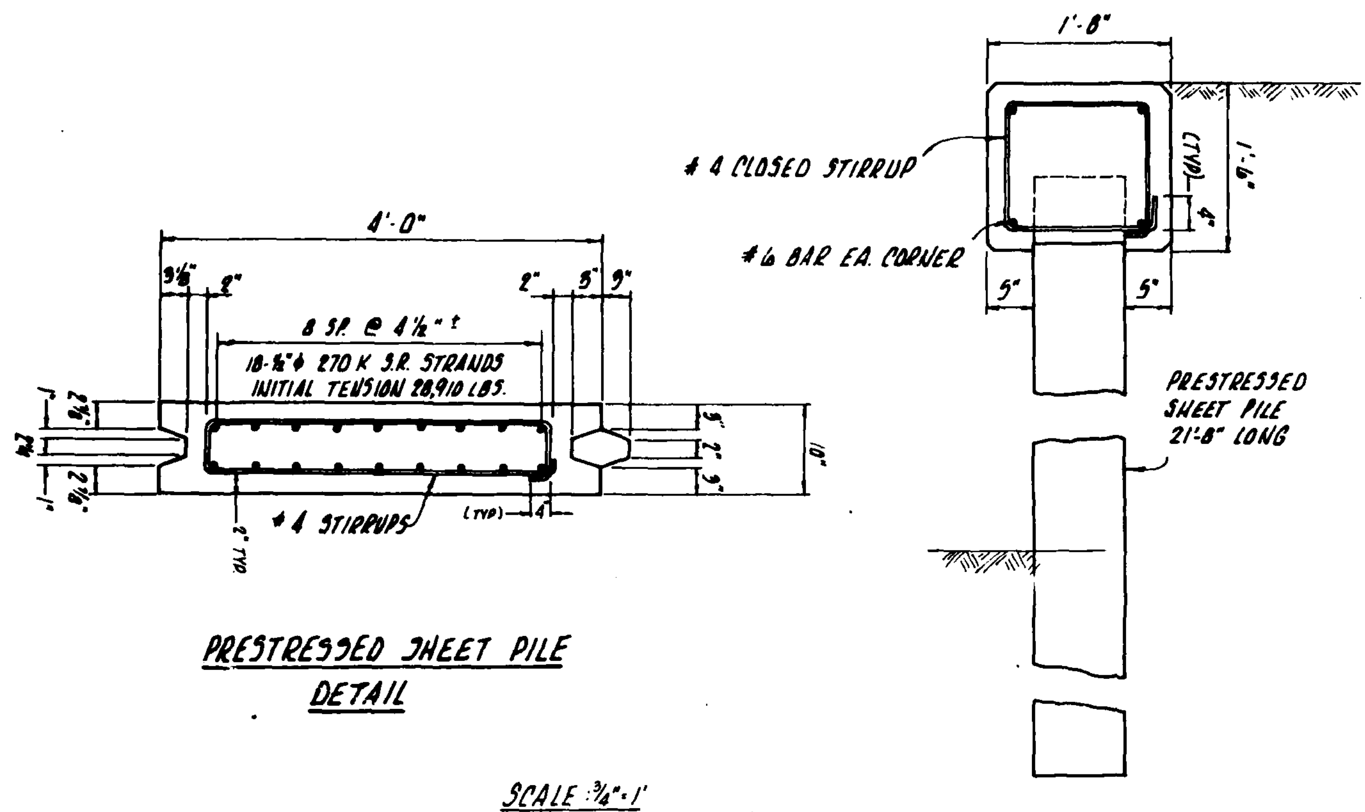
A substantial portion of the waterfront (3,403') at Naval Station Pensacola is protected by Seawall 384, which was constructed in 1931. Seven ramps which at one time provided access to Pensacola Bay for seaplanes from the upland, bridge across the seawall. Figure 9 illustrates in section the construction of Seawall 384. The seawall is a poured in place reinforced concrete structure supported by timber piles. According to Navy drawings, lateral support to the seawall is provided by a poured in place concrete slab and anchor as shown in Figure 9. At 40' to 50' intervals, vertical tongue-and-groove joints were formed in the wall. Numerous outfalls exist through Seawall 384. Following the passage of Hurricane Frederick in 1979, several repairs were made to the seawall. Part of these repairs was the replacement of a segment (station 6+00 to 7+10) of the seawall with prestressed concrete sheet piles as shown in Figure 10 (also see NAVFAC drawing No. 5068068). The ramps bridging this seawall are of two concrete types - pile supported and slab on grade with side retaining walls. Base personnel have indicated that these ramps have not been used for seaplane operation for many years.

4.2.2 Observed Inspection Conditions

Seawall 384 appears to be in overall good condition. Hairline cracks, rust stains, scaling and minor spalls were noted in isolated areas throughout the length of the wall. From station 0+00 to station 2+10 concrete rubble and rock were observed in front of the wall up to the top of the cap as shown in photos 1 and 2. The grades in other areas fronting Seawall 384 vary in height from 4' below mean sea level to 3' above mean sea level. Marine growth on the wetface of the seawall was limited to green algae and scattered barnicles typically less than ½" deep. Throughout the length of the wall the tongue-and-groove joints generally appeared to be in poor condition whereas the construction joint seal had deteriorated and allowed for material to seep through the wall. Photo 3 shows a typical tongue-and-groove joint (in plan). Throughout most the length of the wall rust stains were noted below what appeared to be mounts for hardware as shown in photo 4. Deposits of material were noted at the toe of some areas of the wall, below outfalls as shown in photo 4. Spalling of the concrete was predominantly noted in the area of tongue-and-groove joints as shown in photo 5. At station 24+80 heavy scaling of the concrete walls surface was noted; photo 6 shows this condition. The concrete sheet pile segment of the seawall appeared to have been sealed with grout at the joints of the sheets but the seal terminates at 2' above the berm, as shown in photo 7. It appears that the berm was located at the bottom of the seal at the time the seal was placed.



FROM Y&D DRAWINGS NO. 103817 103818 103819	PREPARED BY ARTHUR V. STROCK & ASSOCIATES, INC. 829 S.E. 9th ST. DEERFIELD BEACH, FLORIDA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C. SEAWALL 384 TYPICAL SECTION (ORIGINAL WALL) FIGURE 9



FROM NAVFAC DRAWING NO. 5068068

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SEAWALL 384
TYPICAL SECTION
(REPLACED SEGMENT)

FIGURE
10

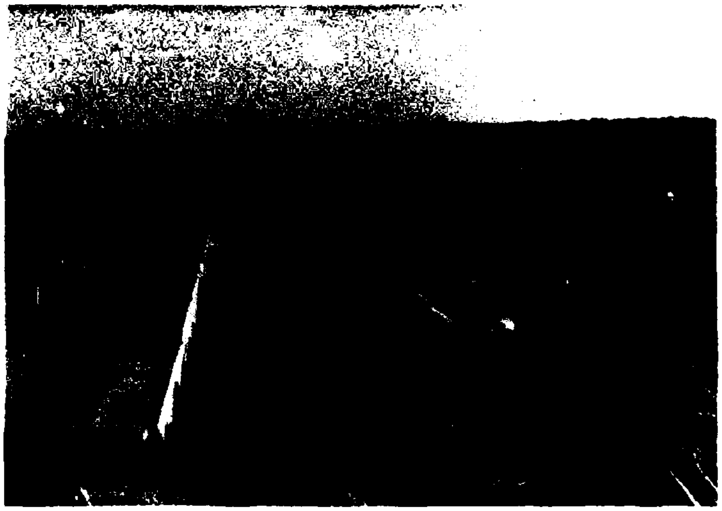


PHOTO NO. 1

Seawall 384 - Rock rubble at face of wall at station 0+00 thru 1+67.

PHOTO NO. 2

Seawall 384 - Perspective view of seawall from station 1+67.

PHOTO NO. 3

Seawall 384 - Plan view of typical tongue-and-groove joint.

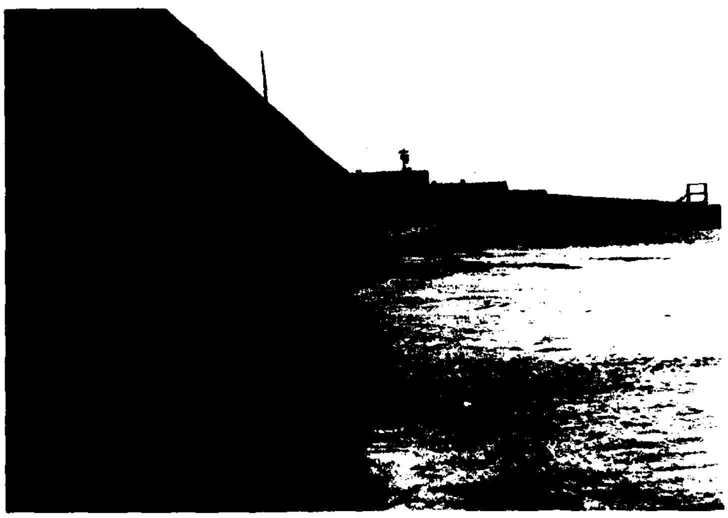


PHOTO NO. 4

Seawall 384 - Typical view of wetface and material deposits below outfall.

PHOTO NO. 5

Seawall 384 - Joint spall and hairline cracks at station 15+20.

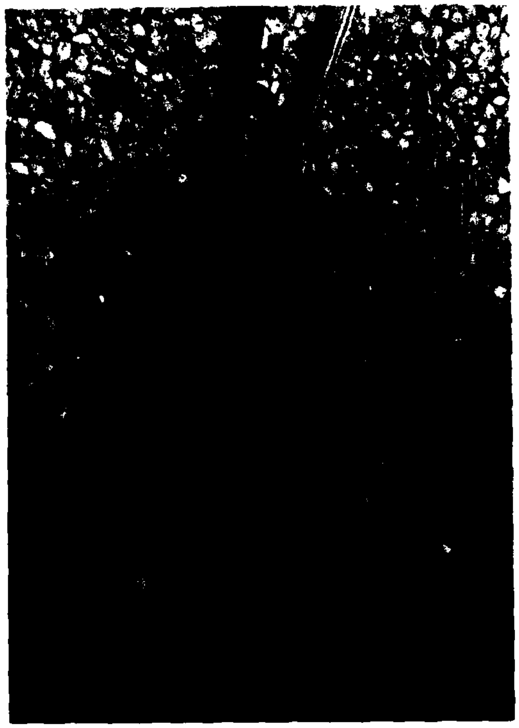


PHOTO NO. 6
Seawall 384 - Heavy scaling at station 24+80.

PHOTO NO. 7

Seawall 384 - Concrete sheet pile with grout at joint at station 7+00.

Electrical conduit wires and junction boxes were also noted; the conduit in the boxes were severely corroded where they are exposed. Photo 8 shows a typical junction box; this box was located at a tongue-and-groove joint. The outfalls in the wall appear to be generally well sealed and did not appear to be leaking soil in general. Photo 9 shows a typical outfall at station 23+00. A 4 to 6 inch "lip" was noted at the berm (mudline) along much of the seawall (stations 12+30 to 28+20).

The concrete ramps were also inspected. Two types of ramps were observed; pile supported and filled ramps. All the ramps typically were noted to have spalling along edge beams and at minimum, hairline cracks in the surface of the ramp. In some instances where the pile supported ramps were bearing on top of Seawall 384, map type cracking was observed in the wetface of Seawall 384 as shown in photo 10.

Appendix A contains drawings illustrating in plan the stations assigned to this facility. Appendix B contains transcripts of the field observations made in conjunction with the facility inspection.

4.2.3 Structural Condition Assessment

Seawall 384 appears in good structural condition. Although no signs of structural failure appear eminent, evidence that the seawall is losing its ability to retain upland material was noted. In particular, the tongue-and-groove joints are in poor condition and do not provide a proper seal to the seawall. Furthermore, the toe of the seawall may be undermined where a "lip" was observed at the berm. Navy drawings indicate that the top to toe height of the seawall is 9 to 14 feet (see Figure 9). The top to toe height measured in the field was as much as 13 feet in the area where a "lip" was observed (see Appendix A for soundings). Based on these observations and the Navy drawings, a condition of inadequate toe penetration may exist. Although the wall appears stable, material may be seeping under the toe of the wall.

The ramps appear in poor to good condition but do not appear to be in danger of collapse. The ramps appear stable under dead loads only. Detailed structural analysis or load rating is beyond the scope of this report.

4.2.4 Recommendations

The tongue-and-groove joints of Seawall 384 should be cleaned of the deteriorated seal material and resealed with a non-shrinking, non-oxidizing permanent elastic sealant. The joints of the concrete sheet piles should be sealed with a non-shrinking grout from the bottom of the existing seal to at least 2' below the berm.

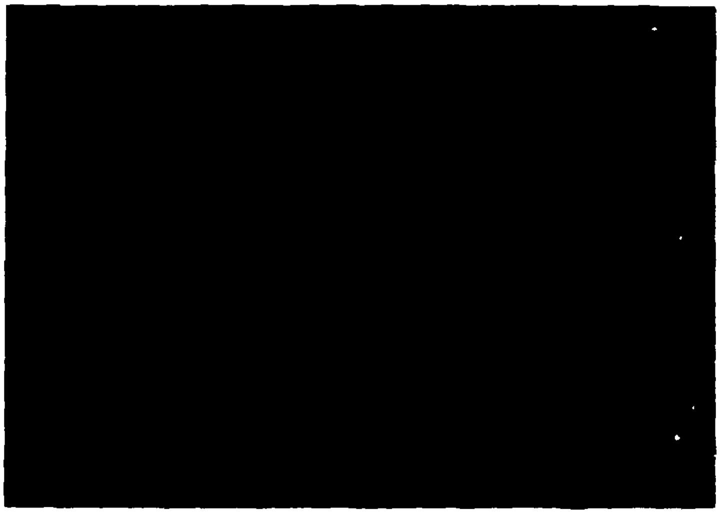


PHOTO NO. 8

Seawall 384 - Electrical box and conduit at joint at station 9+20.

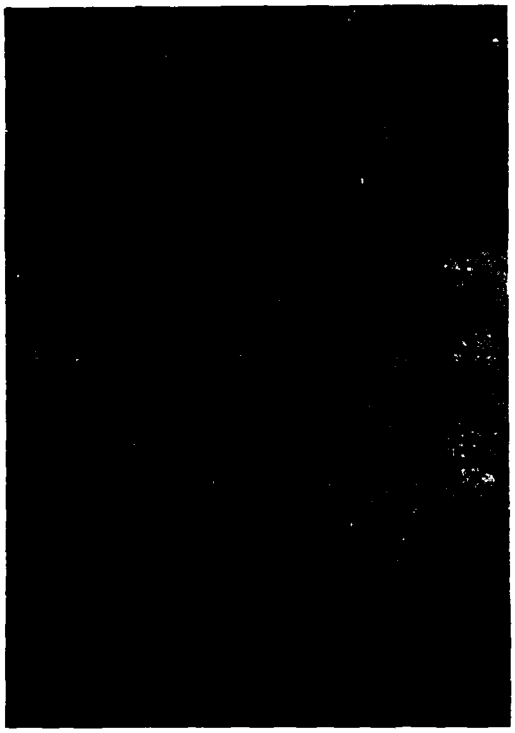
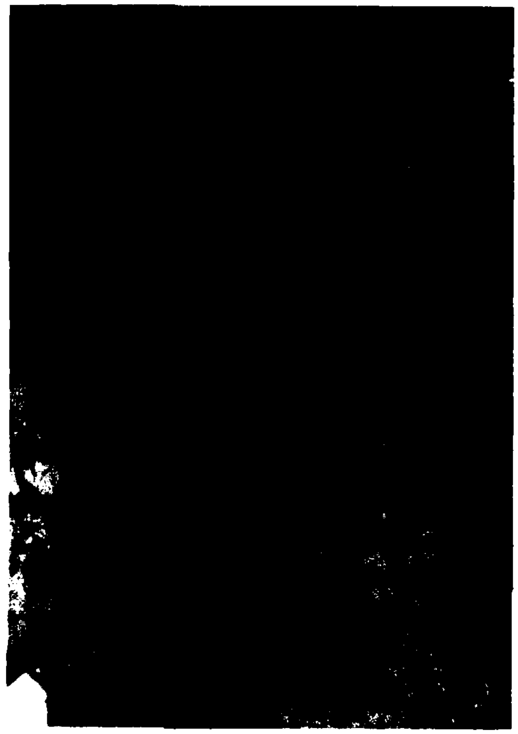


PHOTO NO. 9

Seawall 384 - Typical outfall and moderate scaling (station 23+00).



<u>PHOTO NO. 10</u>

Seawall 384 - Map cracking under ramp.
(stations 7+10 thru 7+90)

Further inspection is also warranted. The pavement immediately behind the seawall should be bored at least at 200 foot intervals (along the wall) to check for loss of material underneath the pavement. If loss of material underneath the pavement has occurred then the undermined pavement should either be closed to traffic or removed and replaced following placement of fill. The depth of the toe of the wall should also be investigated by water jet probing to check for undermining and seepage under the wall - a toewall may be required if penetration is inadequate.

Prior to the use of the ramps bridging Seawall 384, a detailed structural analysis and load rating should be performed. Depending on the expected loads, repairs may be necessary. The ramps do not appear to warrant repair if they are not used or are not planned for future use. However, if no repairs are made then further deterioration may be expected. To deter the deterioration of the reinforced concrete by corrosion of reinforcement, the ramps should be restored. This restoration should include the removal (by chipping away) of loose and broken concrete, sandblasting and coating (with epoxy) the corroded reinforcement and then restoring the original member dimensions with gunite or formed and poured concrete. This work is estimated to cost $50,000 per ramp. Without this work the average expected life of the ramps is 5 years. With completion of the above restoration the expected life of the ramps is 25 years. This repair work may be considered elective in terms of the expected use of the ramps.

With the repair of the joints and repair of the pavement and toe (as deemed necessary by further inspection) the expected life of the seawall is 25 years. Without these repairs the expected life is less than 5 years - during which time the pavement is expected to fail due to loss of material from behind the wall. The estimated cost of the joint (seal) repairs is $10,000 (see Appendix D).

Both the ramps and seawall should be inspected every 4 years. The pavement behind the seawall should be inspected annually to check for signs of settlement.

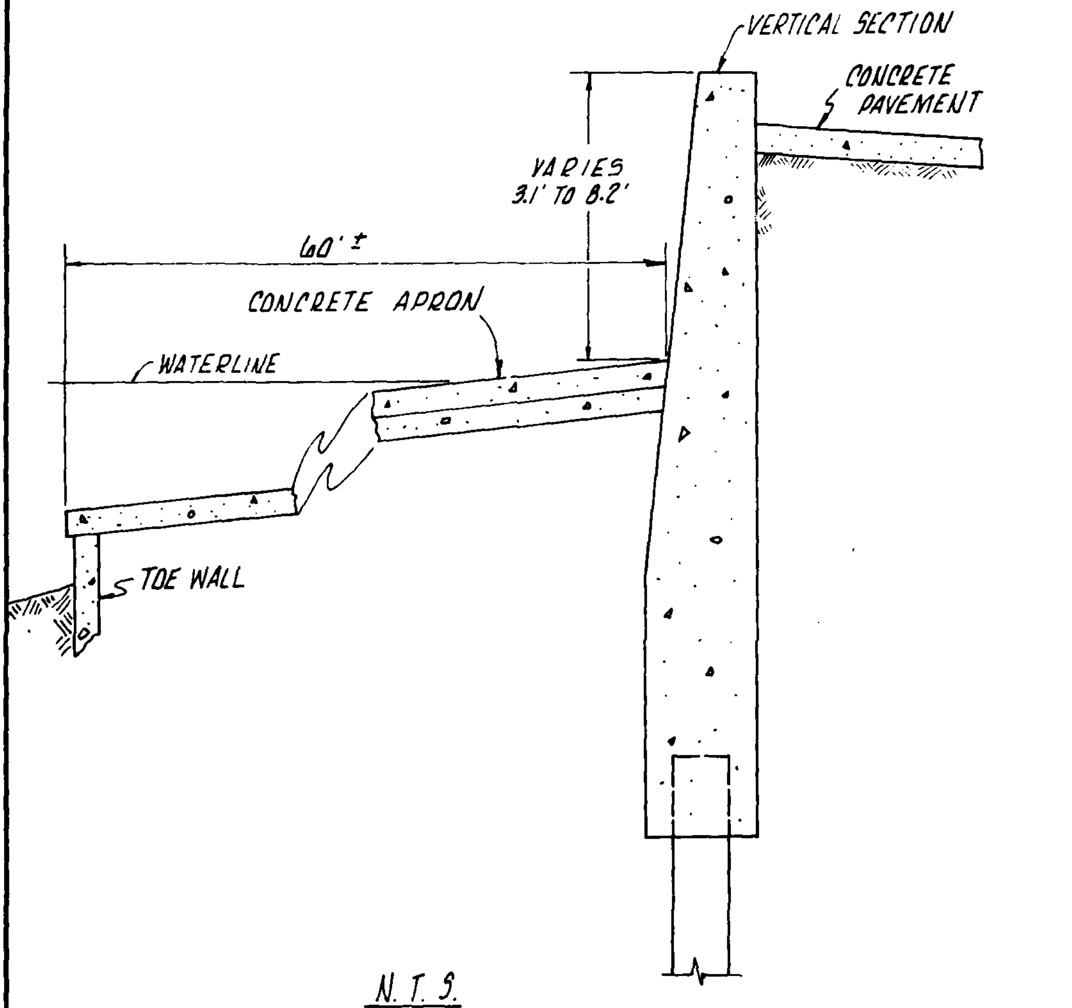
4.3 SEAWALL 1824

4.3.1 Description

Seawall 1824 protects 2,409 feet of the waterfront at Naval Air Station Pensacola; the seawall was constructed in 1924. Seawall 1824 appears very similar in construction to Seawall 384, however a concrete apron has been constructed in front of the vertical portion of the seawall from stations 34+03 to 51+64. This concrete apron ("concrete beach") extends approximately 60' waterward from the vertical face of the seawall. No structural details of the seawall are available from government files. The toe of the apron appears to be supported by a shallow toe wall. A cross-section of Seawall 1824 is shown in Figure 11 based on field observations and the structural details of Seawall 384 (assumed to be applicable). A total of six (6) reinforced concrete ramps bridge over Seawall 1824. Base personnel have indicated that these ramps have not been used for seaplane operations for many years.

4.3.2 Observed Inspection Conditions

Seawall 1824 appears to be in overall good condition. The vertical face of the seawall was noted to have little evidence of deterioration and no notable signs of failure. Photo 11 shows a typical section of the vertical face of Seawall 1824 in good condition. As shown in this photograph, some areas of the apron were covered with sand. Marine growth was limited to green algae and barnacles less than 1" thick. The predominant deficiency noted in the vertical section of Seawall 1824 was the absence of a seal and spalling at the tongue-and-groove joints, this condition was noted throughout the length of the seawall. Some scaling and hairline cracks were also commonly observed in the vertical section of the seawall. The concrete apron appeared to be in fair condition; longitudinal and transverse failure cracks were noted throughout it's length as shown in photo 12. In some areas it appeared that a secondary slab had been poured over the initial slab as a maintenance effort. The top of the apron at the vertical face of the seawall appeared to have settled as evidenced by a gap between the edge of the apron and the vertical face of the wall. Traces of seal material were also noted on the vertical face above the apron. Photo 13 shows a typical example of this condition. Photo 14 shows an elevation view of the toe of the apron which appeared to bear on top of a toe wall. The toe of the apron was covered with sand throughout most of the length of the seawall. Where the toe was exposed it was typically observed in about four (4) feet of water at about sixty feet from the vertical face of the seawall. The underwater portion of the apron appeared to have hollow cavities around joints which were open and without a seal. Photo 15 shows



NOTE:
SECTION IS BASED ON OBSERVATIONS; DETAILS ARE SPECULATIVE AND APPROXIMATE ONLY

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SEAWALL 1824
TYPICAL SECTION

FIGURE 11

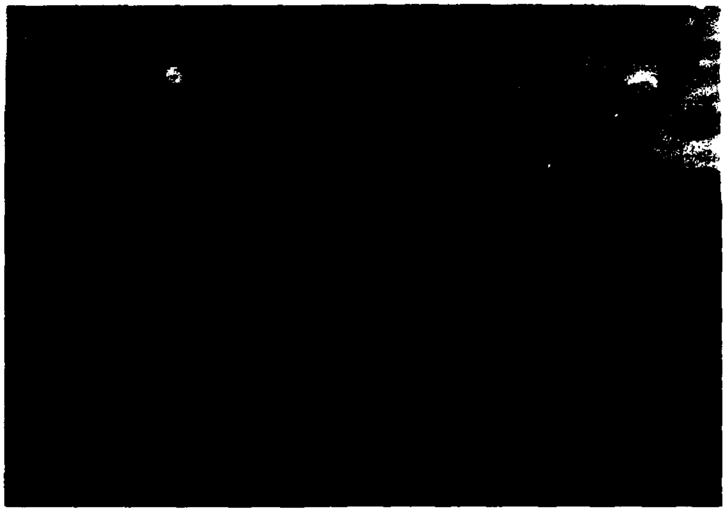


PHOTO NO. 11

Seawall 1824 - Vertical section of seawall in good condition.

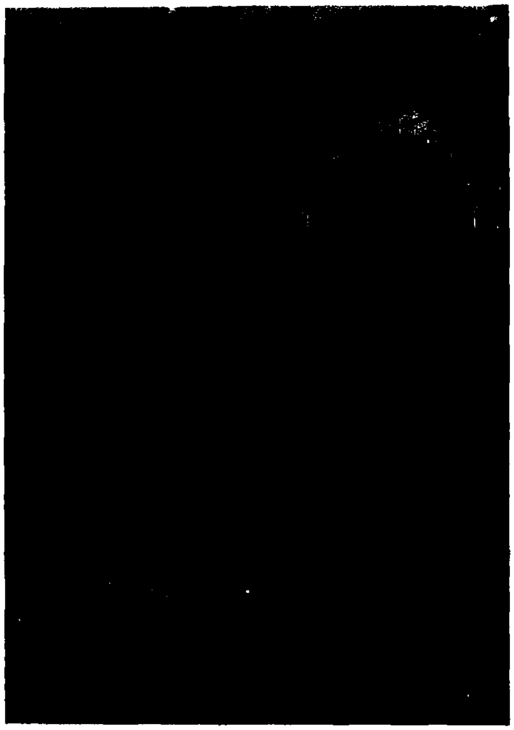


PHOTO NO. 12

Seawall 1824 - Typical transverse and longitudinal settlement cracks in concrete apron.

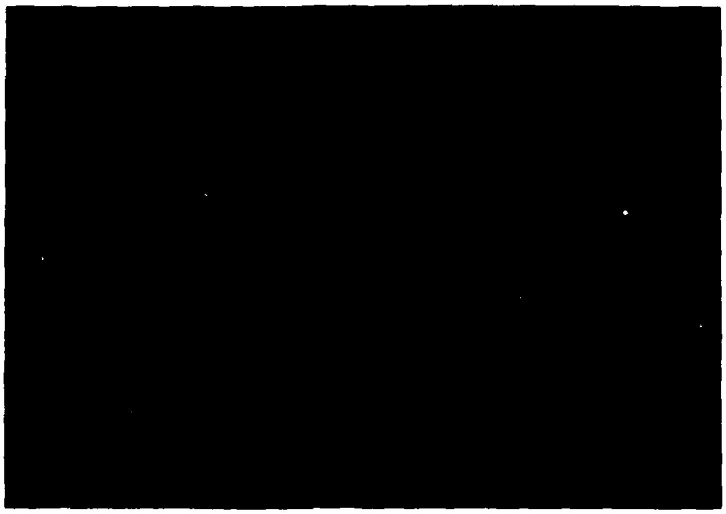


PHOTO NO. 13

Seawall 1824 - Settlement of top of concrete apron at vertical face of wall.

PHOTO NO. 14

Seawall 1824 - Typical elevation view of concrete apron toe.

an inspector standing at the toe of the apron and also shows a joint spall along a joint of the apron. A drainage trough was observed at station 48+75; the concrete trough was noted to have settlement cracks as shown in photo 16.

The concrete ramps bridging Seawall 1824 were also inspected. Two types of ramps were observed; pile supported and filled ramps. The ramps generally were noted to have transverse and longitudinal cracks in the surface of the ramp. Spalls, cracks and exposed reinforcement were commonly observed in the caps and superstructure of the ramps (see Photo 12).

Appendix A contains drawings illustrating in plan the stations assigned to this facility. Appendix B contains transcripts of the field observations made in conjunction with the facility inspection.

4.3.3 Structural Condition Assessment

The vertical face of Seawall 1824 appears to be in good condition except at the tongue-and-groove joints. These joints are commonly in poor condition where a proper seal is lacking in the seawall. Although the concrete apron appears to have stabilized the berm in front of the vertical wall, the apron appears in predominantly poor condition. In spite of it's poor condition, the concrete apron does not appear to be in any eminent danger of failure in it's function of providing protection to the vertical wall.

The ramps appear in poor to good condition but do not appear to be in danger of collapse. The ramps appear stable under dead loads only. Detailed structural analysis or load rating is beyond the scope of this report.

4.3.4 Recommendations

The tongue-and-groove joints of Seawall 1824 should be cleaned of the deteriorated seal material and resealed with a non-shrinking, non-oxidizing permanent elastic sealant. With this repair the expected life of the seawall is 25 years. Without these repairs the expected life is less than 5 years - during which time the pavement is expected to fail due to loss of material from behind the wall. The estimated cost of the joint (seal) repairs is $7,500 (see Appendix D).

Although the concrete apron is in poor condition it does not appear cost-beneficial to make any repairs at this time. The apron should be repaired at such time that it no longer provides the toe protection required for the stabilization of the vertical wall.

Prior to the use of the ramps bridging Seawall 1824, a detailed structural analysis and load rating should be performed. Depending on the expected loads, repairs may be necessary. The

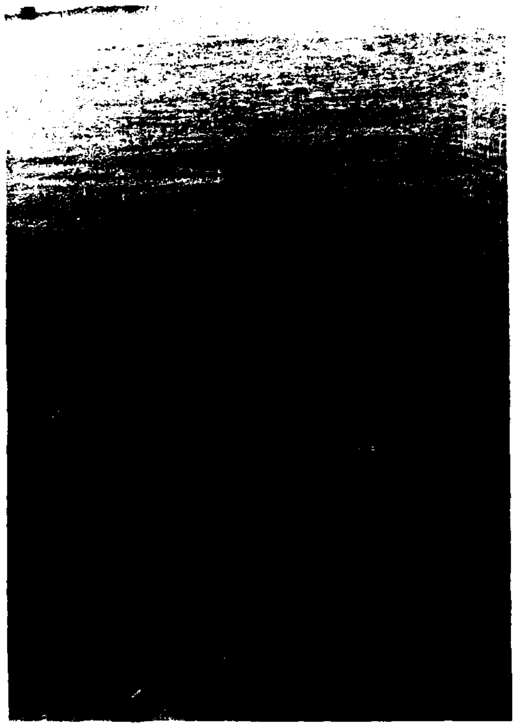


PHOTO NO. 15

Seawall 1824 - Joint spall at station 4+00
and perspective of concrete apron.

PHOTO NO. 16

Seawall 1824 - Drainage trough at station 48+75.

ramps do not appear to warrant repair if they are not used or are not planned for future use. However, if no repairs are made then further deterioration may be expected. To deter the deterioration of the reinforced concrete by corrosion of reinforcement, the ramps should be restored. This restoration should include the removal (by chipping away) of loose and broken concrete, sandblasting and coating (with epoxy) the corroded reinforcement and then restoring the original member dimensions with gunite or formed and poured concrete. This work is estimated to cost $50,000 per ramp. Without this work the average expected life of the ramps is 5 years. With completion of the above restoration the expected life of the ramps is 25 years. This repair work may be considered elective in terms of the expected use of the ramps.

Both the ramps and seawall should be inspected every 4 years. The pavement behind the seawall should be inspected annually to check for signs of settlement.

4.4 PIER 3238

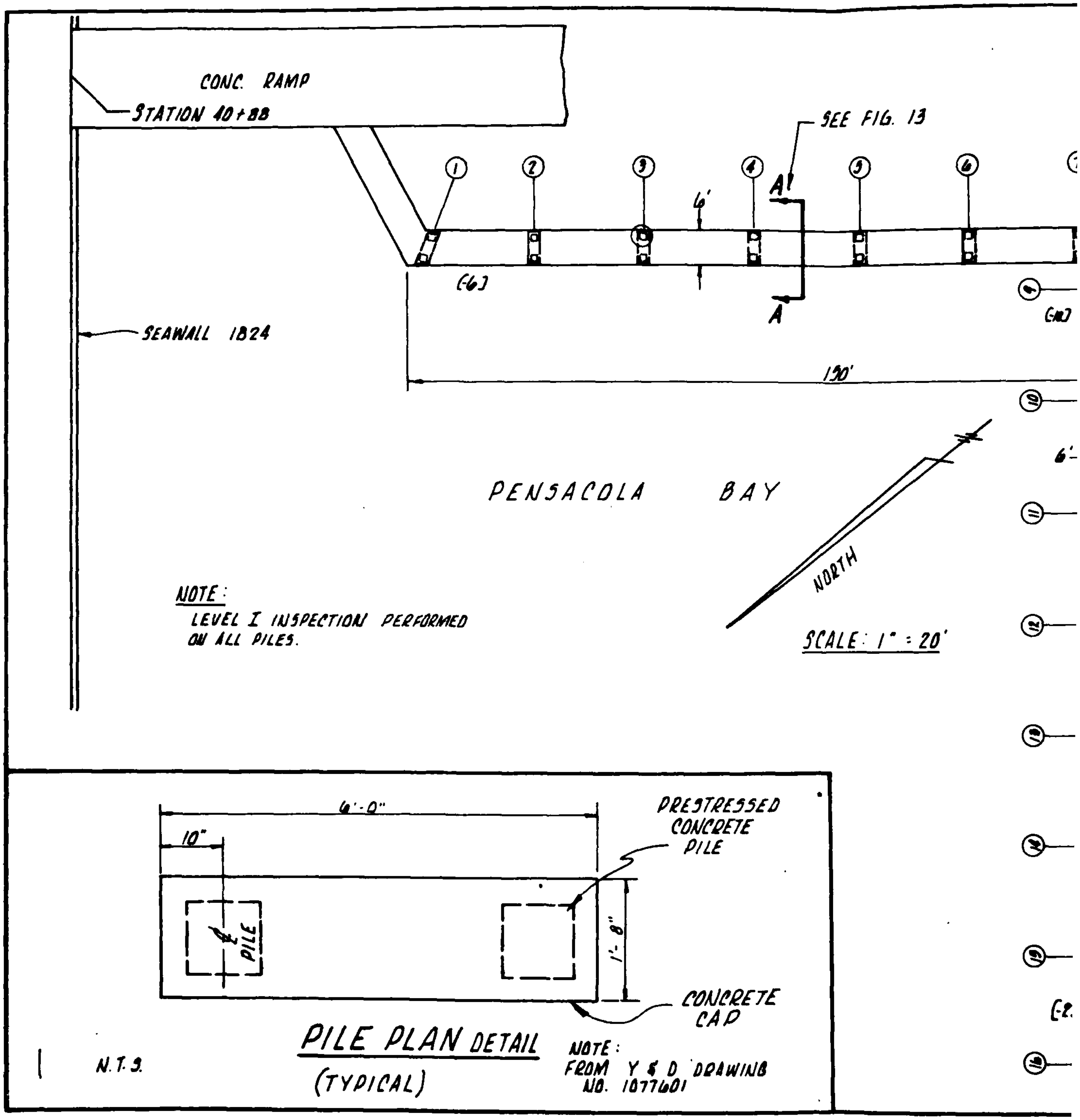
4.4.1 Description

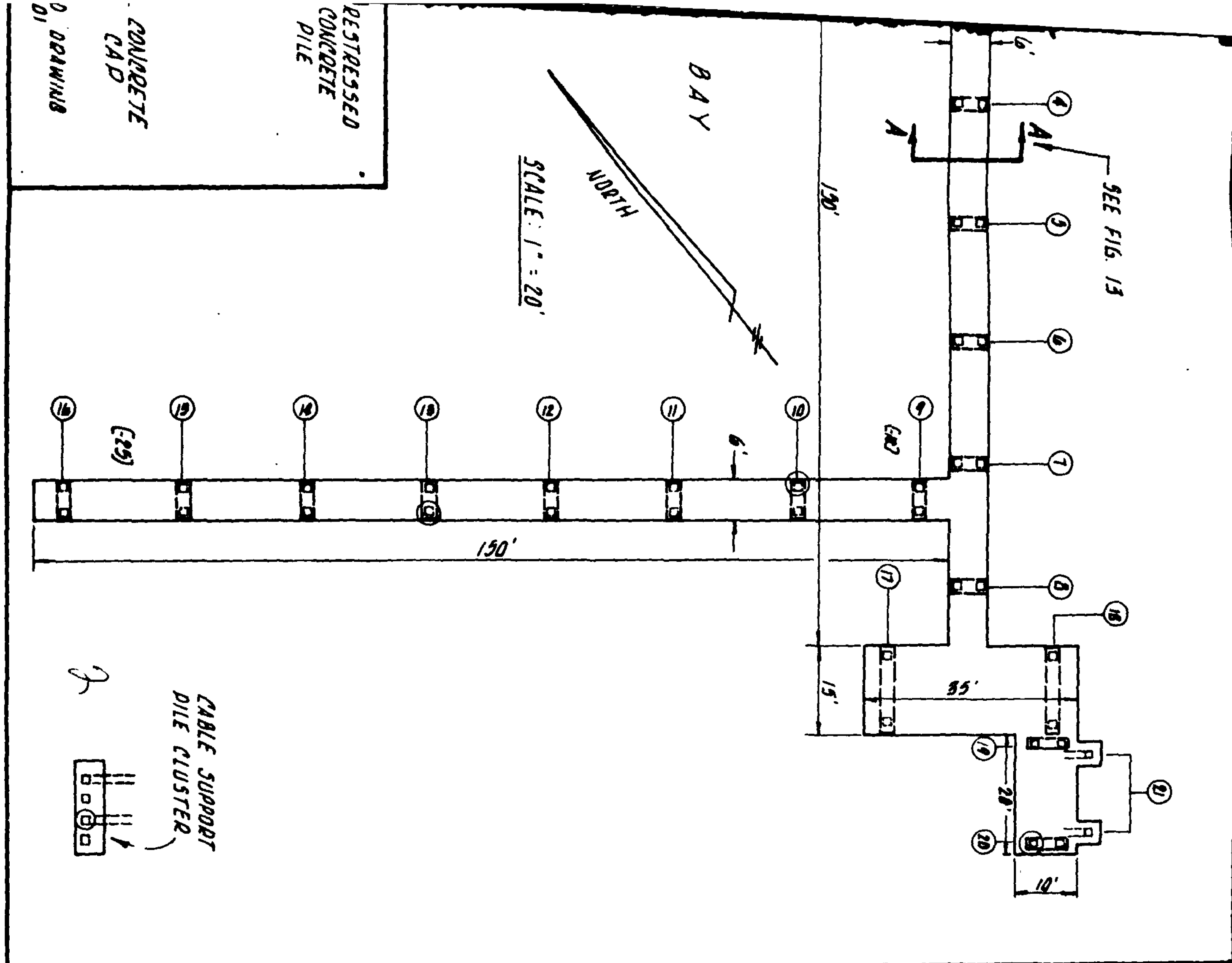
Pier 3238 serves as the sea survival training pier at the Pensacola Naval Station; the pier was constructed in 1965. The pier consists of a 6' wide catwalk supported by 12" prestressed concrete piles and a platform used for training purposes supported by 16" square prestressed concrete piles. Figure 12 illustrates the general layout of the pier. Figure 13 illustrates a typical section at a catwalk bent. The superstructure of the catwalk is comprised of prestressed concrete double-tee units as shown in Figure 13. The platform deck is comprised of prestressed concrete double-tee units supported by bents 17 and 18 and a poured in place deck supported by bents 19,20 and 21. A second level exists above the lower platform over bents 19, 20 and 21. A cable extends from a frame above the second level to a pile cluster to the west (see Figure 12). Following the passage of Hurricane Frederick in 1979, repairs were made to the existing catwalk and platform (see Y+D drawing 1077601).

4.4.2 Observed Inspection Conditions

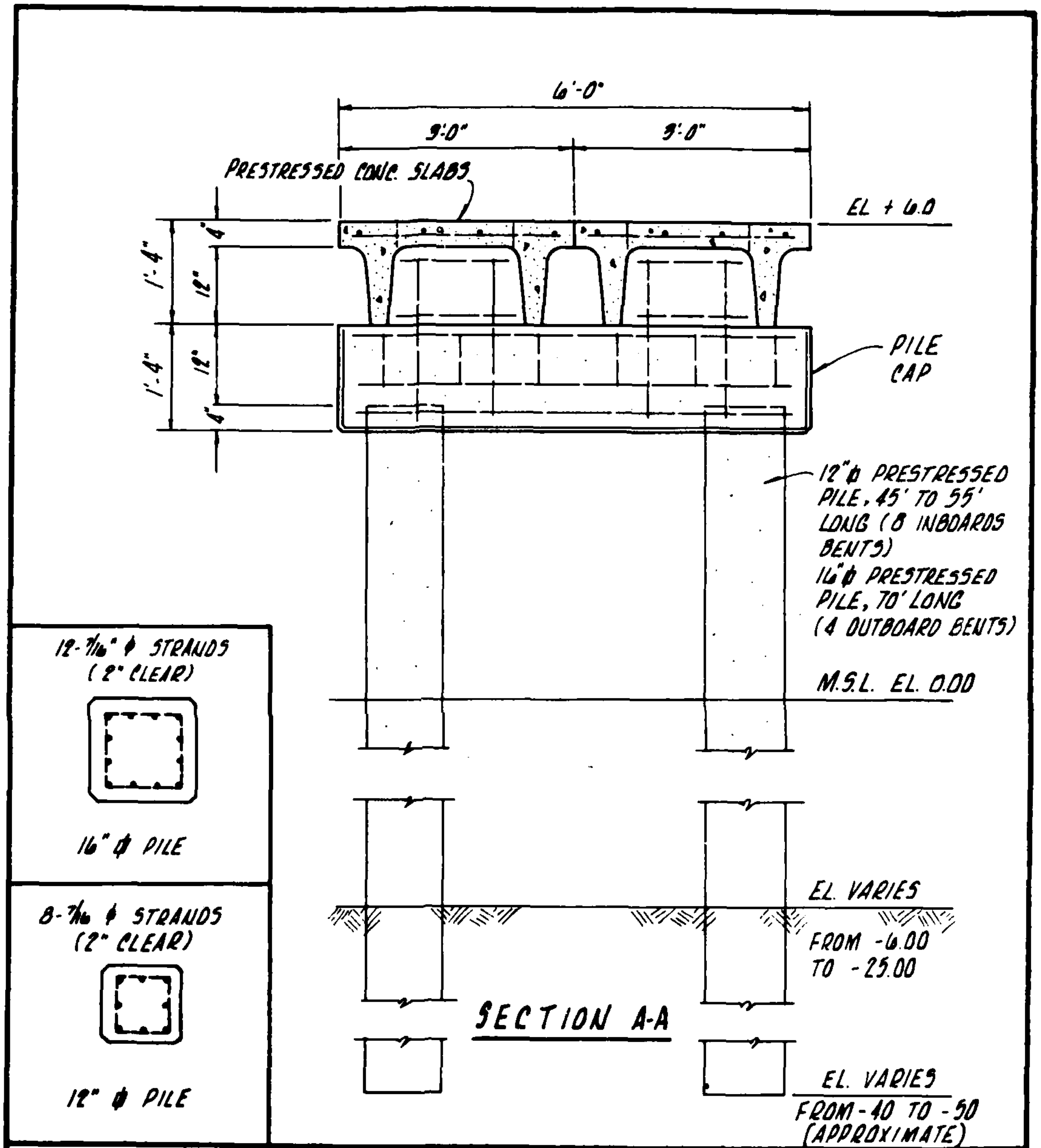
All the piles supporting the catwalk, platform and cable of Pier 3238 appear to be in excellent condition. No significant defects were noted in the piles supporting this pier. Marine growth on the piles was limited to green algae and barnacles typically less than ½" thick. The pile caps and deck units of the pier were noted to have a significant amount of spalling in some areas. In particular, the east-west section of the pier comprised of bents 9 through 16 was noted to have the most significant amount of spalling on the pile caps. Photo 17 shows a perspective view of the platform for sea survival training exercises. At bents 15 and 16 the caps were severely spalled to the extent that the strands of the piles at these bents were exposed and corroded. Photo 18 shows a spall in the edge of the pile cap (bent 9) and also shows rust stains and spalls in the deck unit spanning bent 9. The rust stains shown in photo 18 are typical of those observed in the deck units and caps, except for bents 15 and 16, which appeared in worse condition. Photos 19 and 20 show an uncleaned 12" pile and cleaned 12" pile typical of those supporting Pier 3238. Some deterioration (corrosion, cracks and spalls) was noted in the double-tee units, the platform slab and platform frame.

Appendix A contains drawings illustrating in plan the stations assigned to this facility. Appendix B contains transcripts of the field observations made in conjunction with the facility inspection.


CONC. RAMP
STATION 40+88
SEE FIG. 13
1
2
3
4
5
6
A
A
6'
(-6)
SEAWALL 1824
150'
PENSACOLA BAY
NORTH
NOTE:
LEVEL I INSPECTION PERFORMED ON ALL PILES.
SCALE: 1" = 20'
6'-0"
10"
PRESTRESSED CONCRETE PILE
℄ PILE
1'-8"
CONCRETE CAP
PILE PLAN DETAIL
(TYPICAL)
N.T.S.
NOTE:
FROM Y & D DRAWING NO. 1077601



□ INDICATES CONCRETE PILE □=== INDICATES CONCRETE BATTER PILE (-18) INDICATES APPROXIMATE WATER DEPTH ⊡ INDICATES LEVEL II INSPECTION PERFORMED ON PILE ⑧ INDICATES BENT NUMBER	PREPARED BY ARTHUR V. STROCK & ASSOCIATES, INC. 829 S.E. 9th ST. DEERFIELD BEACH, FLORIDA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C. PIER 3238 PILE PLAN	FIGURE 12



FROM Y & D DRAWING NO. 1077601	PREPARED BY ARTHUR V. STROCK & ASSOCIATES, INC. 829 S.E. 9th ST. DEERFIELD BEACH, FLORIDA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C.	
		PIER 3238 TYPICAL SECTION	FIGURE 13

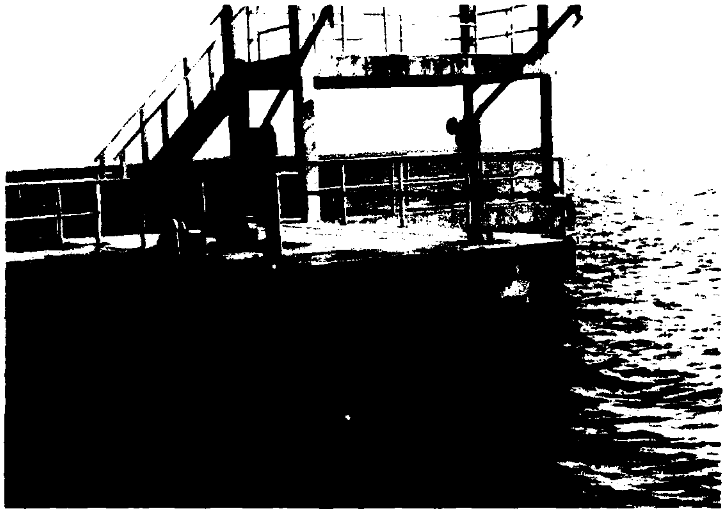


PHOTO NO. 17

Pier 3238 - Perspective view of platform for sea survival training exercises.

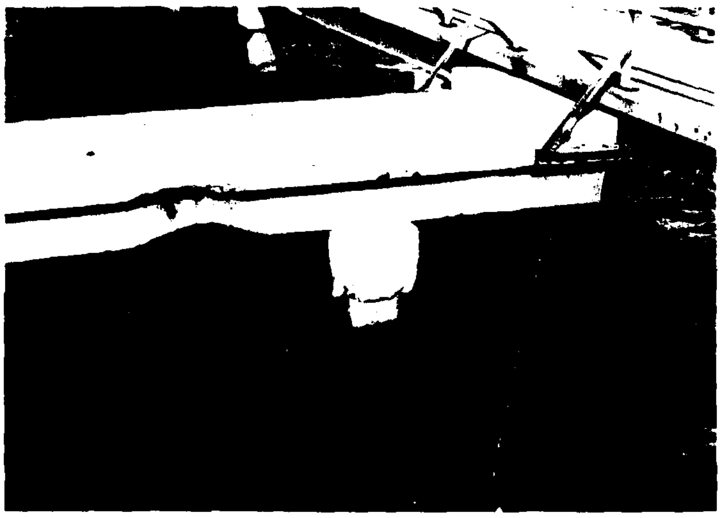


PHOTO NO. 18

Pier 3238 - Spalled edge of pile cap and spalled deck unit at Bent 9.

PHOTO NO. 19

Pier 3238 - Typical uncleaned pile supporting catwalk.

PHOTO NO. 20

Pier 3238 - Typical cleaned pile supporting catwalk.

4.4.3 Structural Condition Assessment

The piles supporting Pier 3238 appeared in excellent condition with no apparent loss of strength or significant deterioration. The piles appear capable of providing the necessary support to the catwalk and platform pedestrian loads (see Appendix C). The pile caps appeared in poor to fair condition. The pile caps appear in various stages of reinforcement corrosion. It appears that chlorides (sea water) have penetrated the concrete and corroded the steel. The corrosion causes expansion of the steel and concrete cover which results in the appearance of cracks and spalls.

4.4.4 Recommendations

The pile caps should be restored. The restoration should include removal of (by chipping away) loose and broken concrete, sandblasting and coating (with epoxy) the corroded reinforcement steel and then restoration of the cap to its original dimensions with gunite or formed and poured concrete. The estimated cost of these repairs is $15,500.

With the repairs the expected life of the pier is 20 years. This is based on the assumption that the superstructure is also properly maintained. Without these repairs the expected life of the pier is less than 5 years, during which time some pile caps are expected to further deteriorate and fail.

The pier piling should be inspected every 4 years. The pile caps, deck units and platform should be inspected by boat annually until such time that suitable repairs are made.

4.5 BULKHEADS 177 & 178

4.5.1 Description

Bulkheads 177 and 178 are gravity structures comprised of granite blocks as shown in Figure 14. These bulkheads were originally constructed in 1852. Bulkhead 177 extends from station 58+12 to station 71+28 and surrounds a small craft berthing basin used typically for tugs and service vessels. Bulkhead 178 extends from station 71+28 to a point approximately 160' under the platform of Pier 303 (see Plan View of Pier 303 which shows the portion of Bulkhead 178 under the platform of Pier 303). According to Navy drawings, the granite gravity structure is supported on a concrete foundation and sheet pilings as shown in Figure 14.

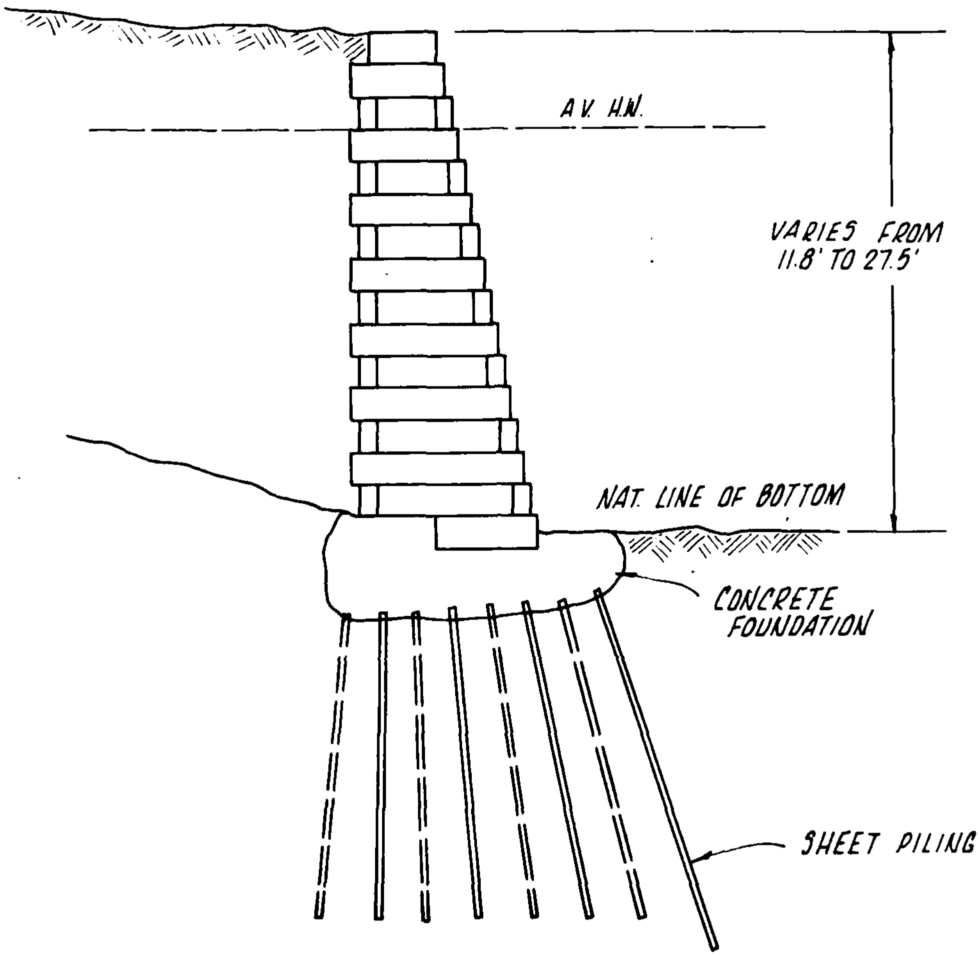
4.5.2 Observed Inspection Conditions

Bulkheads 177 and 178 appear to be in overall good condition. The granite blocks appear to be very stable however, several blocks were noted to be missing from the wetface of the wall. Gaps between blocks and loss of mortar were common conditions noted by the inspectors. Bulkhead 177 appeared to have a greater amount of gaps and missing mortar than Bulkhead 178. Marine growth consisted principally of barnacles and algae with some hydroids. A collapse of the asphalt pavement behind Bulkhead 178 at station 71+50 was observed and is shown in photograph 21. While at the Naval Station a significant amount of rainfall run-off was noted to seep into the low area caused by the collapsed asphalt pavement. Photographs 22 and 23 show perspective views of the wetface of Bulkheads 177 and 178 above the waterline. At the eastern terminus of Bulkhead 178 (station 76+60) a large cavity was noted in the face of the granite bulkhead just below the concrete encasement of the steel sheet pile section of Pier 303, beneath the platform. Figure 15 shows details of the cavity observed at station 76+60.

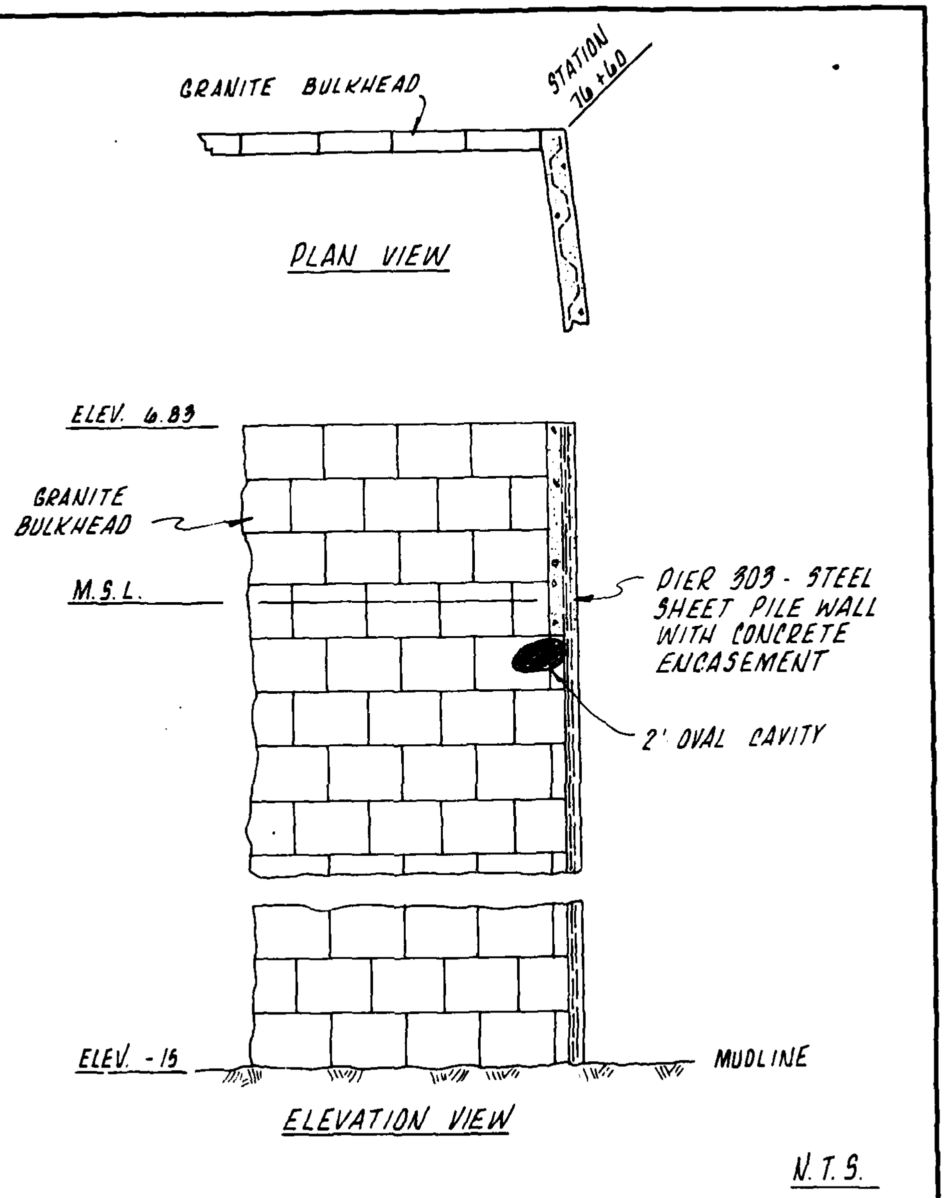
Appendix A contains drawings illustrating in plan the stations assigned to this facility. Appendix B contains transcripts of the field observations made in conjunction with the facility inspection.

4.5.3 Structural Condition Assessment

Although the granite blocks appear stable the bulkhead appears to be allowing material to seep through the gaps (where blocks are missing) and seams (joints) between blocks. Inadequate structural details of the bulkhead exist to perform a meaningful evaluation of the bulkhead. However, it appears that the structure is stable for the existing basin depths.



FROM Y & D DRAWING NO. 2984	PREPARED BY ARTHUR V. STROCK & ASSOCIATES, INC. 829 S.E. 9th ST. DEERFIELD BEACH, FLORIDA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C. BULKHEAD 177, 178 TYPICAL SECTION FIGURE 14



NOTE :
PILES OF PIER 303 PLATFORM ARE NOT SHOWN.

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BULKHEAD 178
CAVITY @ STATION 76+60

FIGURE 15

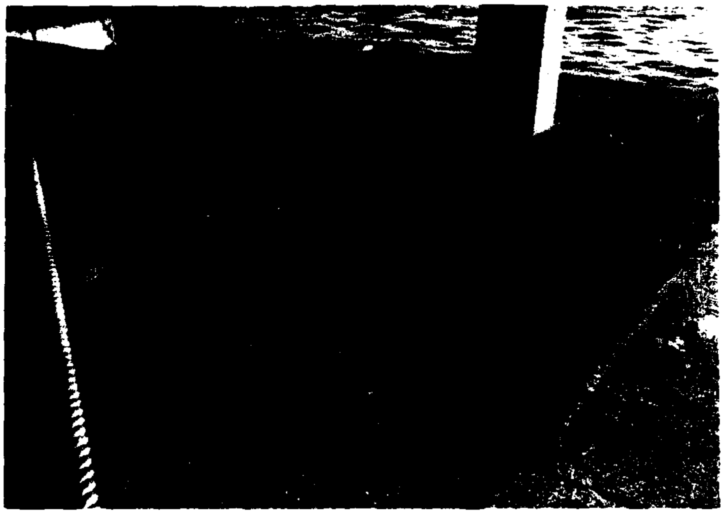


PHOTO NO. 21

Bulkheads 177,178 - Collapsed asphalt pavement at station 71+50.

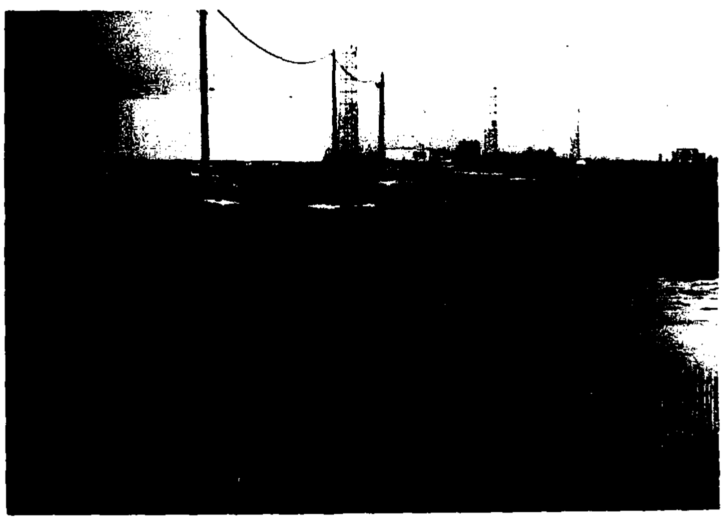


PHOTO NO. 22

Bulkheads 177,178 - View of wetface of bulkhead above waterline opposite Finger Pier 303.

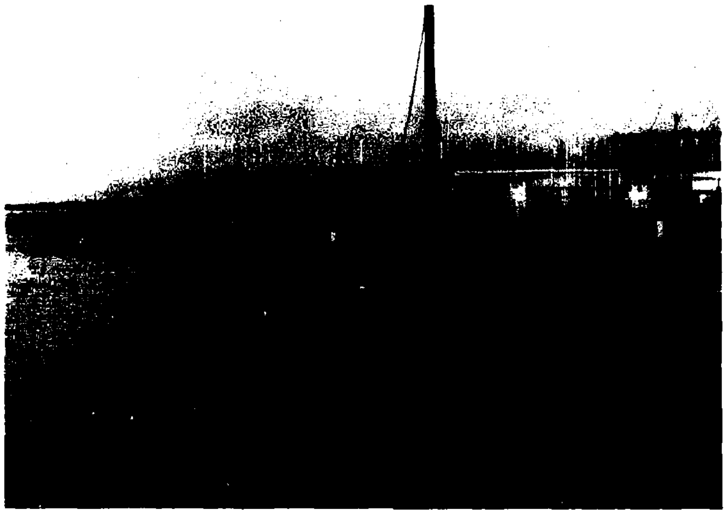


PHOTO NO. 23

Bulkheads 177,178 - View of wetface of bulkheads above waterline near entrance to small craft basin.

4.5.4 Recommendations

The bulkhead should be sealed by pressure sealing of the granite block joints with a non-shrinking, non-oxidizing elastic sealant. The gaps posed by missing blocks in the face of the bulkhead should be filled with concrete by forming the face and pumping concrete into the form. The estimated cost of these repairs is $90,000. With these repairs the expected life of the bulkhead is at least 100 years. Without these repairs the granite blocks will probably remain stable but the bulkheads will further lose their ability to retain material.

Furthermore, repairs to the asphalt pavement on the upland should be made. In conjunction with these repairs filter cloth should be placed against the granite blocks if they can be exposed without unreasonable excavation. The drainage on the upland of the bulkheads should be evaluated and redirected so that run-off does not penetrate upland grades and wash material through the bulkhead.

The bulkheads should be inspected every 4 years. The upland pavement should be inspected annually.

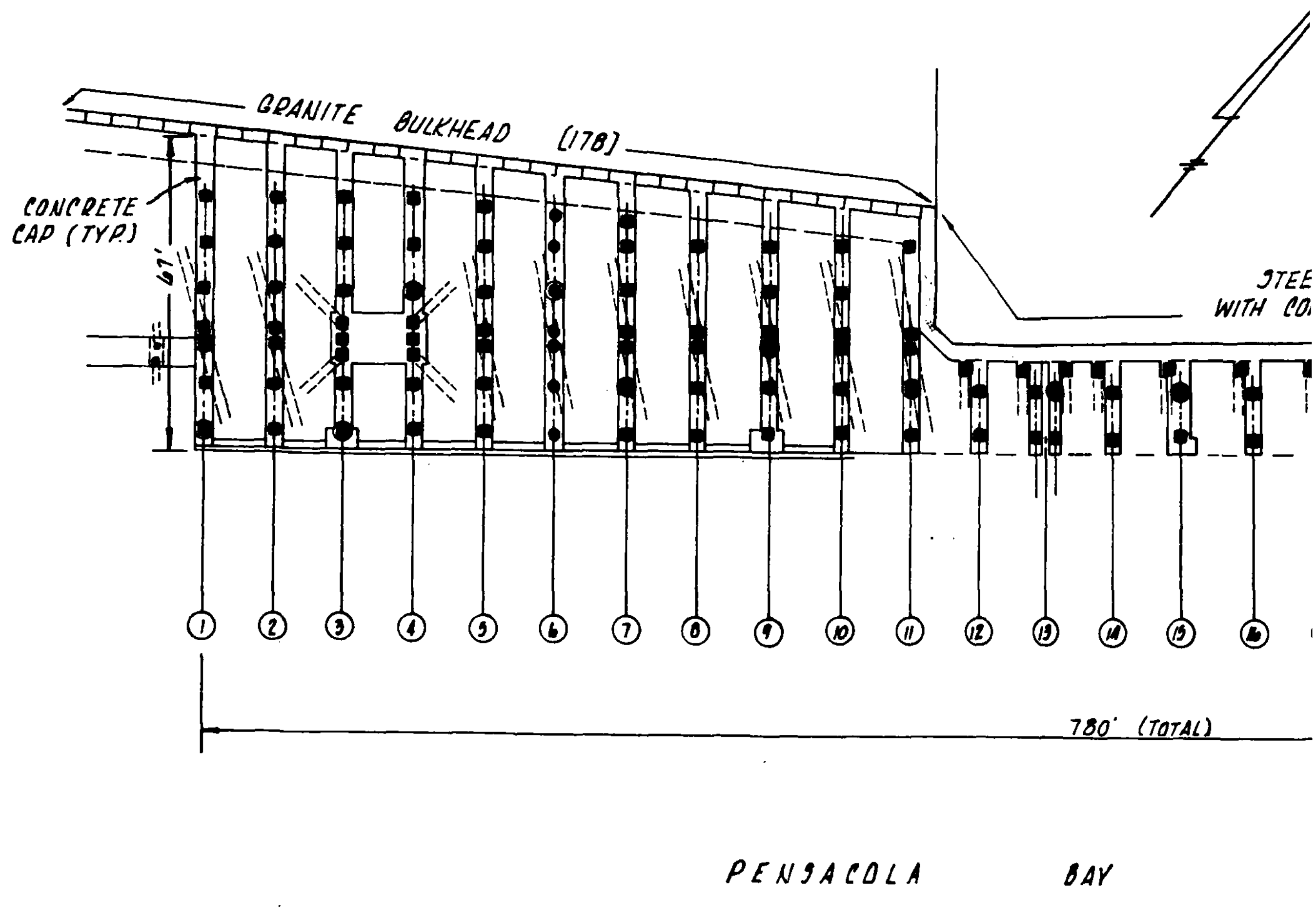
4.6 PIER 303

4.6.1 Description

Pier 303 is comprised of three sections. The first section of Pier 303 is a platform supported by 206-18" square prestressed concrete piles. the platform abuts 160' of the granite Bulkhead 178 and 660' of a steel sheet pile bulkhead with a concrete encasement. The steel sheet pile bulkhead with encasement is considered the second section of Pier 303. The third section of Pier 303 is a new steel sheet pile bulkhead with concrete cap (stations 83+50 to 91+28). This new steel sheet pile bulkhead was constructed following damage to the old structure when Hurricane Frederick passed the area in 1979. The original bulkhead fronting the existing platform was a concrete gravity type structure. In approximately 1940, the steel sheet pile bulkhead was constructed in front of the old gravity structure. During the early 1960's the existing poured in place concrete platform supported by prestressed concrete piles was constructed. Figure 16A and 16B illustrate the pile plan of the Pier 303 platform section. These Figures also illustrate the relative positions of Bulkhead 178, Finger Pier 303 and the steel sheet pile bulkhead with concrete encasement (section 2 of Pier 303). Figure 17 illustrates a typical section through the platform and adjacent structures. An aircraft carrier berths against the platform section of Pier 303. Two camels are positioned between the hull of the carrier and the platform; these camels abut a fendering system against the platform. This fendering system is two separate steel I-beam frames supported by bearing plates and rubber cushions.

4.6.2 Observed Inspection Conditions

The bulkheads of Pier 303 were inspected on a Level I, II and III basis. The prestressed concrete piles supporting the platform of Pier 303 were inspected on a Level I and II basis. The steel I-beam frame fendering system was inspected on a Level I basis only. The prestressed concrete piles supporting the platform of Finger Pier 303 appeared to be in excellent condition. No evidence of significant deterioration was noted by the inspectors. Some minor spalls of the pile caps and bottom of the platform deck were noted. Photos 24 through 28 show typical conditions observed at the platform section of Pier 303. The steel frame fendering system of Pier 303 appeared to be in overall excellent condition but the supports of the westernmost steel frame evidenced a longitudinal displacement of the frame parallel to the face of the pier. Photo 29 is a perspective view of the steel frame fendering system which is connected to the top of the platform and at a point just above the waterline. One type support is a compressive rubber cushion whereas another type support consists of a bearing plate, see photo 30. A longitudinal displacement of the



SCALE : 1" = 30'

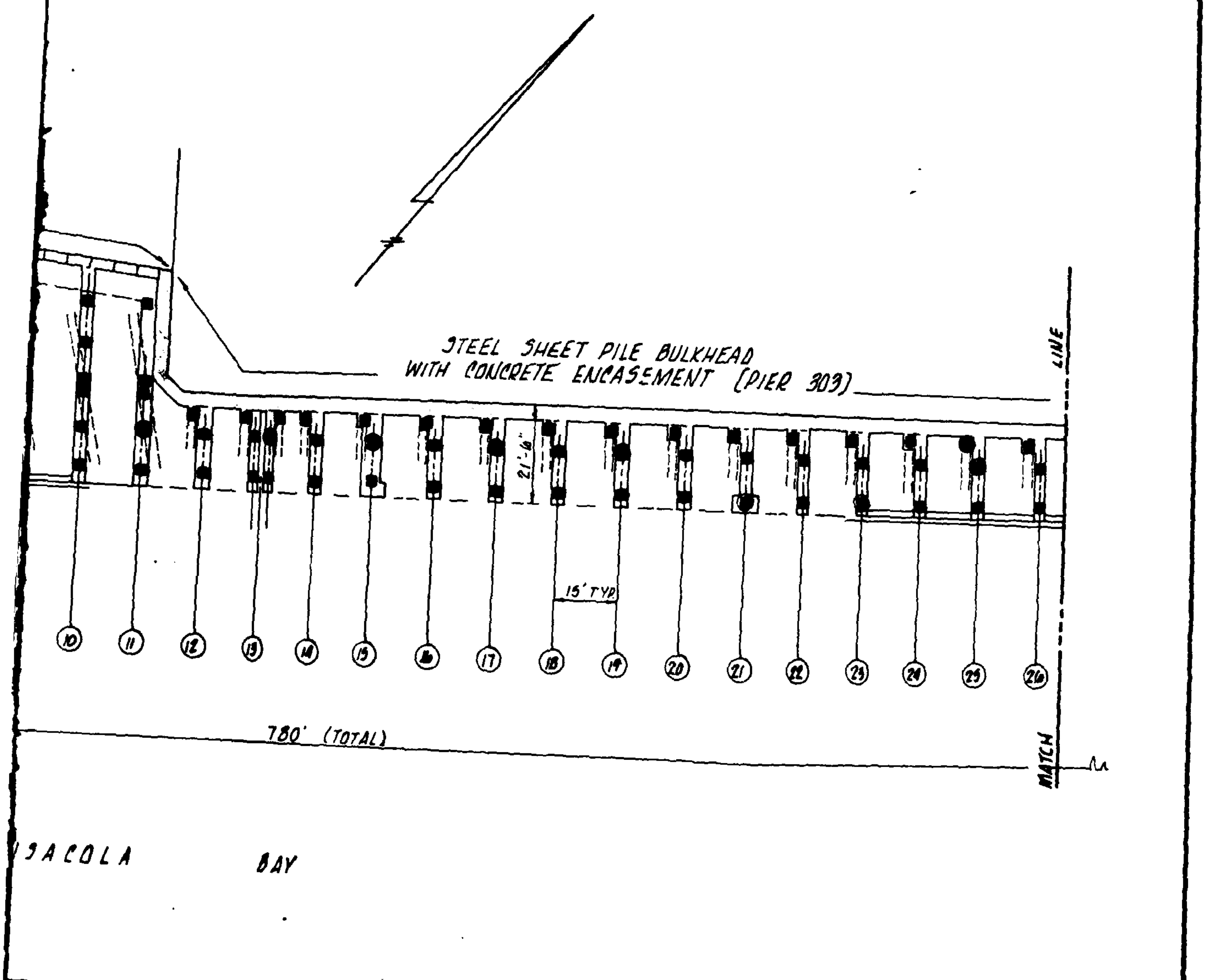
LEGEND

- ① INDICATES BENT ONE OF PIER 303 PLATFORM
- ▪ INDICATES VERTICAL PILE OF PIER 303 PLATFORM
- ● INDICATES LEVEL II INSPECTION PERFORMED ON PILE
- INDICATES BATTER PILE OF PIER 303 PLATFORM

NOTE:
ALL PILES ARE 18" SQUARE PRESTRESSED CONCRETE PILES, LENGTH VARIES.

FROM Y & D

DRAWING NO. 83



S ARE 18" SQUARE
SSED CONCRETE PILES,
VARIES.

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DRAWING NO. 831940

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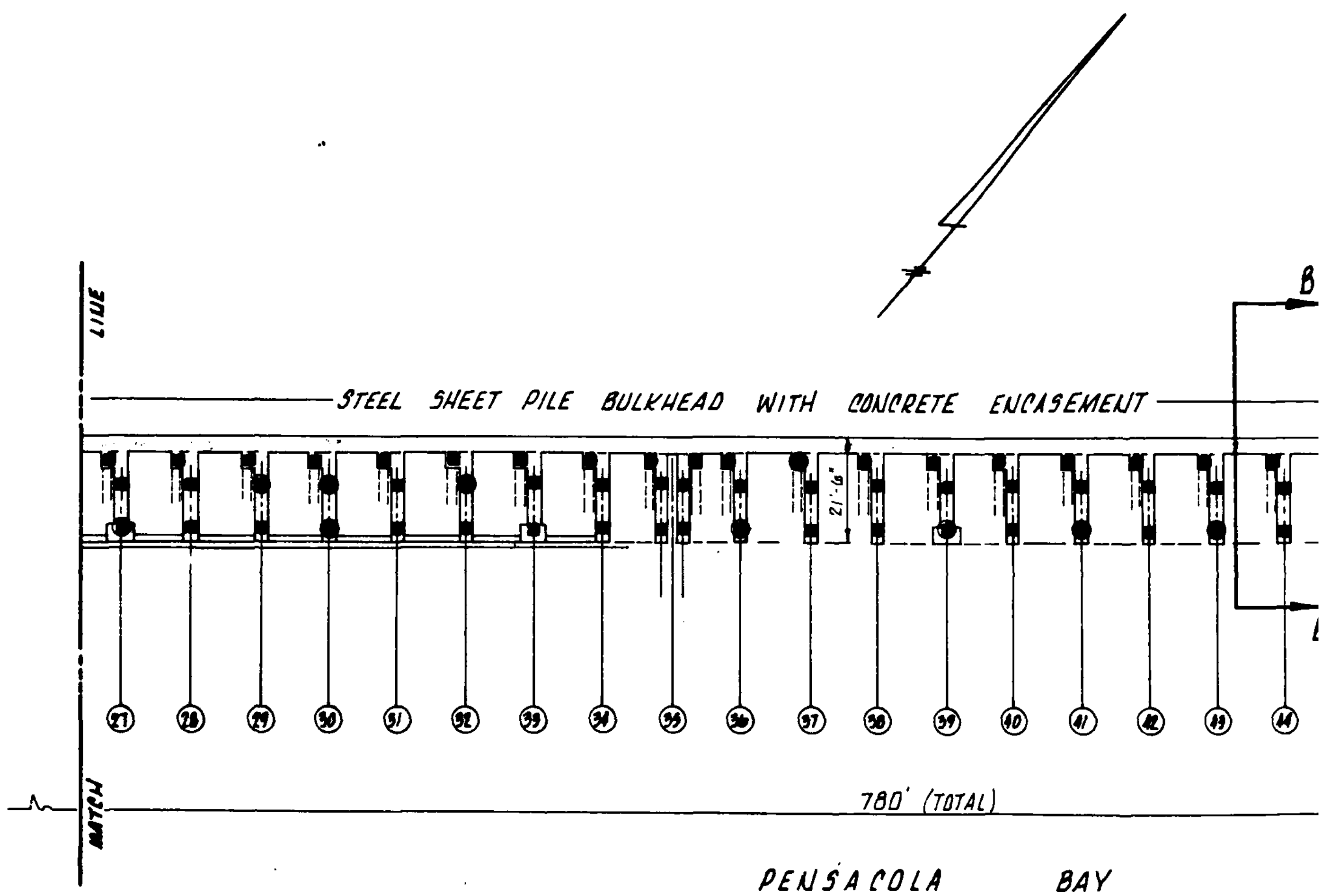
2 PIER 303
PILE PLAN

FIGURE 16 A



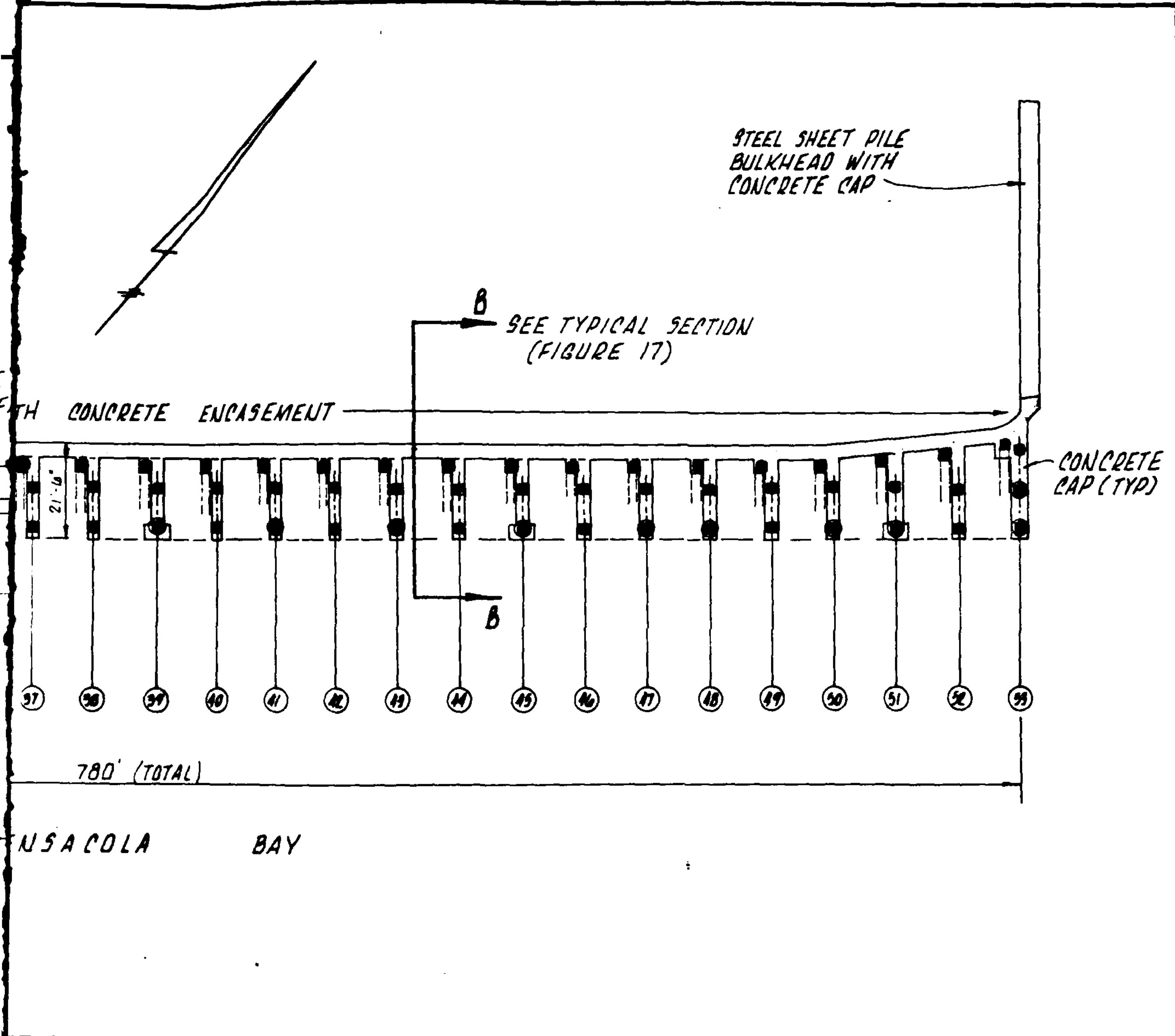
SCALE: 1" = 30'

LEGEND

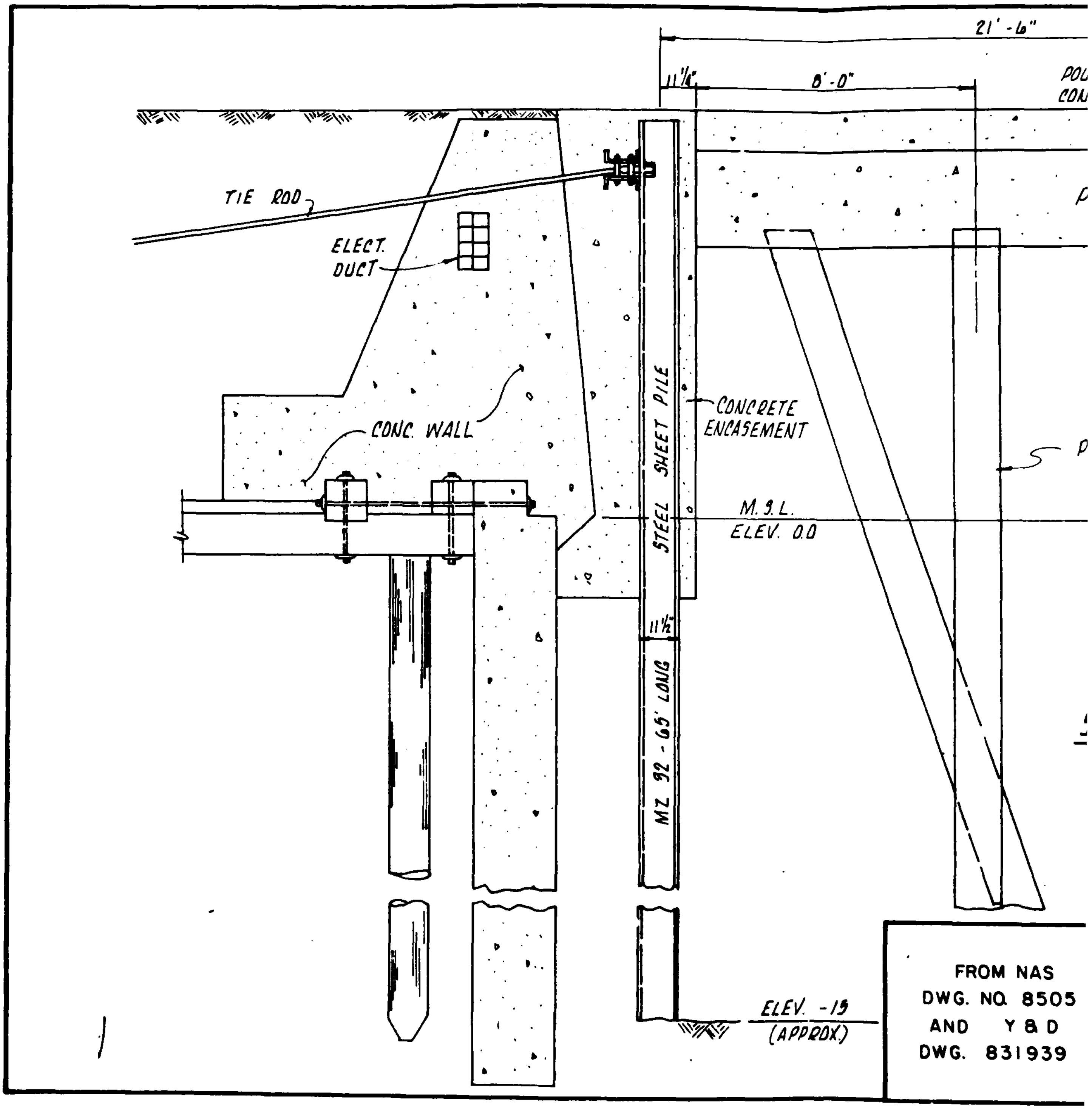
- (37) INDICATES BENT THIRTY-SEVEN OF PIER 303 PLATFORM
- ■ INDICATES VERTICAL PILE OF PIER 303 PLATFORM
- ● INDICATES LEVEL I INSPECTION PERFORMED ON PILE
- INDICATES BATTER PILE OF PIER 303 PLATFORM

NOTE:
ALL PILES ARE 18" SQUARE PRESTRESSED CONCRETE PILES; LENGTH VARIES

FROM Y & D
DRAWING NO. 8319



TE: L PILES ARE 18" UARE PRESTRESSED NCRETE PILES; ENGTH VARIES	FROM Y & D DRAWING NO. 831940	PREPARED BY ARTHUR V. STROCK & ASSOCIATES, INC. 829 S.E. 9th ST. DEERFIELD BEACH, FLORIDA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C. 2 PIER 303 PILE PLAN FIGURE 16B



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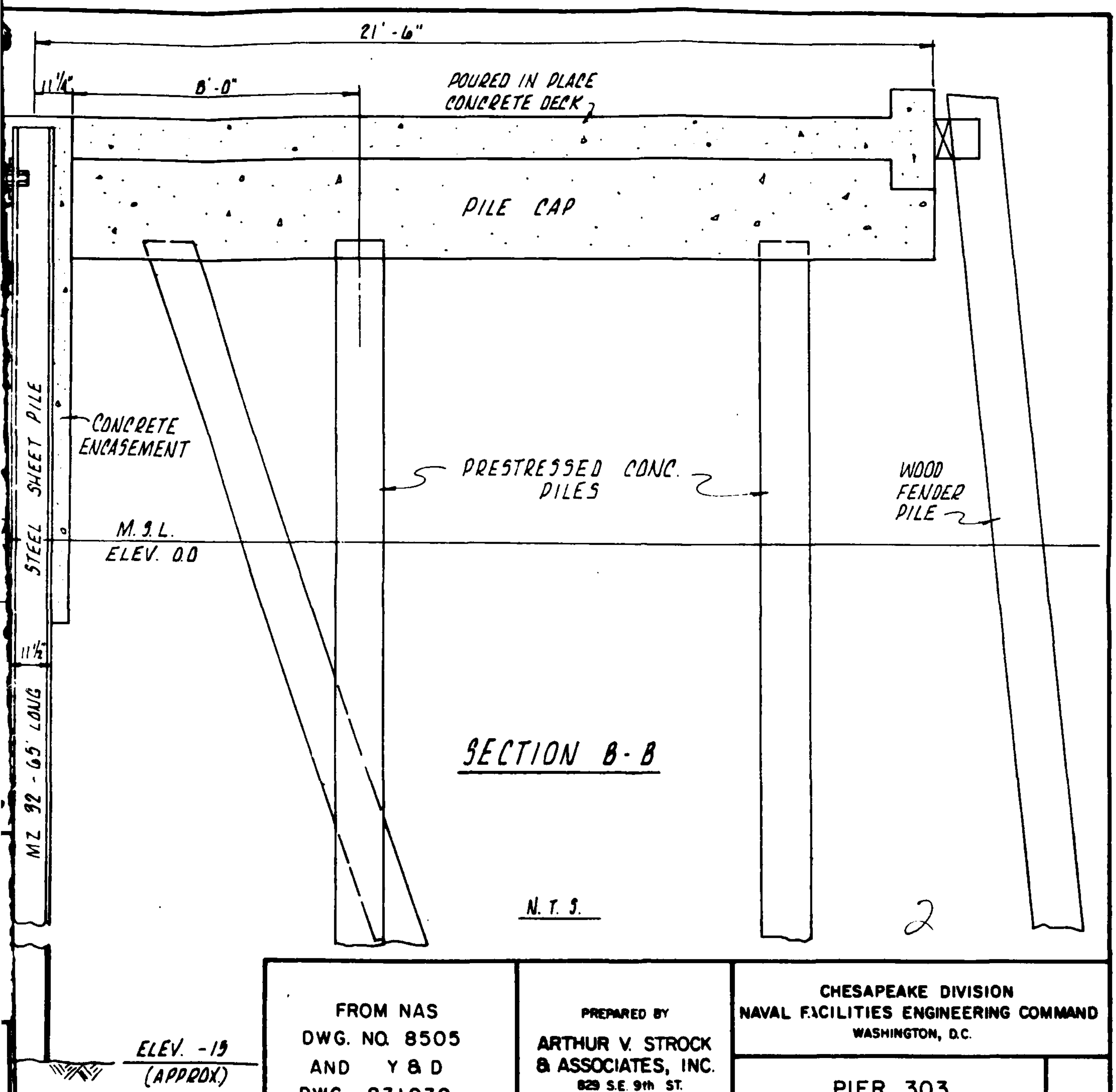
AND Y & D
DWG. 831939



FROM NAS DWG. NO. 8505 AND Y & D DWG. 831939	PREPARED BY ARTHUR V. STROCK & ASSOCIATES, INC. 829 S.E. 9th ST. DEERFIELD BEACH, FLORIDA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C.
		PIER 303 TYPICAL SECTION — FIGURE 17

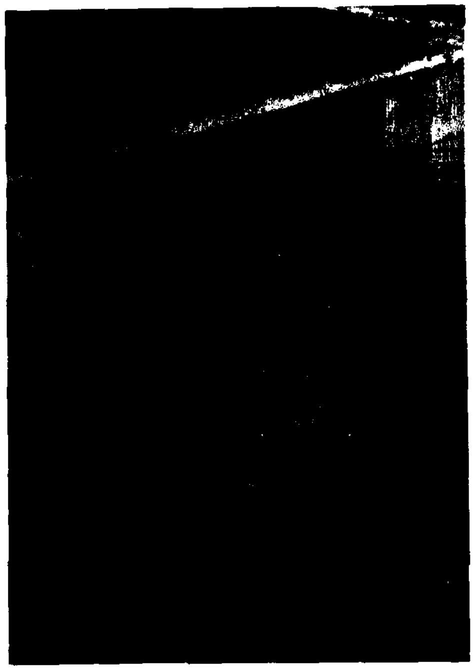


PHOTO NO. 24

Pier 303 - Cap at station 75+00 in good condition (typical).

PHOTO NO. 25

Pier 303 - Typical prestressed concreté pile in good condition - cleaned face below waterline.

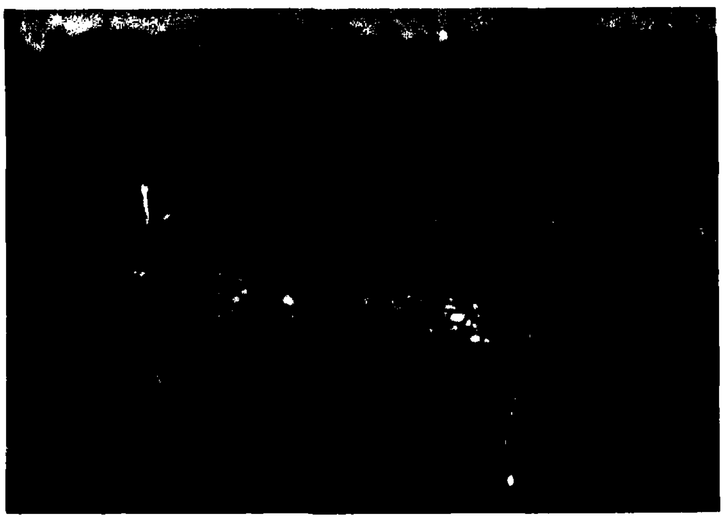


PHOTO NO. 26

Pier 303 - Minor spall and hairline crack at pile cap (vertical face), Bent 34.

PHOTO NO. 27

Pier 303 - Spalled concrete and exposed rebar in pile cap bent 38.

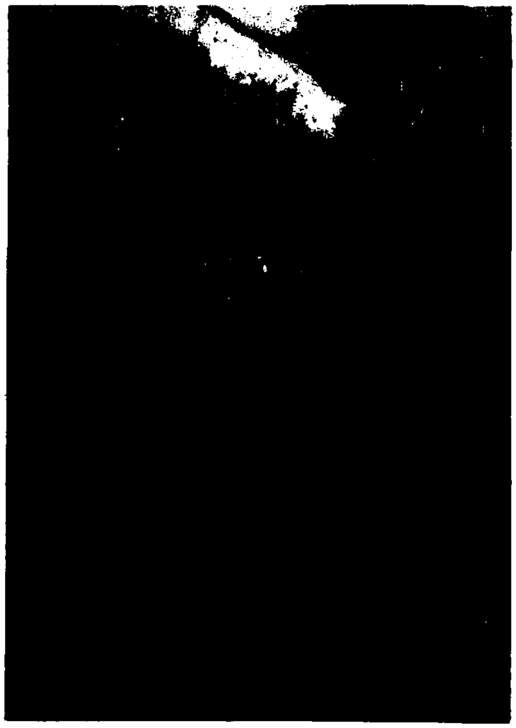


PHOTO NO. 28

Pier 303 - Typical surface spall and exposed reinforcement at bottom of platform deck.

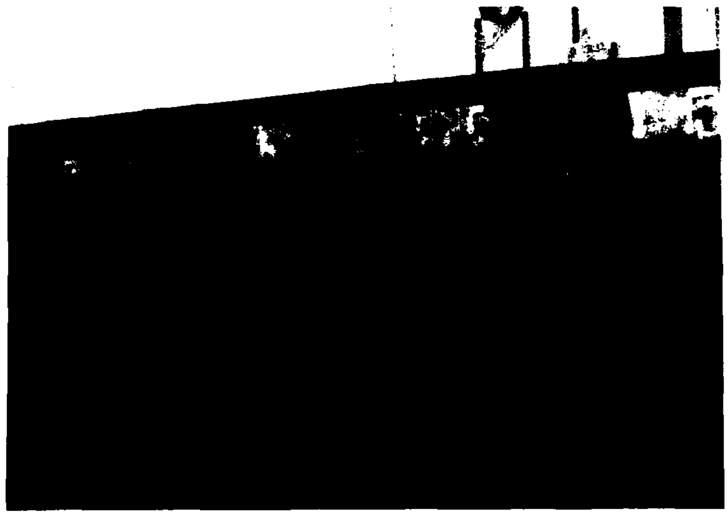


PHOTO NO. 29

Pier 303 - Perspective view of steel frame fendering system.

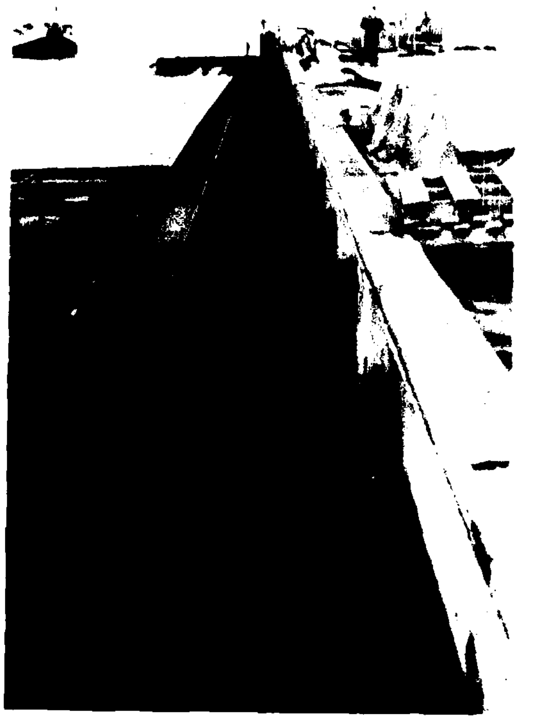


PHOTO NO. 30

Pier 303 - View of steel frame fendering system showing bearing plate and rubber cushions.

westernmost steel frame was evidenced at the top of the frame at supports as shown in photos 31 and 32. Photo 33 shows a horizontal member of the eastern steel frame where the waterward flange appears to have been crushed by impact of the frame with the splintered timber. Other than the longitudinal displacement of the westernmost fendering system and the dented system, the steel beam fendering system appeared to be in very good condition.

The steel sheet piles with concrete encasement abutting the platform of Pier 303 appeared in good condition. Heavy oyster growth was observed in some areas immediately below the concrete encasement on the steel sheet piles. Up to ½" of corrosion products were observed on the steel sheet piles on the surface areas below the concrete encasement. Table 2 contains the ultrasonic thickness testing results for Pier 303 sheet piles abutting the platform (stations 76+60 through 83+15). The average thickness of the flange for this section of the bulkhead was found to be .442 inches; whereas the average thickness of the web was found to be .391 inches. On the average, about 12% of the original thickness of the flange has been lost to corrosion; the average measured web thickness is greater than the manufacturers specified web thickness (see Appendix C). Some minor spalls in the bottom of the concrete encasement were noted along this section of Pier 303; photo 34 shows a typical spall at the bottom of the encasement. The steel sheets were observed to have a heavy oyster growth below the concrete encasement near the ends of the pier at stations 82+86 and 75+00. Near the bottom, marine growth was more commonly noted as algae, hydroids and tunicates. Photo 35 shows a sheet pile cleaned below the concrete encasement. Areas of surface corrosion were evidenced by the appearance of rust stains in patches among the marine growth, which predominantly covered the pile. At station 83+25 two holes were observed in the web of a steel sheet pile just below the encasement. A spall with exposed rebar in the encasement was noted at station 83+35. Photos 36 and 37 shows one of the holes and the spalled encasement respectively; Figure 18 illustrates these observations. Along the new section of the steel sheet pile, the steel sheets appear to be in very good condition. Table 2 shows the ultrasonic testing data for the measurements on the bulkheads of Pier 303 including the new steel sheet pile section which extends from stations 83+50 to 91+28. The sheets appeared in very good condition and only showed minor rust stains and shallow pitting at the surface of the steel sheets. Marine growth was found to consist of young oyster growth and barnacles just below the waterline and very thin algae near the mudline. Photos 38 and 39 show a cleaned new pile just below the waterline and also an uncleaned new pile at the same location.

Significant deficiencies of the new steel sheet pile section are holes in seven sheet piles just above the mudline. These 2" diameter holes were found in pairs in the web and flange of

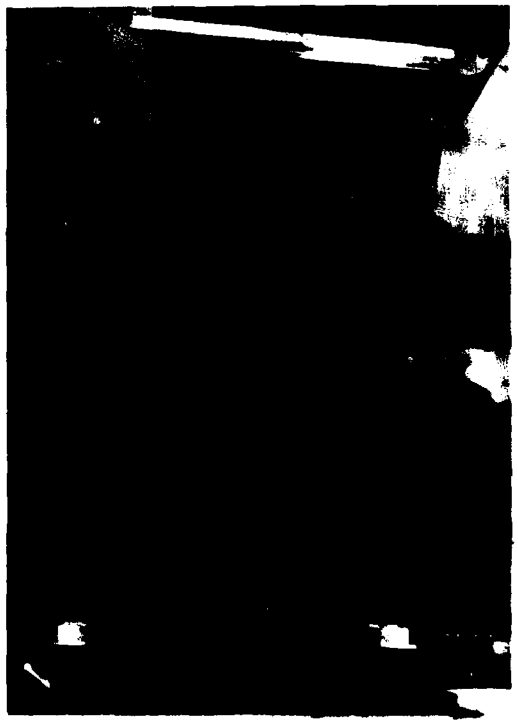


PHOTO NO. 31

Pier 303 - Plan view of rubber cushion showing displacement of steel frame parallel to Pier 303

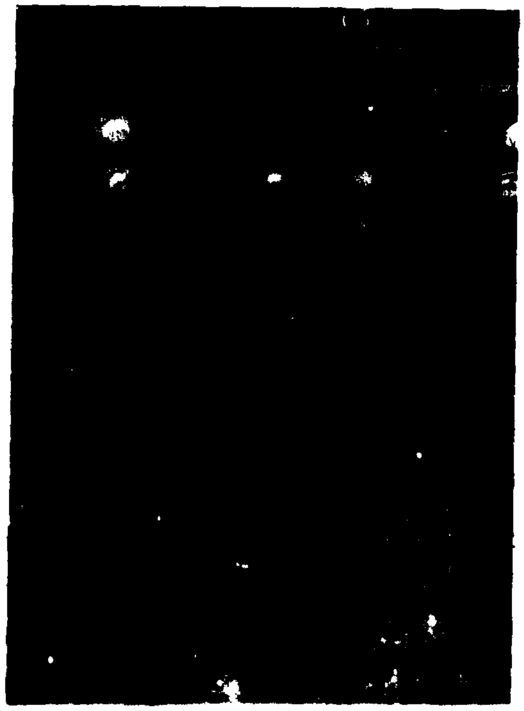


PHOTO NO. 32

Pier 303 - Plan view of bearing plate showing displacement of western steel frame parallel to Pier 303.

TABLE 2

ULTRASONIC TESTING RESULTS
FOR
PIER 303

STATION	LOCATION	FLANGE THICKNESS (inches)	WEB THICKNESS (inches)
91+00	Approximate Mean Low Water	.552	.396
91+00	Within 3' of mudline	.552	.404
89+50	Approximate Mean Low Water	.580	.408
89+50	Mid-point of wall	.579	.390
89+50	Within 3' of mudline	.531	.374
87+50	Approximate Mean Low Water	.534	.375
87+50	Approximate mid-point of wall	.536	.375
87+50	Within 3' of mudline	.590	.413
85+50	Approximate Mean Low Water	.556	.413
85+50	Approximate mid-point of wall	.538	.405
85+50	Within 3' of mudline	.573	.413
83+55	Approximate Mean Low Water	.578	.396
83+55	Approximate mid-point of wall	.544	.420
83+55	Within 3' of mudline	.515	.402
83+15	Within 2' of concrete encasement	.440	.390
83+15	Within 3' of mudline	.445	.467
82+40	Within 2' of concrete encasement	.357	.339
79+90	Within 2' of concrete encasement	.449	.365
76+60	Within 2' of concrete encasement	.413	.389
76+60	Within 3' of mudline	.429	.396

PHOTO NO. 33

Pier 303 - Damaged horizontal member of east steel frame fendering system just above waterline at camel.

PHOTO NO. 34

Pier 303 - Spall at bottom of concrete encasement over steel sheet piles.

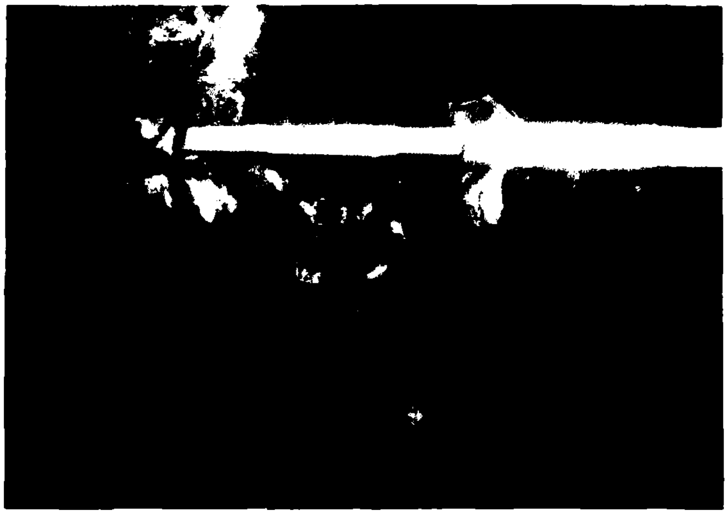


PHOTO NO. 35

Pier 303 - Cleaned pile just below concrete encasement (typical).

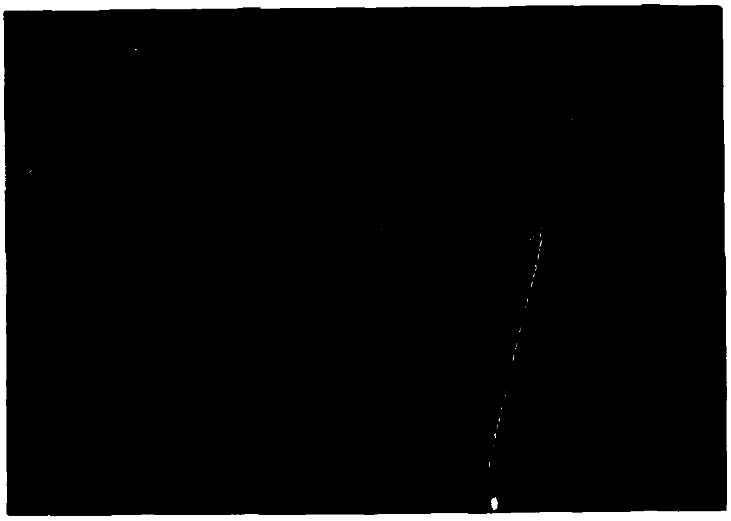


PHOTO NO. 36

Pier 303 - Hole in flange of steel sheet pile 3' below concrete encasement at station 83+25.

PHOTO NO. 37

Pier 303 - Exposed rebar and spall in concrete encasement at station 83+35.

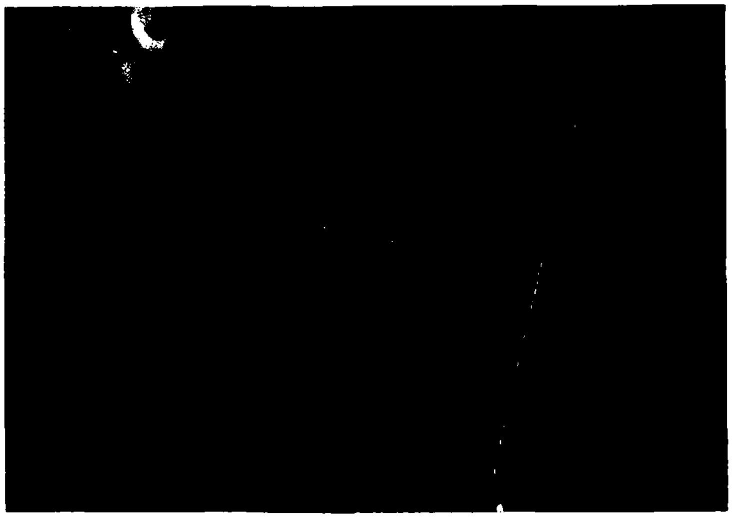
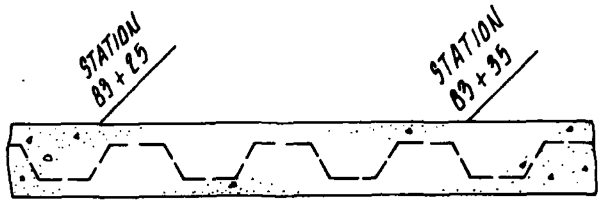
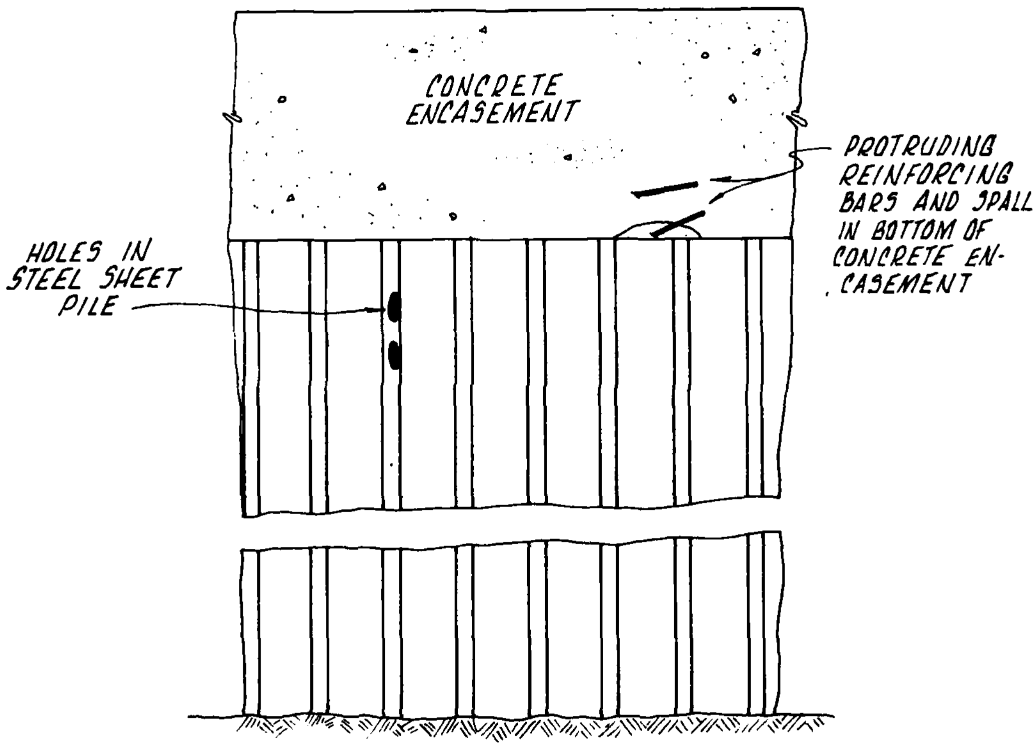


PHOTO NO. 38

Pier 303 - Cleaned steel sheet pile at just below Mean Low Water at station 90+90.



PLAN VIEW



ELEVATION VIEW

N.T.S.

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PHOTO NO. 39

Pier 303 - Uncleaned steel sheet pile
at just below Mean Low Water at station 90+90.

PHOTO NO. 40

Pier 303 - Two inch diameter holes in the
flange of steel sheet pile just above mud-
line, typical at stations 90+59 thru 90+71.

sheet piles between stations 90+59 through 90+71. Figure 19 illustrates the location of these holes. Photo 40 shows a typical pair of these holes which are located near the mud-line on each side of an apparent butt weld in the steel sheet. Following a rain shower, sediment was observed washing through several of these holes as shown in photo 41. No original specifications for these new sheet piles are avilable.

Appendix A contains drawings illustrating in plan the stations assigned to this facility. Appendix B contains transcripts of the field observations made in conjunction with the facility inspection.

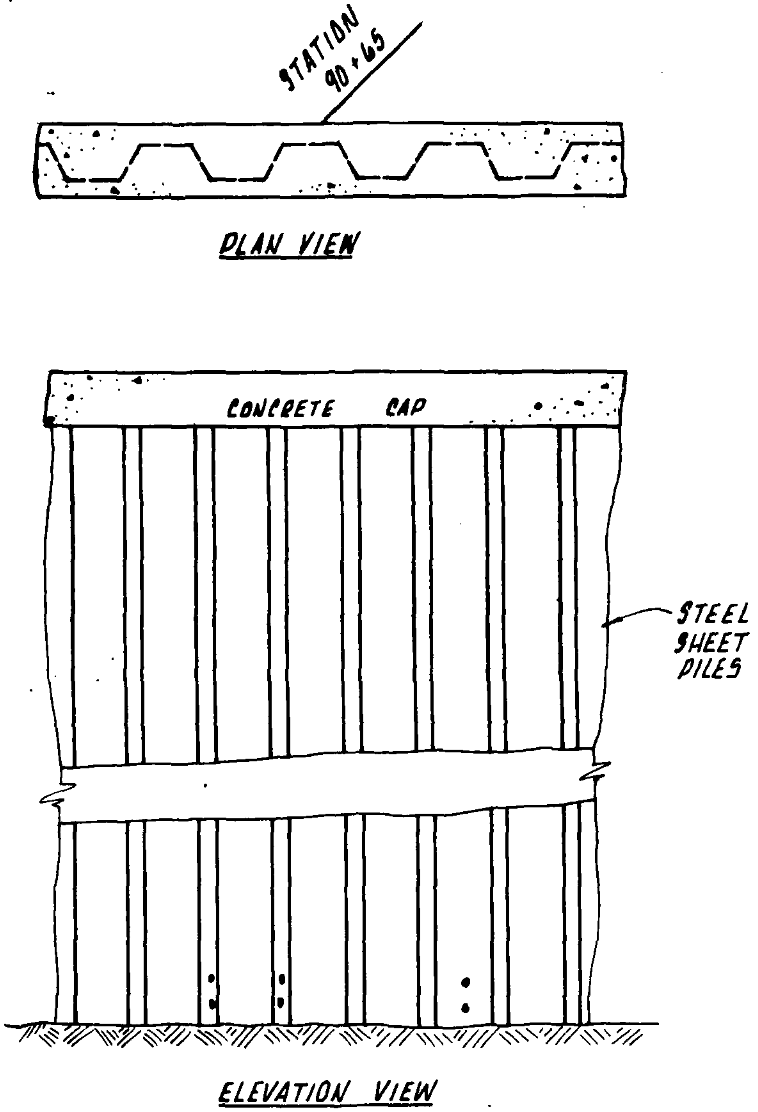
4.6.3 Structural Condition Assessment

Pier 303 appears in good condition. The piles supporting the platform do not appear to have lost any significant strength. The steel sheet piles appear to have lost moderate strength. The entire structure appears capable of supporting the existing soil loads and overburden. The loss of sediment through the bulkheads may ultimately result in the collapse of the pavement areas on the upland. The permanent displacement of the steel frame fendering system indicates inadequate resistance to loads parallel to the platform.

4.6.4 Recommendations

The holes in the steel sheet piles should be sealed. The cost of this work is estimated at $3,500. With these repairs the steel sheet piles have an expected life of 20 years. Without this work the steel sheet piling will continue to corrode and allow material to seep through the bulkhead ultimately leading to the collapse of the upland pavement. The drainage on the upland of the bulkheads should be evaluated and redirected so that run-off does not penetrate upland grades and wash material through the bulkhead. The fendering system of the pier should be evaluated and repaired to resist loads parallel to the platform.

The pier should be inspected every 4 years. The upland pavement area should be inspected annually.



TYPICAL 2" DIA. HOLES OBSERVED AT STATIONS 90+59 THRU 90+71

N.T.S.

NOTE: FOURTEEN HOLES WERE OBSERVED IN SEVEN STEEL SHEETS IN THIS AREA.

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WASHINGTON, D.C.

PIER 303
CUT HOLES

FIGURE 19

PHOTO NO. 41

Pier 303 - Two inch diameter hole leaking sediment following rain typical between stations 90+59 thru 90+71.

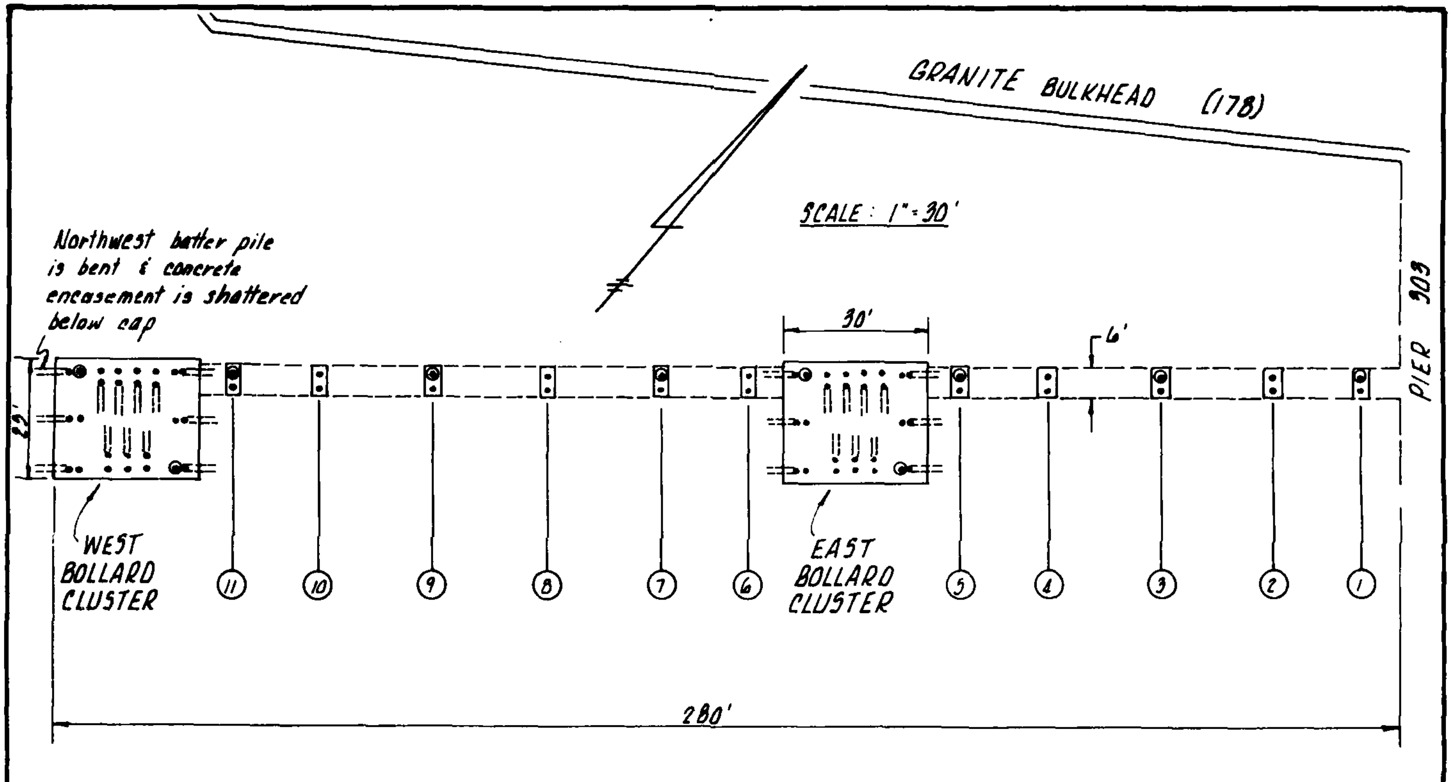
4.7 FINGER PIER 303

4.7.1 Description

Finger Pier 303 is 280 feet long and was constructed during the early 1960's. Finger Pier 303 supports bollards used for mooring support of an aircraft carrier against Pier 303 Finger Pier 303 extends to the west from the west end of Pier 303. Figure 20 shows Finger Pier 303 in plan. The pier is comprised of a catwalk which provides access to two platforms bearing bollards (see photo 42). The catwalk piles are specified on Navy drawings (Y&D Drawing No. 831940) as 10X10 H-bearing piles (10 BP 42). The piles supporting the bollards are specified on Navy drawings as 14X14 H-bearing piles (14 BP 73). All of the steel bearing piles have a concrete encasement which extends approximately 3' below the waterline. A cathodic protection system has been installed to deter corrosion of the steel piles, this system appears to be an impressed current system. The piles supporting the catwalk are all battered (angled to vertical) whereas only a portion of the piles supporting the bollards are battered; see Figure 20.

4.7.2 Observed Inspection Conditions

The piles and concrete encasements supporting Finger Pier 303 appear to be in good condition. Only one pile was noted to have any structural damage; the batter pile at the northwest corner of the west bollard cluster was noted to be bent and the concrete encasement was severely cracked (see photo 43). This damaged pile appeared to have been damaged by impact. Typically, rust stains were not observed on the piles. The observed marine growth consisted of heavy oyster growth (6"± thick) immediately below the waterline and barnacle and algae growth from mid-pile to the berm. Photos 44 and 45 show a typical uncleaned catwalk pile and bollard cluster pile. Ultrasonic thickness test results are given in Table 3 for the piles tested at Finger Pier 303. For the catwalk piles the average measured flange and web thicknesses are .425 and .431 inches respectively. The average measured flange and web thicknesses of the platform piles are .488"and .476" respectively. The original thicknesses of both flange and web of the bollard cluster piles and catwalk piles were .506" and .418" respectively. The bollard cluster piles have experienced about a 5% loss in thickness. The catwalk piles have a greater thickness than a "10BP42" pile; it appears that a "10BP57" pile with .564" thickness may have been substituted. While at the Naval Air Station (1/20/83) a severe storm occurred with gale force winds. During this storm, an aircraft carrier was berthed at Pier 303 and tied to the bollards supported by Finger Pier 303. No apparent damage was noted as a result of this storm. The cathodic protection system appeared to be in good condition; no deficiencies were noted.

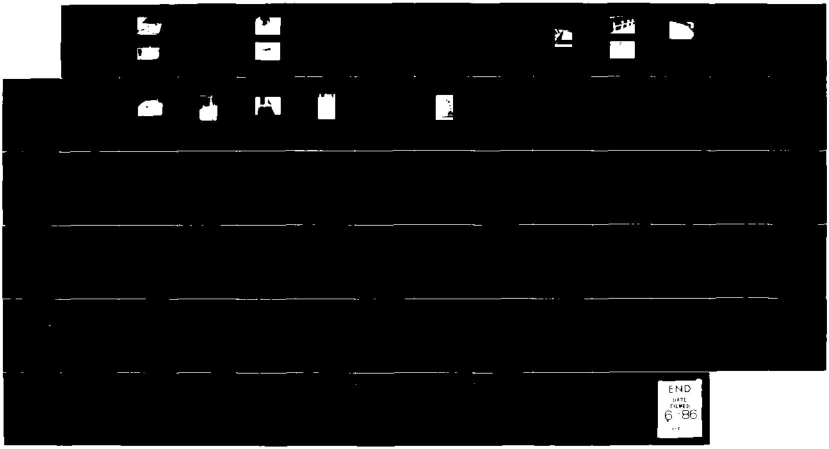


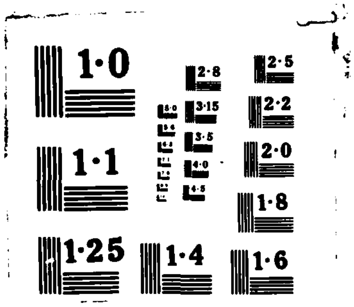
NOTE: ALL DIMENSIONS ARE APPROXIMATE

● INDICATES LEVEL II INSPECTION PERFORMED ON PILE. ① INDICATES BENT ONE ⁚ INDICATES CAP AND PILES (10BP42 - CATWALK 14BP72 - BOLLARD CLUSTERS)	FROM Y & D DRAWING NO. 831940	PREPARED BY ARTHUR V. STROCK & ASSOCIATES, INC. 829 S.E. 9th ST. DEERFIELD BEACH, FLORIDA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C. FINGER PIER 303 PILE PLAN FIGURE 20

AD-A167 358 UNDERWATER FACILITIES AND INSPECTIONS AND ASSESSMENTS AT NAVAL AIR STATIO.. (U) STROCK (ARTHUR V) AND ASSOCIATES INC DEERFIELD BEACH FL JAN 83 2/2

UNCLASSIFIED CHES/NAVFAC-FPO-1-83(1) N62477-81-C-0286 F/G 13/2 NL




1·0
2·8
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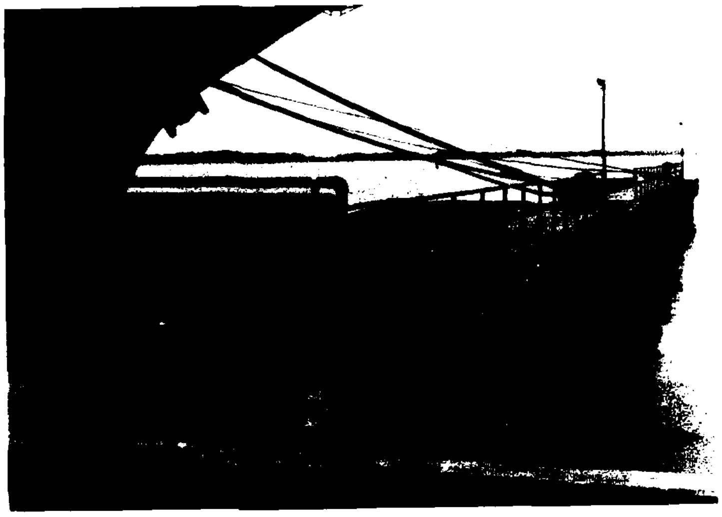


PHOTO NO. 42

Finger Pier 303 - Perspective view of pier.

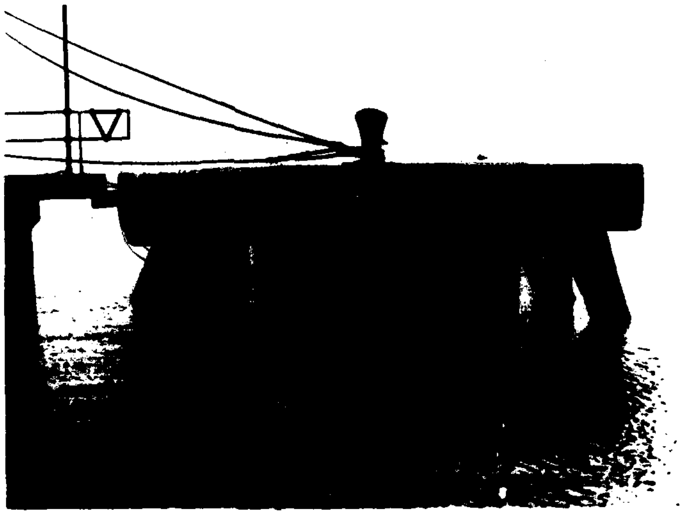


PHOTO NO. 43

Finger Pier 303 - View of west bollard cluster and cracked encasement at northwest batter pile.

TABLE 3

ULTRASONIC TESTING RESULTS
FOR
FINGER PIER 303

BENT/PILE	LOCATION	FLANGE THICKNESS (inches)	WEB THICKNESS (inches)
1/N PILE	Within 2' of concrete encasement	.413	.403
3/N PILE	Within 2' of concrete encasement	.444	.457
5/N PILE	Within 2' of concrete encasement	.451	.399
East Bollard Cluster/ NE Pile	Within 2' of concrete encasement	.473	.460
East Bollard Cluster/ NE Pile	Within 3' of mudline	.489	.467
East Bollard Cluster/ SW Pile	Within 2' of concrete encasement	.483	.459
East Bollard Cluster/ SW Pile	Within 3' of mudline	.501	.462
7/N PILE	Within 2' of concrete encasement	.394	.443
9/N PILE	Within 2' of concrete encasement	.409	.437
11/N PILE	Within 2' of concrete encasement	.438	.449
West Bollard Cluster/ NE Pile	Within 2' of concrete encasement	.497	.509
West Bollard Cluster/ SW Pile	Within 2' of concrete encasement	.484	.498

PHOTO NO. 44

Finger Pier 303 - Typical uncleaned catwalk pile web (with 12" ruler) just below encasement.

PHOTO NO. 45

Finger Pier 303 - A typical uncleaned bollard cluster pile web near bottom.

4.7.3 Structural Condition Assessment

The piles supporting Finger Pier 303 appear in good condition. Typically less than a 10 percent reduction in section modulus has occurred since construction of the Pier; see Appendix C for calculations. The observed performance of the pier during the storm of January 20, 1983 indicates that the pier has adequate strength to provide mooring support to the aircraft carrier. The damaged batter pile appears to act principally as a tension pile under mooring loads of the aircraft carrier. It appears that this damaged pile does not significantly reduce the capacity of the mooring platform and pier under lateral mooring loads of the aircraft carrier. This damaged pile had primarily provided lateral support to the bollard platform for resistance to impact and for mooring loads different than those loads predominantly applied under current mooring conditions. Other than this damaged pile, all other piles appear capable of supporting the full design loads (see Appendix C).

4.7.4 Recommendations

The damaged pile should be removed and replaced. This will require removal and replacement of a portion of the platform superstructure. The estimated cost of this repair is $20,000. The expected life of the pier is at least 25 years with or without the repair. However, without the repair the mooring platform is somewhat limited in its mooring load options.

The cathodic protection system should be inspected annually to check the condition of the anode and the impressed current through each pile (this was not within the scope of work of this task). The piles of the pier should be inspected every four years.

4.8 PIER 302

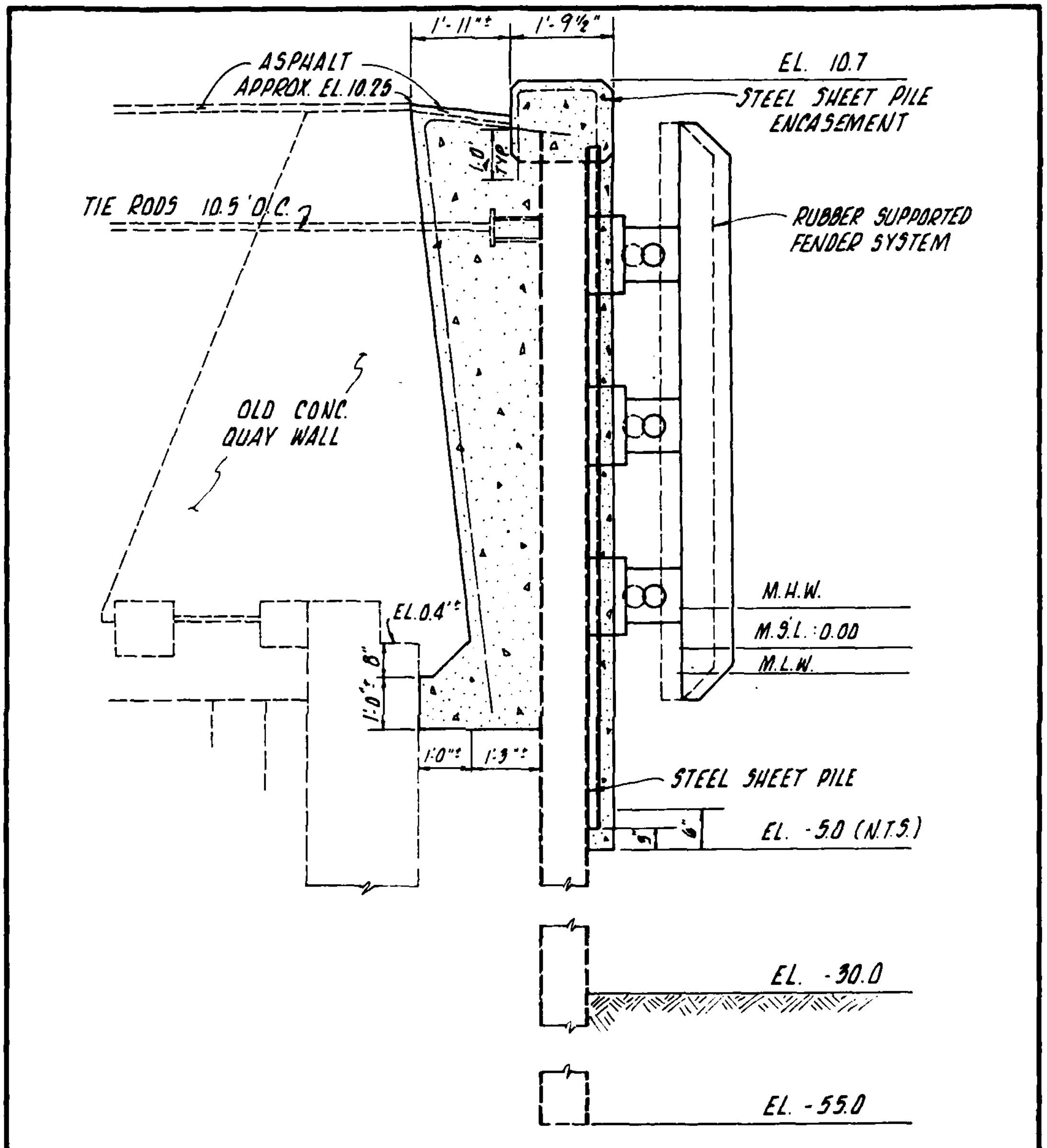
4.8.1 Description

Pier 302 is a ship service area surrounded by a steel sheet pile bulkhead. Figure 21 shows a typical section through the bulkhead. Upland of the steel sheet pile bulkhead exists an old concrete gravity structure which at one time protected the surface area. The steel sheet pile bulkhead was constructed in 1940. Pier 302 is a filled bulkhead with over 1,230 linear feet of steel sheet pile with concrete encasement. The pier length is 581' and the width is 67'. The pier served as a parking area for base personnel at the time of this inspection; see photo 46.

4.8.2 Observed Inspection Conditions

The steel sheet piles of Pier 302 appear to be in good condition. Spotted patches of rust were observed throughout the length of the wall. Marine growth consisted mostly of barnacles and algae near the encasement and of algae near the mudline. The northeast end of the pier, at station 103+50 appeared to have a significantly greater amount of rust stains than other areas of the pier. Ultrasonic thickness testing indicated that the steel sheet piles in the vicinity of station 103+50 had a smaller section remaining than other areas of the pier. While at the base, frequent winds from the easterly sector were noted to cause significant turbulence, wave action and saltwater splashing within the area of station 103+50. Table 4 shows the ultrasonic thickness testing results for Pier 302. Typically, on the average, the steel sheet piles were noted to have a thickness of .505 inches in the flange and .374 inches in the web. Original thicknesses and dimensions of the steel sheets are not known. The concrete encasement over the steel sheet piles appeared to be in good condition. Above the bottom of the concrete encasement, some minor cracks, and holes were noted in the encasement; loss of joint sealant was also noted. Photo 47 shows the concrete encasement and fendering system above the waterline in the vicinity of station 91+50; as shown in the photograph, the encasement appeared in excellent condition above the waterline. Photo 48 shows a minor spall in the bottom of the concrete encasement at station 91+80. During the course of the inspection at the site, heavy rains occurred which resulted in ponding on the upland surface area of Pier 302. The surface area of Pier 302 is typically surfaced with asphalt. Following these heavy rains, water was noted to seep into cracks of the asphalt as shown in Photo 49 in the vicinity of station 92+00.

Appendix A contains drawings illustrating in plan the stations assigned to this facility. Appendix B contains transcripts of the field observations made in conjunction with the facility inspection.



FROM NAVFAC DRAWING NO. 5068040 & NAS DRAWING NO. 8505	PREPARED BY ARTHUR V. STROCK & ASSOCIATES, INC. 829 S.E. 9th ST. DEERFIELD BEACH, FLORIDA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C.	
		PIER 302 TYPICAL SECTION	FIGURE 21

TABLE 4

ULTRASONIC TESTING RESULTS
FOR
PIER 302

STATION	LOCATION	FLANGE THICKNESS (inches)	WEB THICKNESS (inches)
103+50	within 3' of mudline	.480	.314
101+50	within 2' of concrete encasement	.498	.479
101+50	within 3' of mudline	.343	.355
99+50	within 2' of concrete encasement	.527	.348
99+50	within 3' above mudline	.624	.346
97+50	within 2' of concrete encasement	.537	.348
97+50	within 3' of mudline	.564	.370
95+50	within 2' of concrete encasement	.434	.407
95+50	within 3' of mudline	.531	.387
93+50	within 2' of concrete encasement	.495	.386
93+50	within 3' of mudline	.520	.369

PHOTO NO. 46

Pier 302 - Perspective view showing automobile parking in proximity of bulkhead.

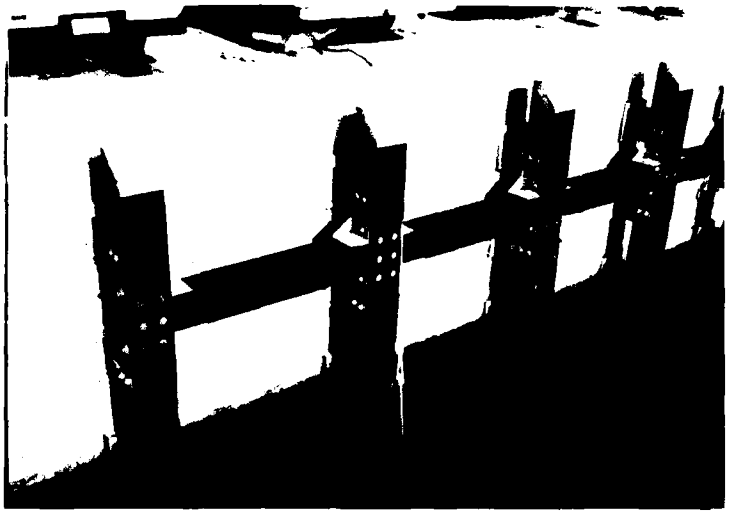


PHOTO NO. 47

Pier 302 - Typical view of concrete encasement in excellent condition above waterline.

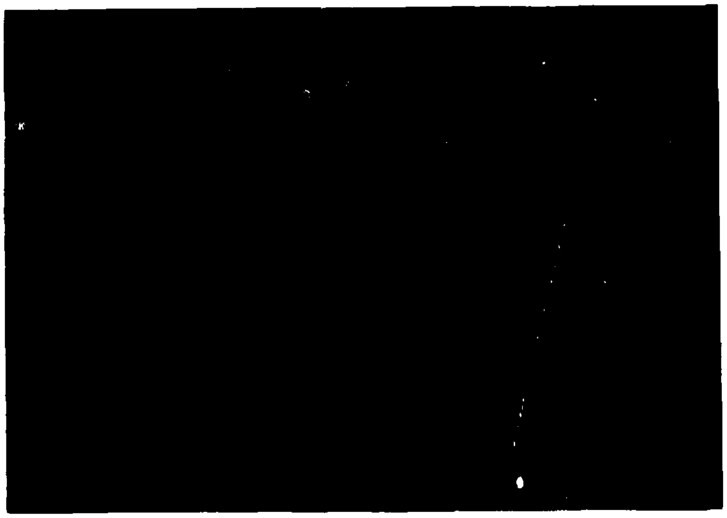


PHOTO NO. 48

Pier 302 - A spall at bottom of encasement at station 91+70.

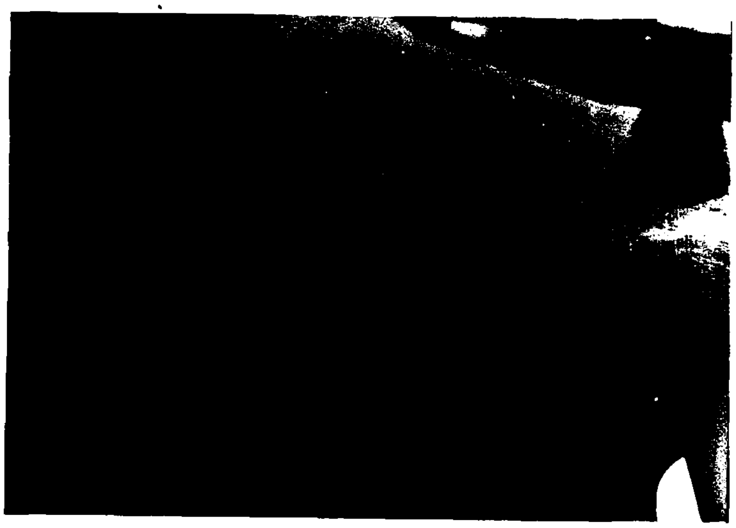


PHOTO NO. 49

Pier 302 - Seepage of ponded water through cracks of asphalt covering service area (note bubbles).

4.8.3 Structural Condition Assessment

Pier 302 appears in good condition. The steel sheet piles appear to have lost minimum strength to corrosion. The concrete encasement appears to provide good protection to the steel sheet piles. The existing soil loads and overburden appear to be within the capacity of the structure. The seepage of water into cracks of the asphalt pavement builds hydrostatic pressure behind the bulkhead and also washes sediment through the bulkhead.

4.8.4 Recommendations

The holes and cracks of the concrete encasement should be sealed with a non-shrinking grout or epoxy. The joints of the encasement should be sealed with a non-shrinking, non-oxidizing elastic sealant. The cost of these repairs is estimated at $11,000. With these repairs the expected life of the pier is 25 years. Without these repairs the expected life of the pier is 10 years - this depends on the condition of the steel sheet piles covered by the encasement.

The drainage on the upland of the bulkheads should be evaluated and redirected so that run-off does not penetrate upland grades and wash material through the bulkhead. The upland pavement should be inspected annually for signs of settlement. The pier should be inspected every 4 years.

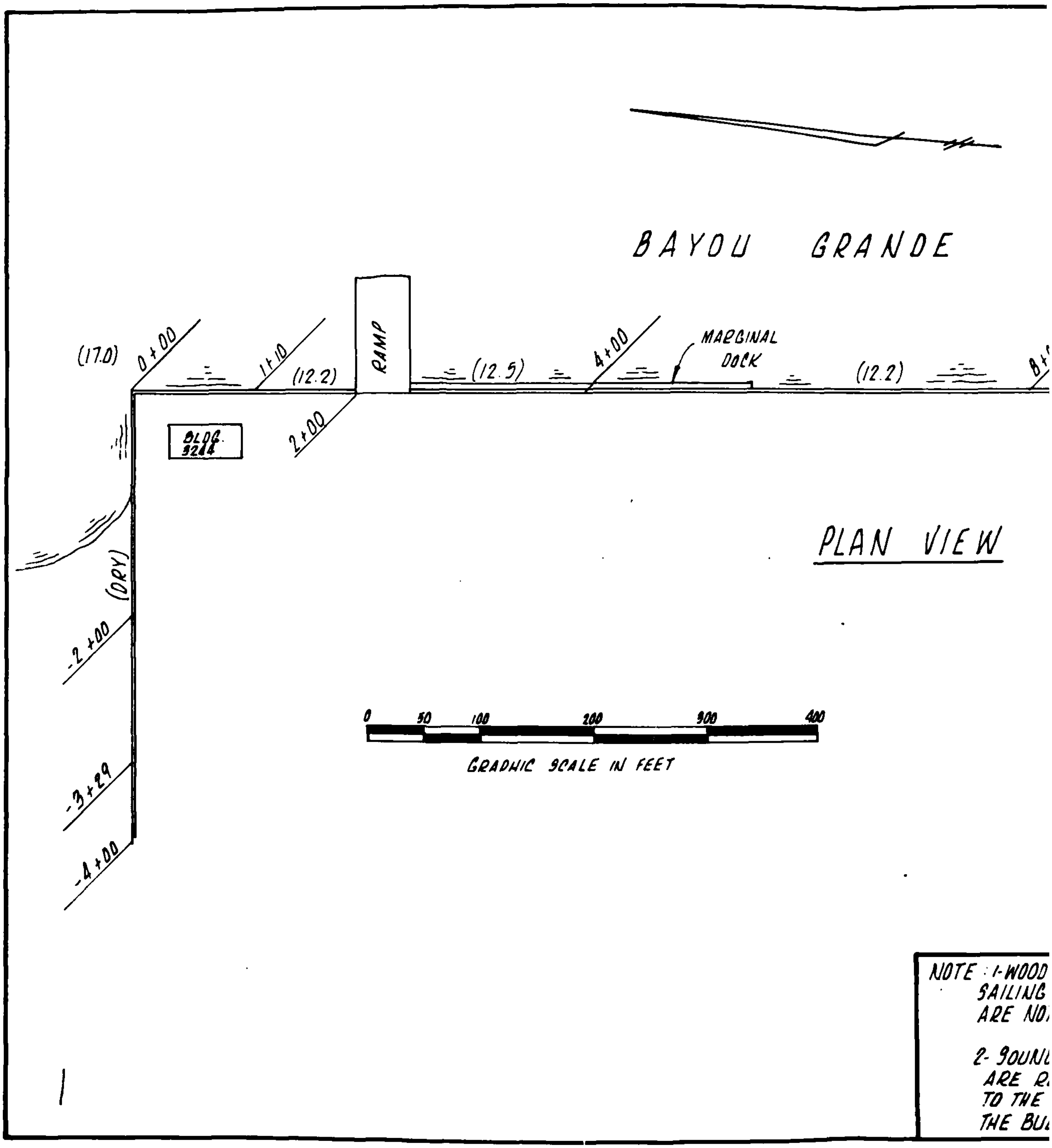
4.9 BULKHEAD 708B

4.9.1 Description

Bulkhead 708B appears to be a reinforced concrete gravity type structure which was constructed in 1943. The bulkhead is 1,875 feet long. No structural details of this facility were available from government files. Bulkhead 708B appears to be of a similar construction as Seawall 384 (see section 4.2). Bulkhead 708B provides soil retention and erosion protection to a concrete pavement on the upland surrounding the base recreational sailing facility, which includes Building 3234. Throughout most of the length of Bulkhead 708B the upland side of the bulkhead is covered with a concrete slab. Currently, plans are underway to repair damage to the bulkhead and slab in the vicinity of Building 3234 (see NAVFAC drawing #5062754). Figure 22 shows the bulkhead location in plan, the stationing assigned to the bulkhead and soundings referenced to the top of the bulkhead. At station 2+00 a concrete ramp exists for the launching of recreational boats. Adjacent to the bulkhead are a wooden marginal walkway and wooden finger piers providing slips for recreational boats.

4.9.2 Observed Inspection Conditions

Bulkhead 708B appears to be in fair condition. The face of the bulkhead was noted to have a large number of hairline cracks but no significant failure cracks were observed. Observed marine growth consisted mainly of oysters and barnacles. Water depths at the wet face were typically noted between 6 inches and 4 feet. As noted above, plans are currently underway to repair two areas of the bulkhead in the vicinity of Building 3234. The two areas proposed for repair are immediately adjacent to vertical tongue-and-groove joints in the bulkhead. These areas cited for repairs are indicative of the observations made throughout the length of the wall at other vertical tongue-and-groove joints. Typically, these joints have experienced loss of construction joint seal material and as a result, the joints appear to be allowing backfill material to seep through the joint resulting in settlement of slabs and loss of material through the bulkhead. An old wood concrete form or wooden toe wall was partially exposed at the toe of the bulkhead in some areas. Photo 50 shows a perspective of Building 3234 and the concrete pavement on the upland behind Bulkhead 708B. Photo 51 shows one of two areas cited for repair at station 0+48; the photo demonstrates the settlement of the concrete pavement behind the bulkhead and also the loss of construction joint seal at the tongue-and-groove joint. Photo 52 shows an apparent lateral displacement of a wall section at a tongue-and-groove joint at station 1+70. Photo 53 shows typical hairline cracks and rust stains in the upper portion



BAYOU GRANDE
(17.0)
0+00
1+10
(12.2)
RAMP
(12.5)
4+00
MARGINAL DOCK
(12.2)
2+00
BLDG. 3244
(DRY)
-2+00
-3+29
-4+00
PLAN VIEW
0 50 100 200 300 400
GRAPHIC SCALE IN FEET
NOTE: 1-WOOD
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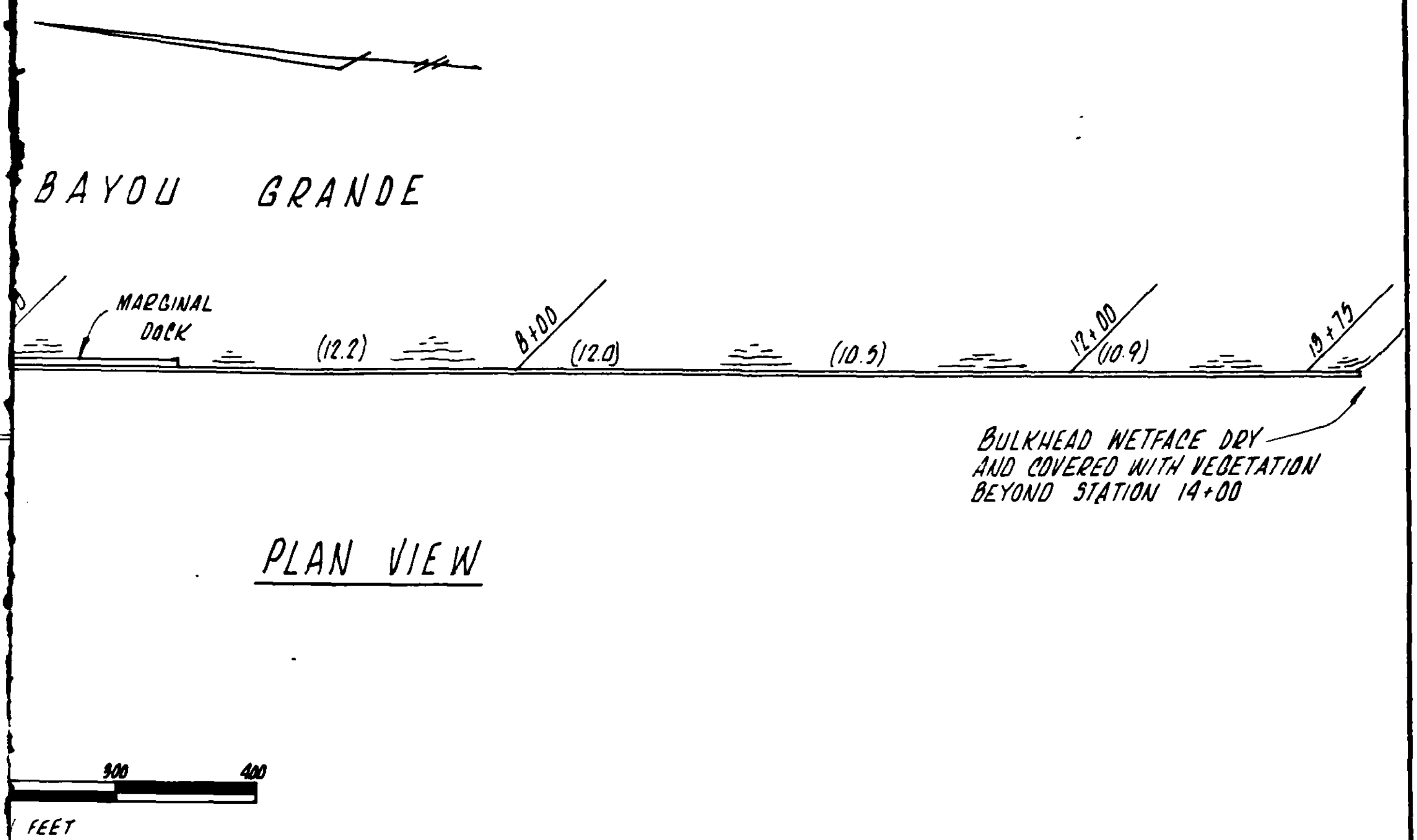

PLAN VIEW

2

NOTE: 1- WOOD PIERS OF SAILING FACILITY ARE NOT SHOWN

2- SOUNDINGS (12) ARE REFERENCED TO THE TOP OF THE BULKHEAD

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BULKHEAD 708 B
STATIONING PLAN

FIGURE 22

PHOTO NO. 50

Bulkhead 708B - Perspective of building 3234 on the upland of Bulkhead 708B.

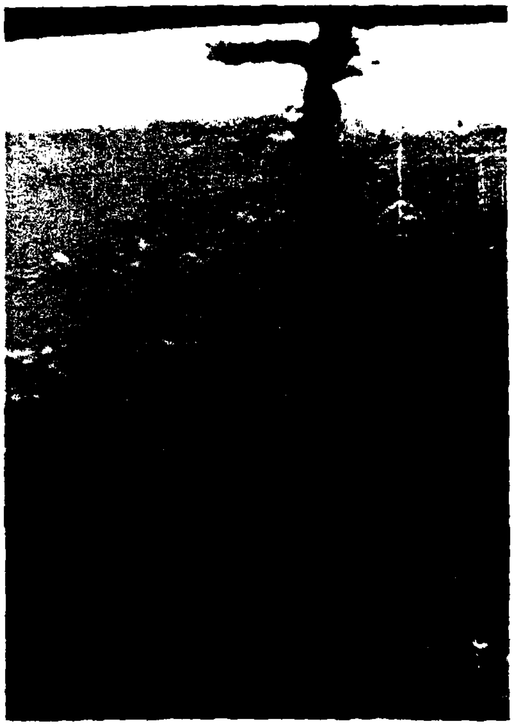


PHOTO NO. 51

Bulkhead 708B - Area cited for repair at tongue-and-groove joint - station 0+48.

PHOTO NO. 52

Bulkhead 708B - Transverse displacement of bulkhead at tongue-and-groove joint at station 1+70.

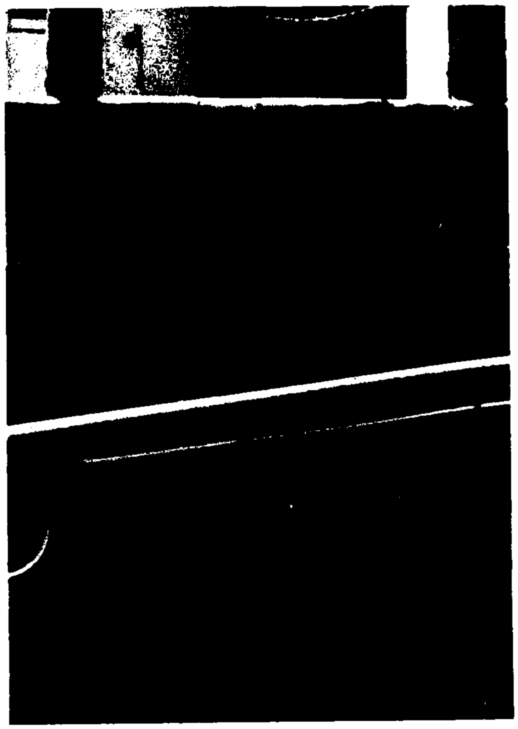


PHOTO NO. 53

Bulkhead 708B - Elevation view of typical wall wetface.

of the bulkhead and also a portion of the wall in typical good condition below the electrical conduits mounted against the wall. Photo 54 shows a close-up of a tongue-and-groove joint with a minor spall at the joint, this is typical. The predominant defect noted throughout the length of the wall was the apparent deterioration of the tongue-and-groove joint seals which appear to be allowing material to seep through the bulkhead.

The ramp at the sailing facility appeared in excellent condition. Appendix B contains transcripts of the field observations made in conjunction with the facility inspection.

4.9.3 Structural Condition Assessment

Bulkhead 708B appears in good structural condition. Although no signs of structural failure appear imminent, evidence that the bulkhead is losing its ability to retain upland material was noted. In particular, the tongue-and-groove joints are in poor condition and do not provide a proper seal to the bulkhead. Furthermore, a condition of inadequate toe penetration may exist. Although the bulkhead appears stable, material may be seeping under the toe of the bulkhead.

4.9.4 Recommendations

The tongue-and-groove joints of Bulkhead 708B should be cleaned of the deteriorated seal material and resealed with a non-shrinking, non-oxidizing permanent elastic sealant. If pavement is to be replaced (as currently planned) then filter cloth should be placed behind the bulkhead, over the tongue-and-groove joints.

Further inspection is also warranted. The pavement immediately behind the bulkhead should be bored at least at 200 foot intervals (along the wall) to check for loss of material underneath the pavement. If loss of material underneath the pavement has occurred then the undermined pavement should either be closed to traffic or removed and replaced following placement of fill. The depth of the toe of the wall should also be investigated by water jet probing to check for undermining and seepage under the wall - a toe wall may be required if penetration is inadequate.

With the repair of the joints and repair of the pavement and toe (as deemed necessary by further inspection) the expected life of the seawall is 25 years. Without these repairs the expected life is less than 5 years - during which time the pavement is expected to fail due to loss of material from behind the wall. The estimated cost of the joint (seal) repairs is $10,000 (see Appendix D).

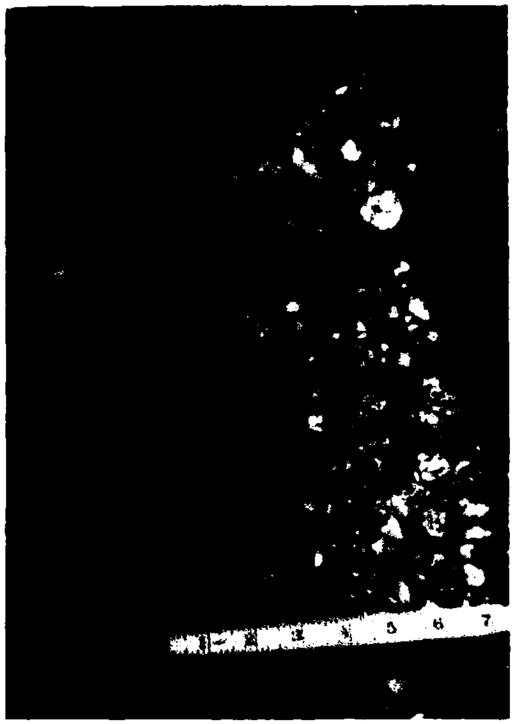
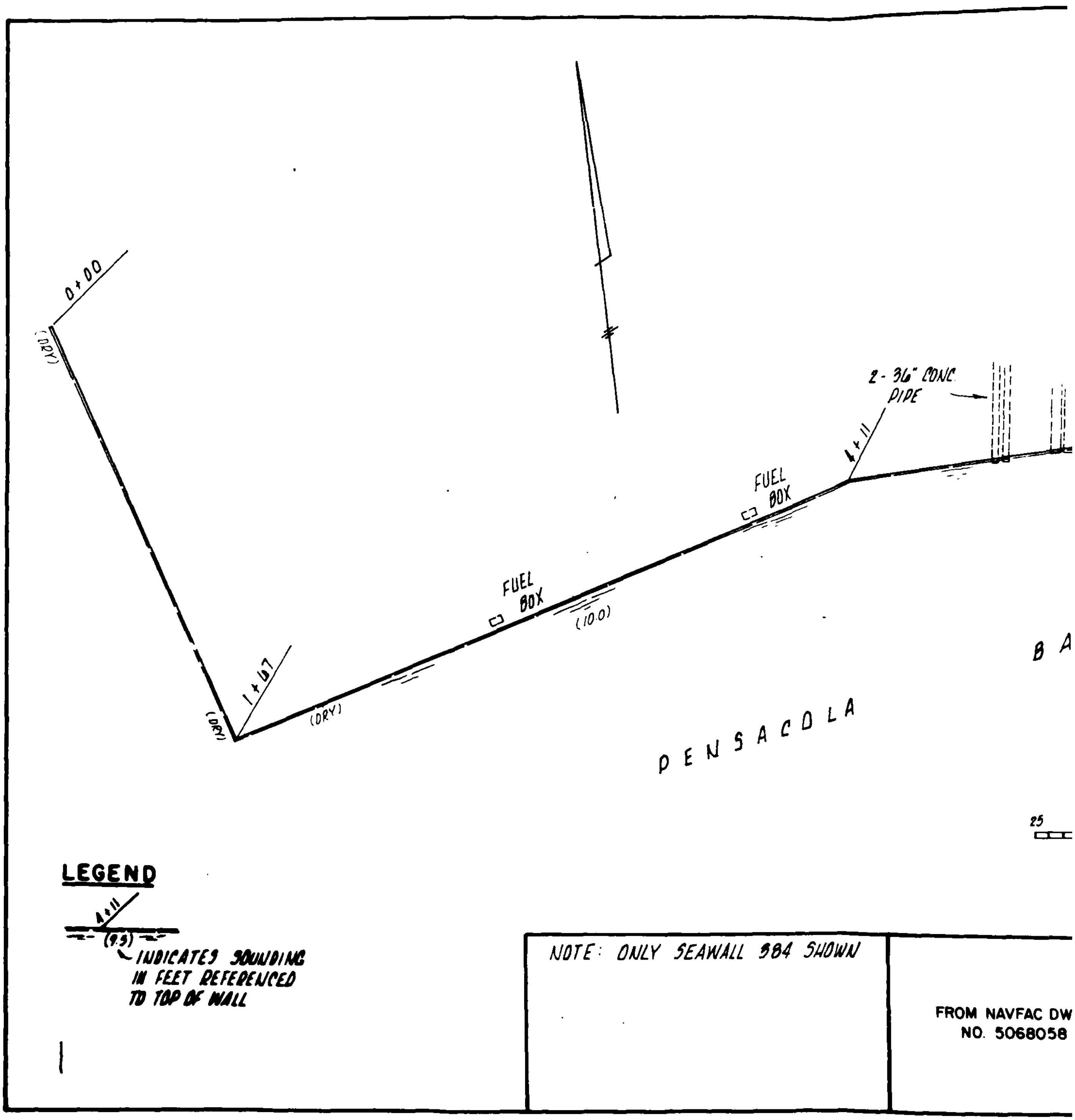


PHOTO NO. 54
Bulkhead 708B - Joint spall at station 1+10

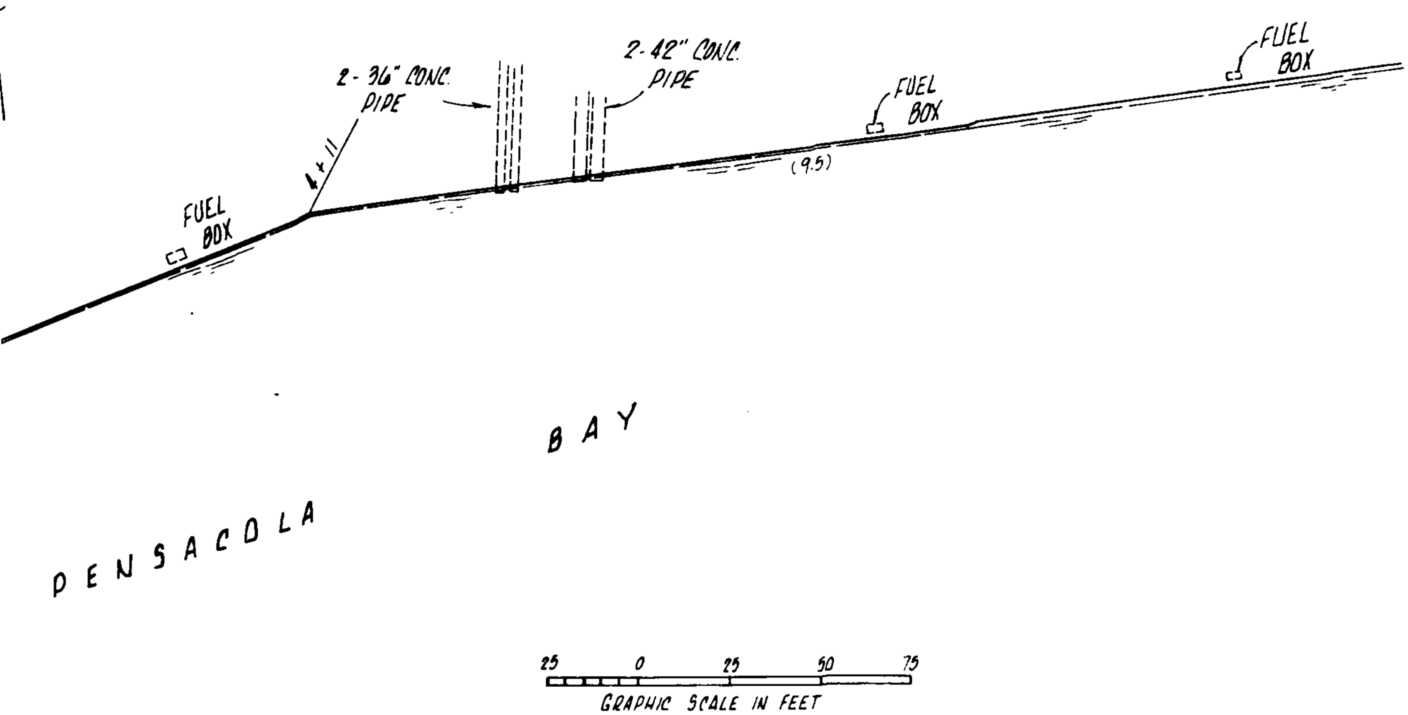
The bulkhead should be inspected every 4 years. The pavement behind the seawall should be inspected annually to check for signs of settlement.

APPENDIX A

STATIONING
PLAN



0+00
(DRY)
1+07
(DRY)
(DRY)
FUEL BOX
(10.0)
FUEL BOX
4+11
2-36" CONC. PIPE
PENSACOLA
BA
25
LEGEND
4+11
(9.3)
INDICATES SOUNDING IN FEET REFERENCED TO TOP OF WALL
NOTE: ONLY SEAWALL 384 SHOWN
FROM NAVFAC DW
NO. 5068058



ONLY SEAWALL 384 SHOWN

FROM NAVFAC DWG. NO. 5068058

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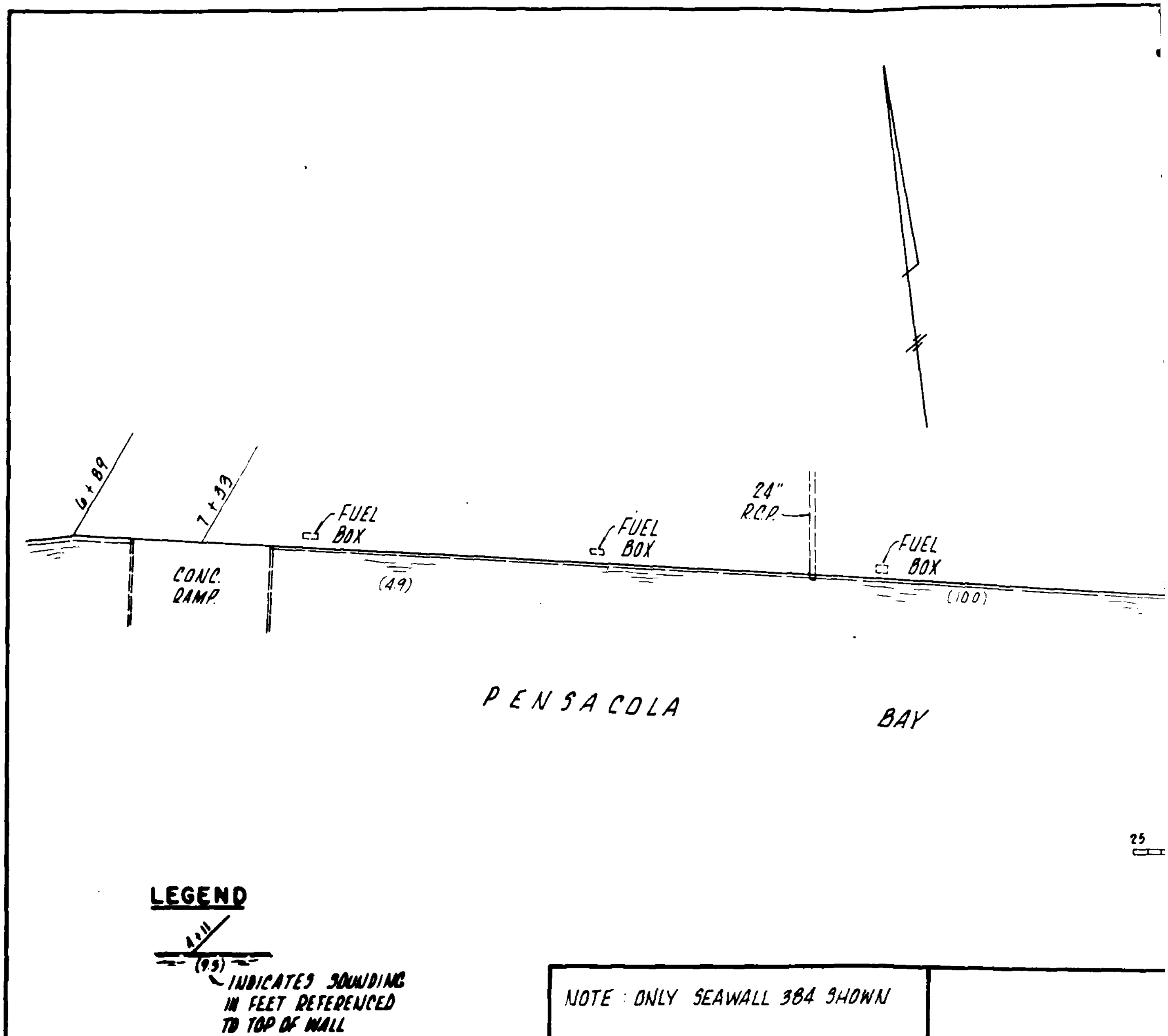
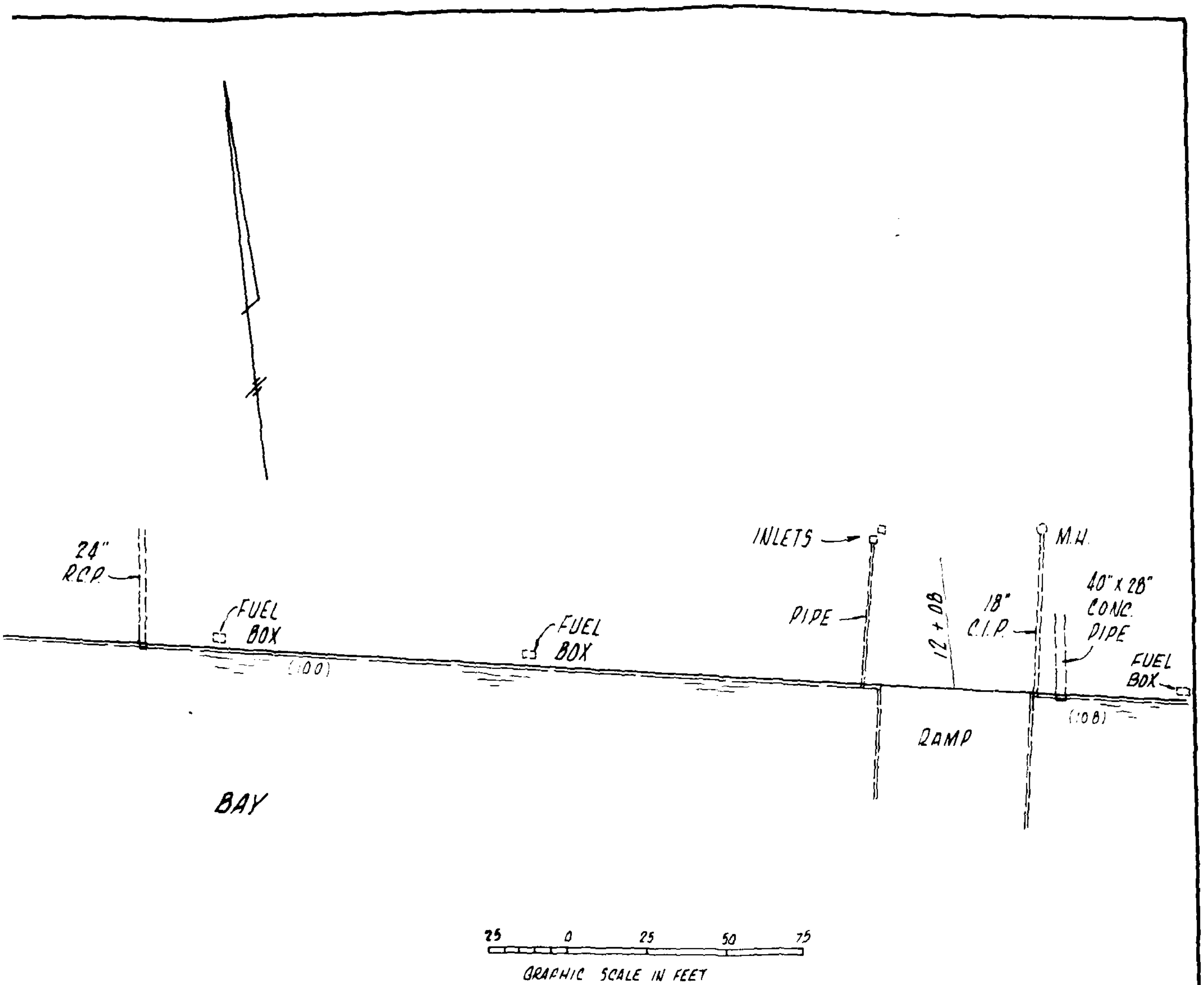
STATIONING PLAN

FIGURE A1



NOTE : ONLY SEAWALL 384 SHOWN

FROM NAVFAC DWG.
NO. 5068059



SEAWALL 384 SHOWN

FROM NAVFAC DWG.
NO. 5068059

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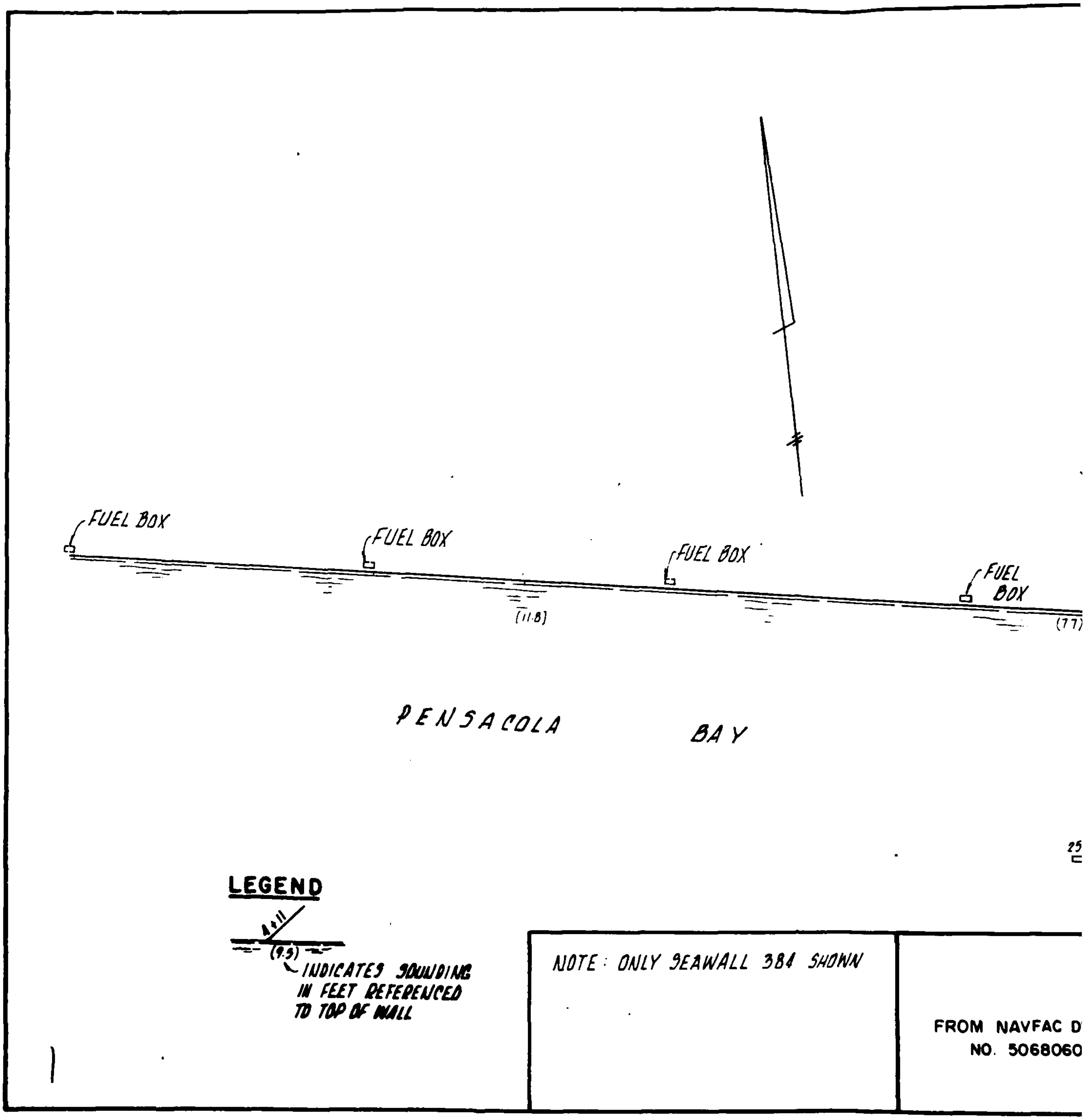
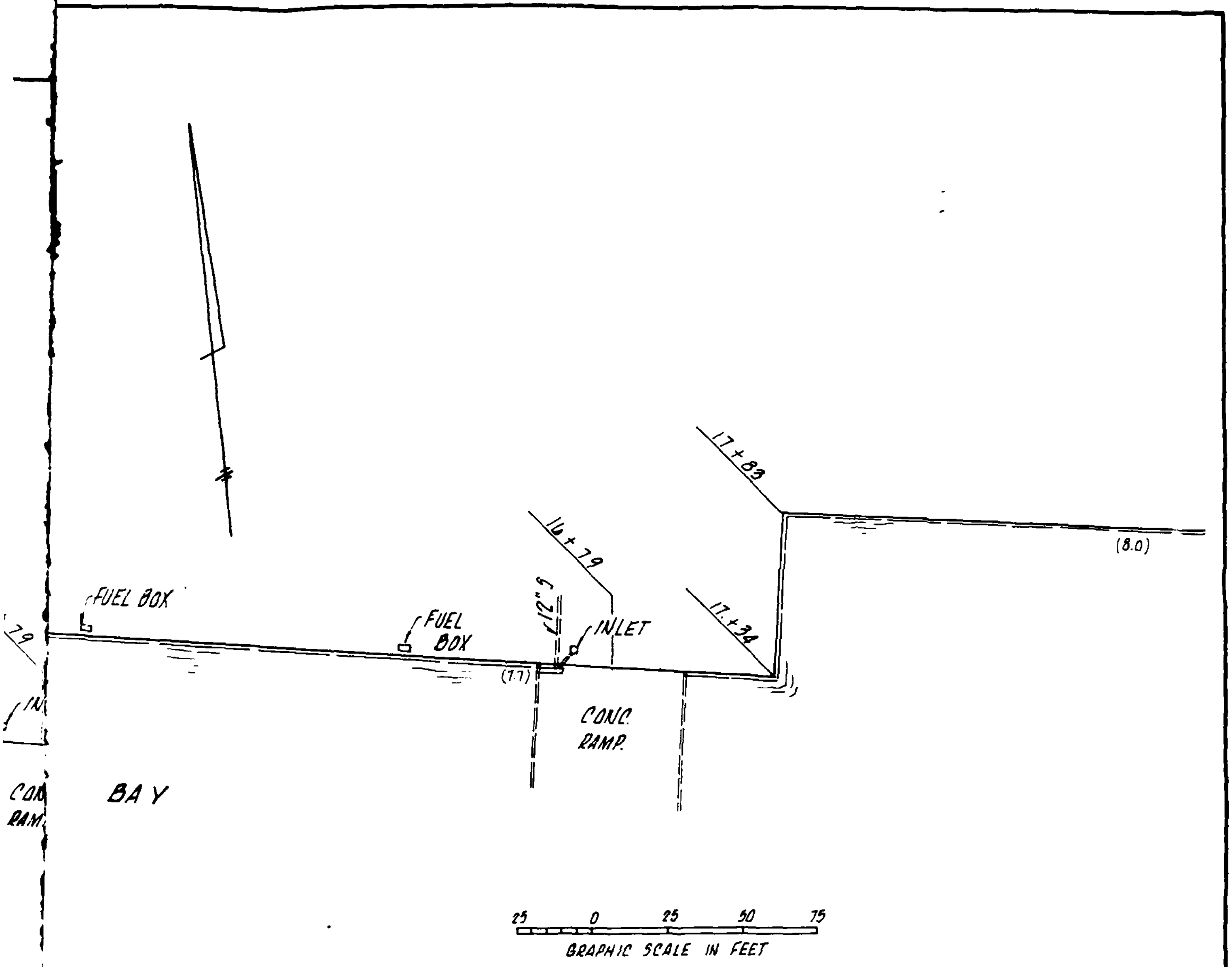
STATIONING PLAN

FIGURE A2


FUEL BOX
FUEL BOX
FUEL BOX
FUEL BOX
(11.8)
(7.7)
PENSACOLA BAY
LEGEND
(9.5)
INDICATES SOUNDING IN FEET REFERENCED TO TOP OF WALL
NOTE: ONLY SEAWALL 384 SHOWN
FROM NAVFAC D
NO. 5068060



LY SEAWALL 384 SHOWN

FROM NAVFAC DWG. NO. 5068060

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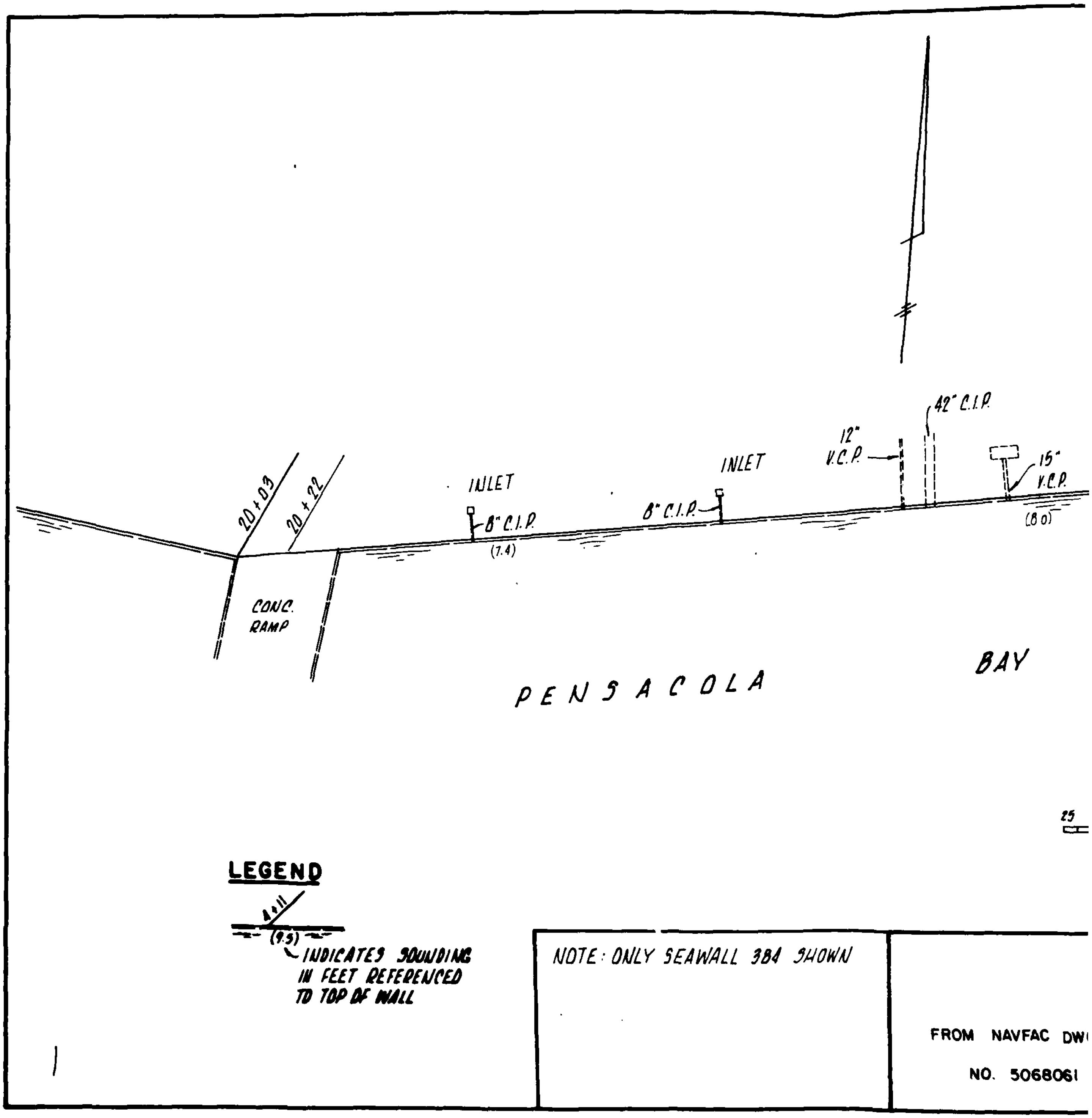
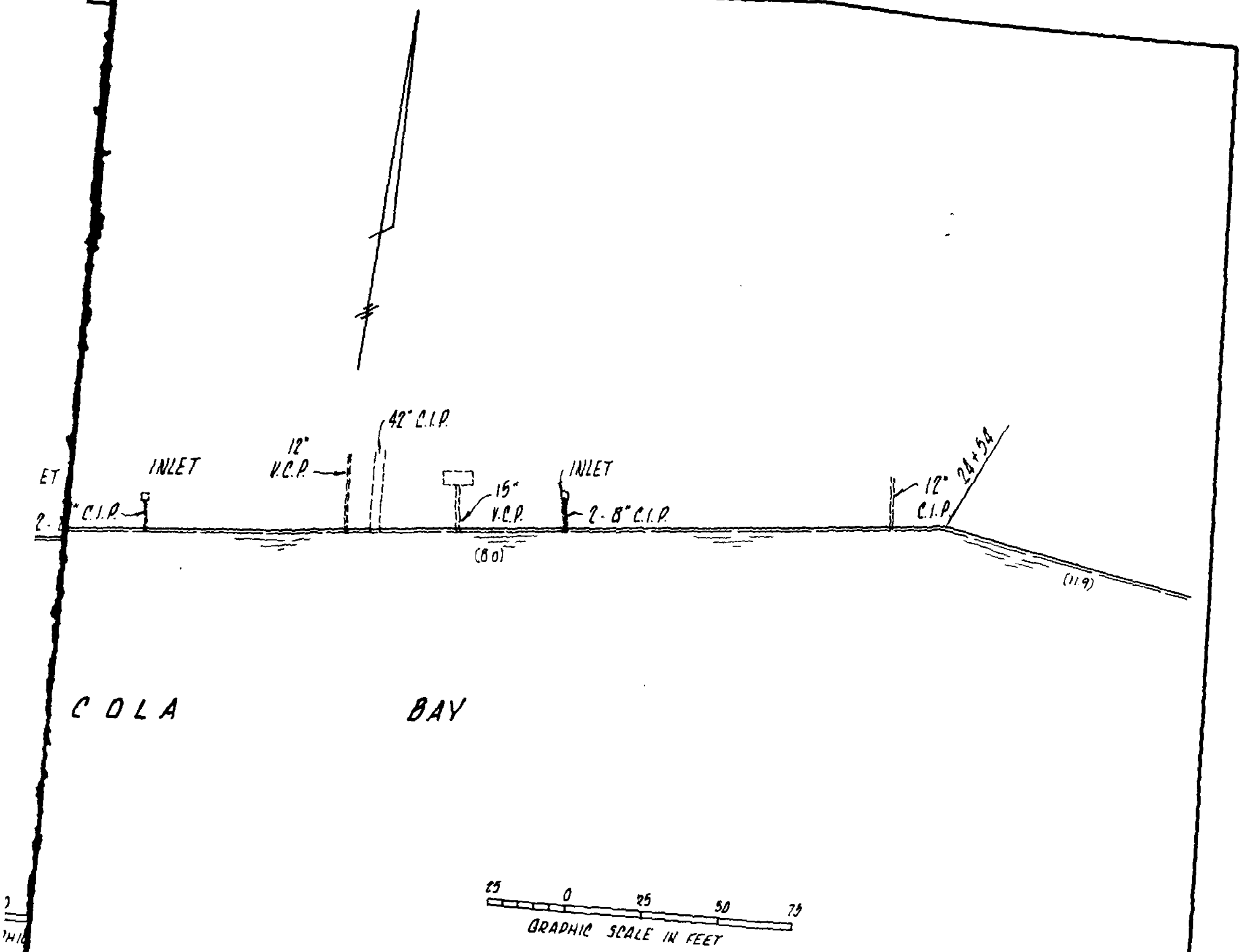
2 STATIONING PLAN

FIGURE A3



20+03
20+22
CONC. RAMP
INLET
8" C.I.P.
(7.4)
INLET
8" C.I.P.
12" V.C.P.
42" C.I.P.
15" V.C.P.
(8.0)
PENSACOLA
BAY
25
LEGEND
4+11
(9.5)
INDICATES SOUNDING IN FEET REFERENCED TO TOP OF WALL
NOTE: ONLY SEAWALL 3B4 SHOWN
FROM NAVFAC DW
NO. 5068061



SEAWALL 384 SHOWN

FROM NAVFAC DWG.
NO. 5068061

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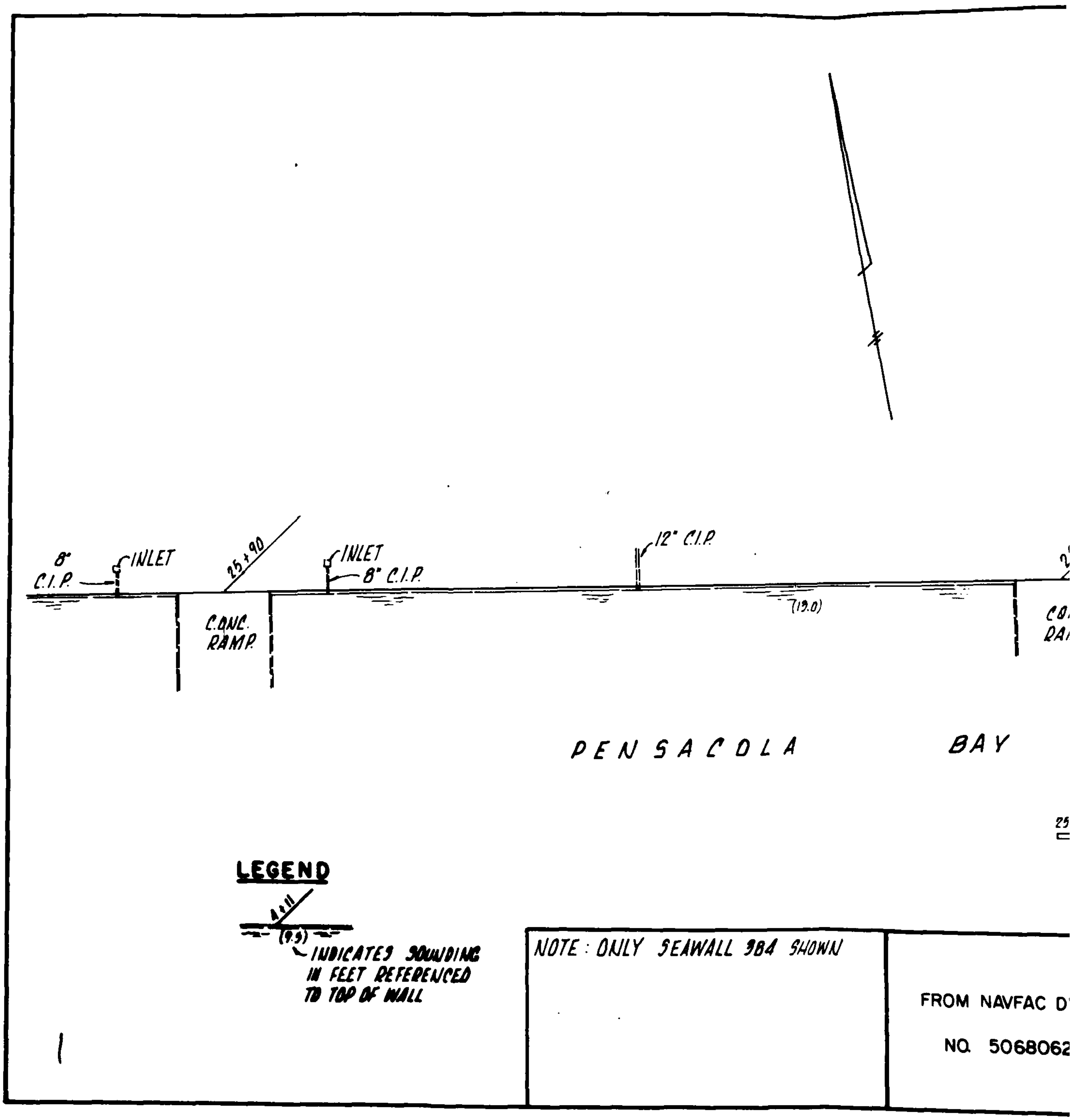
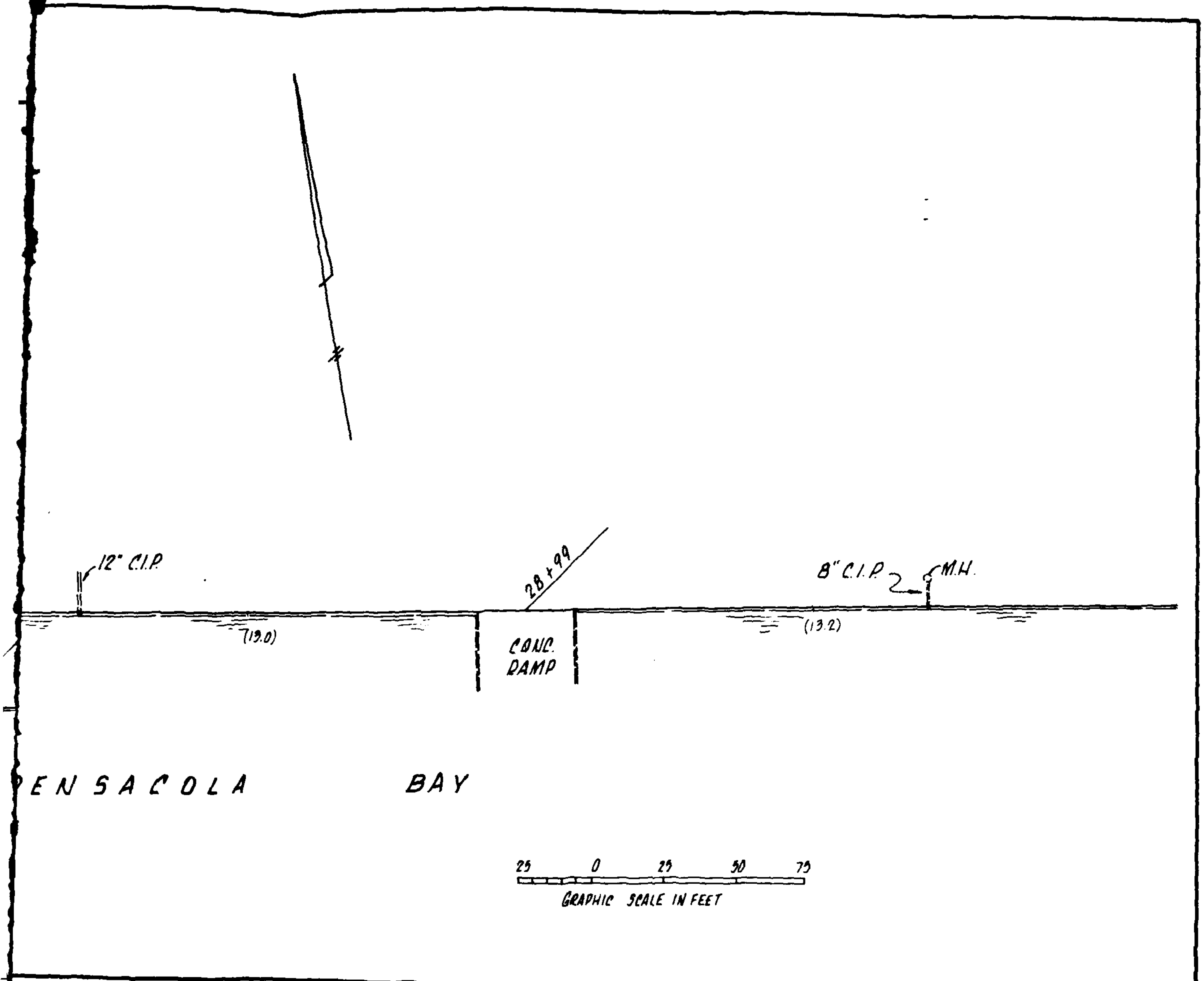
2 STATIONING
PLAN

FIGURE
A 4



8" C.I.P.
INLET
25+90
INLET
8" C.I.P.
12" C.I.P.
(13.0)
CONC. RAMP
PENSACOLA BAY
LEGEND
(9.5)
INDICATES SOUNDING IN FEET REFERENCED TO TOP OF WALL
NOTE: ONLY SEAWALL 3B4 SHOWN
FROM NAVFAC



ONLY SEAWALL 384 SHOWN	FROM NAVFAC DWG. NO. 5068062	PREPARED BY ARTHUR V. STROCK & ASSOCIATES, INC. 829 S.E. 9th ST. DEERFIELD BEACH, FLORIDA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C.	
			2 STATIONING PLAN	FIGURE A5

APPENDIX B

OBSERVATIONS

FACILITY	Page No.
Seawall 384	B2,B3
Seawall 384 Ramps	B4,B5
Seawall 1724	B6-B8
Seawall 1724 Ramps	B9,B10
Pier 3238	B11,B12
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Finger Pier 303	B16
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Bulkhead 708B	B19,B20

PENSACOLA NAVAL AIR STATION
SEAWALL 384

STATION (S)	OBSERVATION
0+00 - 6+00	Poured in place reinforced concrete
0+00 - 6+89	Average depth 0 to 2.5'
1+00	Hairline (crack) in cap
1+67	Bend (in wall)
4+11	Bend (in wall)
6+00	Seal placed on wet face of new concrete sheet pile. Seal does not cover bottom 2'
7+10	(exposed) dowels, dry (berm)
7+10 - 7+90	Map cracking
7+60	Electrical box
7+90	Vertical hairline (cracks)
8+65	Electrical box
9+20	Joint in wall with electrical conduit
9+50	Drain (with) crack thru to top of cap
10+60	Vertical crack
10+80	Electrical box, (water depth) 1'
11+20	Joint
11+50	Aggregate exposed below waterline and immediately above, pitted above waterline; joint, (water depth) 1'
11+80 - 12+30	Ramp
12+30 - 14+00	4 to 6" lip at mudline, (water depth) 2.5'
12+30	Joint
12+65	Joint
13+40	Spall at top
14+00 - 14+30	*12" lip*
14+30	3' joint with spall
15+20	Joint with spall and horizontal hairline (crack)
15+50	(water depth) 4'
16+50	(water depth) 1.5', ramp
16+50	Pollution control box broken up, heavy scaling at joint
16+50 - 17+00	Ramp #5, minor rust stains, photo
17+34	Bend (in wall), hairline crack, light scale
17+50	Minor rust and diagonal hairline (crack)
17+70	Berm at Mean High Water
17+75 - 17+83	Wall patched, photos
17+83	Bend (in wall)
17+95	Tongue-and-groove joint, 1½" gap, photo
18+80 thru 24+20	Rubble at toe of wall
18+08	Berm at Mean High Water (Line)
18+55	Tongue-and-groove joint, sealant deteriorated
18+75	Crack at top of tongue-and-groove joint, photos
19+45	Tongue-and-groove joint, sealant deteriorated, reinforcing bars exposed and rusted, horizontal crack, photo
19+70	Rusted cleat
19+90	Berm at Mean High Water
20+45	Patch at top of wall, photo

SEAWALL 384

STATION (S)	OBSERVATION
20+60	Vertical hairline (crack)
20+75	Tongue-and-groove (joint), deteriorated seal
21+30	Berm at Mean High (Water) Line
21+45	Tongue-and-groove (joint), deteriorated seal
21+50 and 22+00	Tongue-and-groove (joints), seal repaired with tar
22+20	Vertical hairline (crack), (water depth) 0
22+30	Vertical reinforcing bar exposed and rusting
22+50	Water discharge pipe
22+70	Moderate scaling, photo
23+00	Holes in patch around pipe, photo
24+00	Lip approximately 2' (water depth) 3½'
24+20	6" deep hole in wall at bottom of wall at joint
24+54	Bend (in wall), berm, (water depth) 1'
24+75	Heavy rust
24+80	Heavy scaling
25+00	(water depth) 0, photos
25+60	Apparent weep hole coming from under wall, photo
25+75 - 26+06	Ramp, Minor surface spalling
26+12	Vertical hairline (crack)
26+25	(water depth) 1'
26+50	Severe mortar deterioration, photos
26+65 and 26+97	Tongue-and-groove (joints), bad seal
27+40	Out(fall) pipe
27+60	Tongue-and-groove (joint), seal bad, severe scaling with rust stains
28+00	(water depth) 4'
28+20	6" of bottom of lip broken off for 2', light to medium scaling <u>NOTE</u>: underwater photo taken here
28+80	Photo of cleaned wall
28+80 to 29+15	Ramp
29+15 thru 33+50	Rust stains
29+60	Tongue-and-groove (joint), seal bad, photo
30+00	(water depth) 4½'
30+20	Tongue-and-groove (joint), seal bad, light scaling
30+50	Tongue-and-groove (joint), seal bad
31+10	Tongue-and-groove (joint), seal bad
31+60	Vertical hairline (crack), pipe
31+90	Vertical hairline (crack), pipe
32+00	Tongue-and-groove (joint), seal bad, (water depth) 4½'
32+30	Patches of severe scaling above Mean High Water (Line)
32+90	Tongue-and-groove (joint), seal bad, severe scaling by joint
33+00	Vertical hairline (crack)
33+20	Tongue-and-groove (joint), seal bad, minor spalling
33+30	Vertical hairline (crack)
33+50 to 34+03	Ramp, tongue-and-groove (joint), seal bad

SEAWALL 384

STATION (at centerline of ramp)	RAMP OBSERVATIONS
7+33	This ramp is pile supported. Ramp surface appears in good condition. East cap good except for spalled section missing about 30' down. West cap appears O.K. to waterline. Piles appear in good condition with a few cracks. Pile caps appear in good condition with a few cracks. Underside of ramp has minor spalling. Submerged west cap O.K., buried approximately 25' out. Submerged east cap O.K., buried approximately 25' out. End of ramp is buried.
12+08	This ramp is pile supported. Ramp surface and caps in excellent shape, concrete piles and pile caps appear in good condition. Underside of ramp appears in good condition, east submerged cap appears missing. End c ramp is buried under sand, west submerged cap appears O.K. NOTE: This appears to be the best ramp inspected.
16+79	This ramp is pile supported. Ramp and caps above water in excellent shape. One pile cap cracked and spalled, other two have cracks. Concrete pilings appear in good condition. Submerged east side and cap have portion of cap missing, the rest is under sand. Submerged west side and cap have portion of cap missing, the rest is under sand.
20+22	First 4' to 10' of surface of ramp missing or badly broken up (appears to have been mechanically chipped away), rest of ramp surface appears to have been removed leaving pre-existing surface exposed with rusting rebar just under surface. Only small section of cap remains on east side. West side cap is in water over side. Submerged east and west cap laying in sand, broken away.
25+90	This ramp is pile supported. Surface of ramp in good shape except for minor spalls and with exposed rebar along east and west margins of ramp surface. East* and west caps appear in good condition out of water. Piles appear O.K. with a few hairline cracks noted. Underside of ramp O.K. except for 4'X3' area showing exposed steel. Submerged east cap appears O.K., end of ramp appears O.K., submerged west cap appears O.K. *East cap has 2 deep spalls 10' and 15' up from waterline

SEAWALL 384

STATION (at centerline of ramp)	RAMP OBSERVATIONS
28+99	This ramp is pile supported. Ramp surface is in good condition with a number of spalled areas having exposed and rusted rebar, no cracks to waterline. East and west caps have 8' long sections missing at top of ramp. Piles O.K., underside of ramp O.K. except for a 5'X4' area with exposed rebar. Submerged east cap O.K., end buried under sand. Submerged west cap O.K., end buried under sand. End of ramp buried under sand.
33+77	This ramp is pile supported. Surface of ramp appears in excellent condition. East-west caps appear in excellent condition. Pilings show hairline cracks with very little exposed steel. Piling caps show cracks with bottom section of one cap missing, with steel exposed. Underside of deck has exposed steel in several areas. End of ramp O.K., submerged portion of east and west caps O.K. West cap shows cracks and rusted and/or exposed rebar from waterline up 20'.

PENSACOLA NAVAL AIR STATION
SEAWALL 1724

STATION (S)	OBSERVATION
34+03	Wood groin, slab on grade starts at (wood)
34+50	Patch
35+00	Water flowing from joint
35+50	Hairline crack along pour joint
36+00	Outfall, photo
36+50 to 36+80	Ramp
37+25 to 37+35	Water seeping out (of) cracks throughout, photo
37+80	Wall repaired, photos of cracks (on slab, on grade)
38+60 to 38+80	Ramp
38+80	Wood groin
39+00	Bend (in wall)
39+55 thru 40+20	Heavy joint spall, slab edge exposed, cracks throughout
39+55	Underwater photos
40+00	Photo, Mike standing at edge of slab
40+80 thru 41+00	Ramp
41+50	Large crack on slab on grade
41+95	Large crack on slab on grade
42+30	Heavy spall and crack in vertical wall, deep spalling along joint
42+75 and 42+95	Cracks
43+64	Bend (in wall)
43+95	Bad spall with exposed and rusted reinforcing bar
44+08	Crack in vertical wall
44+21	Bend (in wall), badly spalled joint
43+25	1½" vertical crack in toe of apron
43+64	Bend (in wall), joint spalling
44+00	½" crack at toe of footer
44+21	Bend (in wall), 1" crack at footer
44+30	3" vertical crack at footer, crack with spalling
44+85 thru 45+20	Shows cracks
45+40	Crack
45+60	Crack
45+70	Crack with spall
45+90	Crack
46+05	Crack with spall
46+20	Heavy crack with deep spalling
46+35	Crack
46+45	Crack with spall
46+55	Crack
46+65	Crack
46+75	Crack
46+90	Large crack with large spall
47+03	Pipe, large crack
47+95	Large crack with spalling

SEAWALL 1724

STATION (S)	OBSERVATION
48+16	Joint spalled with seal gone
48+26	Bend (in wall)
48+55	Joint with bad spalling and displacement, Level II surface good, slight scaling mid-water, moderate at bottom
48+75	Drain(age) trough with cracks
48+95	Bad joint, badly spalled and displaced
49+25	Joint spalled with exposed steel
49+30	Joint, approximately 2" seperation, seal gone, approximately 4" vertical settlement between slabs at toe
49+50	Patch, hairline crack
49+70	Spall and crack, spalls and cracks, reinforcing bar protruding
48+95 to 50+00	Transverse cracks (on concrete apron), gap between footer and apron
50+10 thru 51+60	Gap approximately 1" between footer and slab, slab on grade, apron
50+10	Joint, very bad spall, seal gone
50+22	Bend (in wall), hairline (crack), spall
50+50	Joint, seal gone, slight joint spall
50+80	Hairline (cracks)
50+90	Joint seal gone, small spall
51+10	Hairline (crack)
51+25	Hairline (crack), joint seal bad and spalled from waterline
51+50 to 51+60	Hairline (cracks), transverse hairline cracks
52+25	Heavy crack with spalled cap and horizontal displacement of approximately ½", pipe sealed
52+30	Tongue-and-groove (joint), horizontal and vertical displacement approximately 1"
52+40	Hairline (crack)
52+50	Platform failed, hairline (crack)
52+80	Pile supported platform in poor condition, piles and pile cap in good condition, spalling of platform midsection, stringers, reinforcing bars exposed and corroded
53+08	Cracked and patched cap, spall, hairline (crack)
53+15	Hairline (crack)
53+25	Diagonal crack, pipe filled with concrete, spall and scaling <u>NOTE</u>: bottom of wall may be exposed here
53+50	Hairline (cracks)
53+60	Joint spall and heavy scaling
53+75	Hairline (cracks)
53+90	Light scale
54+00	Hairline (crack)
54+10	Hairline (crack)
54+25	Tongue-and-groove (joint), joint spall 1' off bottom
54+35	Hairline (cracks)

SEAWALL 1724

STATION (S)	OBSERVATION
54+46	Cracked cap with vertical and horizontal cracks on face
54+50	(Horizontal) crack, hairline (cracks)
54+85	Tongue-and-groove (joint), 1" vertical displacement of wall, joint spall
55+00	Cap broken with exposed broken reinforcing bar
55+15	Bend (in wall), light scaling, hairline (cracks), pipe
55+40	Moderate crack through wall with material missing from cap and exposed broken reinforcing bar, pipe
55+50	Hairline (crack)
56+00	Moderate crack with material missing from cap and rusted reinforcing bar exposed, pop-out
56+60	Hairline (crack)
56+65	Spall, moderate scaling
56+75	Tongue-and-groove (joint), approximately 2" horizontal displacement of wall
57+00	Hairline (cracks)
57+10	Bend (in wall) corner of cap cracked off
57+20	Hairline (crack), pipe
57+30	Hairline (crack)
57+40	Tongue-and-groove (joint) seal fair
57+50	Hairline (crack), Level II cleaning done here
57+62	Hairline (crack)
57+75	Tongue-and-groove (joint), seal bad
57+90	Hairline (crack)
58+01	Tongue-and-groove (joint) O.K.

SEAWALL 1724

STATION (at centerline of ramp)	RAMP OBSERVATIONS
36+70	Ramp surface shows transverse and lateral cracks along east side, cap is missing from approximately 10' down ramp to waterline. East side of ramp severely spalled, cracked and scaling. At waterline, joint surface of ramp is severely cracked with sections missing. Hole exists underwater on east side. Submerged east side of ramp is O.K. End of ramp O.K. West submerged side of ramp is buried to about 3' of water, rest of submerged portion appears O.K. West side above waterline has cap section missing from waterline up 15'; rest of cap is cracked and/or displaced. West side of ramp is moderately spalled with cracks and scaling.
38+70	Ramp surface shows transverse cracks or spalled joints about every 10'. East cap spalled at about mid-distance down ramp. East cap running from mid-ramp to water spalled, cracked and scaling (moderate to severe). Section of cap missing from just above high water line to just below. Spalling, cracks and scaling stops about 15' down ramp into water. South end of ramp appears O.K., west side below water appears O.K. West side above water severe cracks, spalling and scaling are present. At mid-ramp, cap section is displaced from side of ramp approximately 2".
40+88	Surface of ramp shows many transverse and lateral cracks, joint just above high water line shows spalling. West side of ramp in water shows deep scaling and spalling down to waterline. Ramp and apron separated by approximately 4". Cap spalled with exposed rebar 25' from entry. Portion of west ramp below water shows minor spalling in shallow water. Deep water portion in good shape. East deep water wall portion of ramp O.K. At entrance to water cap is broken and cracked. Above water cap shows cracks and spalling with rebar exposed in two locations.
45+31	Cracking on north and south faces with spalls and scaling, footer at toe has been undermined at east end, slab of ramp has transverse cracks where east and west faces are most deteriorated.

SEAWALL 1724

STATION (at centerline of ramp	RAMP OBSERVATIONS
47+57	Good condition below waterline except for 1 vertical hairline crack and 1 diagonal crack approximately ¼" wide along east face of retaining wall. Heavy scaling above waterline and joint spall on top slab, there are 2 slabs, 1 over the other.
51+82	Footer at toe in good condition, cap missing at corner.

PENSACOLA NAVAL AIR STATION
PIER 3238

BENT	OBSERVATION
1	O.K.
2	Spall with rebar exposed at top of west pile
3	Crack on both north and south faces of cap along bottom; level II inspection performed on east pile - O.K. except for minor scaling
4	Minor rust stain in bottom of cap
5	O.K.
6	Minor spalling at cap, pile joint and at end of cap at west end
7	Minor rust stain on top of east pile; spalling and rust at top of cap above west pile
8	North face of cap has crack running between piles
9	Bottom edge of pile cap at south end has broken away, minor rust on east face of cap
10	Top corner at south edge of pile cap has broken away; east face of cap has spall with rust; crack in bottom of pile cap above north pile; level II inspection performed on north pile - O.K.
11	Rust stains surround a rebar protruding from the bottom of the cap between the piles - rebar appears to have been cut-off; cracks noted in cap above north pile
12	Hairline at northwest corner of north pile near top of pile; hairline at southeast corner of south pile near top of pile
13	South face of cap has a severe crack; severe spall and exposed and rusted rebar at north end of pile cap
14	Severe longitudinal cracks along bottom of cap between piles; spall with exposed steel at north-east corner of cap
15	Severe spall at south face and north face of cap with exposed and corroded steel; crack and rust stains along bottom of cap between piles; severe spall at top of south pile
16	Cap appears very similar to cap of bent 15 only worse with badly exposed steel
17	Crack along bottom of cap between piles; cap cracking at bottom, above north pile
18	Crack between piles in bottom of cap on east face of pile cap with minor rust stains
19	Minor crack with rust stains in bottom of pile cap above west pile
20	Minor rust stains and spalling along bottom of pile cap between piles; level II performed on west pile - O.K.

21	North and south batter piles - O.K.
Pile Cluster (at end of descent cable)	O.K.

PENSACOLA NAVAL AIR STATION
BULKHEADS 177,178

STATION (S)	OBSERVATION
58+12	Permeable granite block groin
58+35	Level II, granite face very good condition
58+35 thru 60+40	Light layer of barnacles with algae ¼" thick total
58+50	2" (gap) below waterline, level II granite face very good condition
59+50	Mortar loss at cracked edge of block
59+60	Block missing just below waterline
59+68	Top block, bottom patched
59+80	10' chipped section, patched
60+10	Top block cracked along 1/3 of bottom
60+52	Stains, small corner chip
60+75	Loss of 1/3 of bottom of block at top
60+90	Mortar loss at seam, 15' (horizontal), 3" (vertical)
61+10 to 61+20	1' X 6" chip-outs
61+50	8' (horizontal), 3" (vertical), 4" deep chip
62+20	1' deep chip, 4' (horizontal), 3" (vertical)
62+30	1/3 block lost
63+00	Lower ½ of block lost and ½" gap at seam
63+85	Mortar lost at joint
65+00	6" diagonal crack in corner of block
66+00	Small gap, joint below first block
66+80	Pipe outfall
67+30	1½" gap at seam of above waterline
67+75	Small chip in granite
69+00	5' (horizontal) more or less, 2" (vertical), 1' deep mortar loss
69+15	Granite missing at waterline - 5' (horizontal), 4" (vertical), 4" to 5" deep
69+35	2" to 3" gap at joint at waterline
69+70 thru 73+75	General Note: Typical mortar loss 3" deep at joints (most at 1st joint below top of wall). Typical hardware mounts (doweled into granite) are rusted. Bottom pass ½" to 1" marine growth.
69+70	Stairs, 3'X4' blocks missing, photographed one at 4' (below Mean Low Water) and one at 3' above bottom
69+85	Chips in edge of blocks
70+60	Section filled with grout
71+28 to 71+50	Corner (joints) much mortar missing, light mortar up to 2" gap at joints
71+60	Stairs, some sand leaking at 6' below waterline
71+85	Corner mortar loss
72+30	Mortar looks good
72+85	Minor rust stains
73+10	Looks good below waterline

BULKHEADS 177,178

STATION (S)	OBSERVATION
73+30	Level II, 1½" gap in block, Level II looks good everywhere
73+35 to 73+65	Mortar loss first 3 seams
73+75	Large section of mortar loss and cracks and chipped granite
74+00	2' horizontal crack
75+00	Corner, mortar missing under upper level of blocks
75+15 to 76+60	Okay above waterline
76+60	Granite block wall ends 15' after 10th 6 pile bent at end of granite where it meets sheet pile, below waterline big cavity approximately 2' oval with 2' to 3' depth of penetration

PENSACOLA NAVAL AIR STATION
PIER 303

STATION (S)	OBSERVATION
76+60 thru 83+40	General Note: Typical defects along cap are spalls along edge, along wall are hairline cracks every 100' or so
83+25	1' vertical X 4" wide (hole) in sheet pile 3' below cap
83+35	Reinforcing bars protruding from cap below waterline
83+65	Loose extra fender pile
88+00 to 89+60	Typical conditions, marine growth absent on oxidized areas, film easily removed, when removed cleaned smooth section is visible, minor rust stains
89+60	Broken fender
90+00	Minor rust, ¼" to 1" barnacle and algae typical, nothing unusual
90+65	7 sheets (with) 2 holes in web, fill leaking
83+50 thru 91+28	Barnacles and rust stains along seam of sheet pile, green (algae) coating along face typical

PENSACOLA NAVAL AIR STATION
FINGER PIER 303

BENT	OBSERVATION
1	Level II performed on north pile - piles O.K.
2	Piles O.K.
3	O.K. - Level II performed at north pile - O.K.
4	Slight rust stains noted at bottom of piles
5	O.K.; Level II performed at north pile
6	O.K.
7	Level II performed at north pile; piles O.K.
8	O.K.
9	O.K. - Level II performed at north pile
10	O.K.
11	O.K. - Level II performed at north pile
East Bollard Cluster	Piles appear in good condition - Level II performed at northeast batter pile and southwest batter pile at 2'+ below encasement. Corrosion was noted in northeast batter pile - pitting was also noted.
West Bollard Cluster	Piles appeared in good condition - Level II performed on northeast and southwest piles, steel appeared in good condition; northwest batter was bent, appears to have been hit, concrete encasement is shattered below cap.

PENSACOLA NAVAL AIR STATION
PIER 302

STATION (S)	OBSERVATION
91+65	4' hairline (crack)
91+80	Spall
91+90	Hairline (crack)
92+08	Joint, some seal missing above waterline
92+25	2.5" X 6' spall
92+75	5'X6"X6" deep (spall)
92+85	(Two cracks), 4' wide X 2" deep
93+00	Scaling, spall, rust stains
93+05	Joint
93+25	(Small spall at bottom of encasement)
93+50	Hairline (crack), minor rust
93+60 to 93+70	Concrete encasement, spall, minor rust
94+00	Minor rust
94+10	Joint, 5' vertical spall
94+40	Minor spall
94+75	Chip at base
95+00	Minor rust
95+05	Joint seal patched with grout
95+50	Small hole
95+75	3' (vertical) 1" (wide) cracks
96+05	Joint, spall
96+25	1" wide crack, 4'X6" (spall)
96+50	10' long 1" wide crack
96+80	10' crack
96+90	2'X1" crack
96+65	Exposed reinforcing bar, 4" wide crack
97+09	Face patched with grout- 2' diagonal cracks
97+40	15' (wide) minor spall
97+62	Spall
97+74	4' exposed steel beam
97+55	15' minor spall
98+35	4' hairline (crack), 8' crack, 2" wide
98+45	8' X 2" wide crack
98+70	Joint, seal missing, 10"X2" deep (crack), 6" above (bottom of encasement)
98+75	2" diameter hole, 2" deep
98+95	Hairline (crack), 10"X½" deep (crack) 4" above (bottom of encasement)
99+10	Cavity at fender mount
99+50	4' (wide) X 3" minor spall
99+70	Minor spall
99+90	4' minor spall
100+65	Joint, seal does not fill joint
100+80	Minor spall
100+90	2' hairline (cracks)
101+20	6' X 2" wide crack
101+50	Minor rust

PIER 302

STATION (S)	OBSERVATION
101+65	Minor rust stains
101+75	2' X 3" minor spall
101+98	2" diameter hole approximately 3" deep, 8'X3" wide crack 4' (above bottom of encasement)
102+30	3' chipped section with exposed reinforcing bar
102+50	Minor spall
102+70	Joint, seal does not fill joint, minor spall 2' (above bottom of encasement)
102+95	2" crack, 15' (wide), 4" (above bottom of encasement)
103+05	Vertical hairline (crack), minor spall
103+40	Hairline (crack)
103+50	End of wall

PENSACOLA NAVAL AIR STATION
Bulkhead 708B

STATION (S)	OBSERVATION
0+05	Severe scaling
0+20	Level II - O.K.
0+25	Hairline (crack)
0+30	Hairline (crack)
0+35	Hairline (crack)
0+40	Hairline (crack)
0+48	Tongue-and-groove joint, 1½" gap in joint
0+60	Hairline (cracks), slab behind wall appears to have settled
0+85	¼" cracks
0+90 - 1+10	Wood footer at toe
1+10	Stairs, tongue-and-groove joint, 1½" gap
1+30	Hairline (cracks)
1+50	Hairline (cracks)
1+70	Tongue-and-groove (joint), wall displaced 1½" at joint
2+00	Ramp
2+50	Hairline (crack)
2+60	Hairline (crack)
2+65	Wood footer or old form
2+90	Tongue-and-groove joint
3+20	Level II - O.K.
3+48	Tongue-and-groove joint, 1" gap, sand at bottom, no seal
4+10	Tongue-and-groove joint, no seal
4+35	Clean sand, prop wash - NOTE: Sand noted in many areas by boats, assumed caused by prop wash - no undermining apparent
5+40	Hairline (crack)
5+85	Tongue-and-groove joint
6+30	Hairline (crack), Level II - O.K.
6+35	Hairline (crack)
6+65	Level II, spall at top
6+80	Minor spalling
7+00	Hairline (crack), minor spalling
7+25	Hairline (crack)
7+35	Hairline (crack)
7+45	Hairline (crack)
7+70	Joint mortar missing
7+80	Hairline (crack)
8+00	Hairline (crack)
8+10	Hairline (crack)
8+40	Hairline (crack)
8+50	Hairline crack runs all the way down, oyster growth, sand bottom
8+70	Hairline crack to bottom
8+80	Hairline crack to bottom, minor pop-out, little steel exposed
8+85	Sink, tongue-and-groove
9+10	Hairline crack to bottom
9+20	Smooth concrete, no spalling

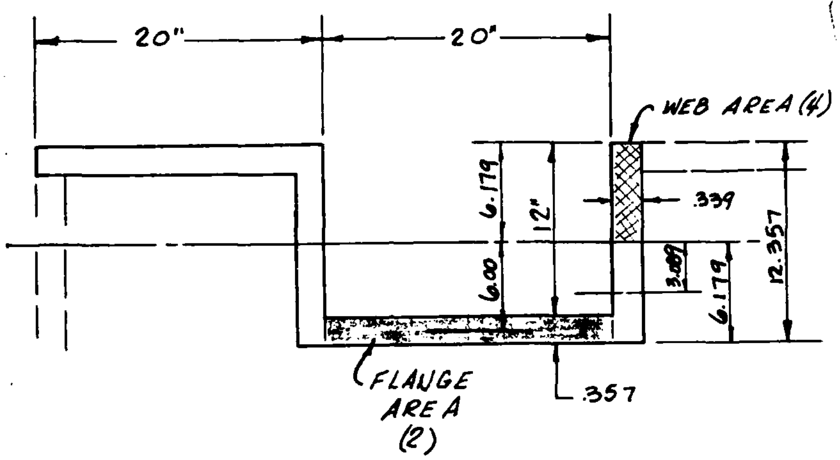
BULKHEAD 708B

STATION (S)	OBSERVATION
9+30	Hairline (crack) to bottom
9+35	Hairline crack, sand bottom, oyster growth
9+45	Seam material falling out
9+60	Hairline (crack)
9+70	Hairline (crack) from top to 10' above waterline
10+25	Hairline (crack), small pop-out, minor rust stains
10+35	Hairline (crack) all way (top of cap to mudline)
10+45	Hairline (crack) all way (top of cap to mudline)
10+85	Hairline crack, exposed steel
10+90	Hairline (crack) to waterline
11+00	Hairline (crack) all way (top of cap to mudline)
11+10	Hairline (crack)
11+15	Pop-out exposed steel
11+45	Hairline (crack)
11+50	Hairline (crack)
11+55	Hairline (crack)
11+60	Hairline (crack)
11+75	Hairline (crack)
12+00	Sink hole
12+15	Hairline (crack)
12+25	Good condition, hairline (crack)
12+45	Sink hole, tongue-and-groove (joint), minor pop-out, soft bottom, hairline (crack)
12+65	Hairline (crack)
12+80	Hairline (crack)
12+90	Hairline (crack)
13+25	Hairline (crack)
13+35	Hairline (crack)
13+40	Minor pop-out
13+50	Hairline (crack)
13+75	Hairline (crack)
-0+10 to -0+20	Hairline (cracks)
-0+30	Tongue-and-groove joint, seal missing
-0+45	Patched hairline (crack)
-0+55	Hairline (crack)
-0+65	Hairline (crack)
-0+90	Tongue-and-groove joint, no seal
-1+50	Tongue-and-groove joint, some seal remains
-1+75	Hairline (crack)
-1+85	Hairline (crack)
-2+10	Joint
-2+30	Hairline (crack)
-2+50	Hairline (cracks)
-2+75	Joint
-3+10	Hairline (crack), drainage pipe, sink hole behind wall
-3+29	End of wall

NOTE: At station -3+29 concrete wall ends, wood wall begins. Wood wall appears in fair to poor condition. At station -4+00 hole exist in wood wall (3'X2'). Beyond hole, it appears the wood wall supports minimum over-burden, less than 1 foot.

APPENDIX C

STRUCTURAL
EVALUATION
NOTES



$$\text{WEB AREA} = \frac{(12.357 \times .339)}{2} = 2.095 \text{ Square inches}$$

$$\text{FLANGE AREA} = (20 \times .357) = 7.140 \text{ Square inches}$$

$$I = 2\left[7.140 \text{ Sq. In.} \times (6 \text{ in.})^2\right] + 4\left[2.095 \text{ Sq. In.} \times (3.089 \text{ in})^2\right]$$

$$= 514.080 \text{ In.}^4 + 79.974 \text{ In.}^4$$

$$= \underline{594.054 \text{ In.}^4}$$

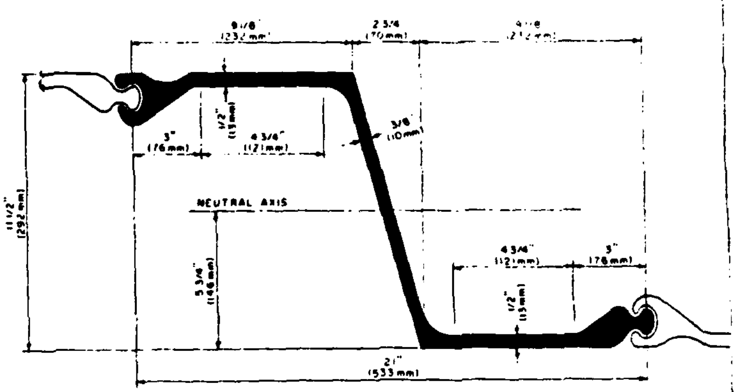
$$S = \frac{I}{C}\ ; \quad C = \frac{12.36}{2} = 6.18; \quad I/\text{Ft.} = 594 \text{ In.}^4 / \left(\frac{40}{12}\right)$$

Implies $S = 178.2 \text{ In.}^4 / 6.18 \text{ In.}$ $\qquad I = 178.2 \text{ In.}^4$

$S/\text{Ft.} = 28.83 \text{ In.}^3 \equiv S_1$

PIER 303 - "OLD" STEEL SHEET PILES
SECTION PROPERTIES

PZ32



SHAPE	DRIVING DISTANCE PER PILE	THICKNESS WEB	THICKNESS FLANGE	WEIGHT Per Lineal Foot of Pile	WEIGHT Per Square Foot of Wall	SECTION MODULUS Per Pile	SECTION MODULUS Per Foot of Wall	AREA	MOMENT OF INERTIA	SURFACE AREA Incl. Inter-lock	SURFACE AREA Coat-ing Area*
	INCHES	INCHES	INCHES	POUNDS	POUNDS	$INCHES^3$	$INCHES^3$	$INCHES^2$	$INCHES^4$	FT^2/FT	FT^2/FT
PZ38	18	3/8	1/2	57.0	38.0	70.2	46.8	16.8	421	5.52	5.06
PZ32	21	3/8	1/2	56.0	32.0	67.0	38.3	16.5	386	5.52	5.06
	mm	mm	mm	MASS Per Lineal m of Pile kg	MASS Per Square m of Wall kg	Per Pile $mm^3 \times 10^3$	Per m of Wall $mm^3 \times 10^3$	$mm^2 \times 10^3$	$mm^4 \times 10^6$	m^2/m	m^2/m
PZ38	457	10	13	84.8	186	1150	2520	10.8	175	1.68	1.54
PZ32	533	10	13	83.3	156	1100	2060	10.6	161	1.68	1.54

FROM: "STEEL SHEET PILING HANDBOOK"
by United States Steel (USS) Co.

PIER 303 - "OLD" STEEL SHEET PILES
ORIGINAL SECTION PROPERTIES

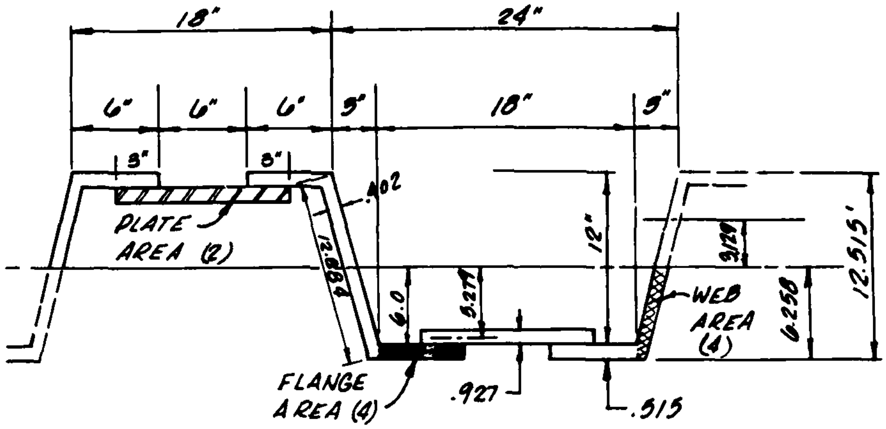


PLATE AREA = (12 X .927) = 11.124 Square inches

FLANGE AREA = (6 X .515) = 3.090 Square inches

WEB AREA = $\frac{(12.884 \times .402)}{2}$ = 2.590 Square inches

$$I = 2\left[11.124 \text{ Sq. In.} \times (5.279 \text{ in.})^2\right] + 4\left[3.090 \text{ Sq. In.} \times (6 \text{ in.})^2\right] + 4\left[2.590 \times (3.129 \text{ in.})^2\right]$$

$= 620.004 \text{ in.}^4 + 444.960 \text{ in.}^4 + 101.431 \text{ in.}^4$

$= \underline{1166.395 \text{ In.}^4}$

$I/\text{ft.} = \frac{1166.4 \text{ in.}^4}{3.5'} = \frac{333 \text{ in.}^4}{\text{ft.}}$

and $S/\text{ft.} = \frac{333 \text{ in.}^4}{(12.5''/2)} = 53.3 \text{ in.}^3/\text{ft.}$

PIER 303 - "NEW" STEEL SHEET PILE
SECTION PROPERTIES

Date : 2/28/83 Run - 1

Comm # : 745801
Jobname : PENSACOLA NAVSTA
By : RIK
PIER 303

Data Specified as Input

Cap Elevation = 10.7'
Berm Elevation = -30'
Water Surface Elevation = -1'
Tierod Elevation = 6'
Pile Spacing = 1'
Pile Face = 1'
Saturated Unit Weight of Soil = 120 pcf
Unit Weight of Water = 64 pcf
Active Soil Pressure Coeff (Ka) = .3
Passive Soil Pressure Coeff (Kp) = 3
Distance below Berm to Point of Contraflexure 4'

Design Requirements (Output)

F1 =	-2464	A1 =	3.10	M1 =	-7638
F2 =	-12215	A2 =	21.50	M2 =	-262618
F3 =	-7064	A3 =	26.33	M3 =	-186029
F4 =	-2729	A4 =	38.00	M4 =	-103702
F5 =	0	A5 =	42.01	M5 =	0
F6 =	0	A6 =	48.09	M6 =	0
F7 =	11174	A7 =	50.11	M7 =	559988

Sum of Moments = 0

Elevation of Equal Active-Passive Pressure = -36.01'
Required Tieback Force = 13298#
Minimum Required Pile Length = 58.9'
(Sheet Piling)

Maximum Moment (flexible pile) = 127127 #
Elevation of Maximum Moment = -15.71'

Maximum Moment (stiff pile) = 194106 #
Elevation of Maximum Moment = -19.73'

PIER 303 - COMPUTER EVALUATION

S old = 28.83 in.^{3}/ft. (approximation, based on measurements)

S new = 53.3 in.^{3}/ft. (approximation, based on measurements)

From computer output M = 127 K'

Assume Fy = 36 ksi

and Fa = 21.6 ksi = .6 fy (Ref. AISC Manual)

$\Rightarrow$ S req'd. = M/Fa = $\frac{127k' * 12''/ft.}{21.6 \text{ ksi}}$

$\Rightarrow$ S req'd. = 70.6 in.^{3}

$\Rightarrow$ SSP wall is not carrying full soil load - gravity structure carries part of load i.e. acts as a relieving platform *

$$\Delta S/\% = \frac{38.3 \text{ in.}^3 - 28.3 \text{ in.}^3}{38.3 \text{ in.}^3} \times 100\% = 26\%$$

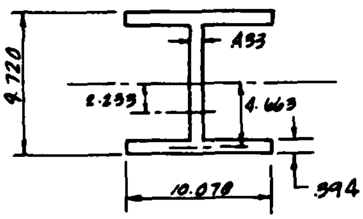
* Detailed analysis beyond the scope of task

BEARING PILES

BENTS 1,3,5,7,9,11 (10 X 10's) AVG. F .425 W .431

W. BOLL., E. BOLL., SW PILE (14 X 14's)
AVG. F .488 W .476

10x 10



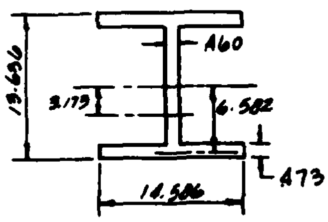
Area Web=$\left(\frac{8.923'' \text{X} .443''}{2}\right)$=1.978 Sq. In.

Area Flange = (10.078" X .394")
= 3.971 Sq. In.

I= 2 [1.978 Sq. In. X (2.233 in.)^2]
+ 2 [3.971 Sq. In. X (4.663 in.)^2]
= 19.726 in.^{4} + 172.687 in.^{2}

10 X 10 I= 192.413 in.^{4}

14x 14



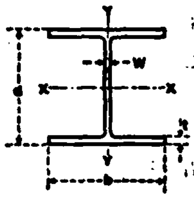
Area Web=$\left(\frac{12.690'' \text{ X } .460''}{2}\right)$=2.919 Sq. In.

Area Flange = (14.586" X .473")
= 6.899 Sq. In.

I=2 [2.919 Sq. In. X (3.173 in.)^2]
+ 2 [6.899 Sq. In. X (6.582 in.)^2]
= 58.758 in.^{4} + 597.767 in.^{4}

14 X 14 I=656.525 in.^{4}

FINGER PIER 303 - BEARING PILE SECTION PROPERTIES



H BEARING PILES

DIMENSIONS AND PROPERTIES FOR DESIGNING

Section Number and Nominal Size	Weight per Foot	Area A	Depth d	Flange Width b	Flange Thickness t	Web Thickness W	Axis X-X I	Axis X-X S	Axis X-X r	Axis Y-Y I'	Axis Y-Y S'	Axis Y-Y r'
	Lb.	In.²	In.	In.	In.	In.	In.⁴	In.³	In.	In.⁴	In.³	In.
	117	34.44	14.234	14.885	.805	.805	1228.5	172.6	5.97	443.1	59.5	3.59
BP 14	102	30.01	14.032	14.784	.704	.704	1055.1	150.4	5.93	379.6	51.3	3.56
14x14½	89	26.19	13.856	14.696	.616	.616	909.1	131.2	5.89	326.2	44.4	3.53
	73	21.46	13.636	14.586	.506	.506	733.1	107.5	5.85	261.9	35.9	3.49
BP 12	74	21.76	12.122	12.217	.607	.607	566.5	93.5	5.10	184.7	30.2	2.91
12 x 12	53	15.58	11.780	12.046	.436	.436	394.8	67.0	5.03	127.3	21.2	2.86
BP 10	57	16.76	10.012	10.224	.564	.564	294.7	58.9	4.19	100.6	19.7	2.45
10 x 10	42	12.35	9.720	10.078	.418	.418	210.8	43.4	4.13	7 [illegible]	14.2	2.40
BP 8 8 x 8	36	10.60	8.026	8.158	.446	.446	119.8	29.9	3.36	40.4	9.9	1.95

AMERICAN INSTITUTE OF STEEL CONSTRUCTION

FINGER PIER 303
AISC MANUAL SPECIFICATIONS

Check Section Modulus

10 X 10 $S=I/C= \dfrac{192 \text{ in.}^4}{(9.72 \text{ in.}/2)} \Rightarrow S=39.5 \text{ in.}^3$

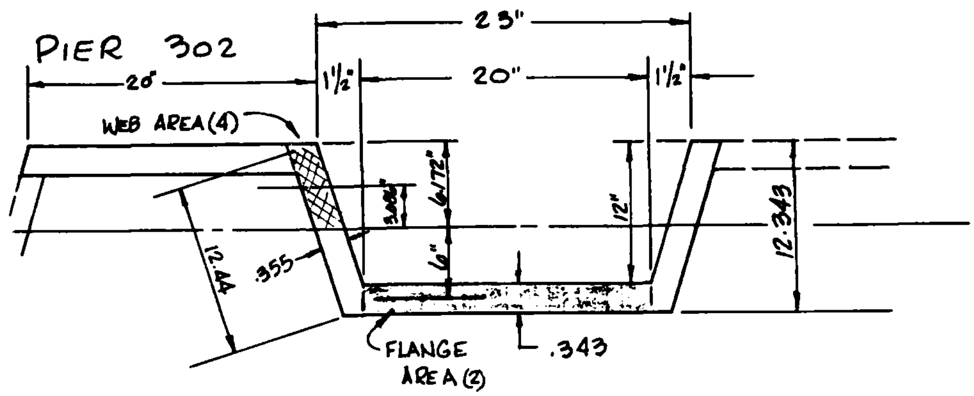
$S_o = 43.4 \text{ in.}^3 \Rightarrow \Delta S/_{\%}= \dfrac{43.4 \text{ in.}^3 - 39.5 \text{ in.}^3}{43.4 \text{ in.}^3} \times 100\%$

$\Rightarrow \Delta S/_{\%}= 9\%$

14 X 14 $S=I/C= \dfrac{656.5 \text{ in.}^4}{13.636 \text{ in}/2} \Rightarrow S=96.3 \text{ in.}^3$

$S_o = 107.5 \text{ in.}^3 \Rightarrow \Delta S/_{\%}= \dfrac{107.5 \text{ in.}^3 - 96.3 \text{ in.}^3}{107.5 \text{ in.}^3} \times 100\%$

$\Rightarrow \Delta S/_{\%}= 10.4\%$



$$\text{WEB AREA} = \frac{(12.44 \text{ X } .355)}{2} = 2.208 \text{ Sq. In.}$$

FLANGE AREA = (20 X .343) = 6.860 Sq. In.

$$I = 2\left[6.860 \text{ Sq. In. X } (6 \text{ in.})^2\right] + 4\left[2.147 \text{ Sq. In. X } (3.086)^2\right]$$

$= 493.920 \text{ In.}^4 + 81.784 \text{ In.}^4$

$= \underline{575.707 \text{ In.}^4}$

$\Rightarrow$ I/Ft. = 576 in.^{4}/3.58' = 161 in.^{4}/ft.

$$\text{and} \Rightarrow \text{S/Ft.} = \frac{161 \text{ in.}^4/\text{ft}}{(12.343''/2)} = 24.7 \text{ in.}^3/\text{ft.}$$

From Computer Output M=145K', assume fa=21.6 ksi

$$\Rightarrow \text{S req'd.} = \frac{145\text{K}'*12''/1}{21.6 \text{ ksi}} = 80.6 \text{ in.}^3/\text{Ft.}$$

⇛SSP wall is not carrying full soil load - gravity structure carries part of load i.e. acts as a relieving platform *

* Detailed analysis beyond the scope of task

PIER 302 - SECTION PROPERTIES AND EVALUATION

Date : 2/28/83 Run - 2

Comm # : 745801
Jobname : PENSACOLA NAVSTA
By : RIK
PIER 302

Data Specified as Input

Cap Elevation = 10.7'
Berm Elevation = -31'
Water Surface Elevation = -1'
Tierod Elevation = 8'
Pile Spacing = 1'
Pile Face = 1'
Saturated Unit Weight of Soil = 120 pcf
Unit Weight of Water = 64 pcf
Active Soil Pressure Coeff (Ka) = .3
Passive Soil Pressure Coeff (Kp) = 3
Distance below Berm to Point of Contraflexure 4'

Design Requirements (Output)

F1	=	-2464	A1	= 5.10	M1	=	-12567
F2	=	-12636	A2	= 24.00	M2	=	-303264
F3	=	-7560	A3	= 29.00	M3	=	-219240
F4	=	-2831	A4	= 41.04	M4	=	-116170
F5	=	0	A5	= 45.12	M5	=	0
F6	=	0	A6	= 51.46	M6	=	0
F7	=	12156	A7	= 53.57	M7	=	651240

Sum of Moments = 0

Elevation of Equal Active-Passive Pressure = -37.12'
Required Tieback Force = 13335 #'
Minimum Required Pile Length = 60.5'
(Sheet Piling)

Maximum Moment (flexible pile) = 145099#'
Elevation of Maximum Moment = -15.52'

Maximum Moment (stiff pile) = 221727#'
Elevation of Maximum Moment = -19.78'

PIER 302 - COMPUTER EVALUATION

PIER 3238 - Catwalk

DEAD LOAD

Deck Units 2(250 in.^{2}/144 □"/□') *20'*150 pcf=10.4K

Pile Cap (6'*1.3'*1.3') * 150 pcf = 1.5K

Piles 2(1'X1'X50') X 150 pcf =15.0K

26.9K

LIVELOAD

100 psf (6'*20') = 12K

⇛ working load - 38.9K/2 piles

∴ 19.5 K/pile $<$10 ton/pile

This is a light load

PILE BEARING CAPACITY

Pier 3238 — Ref NavFac Drawing No. 5068083
Design load is

16"□ — 35 ton, F'_c=5 ksi
35T ≪ (.9) 187K

12"□ — 25 ton estimated, F'_c=5 ksi
25T ≪ (.9) 105K

⇒ no structural limitation
see attached table

Pier 303

18"□ — Design 50 tons, see attached table
50T ≪ (.9) 236K

⇒ OK

Finger Pier 303

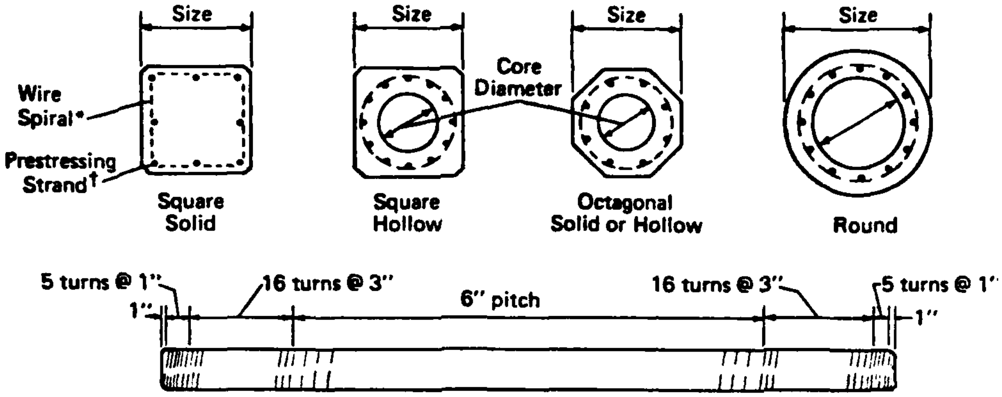
14 BP 73 — Assume K=1.0; l≐50'
$r=\sqrt{I/A}$; I=656.5 in.^{4}; A=2(2.92 in.^{2}+6.9 in.^{2})
⇒A=19.64 in.^{2}
⇒r=5.78 in. ⇒ $kl/r = \frac{50'*12"/ft.}{5.78\ in.} \doteq 104$
From AISC manual Fa=12.47 ksi
⇒Pa=12.47 ksi*19.64 in.^{2}
⇒Pa=245K=122 Tons
Design = 50 Tons ⇒OK

10 BP 42 — Assume K=1.0; l=50'
$r=\sqrt{I/A}$; I=192.4 in.^{4}; A=2(1.98 in.^{2}+3.97 in.^{2})
⇒A=11.9 in.^{2}
⇒$r=\sqrt{I/A}$ = 4.02 in.
⇒ $kl/r = \frac{50'*12"/ft.}{4.02\ in.} = 149$
⇒Fa=6.73 Ksi
⇒Pa=6.73 Ksi * 11.9 in.^{2}
=80.1 K ≐ 40 tons
>20 Tons = Design

PILE LOADS

PILES

Table 3.5.1 Section properties and allowable loads of prestressed concrete piles



*Wire spiral varies with pile size.
†Strand pattern may be circular or square.

Typical Elevation*

Size in.	Core Dia. in.	Section Properties[1]						Allowable Concentric Load, Tons[2] f'_c			
		Area in.^{2}	Weight plf	Moment of Inertia in.^{4}	Section Modulus in^3	Radius of Gyration in.	Perimeter (ft.)	5000	6000	7000	8000
Square Piles											
10	Solid	100	104	833	167	2.89	3.33	73	89	106	122
12	Solid	144	150	1728	288	3.46	4.00	105	129	152	176
14	Solid	196	204	3201	457	4.04	4.67	143	175	208	240
16	Solid	256	267	5461	683	4.62	5.33	187	229	271	314
18	Solid	324	338	8748	972	5.20	6.00	236	290	344	397
20	Solid	400	417	13,333	1333	5.77	6.67	292	358	424	490
20	11"	305	318	12,615	1262	6.43	6.67	222	273	323	373
24	Solid	576	600	27,648	2304	6.93	8.00	420	515	610	705
24	12"	463	482	26,630	2219	7.58	8.00	338	414	491	567
24	14"	422	439	25,762	2147	7.81	8.00	308	377	447	517
24	15"	399	415	25,163	2097	7.94	8.00	291	357	423	488

FROM: PORTLAND CEMENT INSTITUTE HANDBOOK

PILE LOADS

APPENDIX D

REPAIR COST
ESTIMATE
BREAKDOWN

SEAWALL 384

Seal - T & G Joints

10'/joint @ $40/Ft.=$400/joint

~ 21 joints @ $400/joint=$8400

Seal - Prestressed Pile Joints

110'@ $15/ft.=$1650
⇒Total = $1650 + $8400

≐ $10,050, say $10,000

SEAWALL 1824

T & G Joints - 10'/joint @ $40/ft. @ 18 joints
= $7,200, say $7,500

BULKHEAD 177,178

Seal - Estimate 1781' needs to be sealed
⇒1781' @ $40/ft.=$71,240

Pour

@ Station 76+60, 59+60, 60+10, 62+30
⇒ Pour 4 "blocks" @ $4000/block
= $16,000
TOTAL = $87,240, say $90,000

PIER 303

Seal 16 holes
Fabricate and install metal plug
then seal with water epoxy
Materials $100/hole @ 16 holes = $1600
Labor - 2 divers @ $40/hr @ 8hr./day @ 3 day = $1920
$3520

Say $3500

FINGER PIER 303

Mobilization & Demobilization - Crane & Barge	=	$10,000
Break away cap and pull pile (3 days @ $1500/day)	=	4,500
Provide and drive new Pile (80' @ $20/ft.)	=	1,600
Form and place cap and encasement	=	4,000
		$20,100
	Say	$20,000

PIER 302

8 joints @ 15'/joint @ $40/ft.	=	$ 4,800
150' misc. cracks @ $40/ft.	=	6,000
		$10,800
	Say	$11,000

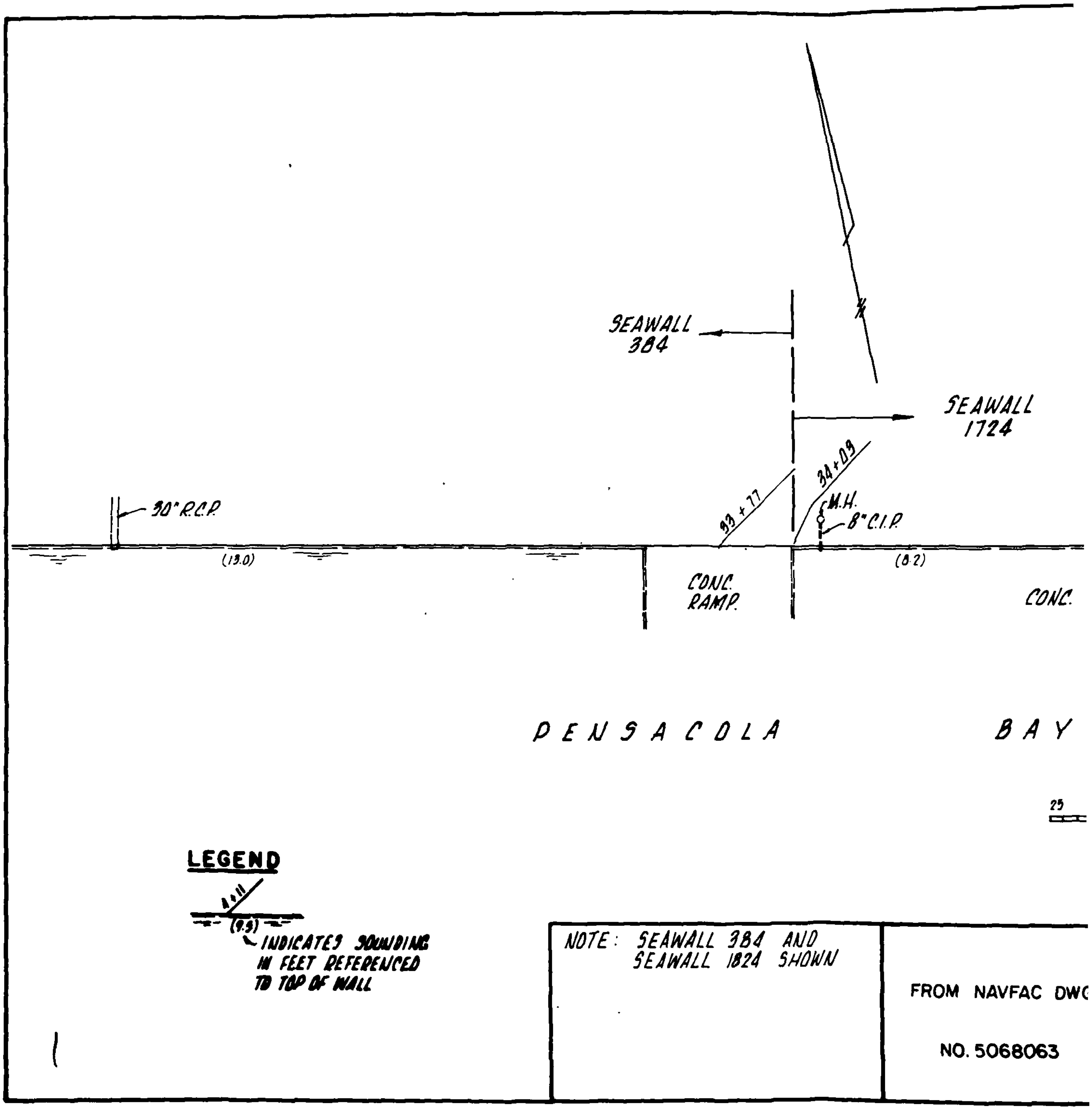
BULKHEAD 708B

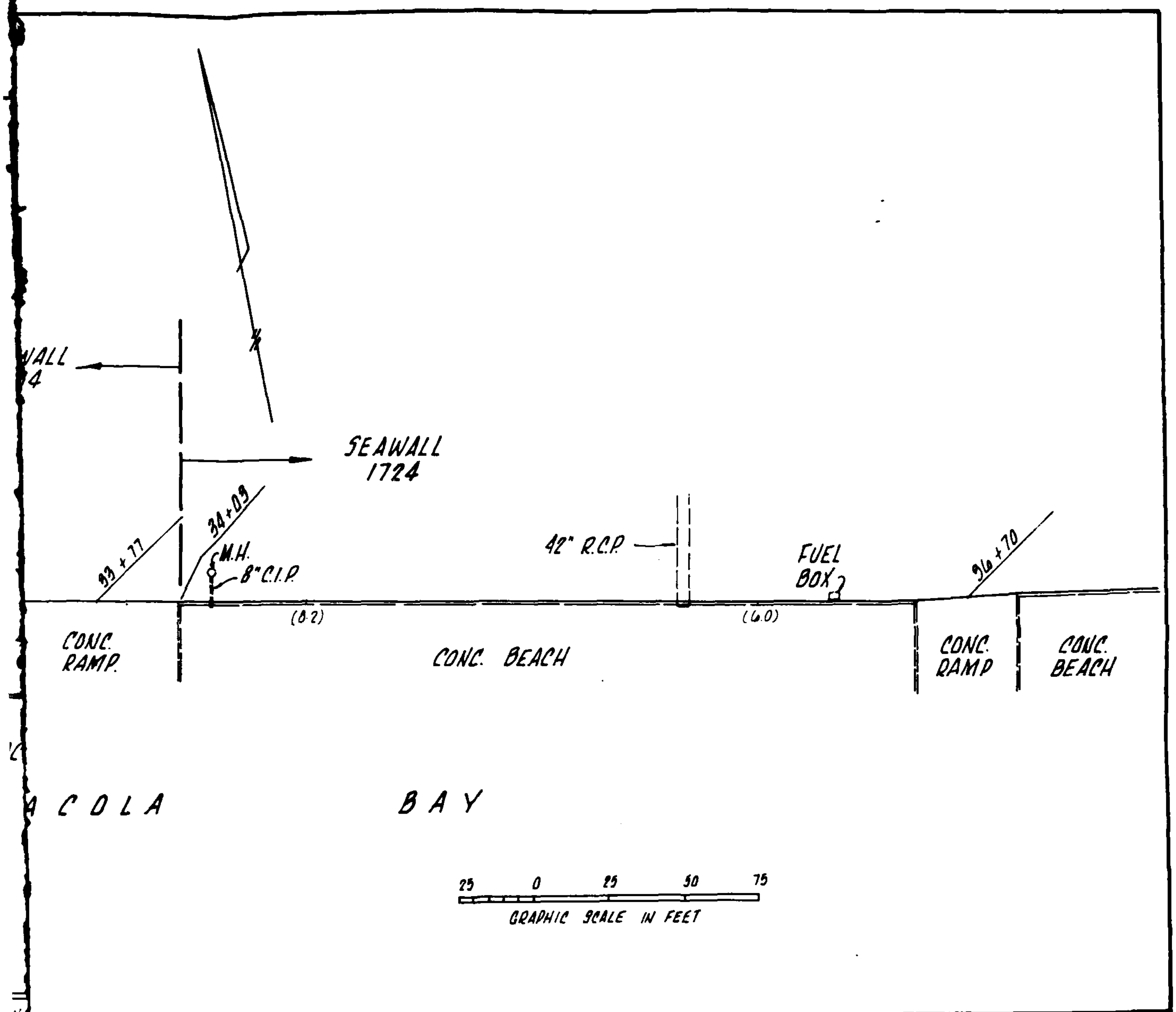
15 T & G joints @ 15'/joint @ $40/ft.	=	$ 9,000
	Say	$10,000

PIER 3238

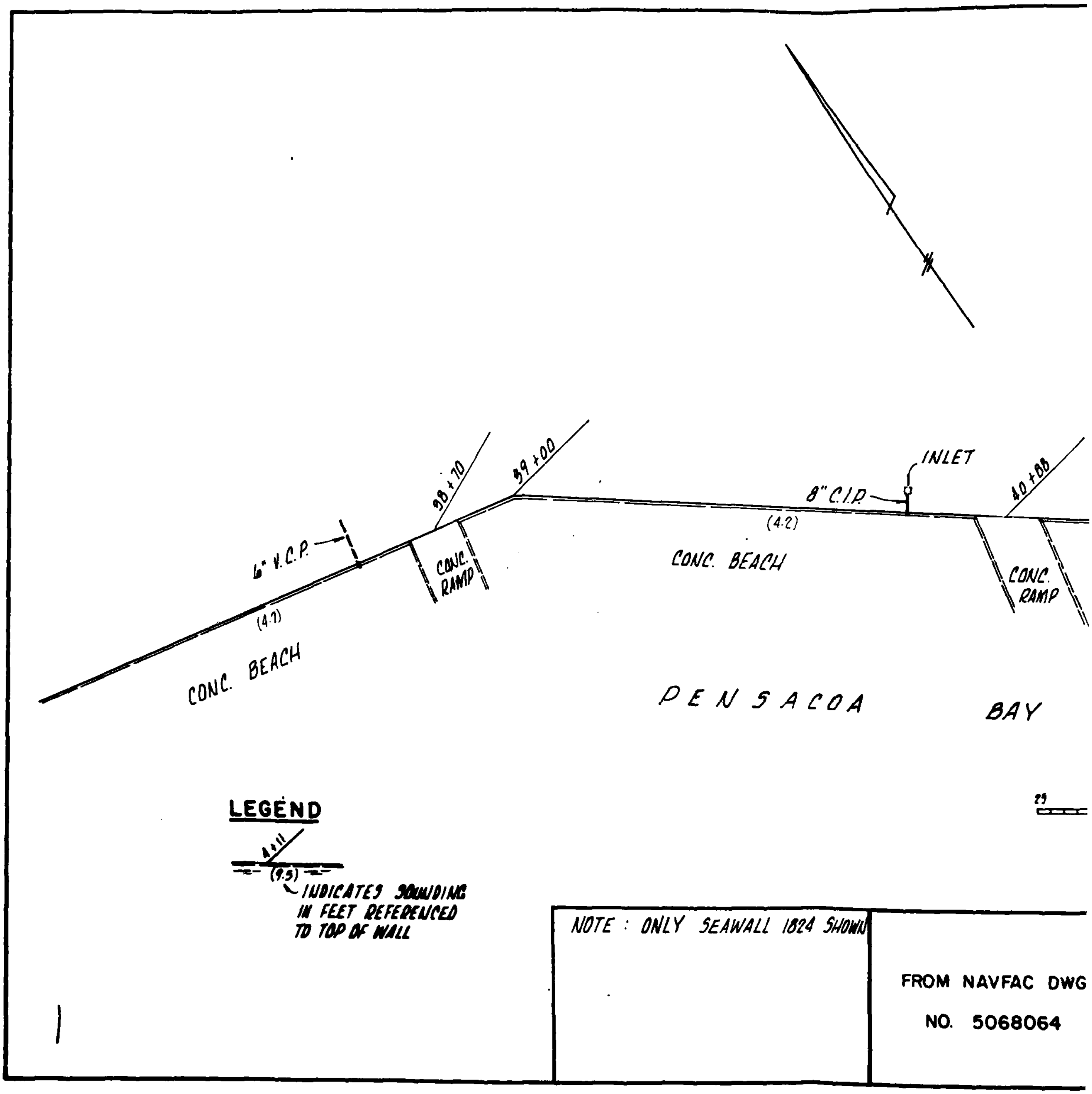
Repair Cap (6'X1.3')/face @ 3 $\frac{\text{faces}}{\text{cap}}$ @ $30/sf = $702/cap

Mobilization and Demobilization	=	$ 5,000
15 caps @ $702/cap	=	$10,530
		$15,530
	Say	$15,500

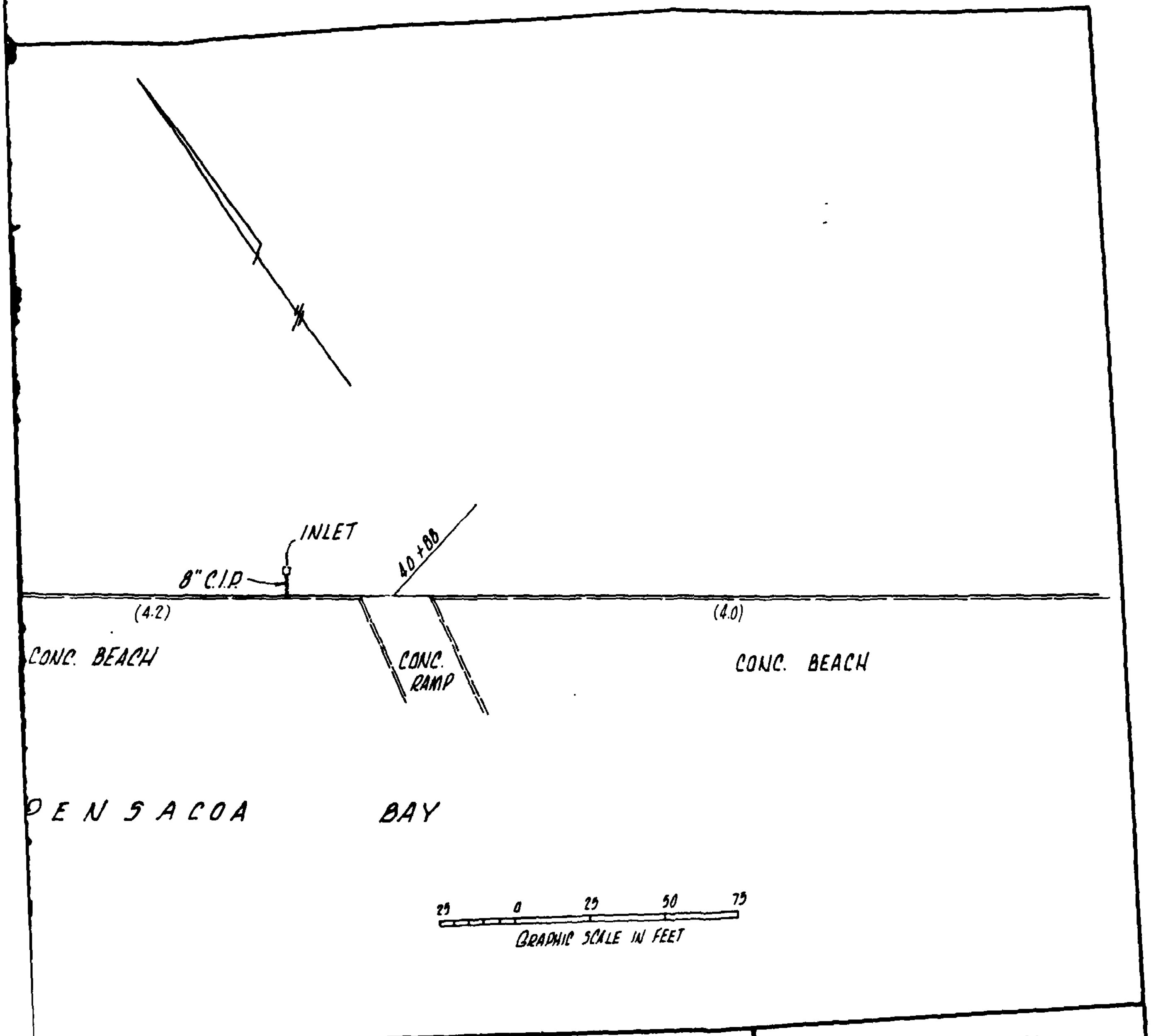

SEAWALL 384
SEAWALL 1724
33 + 77
34 + 09
M.H.
8" C.I.P.
30" R.C.P.
(13.0)
(8.2)
CONC. RAMP
CONC.
P E N S A C O L A B A Y
LEGEND
(9.3)
INDICATES SOUNDING IN FEET REFERENCED TO TOP OF WALL
NOTE: SEAWALL 384 AND SEAWALL 1824 SHOWN
FROM NAVFAC DWG
NO. 5068063



AWALL 3B4 AND WALL 1824 SHOWN	FROM NAVFAC DWG. NO. 5068063	PREPARED BY ARTHUR V. STROCK & ASSOCIATES, INC. 829 S.E. 9th ST. DEERFIELD BEACH, FLORIDA	CHESAPEAKE DIVISION NAVAL FACILITIES ENGINEERING COMMAND WASHINGTON, D.C. 2 STATIONING PLAN	FIGURE A 6



38 + 70
39 + 00
INLET
8" C.I.P.
40 + 88
6" V.C.P.
(4.1)
(4.2)
CONC. RAMP
CONC. BEACH
CONC. BEACH
CONC. RAMP
PENSACOA BAY
LEGEND
(9.5)
INDICATES SOUNDING IN FEET REFERENCED TO TOP OF WALL
NOTE : ONLY SEAWALL 1824 SHOWN
FROM NAVFAC DWG
NO. 5068064



LY SEAWALL 1824 SHOWN

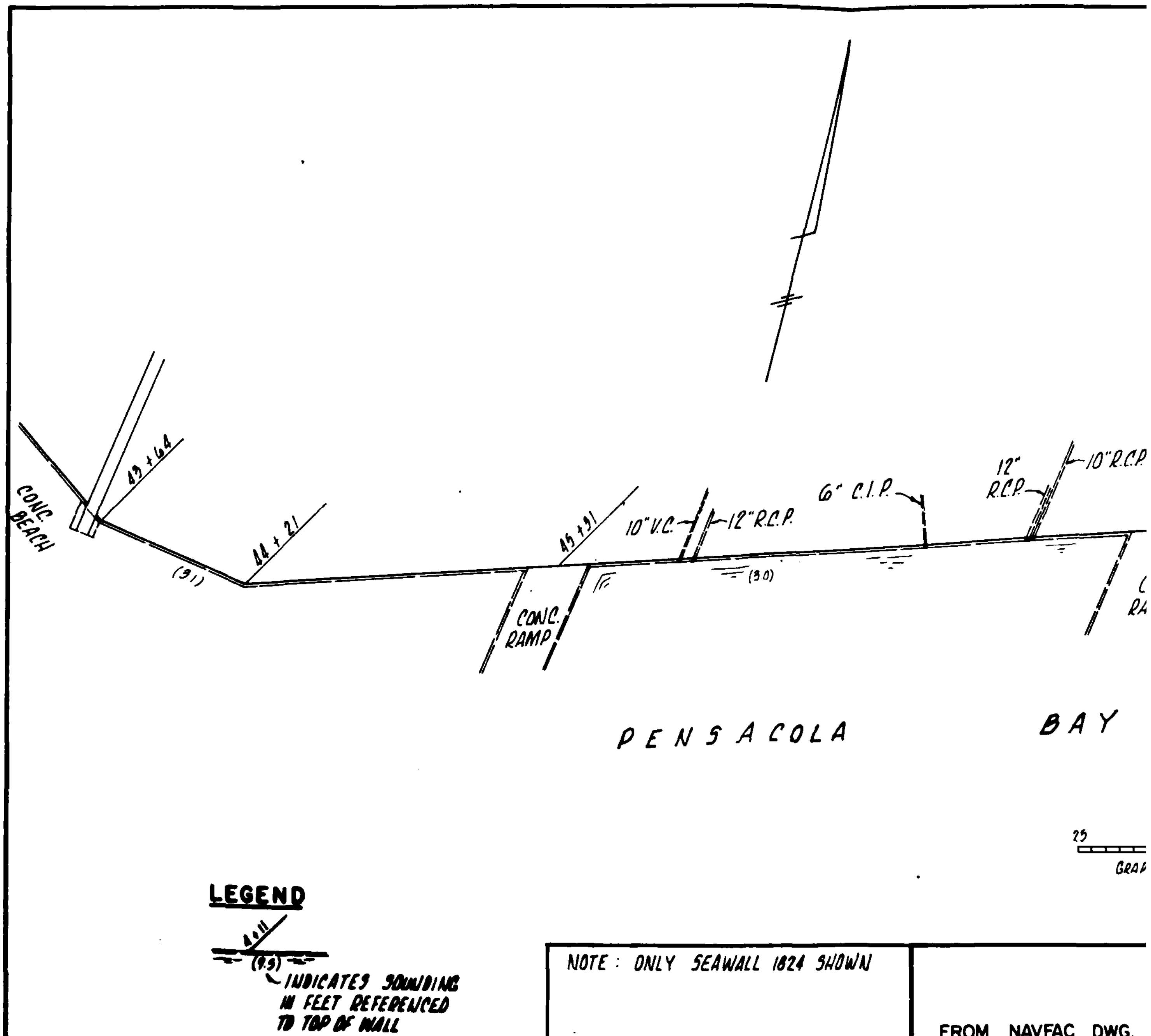
FROM NAVFAC DWG. NO. 5068064

PREPARED BY
ARTHUR V. STROCK & ASSOCIATES, INC.
829 S.E. 9th ST.
DEERFIELD BEACH, FLORIDA

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

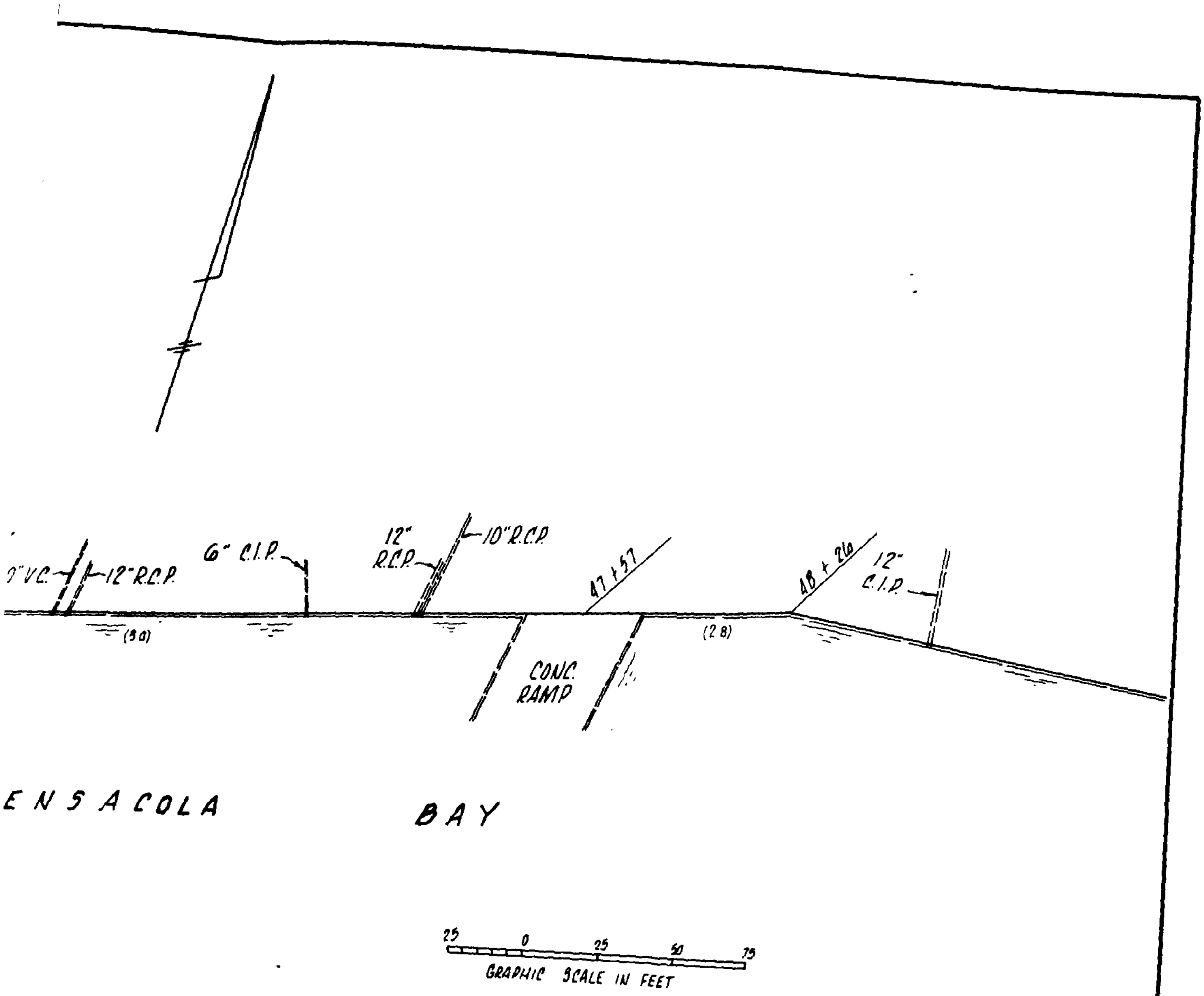
2 STATIONING PLAN

FIGURE A 7



NOTE : ONLY SEAWALL 1824 SHOWN

FROM NAVFAC DWG. NO. 5068065



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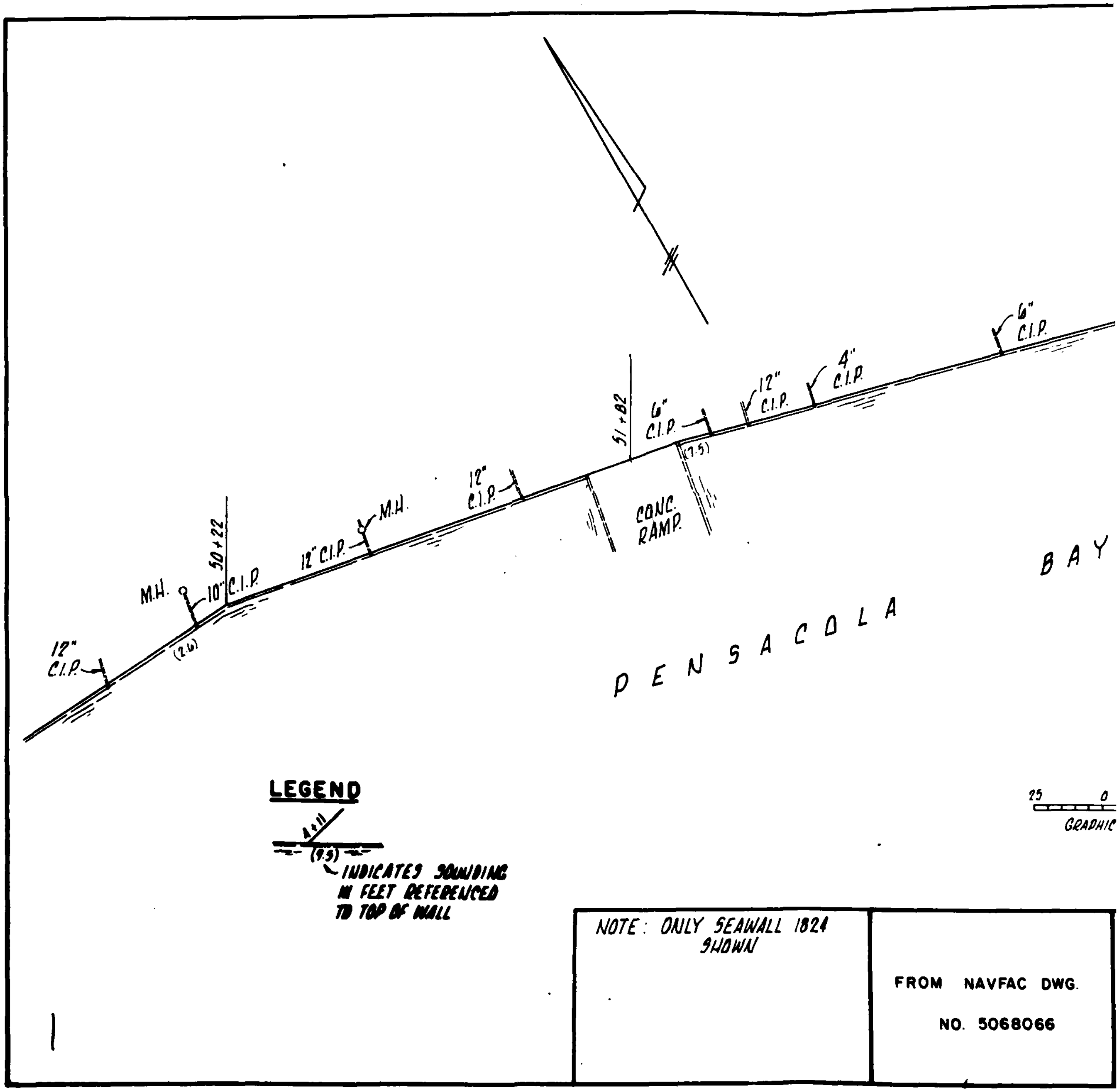
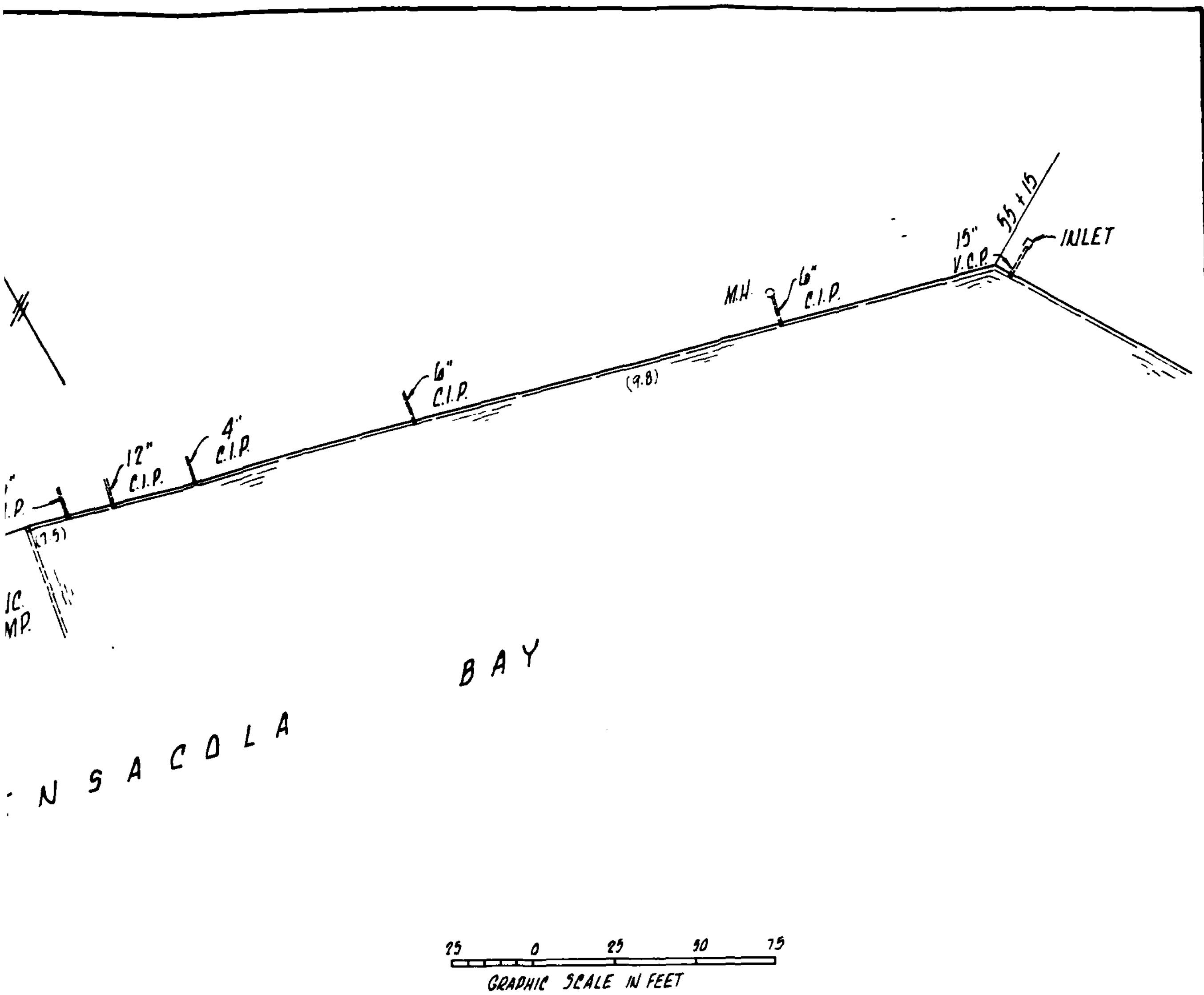
2 STATIONING PLAN

FIGURE A8



12" C.I.P.
M.H.
10" C.I.P.
50+22
(2.6)
12" C.I.P.
M.H.
12" C.I.P.
51+82
CONC. RAMP
6" C.I.P.
(7.5)
12" C.I.P.
4" C.I.P.
6" C.I.P.
PENSACOLA BAY
LEGEND
(9.5)
INDICATES SOUNDING IN FEET REFERENCED TO TOP OF WALL
25
0
GRAPHIC
NOTE: ONLY SEAWALL 1824 SHOWN
FROM NAVFAC DWG. NO. 5068066



ONLY SEAWALL 1824 SHOWN

FROM NAVFAC DWG.
NO. 5068066

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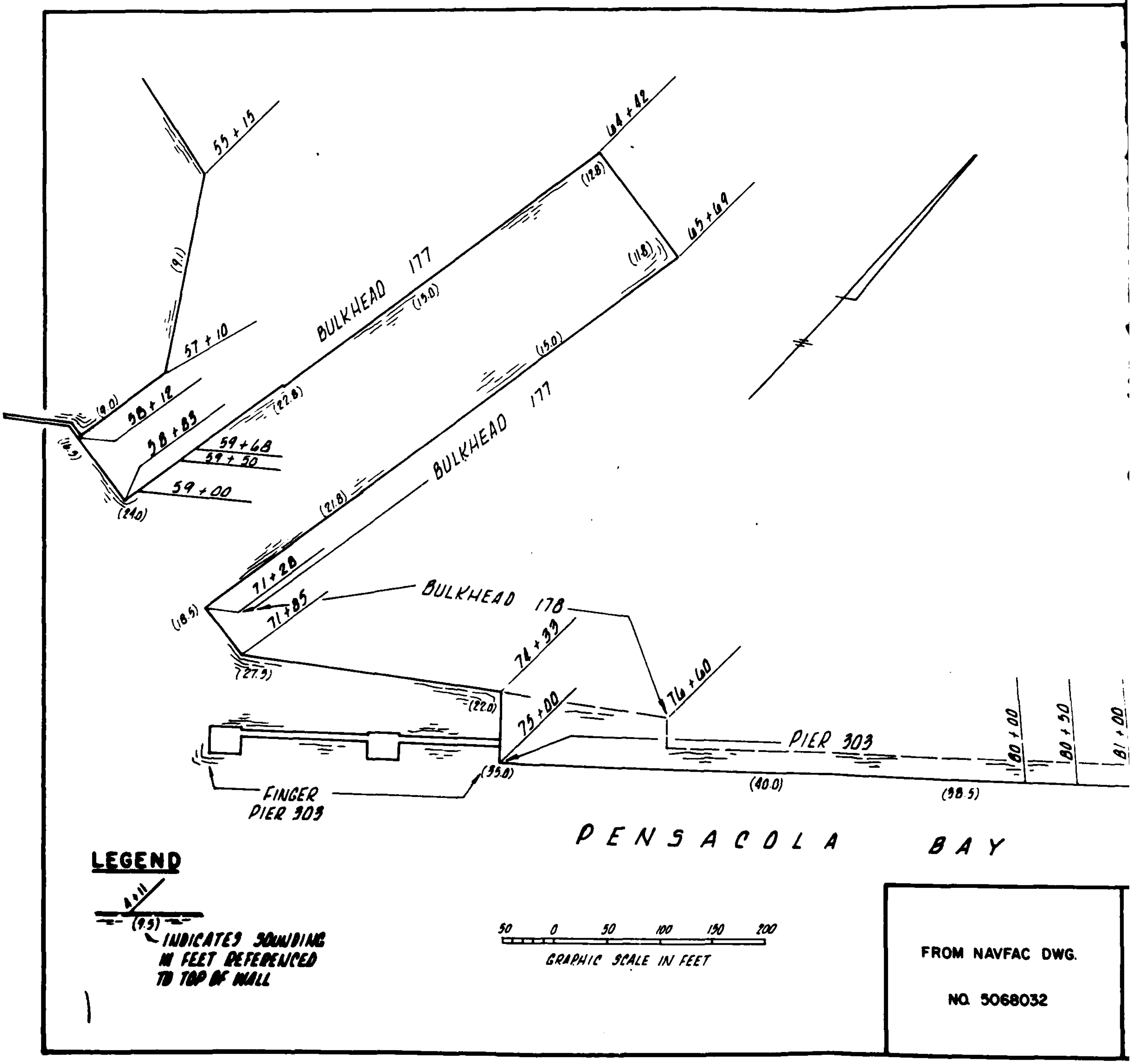
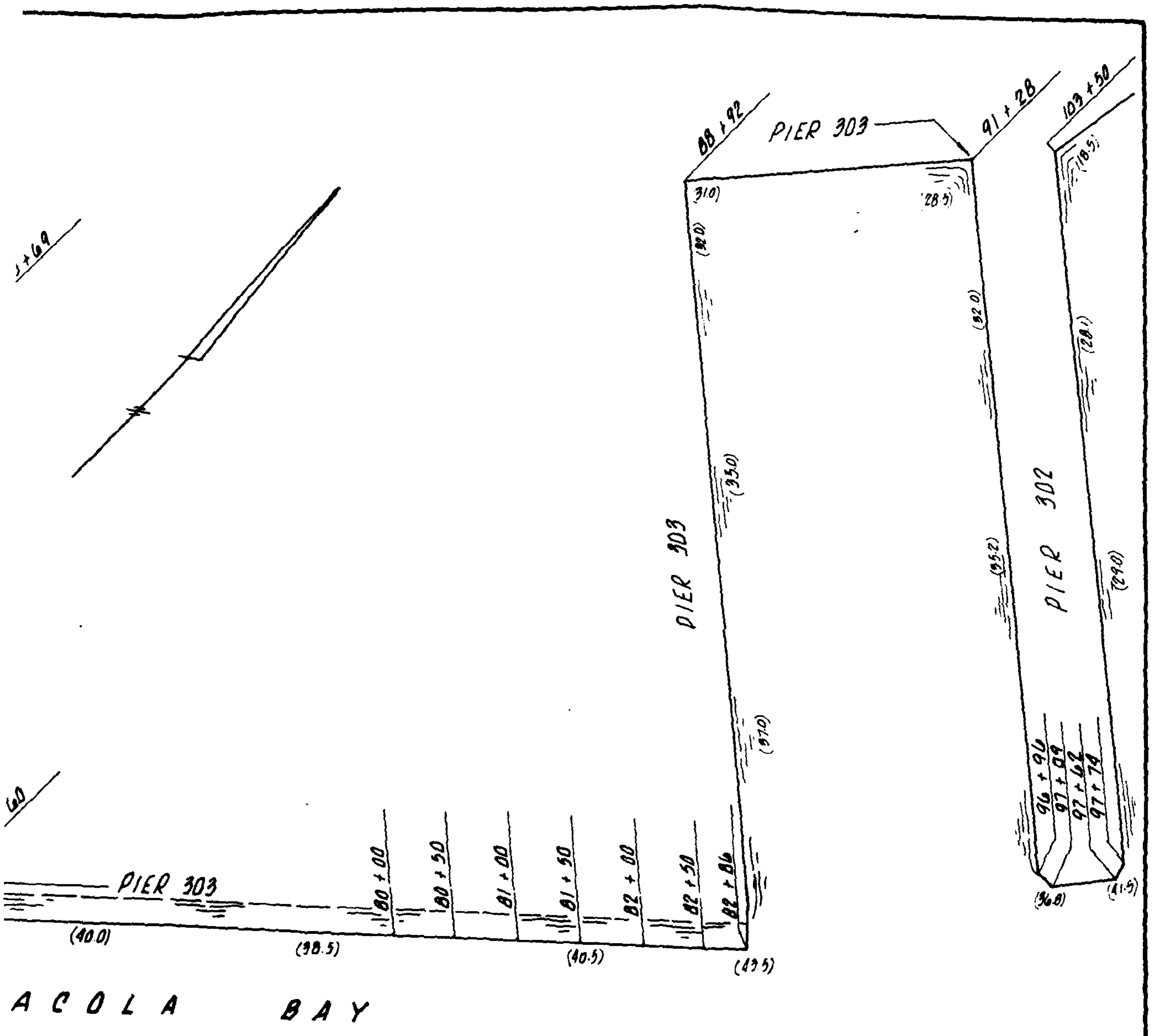
CHESAPEAKE DIVISION
NAVAL FACILITIES ENGINEERING COMMAND
WASHINGTON, D.C.

2 STATIONING PLAN

FIGURE A9



55 + 15
64 + 42
65 + 69
(12.8)
(11.8)
(9.1)
BULKHEAD 177
(15.0)
57 + 10
(9.0)
58 + 12
58 + 85
(22.8)
59 + 68
59 + 50
59 + 00
(24.0)
BULKHEAD 177
(21.8)
71 + 28
71 + 85
(18.5)
BULKHEAD 178
74 + 33
76 + 60
(27.9)
(22.0)
75 + 00
PIER 303
80 + 00
80 + 50
81 + 00
(35.0)
(40.0)
(38.5)
FINGER
PIER 303
PENSACOLA BAY
LEGEND
(9.5)
INDICATES SOUNDING IN FEET REFERENCED TO TOP OF WALL
50 0 50 100 150 200
GRAPHIC SCALE IN FEET
FROM NAVFAC DWG.
NO. 5068032



150 200

FROM NAVFAC DWG.

NO. 5068032

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DEERFIELD BEACH, FLORIDA

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

2 STATIONING PLAN

FIGURE A10

END

DATE

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