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TECHNICAL FEASIBILITY STUDY OF NAVY PIER CONCEPTS
CONCEPT 2 THE FLOATING... (U) LIN (T Y) INTERNATIONAL SAN
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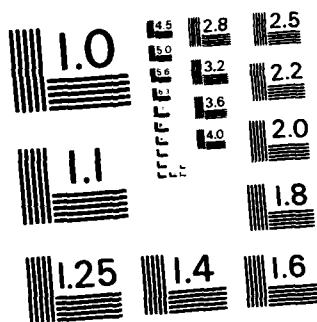
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NAVY PIER CONCEPTS
REPORT No. 2/83

ADA 131 122

THE
FLOATING MARINA
PIER

SUBMITTED TO:

DEPARTMENT OF THE NAVY

OFFICE OF NAVAL RESEARCH
ARLINGTON, VIRGINIA

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SUBMITTED BY:

T.Y. LIN INTERNATIONAL

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20. ABSTRACT (Continue on reverse side if necessary and identify by block number) The concept of a floating marina pier is presented. This pier concept offers the benefits of off-site construction, less critical waterfront site selection, tidal independence, and reduced fendering requirements. Disadvantages include high initial cost, increased inspection requirements during service, and a vulnerability to sinking. Design challenges are seen for the connection between modules, anchoring systems, service distribution, and the shore-pier interface.		

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THE FLOATING MARINA PIER

1. INTRODUCTION

The floating marina pier which was developed and presented to ONR in September, 1981, is the second of three concepts selected for further study under the contract for 1982/83. Interestingly, all three concepts of the future Navy pier are common in their basic features of flotation and mobility. The difference between them is only in their application. The emergence of a floating and mobile pier concept is perhaps a logical next stage in the evolution of a Navy pier, since the fixed position and fixed level pier have been with us for a long time, and any improvement thereon at this time would at best be incremental and not generic in nature. Increasing the working area of a deck by adding another deck on the fixed pier can also be considered an improvement in the former category.

The floating pier will solve the traditional problems of a fixed pier — congestion on deck, slow turnaround time, inadequate services, and heavy maintenance costs, particularly from the replacement of the camel fendering system.

Although a floating pier is new to the Navy, the state of the art pertaining to the design and construction of a floating vessel is not. There are numerous instances and experiences that have proven the feasibility of a floating vessel being used as a pier. The Navy itself has built several prestressed concrete floating piers measuring 500' x 100' to serve as docks for seaplanes in the early 50's, in Honolulu, Alameda and San Diego. Several concrete floating dry docks were also built in WW II, and these had operated continuously for forty years. Recent examples of concrete vessels include the 65,000-ton LPG barge for ARCO that measures 136' wide by 460' long, now operating off Indonesia, and the floating pier terminal measuring 30' x 100' x 700', which is under construction in Tacoma, Washington, for service in Valdez, Alaska.

The format of this report will be similar to the first one on the floating expeditionary pier. It will investigate the general validity of the design, etc., that were used in the development of the concept in 1981, study the feasibility of materials and construction methods, and identify problem areas.

2. THE ADVANTAGES OF THE FLOATING MARINA PIER

The advantages of a floating pier for Naval applications have been brought out and discussed in published materials, particularly the study on floating piers submitted by T.Y. Lin International to the Naval Civil Engineering Lab in Port Hueneme in 1982. For the sake of the completeness of this report, the major advantages described in the study may be summarized as follows:

- a) The pier can be constructed offsite, then towed to site for installation when completed. The advantages here are the reduction of construction time, cost, and traffic congestion at the site.
- b) A floating pier, particularly one that can be extended far out into deeper waters, does not take up as much valuable waterfront real estate as a fixed pier. It is also not as demanding as a fixed pier in the selection of a suitable site.
- c) By being able to move up and down with the changes of water level, it will avoid the customary and considerable changes in the length of mooring lines, brows, hoses, electric cables and other utility lines that connect the pier with the ship, thus reducing the wear and tear, as well as the knotting and the pinching of these lines due to excessive coiling and stretching that are common with the fixed pier.
- d) The near constant pier/ship level relationship will make it possible to use fixed position modern fendering systems, instead of the traditional camel fendering system which is subjected to heavy maintenance problems and costs.
- e) It can provide a second deck without much additional cost. This advantage has been recognized, as evidenced by the double-deck pier being considered for Charleston Naval Station. For the floating pier, the addition of the second deck is almost "natural." This is because of the available space between the main deck, which is located 16' above the waterline, and the buoyancy chamber, which is below the waterline. This space serves no purpose other than to support the main deck and could easily be turned into a second deck with little additional effort and cost. The second deck will go far in relieving congestion, not only in traffic but also in the utility pipelines, and will perform additional functions such as providing space for training facilities, workshops, and parking. It will undoubtedly increase the efficiency of the pier and heighten the combat readiness of the docked ships.
- f) It will increase cost effectiveness of the pier by making it reuseable. Thus, a floating pier that has become obsolete in one location need not be disposed of by demolition, but could be relocated at another site where it could be useful.

- g) Its mobility and relocatability will enhance the deterrent effect of the Navy in general. To maximize this effect, the pier should be located in a relatively large body of inland or sheltered water. There is a number of such waters around the USA, e.g., the Great Lakes that feed into the Hudson River, Puget Sound in Oregon, the San Francisco Bay, the Sacramento Delta area in California, and the Galveston Bay in Texas.
- h) The marina pier concept will cater well to the future expanded Navy because it can be modularized, mass-produced and made up to almost any size within physical limits by assembling the number of modules required. The major limitation is the capacity of the anchoring system and the incidence and magnitude of wave and seiche action at the site.
- i) Better earthquake resistance can be expected in a floating pier.

There are, of course, also disadvantages to this concept. The first would be its vulnerability to blockage if the ships' access to the sea is cut by enemy action. This would be true for the traditional Navy ships. However, for the marina pier operating in the future, the Navy ships it was to cater for and as projected in the original concept, have been envisioned to be vessels capable of skimming over the water surface at great speed like a hydrofoil. The blockage of the waterways would have to be fairly complete before vessels of this type could be stopped.

Other disadvantages exist with the floating pier concept which must be considered in a final evaluation. Some of these are: high initial cost, as high quality control during construction and larger quantities of materials are required; more inspection to ensure seaworthiness is necessary; vulnerability to accidental sinking and relative movement at the pier-shore interface which may pose some problem to the design.

3. DESIGN CONSIDERATIONS

The design parameters that have been established for the marina pier are as follows:

- a) Dead and live loads. Refer to Appendix A, Sheet A-1.0 for loading criteria.
- b) Wave loads according to Sea State 2; e.g., a wave height of 3' maximum and a wave length equal to the pier length parallel to the direction of the incident wave. The design procedure for wave loading follows that of the ABS rules and classing of vessels.
- c) Wind loads according to Sea State 2; e.g., a maximum wind velocity of 50 mph. The wind forces and their distribution are done according to the NAVFAC's Design Manual DM 26.

- d) Current of 3 knots at water surface decreasing linearly to zero at the water bottom and constant current velocity from pier end to shore. The current loadings are estimated using the new Navy DM 26-6 method. Wave, wind and current forces are assumed to be acting concurrently when full berthing at the floating pier (Scheme 2) occurs.
- e) Double deck spine pier only.
- f) Safety considerations.
- g) Modular construction.
- h) Maximum draft of 27.5' due to dead and full live load using hard rock concrete. Water depth of 40'. If lightweight concrete is used, the draft will be reduced to 24.5', thereby extending the usefulness of the pier to more shallow sites.
- i) Self-sufficiency.

4. DESCRIPTION OF THE PIER

Two marina pier schemes, each with 12-berth capacity, are presented. Scheme 1 is similar to the original concept of 1981 that caters for the projected future Navy ships that can speed like a hydrofoil, and maneuver under water like a submarine. In the second scheme, the plan configuration is altered to cater for the present destroyer class vessels. These two schemes are shown respectively in Drawings 1 to 7 inclusive.

The structural configuration of the pier is generally as shown in the original concept. The elevation of the main deck is set at 17' above water. The draft of the finger piers, which are made up basically of buoyancy chambers, is 13' on the average, while the draft of the main spine pier is 27'. As mentioned previously, the pier is to be made up of several modules. For each unit to be self-floating, the finger and spine piers will have to be enclosed around the periphery to keep the water out. Connection details between the modules when they are maneuvered into position are shown in Drawing 11. For Scheme 2 where the pier is shown to service the present ships, the finger piers are set at 330' clear apart to meet the requirements of the design manual.

Service lines are led to each finger pier from the second deck of the spine pier, either through a servicing tower as shown in the original concept, or by pipe troughs under the finger piers, as shown in Drawing 11. If the servicing tower to the stern or the bow of a ship is envisaged, the utility inlets of the ship will have to be located accordingly.

The pier is anchored by two pylons, one at each end of the spine. As envisaged, the pylon consists of a hollow circular shaft or shaft of other cross-sections, which is inserted into a base structure on the floor basin. The advantage of this is the simplicity of the process of installing and removing anchors, thereby enhancing the relocatability of the pier. The procedure of anchoring or deanchoring consists of no more than inserting into or removing the vertical shaft from the base structure. See Drawing 10 for possible details.

The pier main deck is connected to the shore by one 20' wide ramp and the second deck by two 10' wide ramps, one for ingress and the other for egress. These ramps are connected to a pinned joint at the shore end, and a sliding joint at the pier that are designed to accommodate relative vertical and horizontal movements.

The availability of the second deck will provide the necessary space required to make the pier as self-sufficient as possible. There will be ample space for storage, living accommodations for the maintenance and control crew, etc.

It is also possible to convert the spine deck into a runway for Navy planes, similar to the conversion of the finger pier in the expeditionary pier concept into runways. In this case, the pylon must not protrude above the deck level, and service lines must be kept below the main deck.

5. VALIDITY OF DESIGN

Based on the design considerations listed above, the preliminary design of the pier was carried out and the calculations enclosed as Appendix A at the end of this report. As for the expeditionary pier, this design was checked to ensure that the structural system as quantified in the drawings, will be adequate to withstand assumed construction, towing and operating conditions and that its flotation and naval architectural characteristics are satisfactory. For this pier the major design problems are centered around:

1. The connection between the modules;
2. The anchoring system;
3. The supply of services for a high concentration of ships that the pier can accommodate; and
4. The shore-pier interface.

The rest of the design is seen to be within the state of the art.

6. ANALYTICAL AND DESIGN FEASIBILITY

The feasibility of design of this pier is similar to that described for the expeditionary pier in Report No. 1. Deficiencies in the state-of-the-art technology are nevertheless present, particularly in relation to the major problem areas as pointed out in the previous section. This will be commented upon in the following.

6.1 Connection Between Modules

Connections between prefabricated modules of a large floating structure are within the state of the art. The difficulty in our case is with the size and lightship draft of the modules involved. Our design has called for module connection between spine and spine of similar draft, and finger to spine modules of different drafts. This last connection requires ballasting of the finger module to achieve the same draft as the spine pier. The two modules assembled are first of all maneuvered into position and secured in the relative position by a set of temporary ties on the main deck. The two modules to be connected are then pulled together by tightening the adjustable ties at the sides of the connecting face as shown. This will provide sufficient force for the two connecting faces to seal off the trench between the two adjacent bulkheads along the joint. Water is pumped out of the trench to permit prestressing couplings and other reinforcing steel to be connected or spliced. The trench is then concreted, and when the concrete has acquired the necessary strength, the connecting tendons are stressed.

6.2 The Pylon Anchoring System

The purpose of the pylon is to restrain the pier against horizontal movement while permitting it to move vertically with the change in water level. It consists of a larger-diameter hollow vertical shaft standing freely in a sleeve that forms part of a piled base structure. It is important to keep the floating pier aligned with the pylon base structure when the pylon is being positioned. The tentative anchoring system is shown in Drawing 10.

6.3 Utility Lines

In view of the number of ships that could be serviced by this pier and of the need for the pier to be as self-sufficient as possible, it is envisioned that the pier will be equipped to generate most if not all of the services, such as steam, power, sewage disposal, and potable and fire-fighting water supplies. Because of the size of the supply system, the flexible connection of the utility lines between the pier and the shore would be an area of major design effort, as this connection must accommodate the full range of vertical movements due to water action, and limited horizontal displacements caused by environmental loads. Refer to Dwgs. 8 & 9.

7. MATERIAL FEASIBILITY

Normal weight concrete has been used in this design as the prime structural construction material. No difficulty is foreseen in the use of this material, which has been proven in marine applications. Permeability and abrasion of the pier must be kept to a minimum in order to ensure long-term durability. The standard way to watertightness is to produce good concrete, provide proper curing, and coat the exterior face of the construction joints with sealant. Lightweight concrete could be used to advantage for floating structures. Lightweight concrete can now be made with high working strength (7,000 psi) and resistance against abrasion. Concrete in the sea inhibits marine growth due to its alkalinity, although growth does take place. Where this problem exists, the surface could be coated with marine growth inhibitors commonly used for offshore work. Other materials that can be used include high strength steel tendons and wires, mild steel and wire mesh, all of which are readily available today.

The materials that may require some research will be those connected with the construction of the pylon anchoring system and materials used in the connection of the utility lines across the shore-pier interface. The pylon shaft requires a large cross-section and can be exceedingly heavy. Its movement will require the use of very heavy cranes, which may not be easily available at a particular site. The design tentatively calls for 13' outside diameter normal weight concrete pylons weighing 300 tons each. As an unconventional alternative, high strength polymer concrete with three times the conventional strength may be used to reduce weight.

8. CONSTRUCTIONAL FEASIBILITY

The comments that were made for the expeditionary pier are mostly equally applicable in the case of the marina pier. Generally it is envisioned that the pier modules will be built by the flood basin method, as described for the expeditionary pier. The same flood basin method may be used to construct the base structure of the pylon anchors and the pylons.

9. ALTERNATIVE ANCHORING SYSTEM

It will be possible to anchor the marina pier by the conventional anchor and mooring line methods. However, the anchoring system will be extensive, requiring perhaps 16 or 18 anchoring points, making the installation and removal of the anchors a lengthy process. Another difficulty is the possible excessive excursion of the pier while on station, and the problem it poses to its connection with shore.

It may also be possible for this marina pier to incorporate the stiff-legged anchoring system as described for the expeditionary pier.

10. COST CONSIDERATIONS

Based on the concept presented herein and the unit prices, as listed in the following, the cost of the pier may be estimated as follows:

For hard rock concrete the overall unit cost is taken at \$900/C.Y. of concrete. This order-of-magnitude cost includes concrete, steel, ramps, construction, tow and connection, fenders, plus an additional allowance of 10% for miscellaneous items. The cost of the pier structure then works out to be \$85 million for approximately 95,000 C.Y. of concrete.

The cost of the pier structure, if constructed of lightweight concrete assumed at \$1,000/C.Y. of concrete, will amount to \$95 million. On a unit cost basis, this works out to be \$164 per sq. ft. for a total of 578,000 sq. ft. of deck span including the second deck for Scheme 2.

11. CONCLUDING REMARKS

The development of this concept had produced a number of novel features which may have important impact on the development of floating piers in general. The most significant ones are the pylon anchoring system, and the unprecedented magnitude of vessel size.

The technological feasibility of the marina pier has been demonstrated in this brief exercise. The initial cost is high compared to the conventional pier. It does offer important advantages, as described in this report if conditions exist to justify its construction.



INTERNATIONAL
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133

PROJECT: Navy Pier

ITEM: Floating Marina Pier

DESIGN: Summary, Scheme 2

DATE: IV 83 RM

SHEET:

A-1.0

OF

REVISION:

Appendix A ; Scheme 2

Geometry

Size : Pylon = $13'\phi_o$, $11'\phi_{in}$ x ~55'
Spine ~1000 x 160'
Fingers ~540' x 80' each

Hardrock : Dead weight : 203360^T
Lightship draft : 17.3'
Live load draft : 10.5' @ Full load
Total : 27.8'

Light weight : Dead weight : 179000^T
Lightship draft : 14.0'
Live load draft : 10.5' @ Full load
Total : 24.5'

Center of Gravity and Buoyancy located at the
Geometric center of the pier (x, y plane).

Pier Period $x-x$ = 9.3 sec (Hardrock concrete)

Loading

Sea State 2 : Max. wave height : 3'
Wind Velocity : 50 mph
Current : 3 knots

Materials

Hardrock concrete @ 150 pcf or

Lightweight concrete @ 120 pcf

FC = 7000 psi For Hardrock and Lightweight concrete

Prestressing steel : Uncoated seven wire strand

ASTM A416, $58'\phi$, F_{pu} = 270 psi

Reinforcing steel : ASTM A615 Grade 60



INTERNATIONAL
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133.

PROJECT: Navy Pier

ITEM: Floating Marine Pier, Sec. 1

DESIGN: Surface Area & Floatation

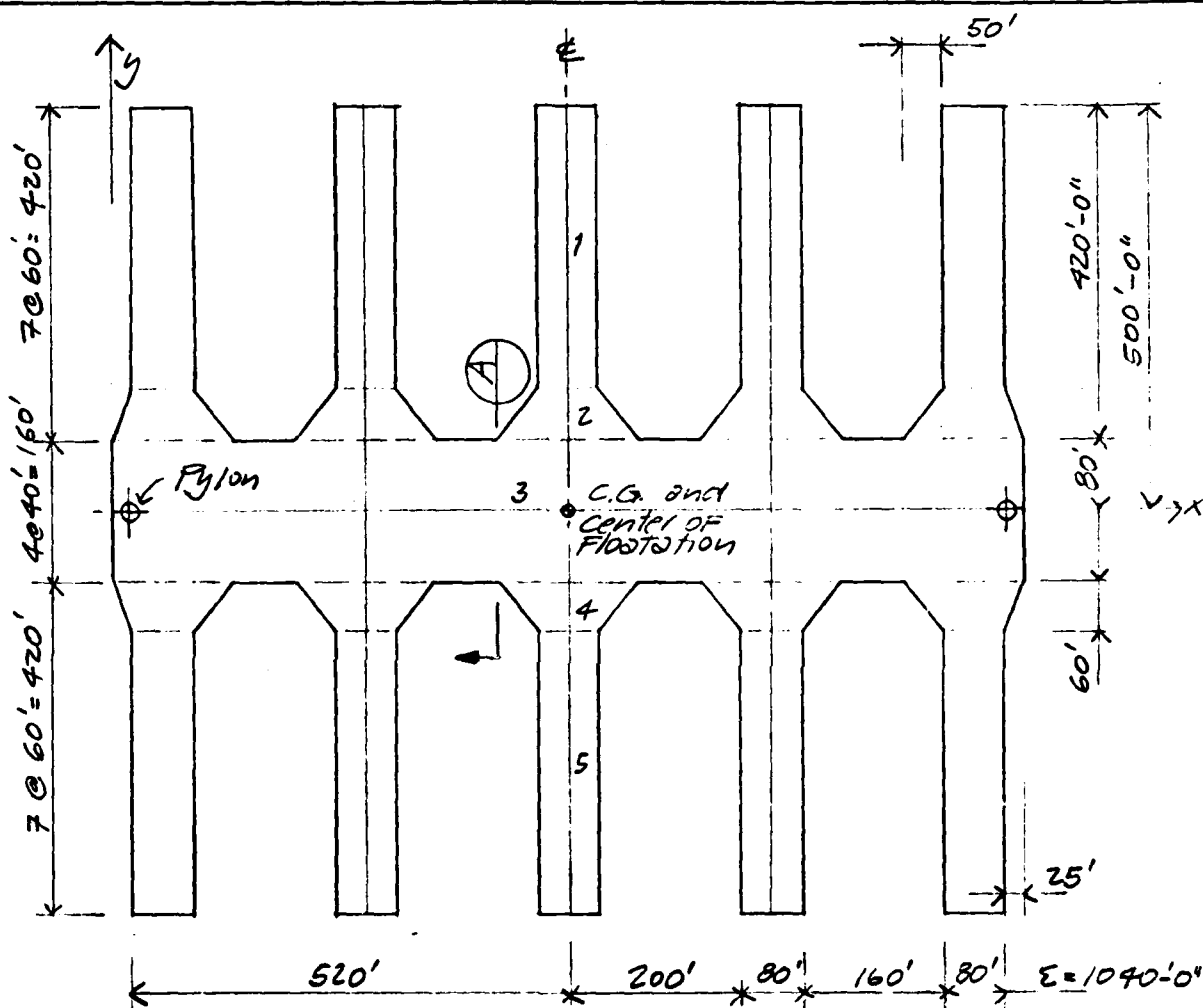
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SHEET

A-1.1

OF

REVISION



	Area ϕ	y (")	$Ay (F^3)$	$Ay^2 (F^4)$	$Iy_0 (F^4)$
1	144000	820	118×10^6	9.68×10^{10}	1.55×10^9
2	42000	614	25.8×10^6	1.58×10^{10}	1.18×10^7
3	174400	500	87.2×10^6	4.36×10^{10}	3.72×10^8
4	42000	386	16.2×10^6	6.25×10^9	1.18×10^7
5	144000	180	25.9×10^6	4.67×10^9	1.55×10^9
	546400		293×10^6	170.6×10^9	

$$I_{xx} = 170.6 \times 10^9 - 546400 \times 500^2$$

$$= 3.4 \times 10^{10}$$



STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133

PROJECT: *NAVIN PIERS*

ITEM: *Floating Marina Pier*

DESIGN: *loading, Scheme 1*

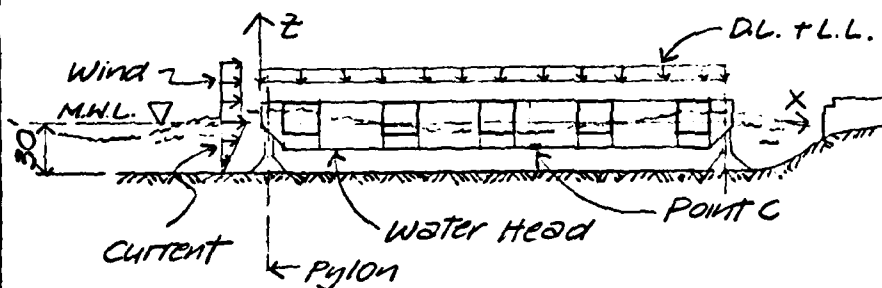
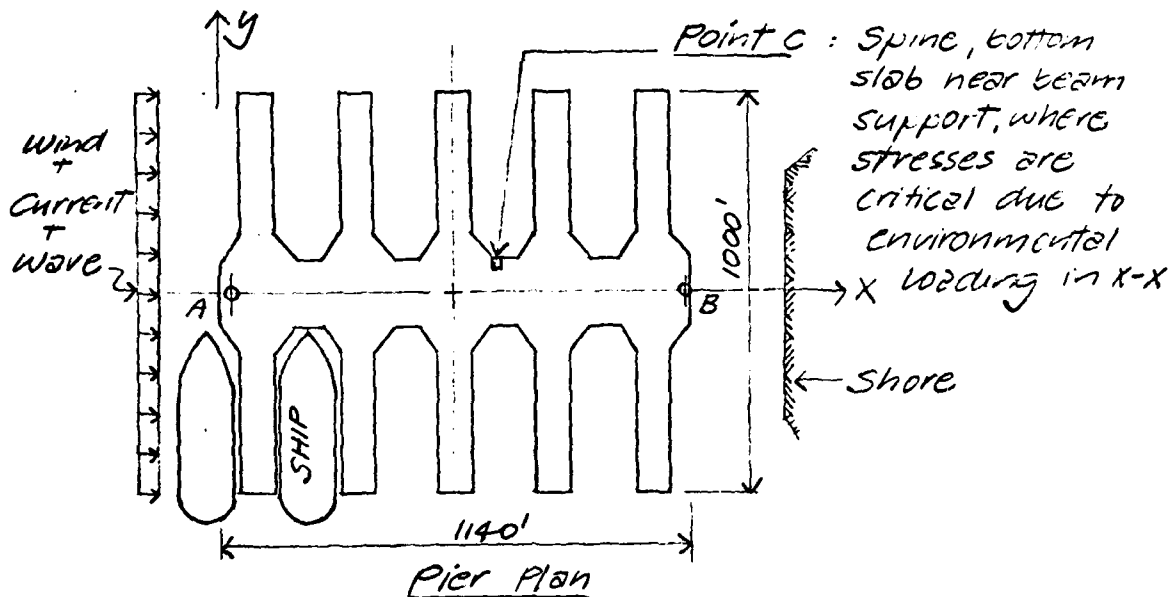
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A-1.2

OF

REVISION:



loads in the Structure :

- a) Permanent : - Dead load concrete + pavement
- Water head due to Dead load
- b) Live load : - 400 psf @ top deck , 150 psf @ middle deck
- 200 psf @ bottom deck
- water head due live load, mooring of vessels
- c) Deformation: - Prestressing
- Δ Temperature , shrinkage , creep (ignore now)
- d) Environmental: - wave load : 3' high wave, $L = 1140'$
(Sea State 2) - Current @ 3 knots @ water surface.



STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133.

PROJECT: NAVA PIER

ITEM: Multiple Marina Pier, Sch. 1

DESIGN: ABS Rules For Wave Loads

DATE: 11/83 RM

SHEET:

A-1.3

OF

REVISION:

USE ABS Rules ; Environmental Loads For Sea State 2

Section 6.3.2 Total Bending Moment about y-y axis

$$M_{t,y-y} = M_{sw,y-y} + M_{w,y-y}$$

$M_{sw,y-y}$ = Still Water Bending Moment

$M_{sw,y-y} = 0$ Assuming : Uniform D.L & L.L. distribution

$M_{w,y-y}$ = Maximum Wave Induced Bending Moment

$$= C_2 L^2 B H_e K_6$$

$$K_6 = 1.0 \quad \text{For } C_6 > 0.80$$

$$C_6 = 1.0 \quad \text{assumed}$$

$$\begin{aligned} C_2 &= [6.53 C_6 + 0.57] \times 10^{-4} \\ &= [6.53 \times 1.0 + 0.57] \times 10^{-4} \\ &= 7.1 \times 10^{-4} \end{aligned}$$

$$L = 1040' + 100' = 1140' \quad (\text{x-x axis})$$

$$\begin{aligned} \bar{B} &= (1000' \times 80' + 220' \times 50' + 160' \times 60') / 190' \\ &= 529' \quad (\text{weighted average}) \end{aligned}$$

$$H_e = 26.75'$$

$$\begin{aligned} M_{w,y-y} &= 7.1 \times 10^{-4} (1140')^2 \times 529' \times 26.75' \times 1.0 \times 2.24 \\ &= 29.2 \times 10^6 \text{ K-F} \end{aligned}$$

Determine Allowable Moment that can be resisted by pier @ the 160' wide Section:

See State $\rightarrow$ Max. Wave Height = 3'

$$M_{max,y-y} = \frac{H}{H_e} \times M_{w,y-y} \quad \text{@ Midship}$$

$$= \frac{3}{26.75} \times 29.2 \times 10^6$$

$$= 3.28 \times 10^6 \text{ K-F}$$



INTERNATIONAL
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133

PROJECT: Navy Pier
ITEM: Floating Marina Pier, Sch. 1
DESIGN: Environmental Loads 1-X
DATE: 11 88 AM

SHEET:
A-1.4
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REVISION:

ABS Wave Bending Moment and Shear Distribution

$$M_{max} = 3.28 \times 10^6 \text{ K-F}$$

$$L = 1140'$$

Stations @ 114'-0"

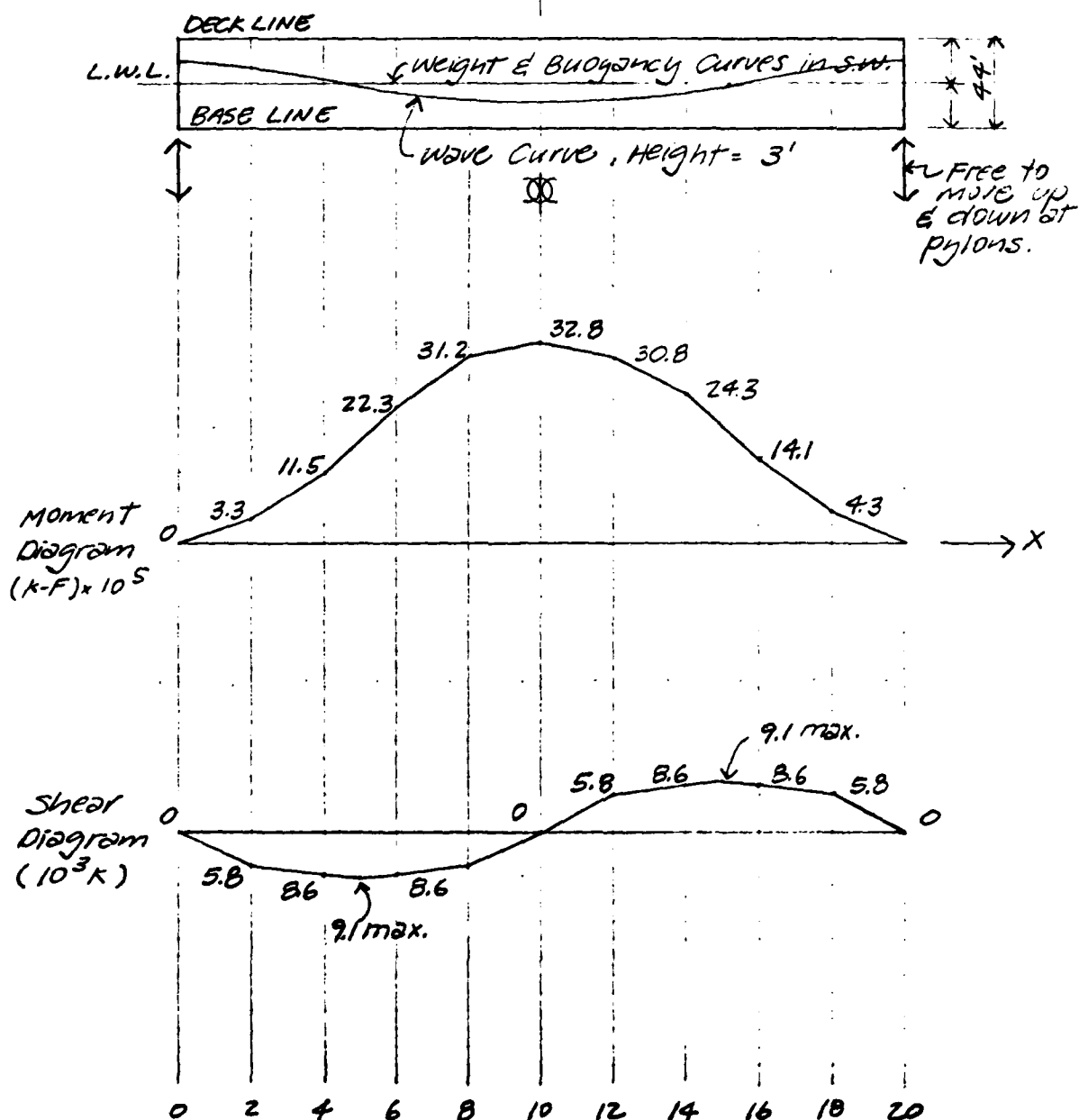
Bending Moment (Wave Height = 3')

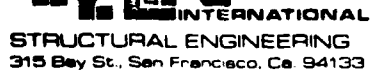
Station		=		K-F
0	0	=	0	
2	$0.10 \times 3.28 \times 10^6$	=	330000	
4	$0.35 \times 3.28 \times 10^6$	=	1150000	
6	$0.68 \times 3.28 \times 10^6$	=	2230000	
8	$0.95 \times 3.28 \times 10^6$	=	3120000	
10	$1.00 \times 3.28 \times 10^6$	=	3280000	
12	$0.94 \times 3.28 \times 10^6$	=	3080000	
14	$0.74 \times 3.28 \times 10^6$	=	2430000	
16	$0.43 \times 3.28 \times 10^6$	=	1410000	
18	$0.13 \times 3.28 \times 10^6$	=	430000	
20	0	=	0	

Shear Force

Station		=		K
0	0	=	0	
2	$(330000/114) \times 2$	=	5789	
4	$[(1150000 - 330000 - 5789 \times 114)2]/114 + 5789$	=	8597	
5	$0.45 \times 1140 + 8597$	=	9110	
6		=	8597	
8		=	5789	
10	0	=	0	
12		=	5789	
14		=	8597	
15		=	9110	
16		=	8597	
18		=	5789	
20	0	=	0	

Wave Moments and Shear Envelopes (y-y axis)
Scheme 1





DATE: 11 83 KM

REVISION:

The diagram is a hand-drawn plan view of a ship's hull section, showing the structure and deck layout. The hull is symmetrical about a central vertical axis. The deck is divided into two levels: the Top Deck and the Utility Deck. The hull is divided into four main sections, numbered 1, 2, 3, and 4. The dimensions are as follows:

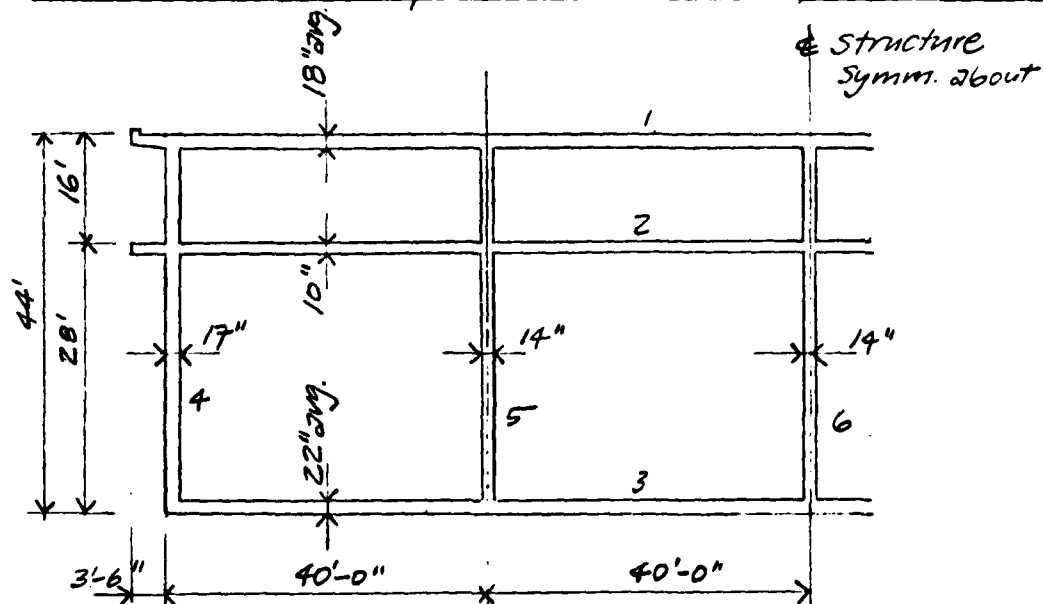
- Overall Width:** 44' (Total), 20.5' (Top Deck), 8.5' (Utility Deck), and 2' (Hull).
- Top Deck:** 15' (Total), 15" (Each side of the central section).
- Utility Deck:** 12" (Total), 15" (Each side of the central section).
- Hull:** 15" (Total), 15" (Each side of the central section).
- Radius:** $R = 28.3$ (Indicated for the curved hull section).
- Deck Structure:** 4' (Total), 4' (Each side of the central section).
- Deck Thickness:** 3'-6" (Total), 3'-6" (Each side of the central section).
- Deck Area:** 40'-0" (Total), 40'-0" (Each side of the central section).
- Deck Labels:** Top Deck, Utility Deck.
- Structure Labels:** 1, 2, 3, 4, 5.
- Coordinate System:** X and Y axes are shown at the bottom left.

	A^{F^2}	y^F	Ay^{F^3}	Ay^{F^4}	Iox	y_t	St
						y_b	S_b
1	101.1	43.4	4384	190158	13		
2	80.9	28.5	2306	65711	7		
3	108.7	2.75	299	822	317		
4	46.9	25.3	1186	30005	5496		
5	<u>23.5</u>	<u>25.3</u>	<u>595</u>	<u>15054</u>	<u>2748</u>		
	361.1	24.3	8770	301750	8591	19.7	
						24.3	

$$I_{xx} = \frac{[(301750 + 8581) - 361.1 \times 24.3^2] \times 2}{2} = 194210 \text{ F}^4$$

$$S_6 = 194210 / 19.7 = 9858^{F3} < 12974^{F3}$$

Critical Section Properties near Midship, Trial Section II



SECTION C (A)
(II)

	$A F^2$	$y F$	$A y F^3$	$A y^2$	I_{ox}	y_t	S_t
1	120.4	43.3	5207	225216	23	y_b	S_b
2	66.8	27.6	1843	50855	4		
3	140.9	0.92	129	118	39		
4	62.3	22	1371	30169	10056		
5	51	22	1122	26684	8282		
6	25.7	22	565	12439	4141		
	<u>467.1</u>	<u>21.9</u>	<u>10237</u>	<u>345481</u>	<u>22545</u>	22.1	13002
						21.9	13121

$$I_{ox} = [(345481 + 22545) - 467.1 \times 21.9^2] \times 2$$

$$= 287342 F^4$$

$$S_b = 287342 / 21.9$$

$$= 13121 F^3 > S_{req} \quad \checkmark$$

$$S_t = 287342 / 22.1$$

$$= 13002 F^3 > S_{req} \quad \checkmark$$

Minimum Allowable Section Modulus at Section (A)

$$M_{max} = [32.8 - \frac{(32.8 - 31.2) \times 90}{114}] \times 10^5$$

@ (A) = 3153648 K-F

$\bar{\sigma}_{max} = \text{Maximum Allowable Bending Stresses:}$
 $= 0.45 F'_c / 2 \quad F'_c = 7500 \text{ psi}$
 $= 1688 \text{ psi}$

Minimum $S_{t,b} = M_{max} / \bar{\sigma}_{max}$
 @ Section (A) = $3153648 / (1688 \times 144 / 1000)$
 $= 12974 \text{ F}^3$

This is the critical section of Pier for wave loading parallel to X-axis because of the high moment and the small width at this location.

Section Modulus provided at this section: See pg.

$S_t = S_b = 13000 \text{ F}^3 > 12974 \text{ F}^3$
 $\therefore$ Satisfactory in bending Myy

Maximum Wave Height Allowable for $S = 20041 \text{ F}^3$

$$H = \frac{M_{max} \times H_e}{M_{wave_y()}}$$

$$M_{max_{yy}} = (S_{t,b}) \bar{\sigma}_{max}$$

$$= 13002 \times 1688 \times 144 / 1000$$

$$= 3.16 \times 10^6 \text{ K-F}$$

$$H = \frac{3.16 \times 10^6}{3.15 \times 10^6} \times 3.0'$$

$$= \underline{\underline{3.01' \text{ Wave}}}$$

Comparison of Cross Sections @ Midship : Scheme 1

A rectangular cross section is preferable compared to a semicircular one. The biggest loading here comes from wave action in the long direction of the pier. The pier here acts like a flexible beam which is stressed to its maximum @ the top and bottom slabs. These stresses account for ~ 75% of the total stress at the bottom slab. Therefore the higher the Moment of Inertia of the Section the more effective it is.

Since the operational depth of water is limited to ~ 40' the pier draft must also be limited to ~ 30' max. Therefore the section height is controlled by this parameter.

A circular section, like Section I, is quite effective for high hydrostatic pressures and torsion but not so for forces that causes bending stresses in one direction (wave action) like in our case. As hull depth is shallow, hydrostatic pressure is small and therefore a circular section is not appropriate.

Forming of a circular shape become more expensive and module connections more difficult than rectangular shapes.



INTERNATIONAL
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133

PROJECT: Navy Piers

ITEM: Floating Marina Pier, Sch. 2

DESIGN: SURFACE AREA & FLOATION

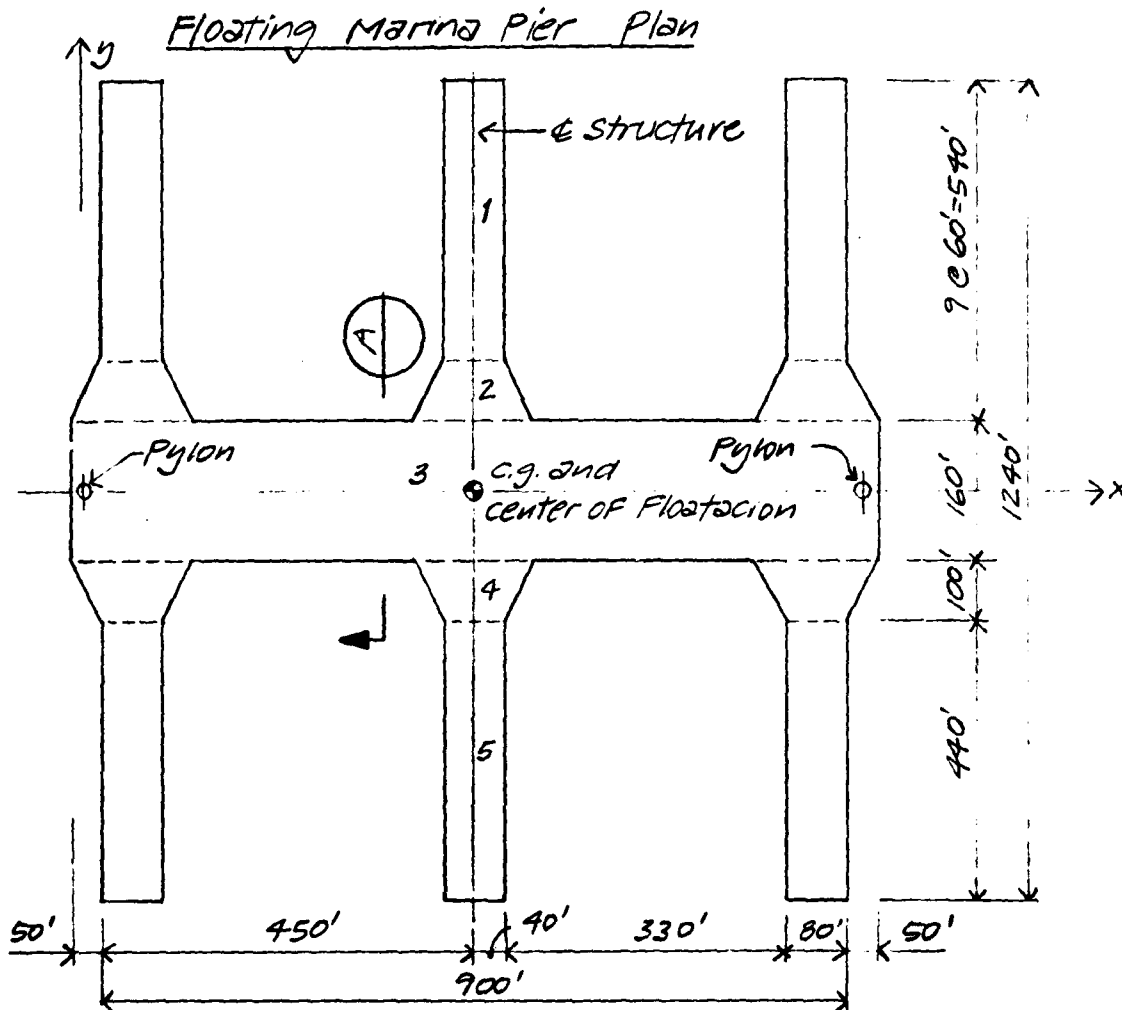
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SHEET:

A-2.1

OF

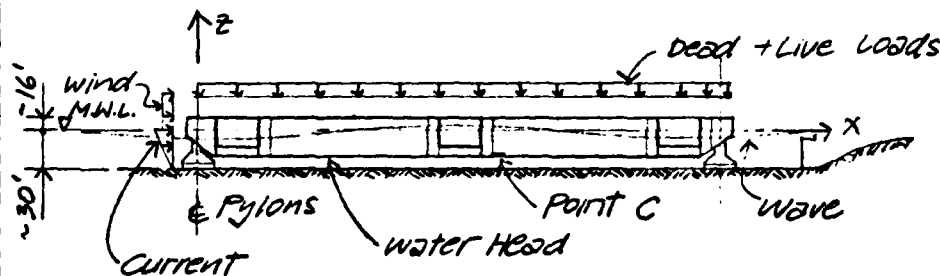
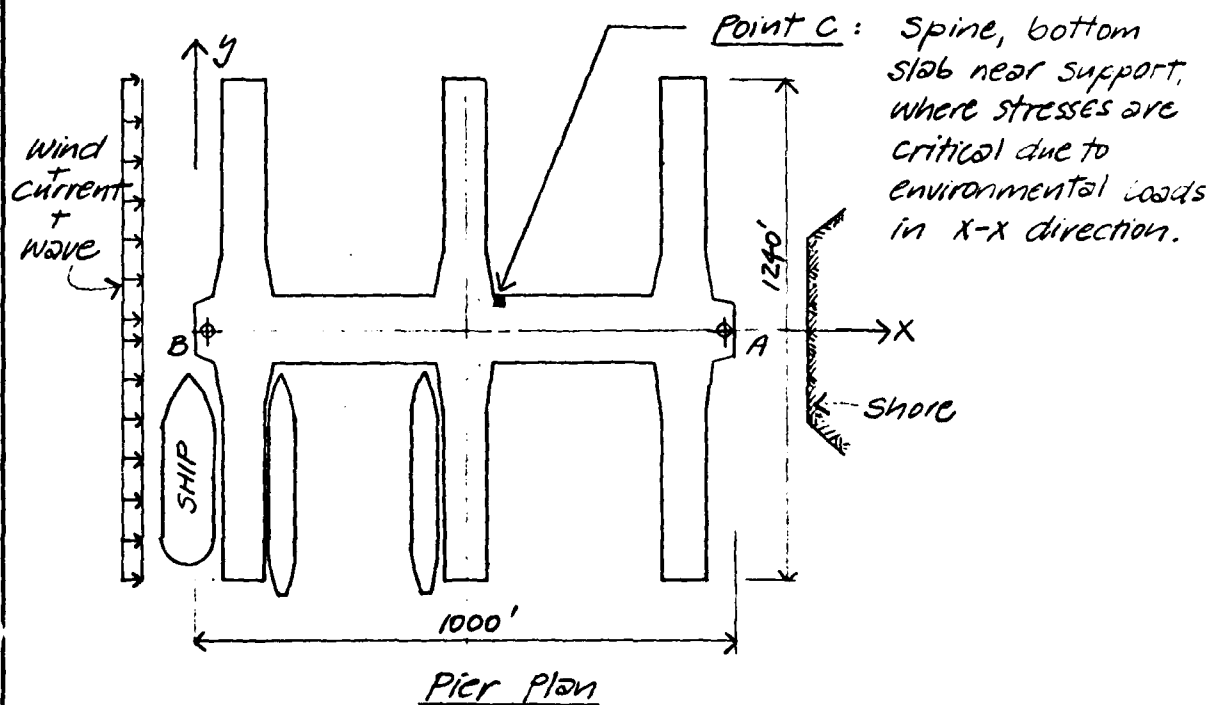
REVISION:



	Area ^φ	y(F)	A _y ^(F³)	A _y ^(F⁴)	I _y ^(F⁴)
1	105600	1020	1.08x10 ⁸	1.1x10 ¹¹	1.7x10 ⁹
2	31500	746	2.35x10 ⁷	1.75x10 ¹⁰	8.6x10 ⁶
3	144000	620	8.9x10 ⁷	5.54x10 ¹⁰	3.1x10 ⁸
4	31500	494	1.5x10 ⁷	7.7x10 ⁹	8.6x10 ⁶
5	105600	220	2.3x10 ⁷	5.1x10 ⁹	1.7x10 ⁹
	418200	620'	2.59x10 ⁸	1.96x10 ¹¹	0.03x10 ¹¹

$$I_{xx} = 1.99 \times 10^{11} - 418200 \times 620^2$$

$$= 3.8 \times 10^{10} \text{ F}^4$$



Loads in Structure :

- a) Permanent : Dead Load : Concrete, mechanical + pavement.
Water Head due to Dead Load.
- b) Live Load : 400 psf @ top deck , 150 psf @ middle deck
200 psf @ lower deck
Water Head due to Live Load, mooring of vessels.
- c) Deformation : Prestressing , Δ Temperature + Shrinkage + Creep
- d) Environmental : Wave Load : 3' high wave , $L = 1000'$
(Sea State 2) Current @ 3 knots @ Water Surface

Structural System :

The system consists of a Floating concrete-box right angle, multiple berth pier kept in place by 2 - Anchor pylons, one at each end of the pier.

The concrete box (see Dwg. 6) carries all external vertical loads: D.L., L.L., water Head, wave Sagging and Hogging, through local and global axial, shear and bending as the pier is free to move vertically at the pylons.

All horizontal forces due to wind, Berthing and current will be transferred to the pylons through axial, and shear deformations in the concrete-box structure. These external loads will be eventually carried to the soil by piles set in the pylon foundation.

State of stresses @ a Critical point

Point C : Critical in vertical loading as global bending stresses (primary stresses) due to wave hogging are highest.

Loading	Stresses at Point C	
	Local	Global
D.L.	Bending _{yy}	—
L.L.	"	—
*1 Buoyancy	"	Axial x,y
Wave S. & H.	—	Bending _{yy}
*2 Wind	—	Axial x,y
*3 Current	—	Axial x,y
Prestressing	Bending _{xx,yy}	Axial x,y

*1 assume no internal pressure due to fluids in tanks

*2 assume same number of ships nested at either side of spine pier (balance horizontal reaction)

Cont' Loads in the Structure:

- d) Environmental : - Wind @ 15 knots
e) Accidental : - Collision (ignore)
- Explosion etc (ignore)

Load Combination: (NAVFAC DM-26)

Group	loading	% Unit All. Stress
a	$D+L+I+E+B+W_b+Be+P$	100%
d	Group a. + 30% W + W_s + Wave + C + S_s + S + T	140%

D: Dead load

L: Live load

I: Impact (ignore)

E: Earth Pressure (non existing)

B: Buoyancy

W_b : Water Pressure

Be : Berthing load

P: Prestressing

W: Wind on Structure

W_s : Wind on Ships

Wave: Hogging or Sagging, Surge, Sway

C: Current

S_s : Surge, Sway (ignore as have fixed moorings)

S: Shrinkage (ignore)

T: Temperature (ignore)



INTERNATIONAL
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133

PROJECT: NAVY PIER

ITEM: Floating Marine Pier, Sch. 2

DESIGN: Environmental Loads X-X

DATE: 11/83 RM

SHEET:

4-2.5

OF

REVISION:

ABS Wave Moments and Shears

Section 6.3.2 (Total Bending Moment about y-y axis due to Sea State 2 to waves in the X direction)

$$M_{t_{xx}} = M_{sw_{xx}} + M_{w_{xx}}$$

$$M_{s_{xx}} = \text{Still Water Bending Moment} \\ = 0 \quad \text{Assuming uniform D.L. \& L.L. distribution}$$

$$M_{w_{xx}} = \text{Maximum Wave Induced Bending Moment} \\ = C_2 L^2 B H_e K_b$$

$$K_b = 1.0 \quad \text{For } C_b > 0.80$$

$$C_b = 1.0 \quad \text{assumed}$$

$$C_2 = [6.53 C_b + 0.57] \times 10^{-4} \\ = [6.53 \times 1.0 + 0.57] \times 10^{-4} \\ = 7.1 \times 10^{-4}$$

$$L = 1000' \quad (X-X \text{ axis})$$

$$B = 418200/900' = 465' \quad (\text{average})$$

$$H_e = [4.5 \times 1000' - 0.00216 \times 1000'^2 + 335] \times 10^{-2} \quad 720 < L \leq 1000' \\ = 26.75'$$

$$M_{w_{xx}} = 7.1 \times 10^{-4} (1000')^2 \times 465' \times 26.75' \times 1.0 \times 2.24 \\ = 19.8 \times 10^6 \text{ K-F}$$

Determine Allowable Moment that must be resisted by pier @ the 160' wide section :

Sea State 2 $\longrightarrow$ Max. Wave Height = 3'

$$M_{max_{xx}} = \frac{H}{H_e} \times M_{w_{xx}} \quad \text{@ Midship}$$

$$= \frac{3'}{26.75'} \times 19.8 \times 10^6 \\ = 2.22 \times 10^6 \text{ K-F}$$



INTERNATIONAL
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315 Bay St., San Francisco, Ca. 94133

PROJECT: Navy Piers

ITEM: Floating Marine Pier, Sch. 2

DESIGN: Environmental Loads: Waves

DATE: 11/83 AM

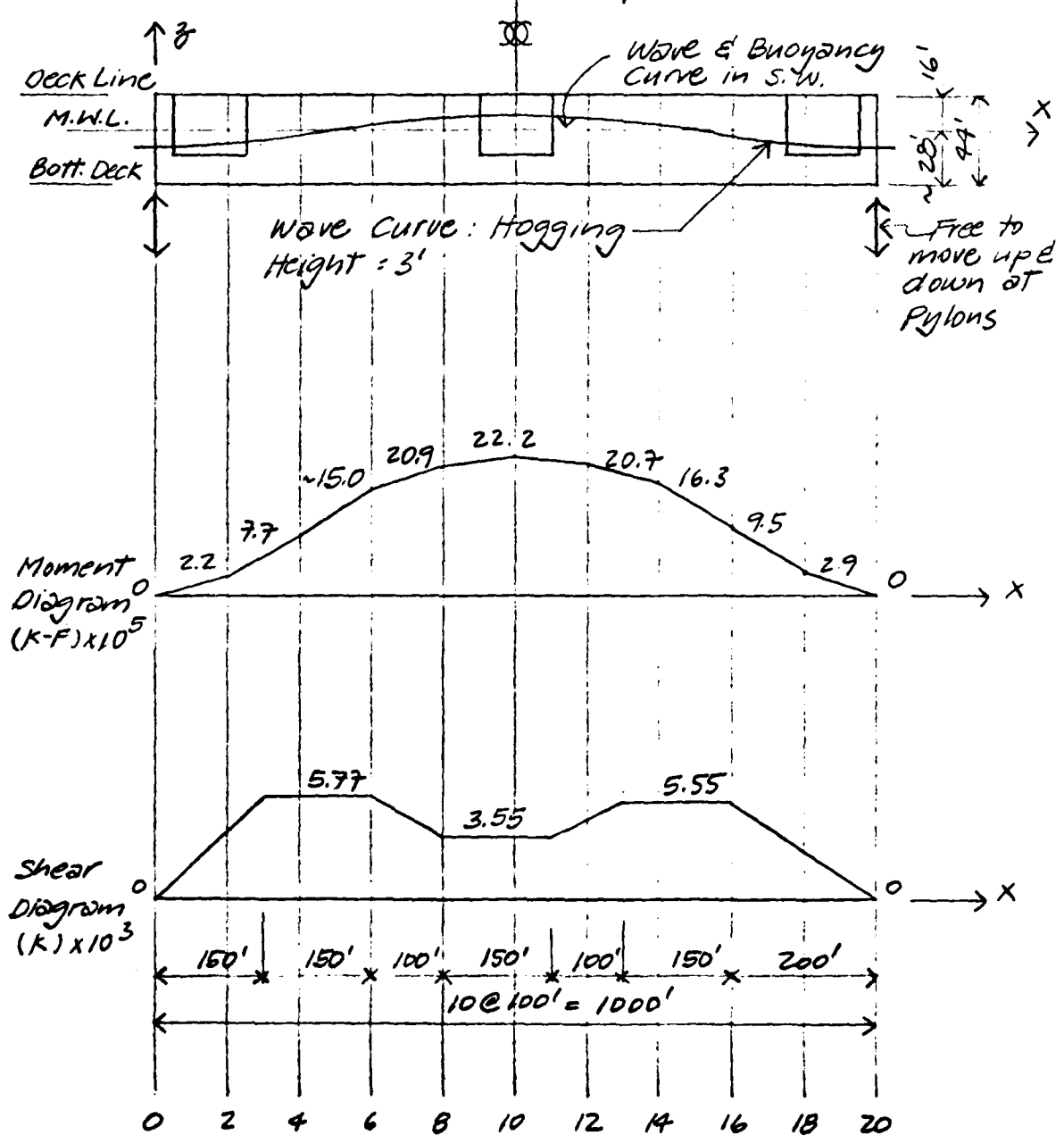
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OF

REVISION:

Wave Moments and Shear Envelopes (X-X direction)



ABS Wave Bending Moment and Shear Distribution, X-X

$M_{max yy} = 2.22 \times 10^6 \text{ K-F}$ $L = 1000'$

Stations at 100'-0" along X axis

Bending Moment (Wave Height = 3')

Station	Bending Moment (Wave Height = 3')	K-F
0	0	0
2	$0.10 \times 2.22 \times 10^6$	= 220000
4	$0.35 \times 2.22 \times 10^6$	= 770000
6	$0.68 \times 2.22 \times 10^6$	= 1496000
8	$0.95 \times 2.22 \times 10^6$	= 2090000
10	$1.00 \times 2.22 \times 10^6$	= 2220000
12	$0.94 \times 2.22 \times 10^6$	= 2070000
14	$0.74 \times 2.22 \times 10^6$	= 1628000
16	$0.43 \times 2.22 \times 10^6$	= 954600
18	$0.13 \times 2.22 \times 10^6$	= 288600
20	0	0

M_{max}
←
at midship

Shear Distribution

$F_{max xx} = F_{syy} + F_{wyy}$

$F_{sxx} = \text{Still Water Shear}$
 $= 0$

$F_{wxx} = KM_{wxx}/L$

$M_{wxx} = \text{Max. Wave Moment X.}$

$= 2220000 \text{ K-F}$

$L = 1000'$

$K = 0 \text{ at ends}$

$= 2.6, 0.15L < y < 0.30L$

$= 1.6, 0.4L < y < 0.55L$

$= 2.5, 0.65L < y < 0.8L$

K	$F_{wxx} (K)$
0	0
2.6	5772
1.6	3552
2.5	5550



STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca 94133

PROJECT: Navy Piers

ITEM: Floating Marine Pier, Sch. 2

DESIGN: Section Properties C A

DATE: 11/83 KM

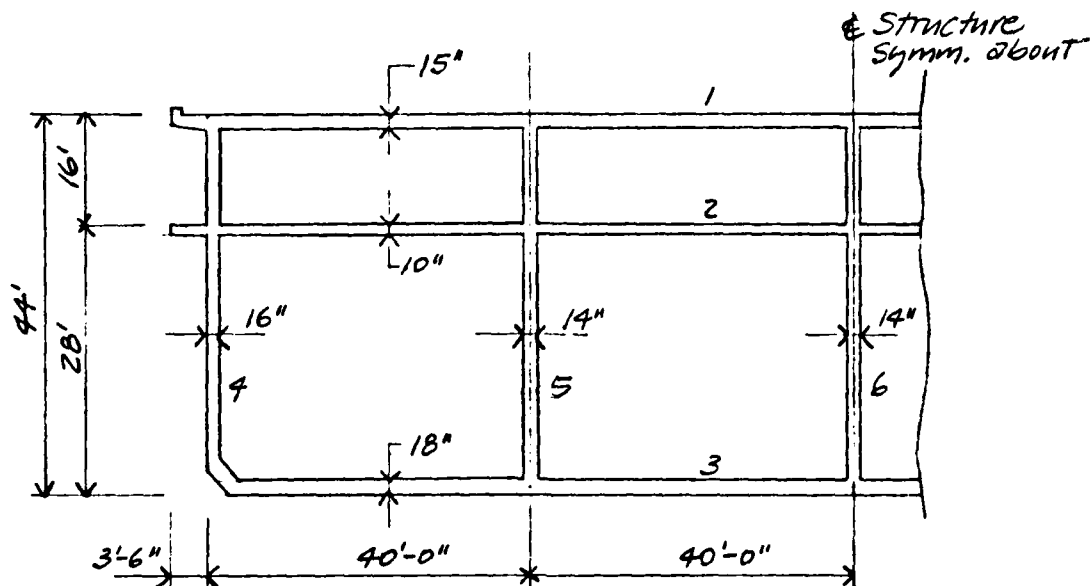
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REVISION:

Critical Section Properties near Midship :



Section C (A)

	A ^{ft}	y ^{ft}	Ay ^{ft}	Ay ² ^{ft}	I _{ox} ^{ft}	y _c	S _t
1	100.5	43.4	4363	189337	13	y _b	S _b
2	66.8	27.6	1843	50855	4		
3	115.4	0.75	86	64	22		
4	58.7	22	1291	28395	9465		
5	51	22	1129	24838	8281		
6	25.7	22	565	12423	4141		
	418.1	22.2	9277	305914	21926	21.8	10971
						22.2	11173

$$I_{ox} = [(305914 + 21926 - 418.1 \times 22.2^2) \times 2] \\ = 243563 \text{ ft}^4$$

$$S_t = 243563 / 21.8 \\ = 10971 \text{ ft}^3 > S_{req} = 9418 \text{ ft}^3 \checkmark$$

$$S_b = 243563 / 22.2 \\ = 11173 \text{ ft}^3 > S_{req} = 9418 \text{ ft}^3 \checkmark$$

Minimum Allowable Section Modulus @ Section (A)

$$M_{max@A} = [2.22 - (2.22 - 2.09)/100' \times 65'] \times 10^6$$

$$= 2136000 \text{ K-F}$$

σ_{bmax} = Maximum Allowable Bending Stresses :

$$= 0.45 F_c / 2 \quad F_c = 7000 \text{ psi}$$

$$= 1575 \text{ psi}$$

Minimum $S_{t,b} = M_{max} / \sigma_{bmax}$
at point (A)

$$= 2136000 / (1575 \times 144 / 1000)$$

$$= 9418 \text{ Ft}^3$$

This is the critical section of the Pier for wave loading parallel to the X-axis because of the high moment and the narrow width at this location.

Section Modulus provided at this Section : See pg.

$$S_t = 11173 \text{ Ft}^3$$

$$S_b = 10971 \text{ Ft}^3 > 9418 \text{ Ft}^3$$

$\therefore$ satisfactory in bending

Maximum Wave Height Allowable For $S_b = 10971 \text{ Ft}^3$:

$$H = \frac{M_{max,xx}}{M_{wave,xx}(3')} \times H_e$$

$$M_{max.} = S_b \times \sigma_{bmax.}$$

$$= 10971 \times 1575 \times 144 / 1000$$

$$= 2.49 \times 10^6 \text{ K-F}$$

$$H = \frac{2.49 \times 10^6}{2.14 \times 10^6} \times 3'$$

$$= \underline{\underline{3.5' \text{ wave}}}$$

Wind Loading in X-X direction

- ASSUME : a) Sea state 2, Wind Velocity of 50 MPH in X-X direction, i.e., @ 90° to the ships
b) All ships nested (12) in the Pier.
c) Ships nested in Leeward side of every finger will take 50% of wind loads on ships at the windward side of the Pier.

12, DD-963 nested at Pier : 6, DD-963, 100% exposure
6, DD-963, 50% exposure

From DM 26 : $F_w = C_{yw} \times \frac{1}{2} P_w V_w^2 A_s$

Wind @ 90° to ship, $C_{yw} = 1.0$

$P_w = 0.00237 \frac{\#-sec^2}{F^4}$ @ 68°F

$V_w =$ Velocity, F/sec @ 40' Above waterline
= 73 F/sec

For DD-963

DM 26.6 Gives data on smaller DDs

DD-692 L=377' h=22' $A_s = 10200 \phi$

DD-931 L=418' h=31' $A_s = 13000 \phi$

Extrapolating

DD-963 L=564' h=39.6' $A_s = 22600 \phi$

At 100% exposure :

$$(F_w)_{DD} = 1.0 \times \left(\frac{1}{2}\right) (0.00237) (73)^2 (22600) \\ = 144 \text{ K/ship (windward)} \quad \leftarrow$$

At 50% exposure :

$$\left(\frac{1}{2}\right) (F_w)_{DD} = 144/2 = 72 \text{ K/leeward ship} \quad \leftarrow$$

For Floating Pier : $A_s = 16' \times 200' = 3200 \phi$

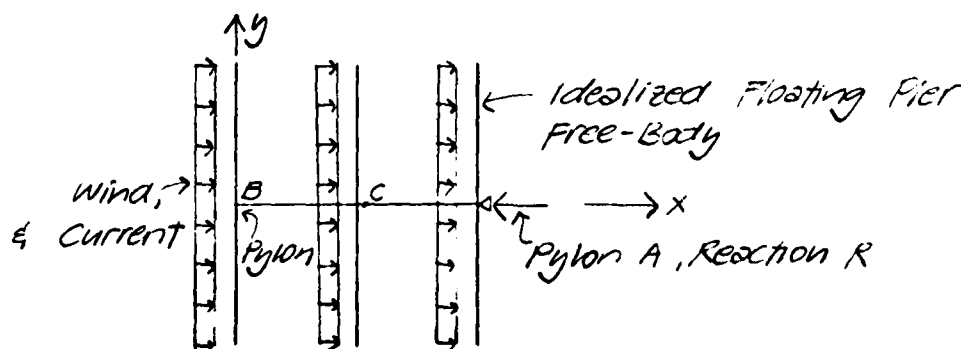
$$(F_w)_{FP} = 144 \times 3200 / 22600 = 20 \text{ K} \quad \leftarrow$$

Cont Wind Loading

At point C

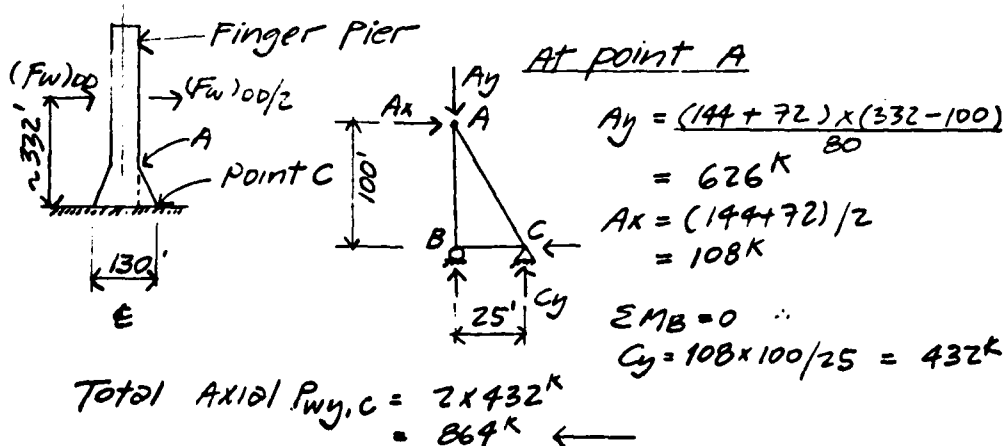
$$\text{Total wind load}_C = \Sigma \text{Tributary } (F_w)_{DO} + (F_w)_{FP}$$

a) Global Axial x @ Point C :



$$\begin{aligned} \text{Total Axial } P_{wx,C} &\approx 2(F_w)_{DO} + 2(F_w)_{DO}/2 \\ &= 2 \times 144 + 2 \times 72 \\ &= 432^K \quad \leftarrow \end{aligned}$$

b) Global Axial y @ Point C : Due to Finger Bending



At Pylon A & B

$$\text{Axial @ Pylons} = [6(144 + 72) + 20]/2 = 658^K \quad \leftarrow$$

Current loading in X-X direction

- Assume: a) Use same assumptions for wind loading
b) 3 knots current
c) Ships at leeward side will pick 33% of current force on those in windward side.
d) All vessels are secured to mooring

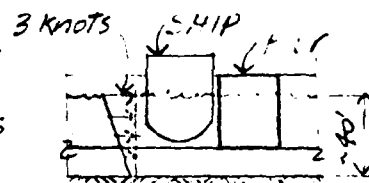
From new DM 26, $F_c = C_{yc} (1/2) \rho_c V_c^2 (LWL) T$

C_{yc} From Fig. 4: When $\frac{W.D.}{T} = \frac{40}{29} \frac{\text{Water Depth}}{\text{Draft}} = 1.4$

Then $C_{yc} \approx 1.9$

$\rho_c = 1.9876 \text{ #-sec}^2/\text{F}^4$ @ 68°F

$V_c = \text{avg. current on hull, F/S}$
 $= (3 + 0.83)/2$
 $= 1.91 \text{ knots}$
 $= 3.22 \text{ F/S}$



$LWL = \text{Length @ Waterline, Ft.}$

$T = \text{Draft, Ft.}$

For DD-963

$$(F_c)_{DD} = 1.9 \times \frac{1}{2} \times 1.9876 \times 3.22^2 \times 530' \times 29' = 301 \text{ K} \leftarrow$$

$$0.33(F_c)_{DD} = 99 \text{ K} \leftarrow$$

For Floating Pier: $T = 25'$ $\frac{W.D.}{T} = \frac{40}{25} = 1.6$

Then $C_{yc} = 1.8$

$V_c = 2.1 \text{ knots}$
 $= 3.55 \text{ F/S}$

$LWL = \text{length @ Waterline, Ft.}$

$T = \text{Draft, Ft.}$

$$(F_c)_{F.P.} = 1.8 (1/2) (1.9876) (3.55)^2 (200') \times 25' = 113 \text{ K} \leftarrow$$

Cont' Current loading in X-X direction :

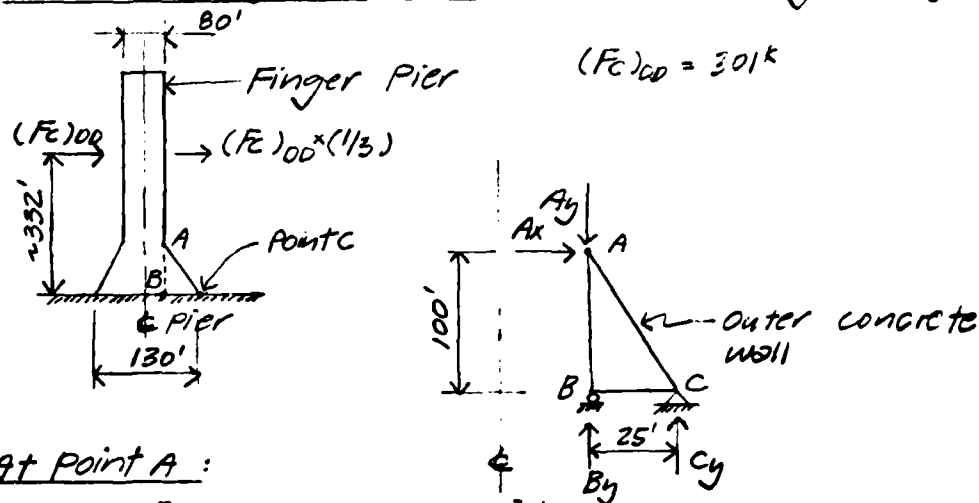
At point C

$$\text{Total Current Load}_C = \Sigma \text{Tributary } (F_c)_{DD} + (F_c)_{F.P.}$$

a) Global Axial x @ Point C :

$$\begin{aligned} \text{Total Axial } P_{x,C} &= 2(F_c)_{DD} + 2(F_c)_{DD} \\ &= 2(301) + 2(99) \\ &= \underline{800K} \leftarrow \end{aligned}$$

b) Global Axial y @ Point C = Due to bending in Finger



At Point A :

$$\begin{aligned} A_y &= [(301 + 99) \times (332 - 100)] / 80 \\ &= 1110 K \end{aligned}$$

$$\begin{aligned} A_x &= (301 + 99) / 2 \\ &= 200 K \end{aligned}$$

$$\begin{aligned} \Sigma M_B = 0 \quad \therefore C_y &= 200 K \times 100' / 25' \\ &= 800 K \end{aligned}$$

$$\text{Total Axial } P_{y,C} \text{ Point C: } 2 \times 800 = \underline{1600 K} \leftarrow$$

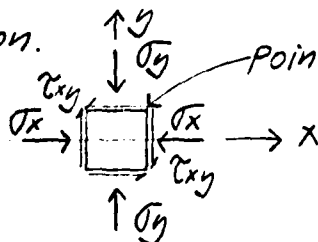
At Pylon A and B:

$$\begin{aligned} \text{Total Axial } P_{x,B} &= [6(301 + 99) / 2 + 113 K] \\ &= \underline{1313 K} \leftarrow \end{aligned}$$

Primary Stresses at Point C : due to Environmental Loads

in X-X direction.

- 1) Wave
- 2) Wind and
- 3) Current



Point C: Bottom slab (X-Y plane)
near bulkhead

State of Principal Stresses @ C

- 1) Wave induced stresses (at Section A, through Pt. C)

$$M_{max} = 2136000 \text{ K-F (due to Hogging)}$$

$$\sigma_{Top, X} = 2136000 / 11173 \text{ F}^3 = 191 \text{ KSF} = 1328 \text{ PSI (T)}$$

$$\sigma_{Bot, X} = 2136000 / 10971 = 195 \text{ KSF} = 1352 \text{ PSI (C)} \leftarrow$$

- 2) Wind X-X Induced Stresses : @ Point C

Axial X : $\sigma_x = P_{wx} / A_y$ $A_y = \text{Transverse Cross Section}$
@ C $= 482 \times 2$
 $= 964 \text{ F}^2$

$$\sigma_x = 432 / 964 = 0.45 \text{ KSF} = 3 \text{ PSI (C)} \leftarrow$$

Axial Y : $\sigma_y = P_{wy} / A_x$ $A_x = \text{Longitudinal Cross-Sect.}$
@ C $= 864 / 187 \text{ SF}^*$ Assume axial load is distributed in a width of 40' only.

$$= 4.6 \text{ KSF} \leftarrow$$

$$= 32 \text{ PSI (C)}$$

$$A_x = (10 + 10 + 5) / 12 \times 40' + 1' \times 44' = 187 \text{ F}$$

- 3) Current X-X Induced Stresses :

Axial X : $\sigma_x = P_{cx} / A_y$ $P_{cx} = 800 \text{ K}$
 $= 800 / 964$
 $= 0.83 = 6 \text{ PSI} \leftarrow$

Cont' Current Induced Stresses :

$$\begin{aligned} \text{Axial } y : \sigma_y &= P_{cy} / A_x & A_x &= \phi \\ & & P_{cy} &= 1600^K \\ \sigma_y &= 1600 / 187 \\ &= 8.6 \times 1000 / 174 \\ &= 59 \text{ psi} \end{aligned} \quad \leftarrow$$

Total Primary Stresses @ Point C due to Environmental Loads :

$$\begin{aligned} \Sigma \sigma_x &= 1352 + 3 + 6 \\ &= 1361 \text{ psi} \end{aligned} \quad \leftarrow$$

$$\begin{aligned} \Sigma \sigma_y &= 32 + 59 \\ &= 91 \text{ psi} \end{aligned} \quad \leftarrow$$

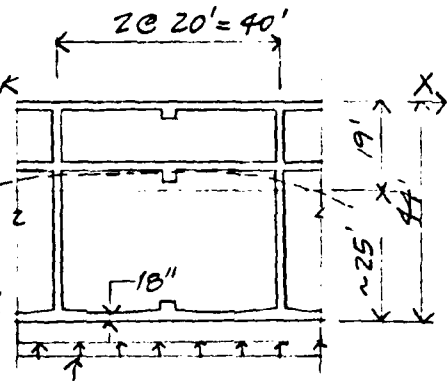
$$\Sigma \tau_{xy} = 0 \text{ due to Global Loading}$$

Secondary Stresses @ Pt. C :

1) Buoyancy Bending :

Longitudinal Slab Span: 20'
t slab = 20" @ support

TOP DECK



2) Static Pressure :

D.L.:

Concrete: $18" / 12 \times 150 = 0.225 \text{ KSF}$

Mech. + Misc. = 0.05

Static Buoy: $27 \times 0.062 = -1.67$

Resultant : -1.4 KSF

Buoyancy @ $25' + 1.5' = 26.5'$
Wave ht.

Dynamic Pressure : Does not control in this case because Sea State 2 is mild.

Slab Section Properties at Pt. C

Haunched Slab, $t = 20"$

t midspan = 16"

$I = 12 \times 20^3 / 12 = 8000 \text{ in}^4$
 $S = 800 \text{ in}^3$

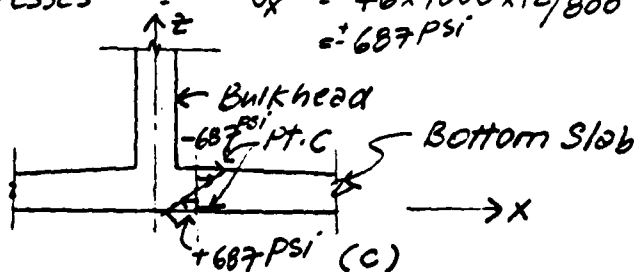
b) Moment : $L_n \approx 19'$, $L_i = 40'$

$L_i / L_n > 2 \rightarrow \text{One-way Slab}$

$M_{max} = W L_n^2 / 11$
 $= 1.4 \times 19^2 / 11$
 $= 46 \text{ K-F}$

c) Stresses

$\sigma_x = 46 \times 1000 \times 12 / 800$
 $= 687 \text{ PSI}$

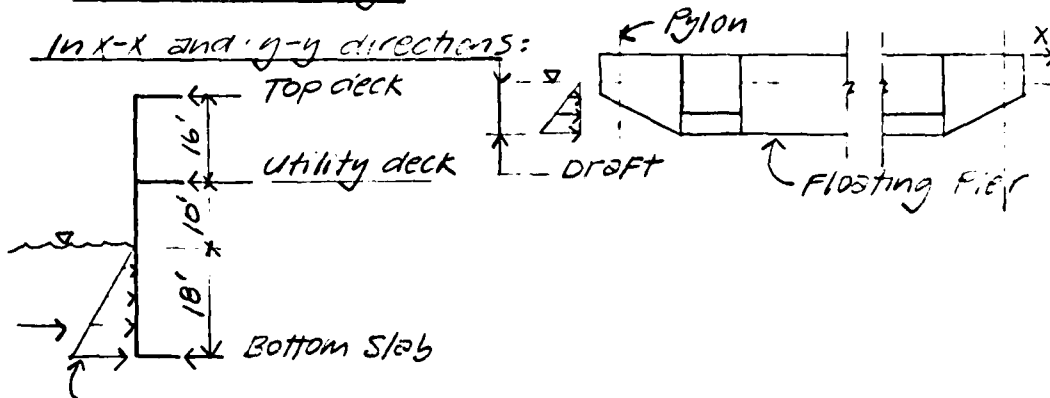


Cont' Buoyancy Secondary Stresses @ Point C

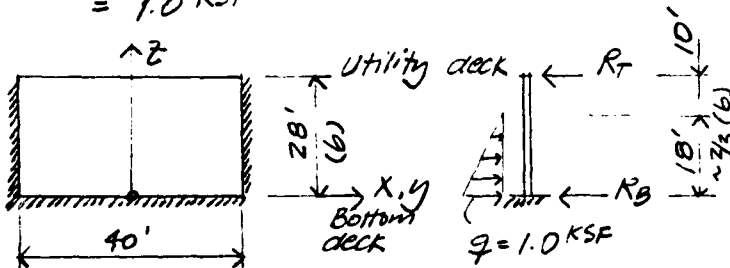
2) Buoyancy Axial Global Stresses:

a) Deck Load only: Draft $\approx 18'$

In x-x and y-y directions:



$$P_B = 0.062 \times 16' \\ = 1.0 \text{ KSF}$$



From Formulas for Stress and Strain by Roark & Young,

Case no. 10 dd : $a/b = 40'/28' = 1.43 \therefore$
 $r_1 \approx 0.29$

At $x=0, z=0 \Rightarrow R_B = r_1 q$
or $y=0$
 $= 0.29 \times 1.0 \times 28'$
 $= 8.1 \text{ k/ft}$ or 8 k ←

Stresses @ Pt. C :

$$\sigma_{x,y} = R_B/A$$

$$= 8 \times 1000 / 240$$

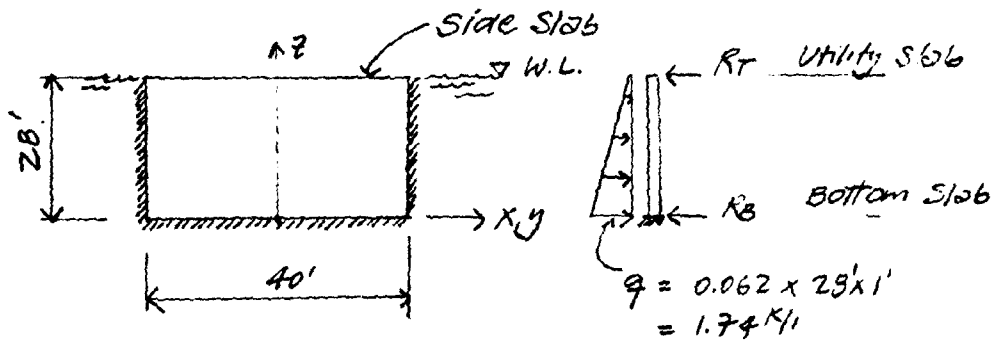
$$= 33 \text{ psi}$$

$A = \text{Unit Area}$
 $= 12" \times 20" = 240 \text{ in}^2$ ←

Cont' Axial Global stresses due to Buoyancy:

b) Dead + Live + Tide : $\swarrow$ D.L. Tide, wave

Total Water Head = $25' + 2' = 27'$ say $28'$



Case no. 9d, At $x=0$ and $z=0$
or $y=0$

$$z/b = 1.43 \rightarrow \sigma_1 = 0.376$$

$$\therefore R_b = \sigma_1 q b \\ = 0.376 \times 1.74 \times 28' \\ = 18.3 K'$$

Stresses @ Point C :

$$\sigma_{x,y} = R_b/A \\ = 18300'/240 \\ = 76 \text{ psi}$$

$$A = \text{Unit Area} \\ = 12" \times 20" = 240"^2$$





INTERNATIONAL
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133

PROJECT: NAMI PIERS

ITEM: Floating Marina Pier, Sch. 2

DESIGN: Summary of Stresses @ C

DATE: 11/83 RM

SHEET:

A-2.19

OF

REVISION:

Summary of Stresses @ Point C : Compression (-)
Tension (+)

Loading	Stresses @ Point C (psi)					
	Local		Global		Σ	
	σ_x	σ_y	σ_x	σ_y	σ_x	σ_y
Dead + Live + Buoyancy	+687	—	-76	-76	-687	-76
Wave x-x (Hogg.)	—	—	-1352	—	-1352	—
Wind x-x	—	—	-3	-32	-3	-32
Current x-x	—	—	-6	-59	-6	-59
					-2048 ^{psi}	-167 ^{psi}



STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca 94133

PROJECT: NAK PMS

ITEM: Floating Marine Pier Sch. 2

DESIGN: X-X PRESTRESSING

DATE: 11 90 RM

SHEET:

A-5.20

OF

REVISION:

Prestressing in X direction :

Partial Prestressing : Assume 80% of Prestress
to offset tensile stresses at Bottom due
caused by Wave Action (Sagging)

$$\sigma_{\text{bottom}} = +1352 \text{ psi}$$

$$\begin{aligned} \text{Prestress} &= -0.80 \times 1352 \text{ psi} \\ &= -1082 \text{ psi} \end{aligned}$$

Total Prestressing Force :

$$\begin{aligned} &= 1082 \times 418 \times 2 \times 144 / 1000 \\ &= 130207 \text{ K} \end{aligned}$$

Assume 270 Grade Prestressing Strands :

$$\begin{aligned} A_{\text{required}} &= 130207 / 270 \times 0.70 \text{ ksi} \\ &= 689 \text{ in}^2 \\ &= 4.8 \text{ ft}^2 \end{aligned}$$

Total % Steel in Gross-Area @ Point C :

Mild = 1% (assume)

$$\text{prest.} = 4.8 / 2 \times 418 = 0.006$$

$$= 0.6\%$$

$$\Sigma = 1.6\% \rightarrow 13.4 \phi \quad \leftarrow$$

Transformed Area of concrete @ Point C :

$$\begin{aligned} E_c &\approx 120 \times 33 \sqrt{7000} \\ &= 3.7 \times 10^6 \text{ psi} \end{aligned}$$

$$E_s = 29 \times 10^6 \text{ psi}$$

$$\begin{aligned} A_T &= 936 \phi + 13.4 (29 / 3.7) \\ &= 1038 \phi \quad \text{or} \quad 11\% \text{ increase in Area} \end{aligned}$$

$$\begin{aligned} \text{Prestress} &= -0.80 \times 1352 / 1.11 \\ &= -974 \text{ psi} \quad \leftarrow \end{aligned}$$

$$\begin{aligned} \text{Wave Load} &= 1352 / 1.11 \\ &= 1218 \text{ psi} \quad \leftarrow \end{aligned}$$



STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca 94133

PROJECT: Navy Piers

ITEM: Floating Marine Pier, Sch. 2

DESIGN: Final Stresses @ Pt. C

DATE: due to X-X Loading

SHEET:

A-2.21

OF

REVISION:

Cont' prestressing in x-direction @ Pt. C :

$$\begin{aligned}\text{Residual Tension} &= \text{Wave (Sag.)} - \text{Prestress} \\ &= 1218 \text{ psi} - 974 \text{ psi} \\ &= 244 \text{ psi} \rightarrow \text{Use mild steel}\end{aligned}$$

Final compressive stresses at point C : X-X Loading

$$\begin{aligned}\sigma_x, \max &= \sum \sigma_x + \sigma_x \text{ prestress, includes transformed area} \\ &= \pm 687 \pm 1218 - 3 - 6 - 974 \\ &= \underline{-2888 \text{ psi}} \quad \leftarrow\end{aligned}$$

$$\begin{aligned}\sigma_y, \max &= \sum \sigma_y + \sigma_y \text{ prestress, assume 400 psi} \\ &= -76 - 32 - 59 - 400 \\ &= \underline{-567 \text{ psi}} \quad \leftarrow\end{aligned}$$

$\tau_{xy} \approx 0$ at bott. slab extreme fibers

Principal Stress at C :

$$\begin{aligned}\sigma_{\max, \min} &= \pm \frac{1}{2} \sqrt{(\sigma_x - \sigma_y)^2 + (2\tau)^2} \\ &= \pm \frac{1}{2} \sqrt{(-2888 + 567)^2 + 0} \\ &= \pm 1161 \text{ psi}\end{aligned}$$

$$\begin{aligned}\sigma_{\max, \min} &= \frac{1}{2} (\sigma_x + \sigma_y) \pm \tau(\max) \\ &= \frac{1}{2} (-2888 - 567) \pm 1161 \\ &= \underline{-2889 \text{ psi}, -567 \text{ psi}} \quad \leftarrow\end{aligned}$$

$$\begin{aligned}F_c \text{ required} &= (\sigma_{\max} / 0.45) \\ &= [2889 / 0.45] \\ &= 6420 \text{ psi}\end{aligned}$$

Use Lightweight concrete $F_c = 7000 \text{ psi}$ $\leftarrow$



INTERNATIONAL
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca 94133

PROJECT: Navy Piers
ITEM: Floating Marine Pier, Sch. 2
DESIGN: Environmental Loads, y-y
DATE: 11/93 RM

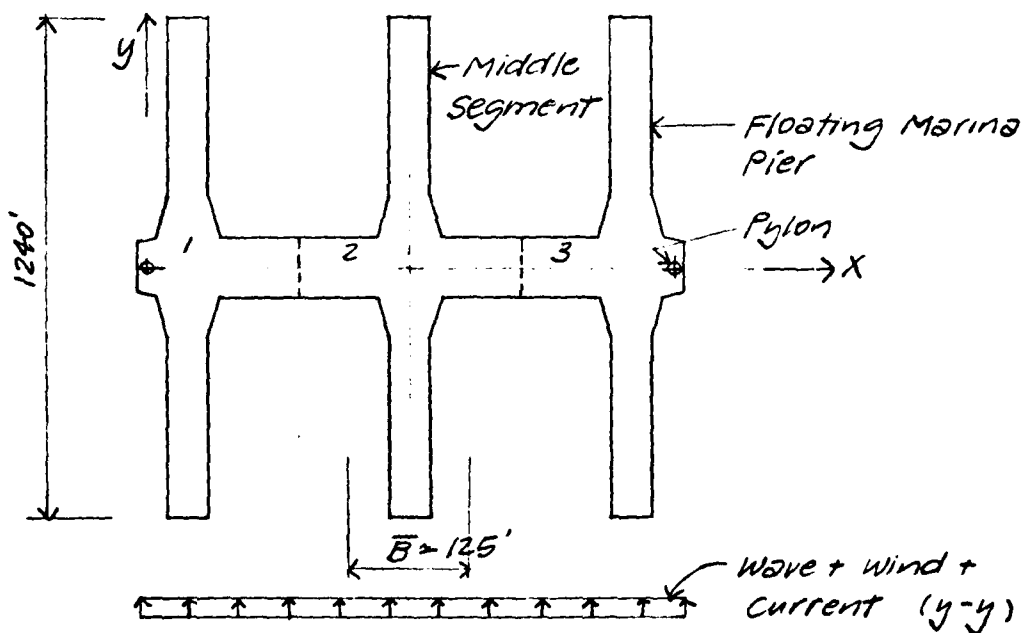
SHEET: 4-2.22
OF
REVISION:

Environmental loading in y-y direction :

Wave loading :

ABS Wave Moments and Shears @ Sea State 2 :

For wave loading in y-y direction one can subdivide the Floating pier in 3 segments. Each segment contains the two opposite Fingers plus the portion of the spine that connects them. Using this assumption then the critical segment would be the widest one which is #2 below.



Wave Moment at Middle Segment (2) :

$$M_{tgy} = M_{syy} + M_{Wgy}$$

M_{syy} = Still water Bending Moment

= 0 $\therefore$ assume relatively uniform D.L & L.L distribution, tendon eccentricity will balance any possible still water moments.



INTERNATIONAL
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca 94133

PROJECT: Navy Piers

ITEM: Floating Marine Piers, Sch. 2

DESIGN: Env. Loading in y-y dir.

DATE: III 83 KM

SHEET:

A-2.23

OF

REVISION:

Cont' wave moments y_y at Segment 2

M_{wyy} at Middle Segment (2)

$$M_{wyy} = C_2 L^2 \bar{B} H_e K_b, \text{ Max. wave moment @ } y = 0$$

$$K_b = 1.0 \text{ For } C_b > 0.80$$

$$C_b = 1.0 \text{ assumed}$$

$$C_2 = 7.1 \times 10^{-4}$$

$$L = 1240'$$

$$\bar{B}_{y=80'} = [440' \times 80' + 100' \times 100' + 80' \times 410'] / 620'$$

$$= 125'$$

$$H_e = 26.75' \quad 1000' < L \leq 1400'$$

$$M_{wyy} \text{ at Spine} = 7.1 \times 10^{-4} \times 1240^2 \times 125 \times 26.75 \times 1.0 \times 2.24$$

$$= 8.18 \times 10^6 \text{ K-F}$$

Maximum moment that can occur at Segment 2 (Hogg.)
Face due to 3' high waves:

$$M_{\text{max}, wyy} \text{ at spine} = \frac{H}{H_e} \times M_{wyy}$$

$$\frac{3'}{26.75'} \times 8.18 \times 10^6 \text{ K-F}$$

$$= \underline{9.2 \times 10^5 \text{ K-F}} \quad \leftarrow$$

σ_{max} = Maximum allowable wave bending stress:

$$= 0.45 F_c / 2$$

$$= 1575 \text{ psi}$$

$$F_c = 7000 \text{ psi (assumed through)}$$



INTERNATIONAL
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315 Bay St., San Francisco, Ca 94133

PROJECT: NSM Piers

ITEM: Floating Marine Pier, S-2

DESIGN: Seismic loads in y-y dir.

DATE: 11/35 KM

SHEET:

A-2.24

OF

REVISION:

ABS Wave Bending Moment and Shear Distribution :

Wave in y-y direction

$M_{max,y} = 9.2 \times 10^5 \text{ K-F @ centerline}$ $L = 1240'$

Stations at 124'-0" along y axis

Station	Bending Moment (Wave Height = 3')		
0	0	=	0
2	$0.10 \times 9.2 \times 10^5$	=	92000
4	$0.35 \times 9.2 \times 10^5$	=	322000
6	$0.68 \times 9.2 \times 10^5$	=	625600
8	$0.95 \times 9.2 \times 10^5$	=	874000
centerline Moment → 10	$1.00 \times 9.2 \times 10^5$	=	920000
12	$0.94 \times 9.2 \times 10^5$	=	864800
14	$0.74 \times 9.2 \times 10^5$	=	680800
16	$0.43 \times 9.2 \times 10^5$	=	395600
18	$0.13 \times 9.2 \times 10^5$	=	119600
20	0	=	0

Shear Distribution

$$F_{max,y} = F_{sy} + F_{wy}$$

F_{sy} = still water shear
= 0

$$F_{wy} = KM/L$$

$$\begin{aligned} M_w &= \text{Max. wave moment} \\ &= 920000 \text{ K-F} \\ L &= 1240' \end{aligned}$$

$K = 0$, ends

$$= 2.6, 186' < y < 372'$$

$$= 1.6, 496' < y < 692'$$

$$= 2.5, 806' < y < 992'$$

$$F_{wy}(K=0) = 0$$

$$(K=2.6) = 1929 \text{ K}$$

$$(K=1.6) = 1187 \text{ K}$$

$$(K=2.5) = 1855 \text{ K}$$



STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca 94133

PROJECT: Navy Piers

ITEM: Floating Marina Pier Sch 2

DESIGN: Envir. Loads in y-y direct

DATE: 11/93 AM

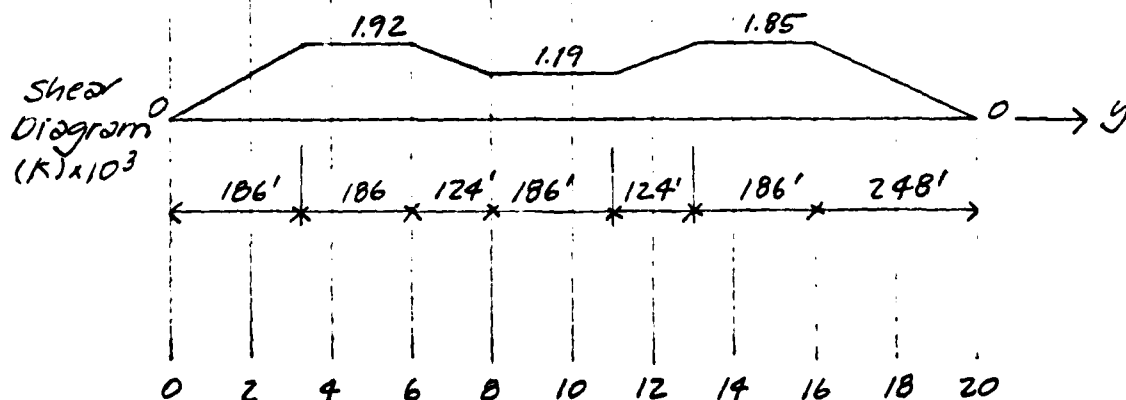
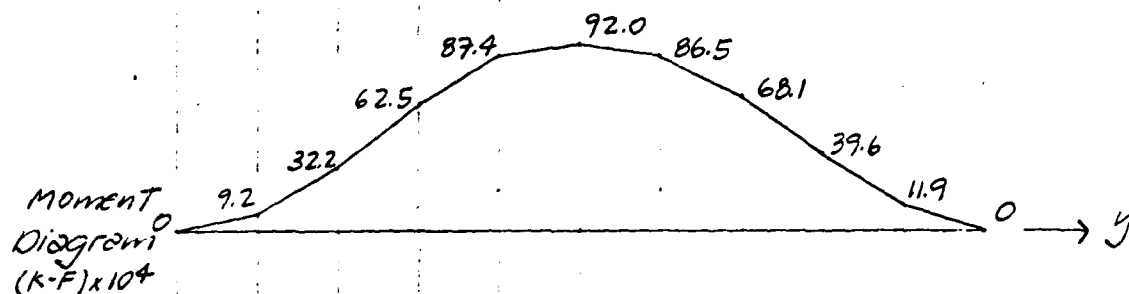
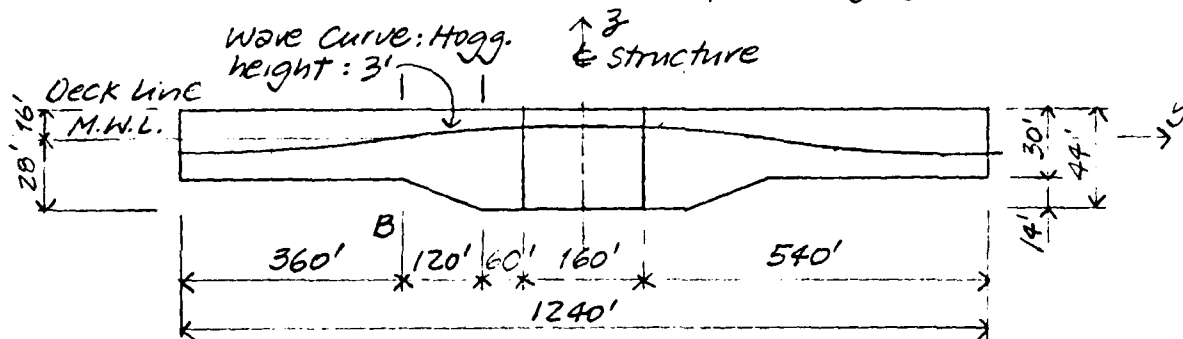
SHEET:

A-2.05

OF

REVISION:

Wave Moments and Shear Envelopes (y-y direction)





INTERNATIONAL
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315 Bay St., San Francisco, Ca 94133

PROJECT: NAVY PIERS
ITEM: Floating Marine Pier Sch. 2
DESIGN: NAVY CONTRACTS in g-g & l-t
DATE: 11/83 AM

SHEET: A-2.26
OF: _____
REVISION: _____

Check Finger Pier Section For Wave Bending

Section at B : $F_c = 7000 \text{ psi}$

$$M_{wyg} \text{ at } B = 322000 + (625000 - 322000) \times 112/124'$$

$$= 595677 \text{ K-F}$$

$$\text{Shear Force}_B = 1920 \text{ K}$$

$$\sigma_{max} \text{ @ } B = \text{Maximum allowable bending stresses}$$

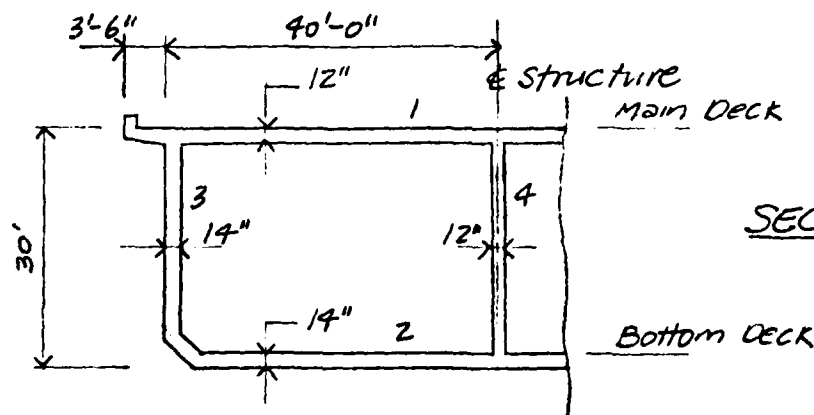
$$= 0.45 \times 7000/2$$

$$= 1575 \text{ psi}$$

Minimum $\sigma_{t,b}$ required at (B)

$$\sigma_{t,b} = 595677 / (1575 \times 144) / 1000$$

$$= 2626 \text{ F}^3$$



SECTION (B)

	A ϕ	y'	Ay F^3	Ay ² F^4	Iox F^4	y _t y _b	σ_t σ_b
1	43.5	29.5	1283	37856	4		
2	46.7	0.58	27	16	5		
3	32.5	15.1	490	7402	2096		
4	13.9	15.1	210	3193	898		
	<u>136.6</u>	<u>14.7</u>	<u>2010</u>	<u>48447</u>	<u>3003</u>	15.3 14.7	2867 2984

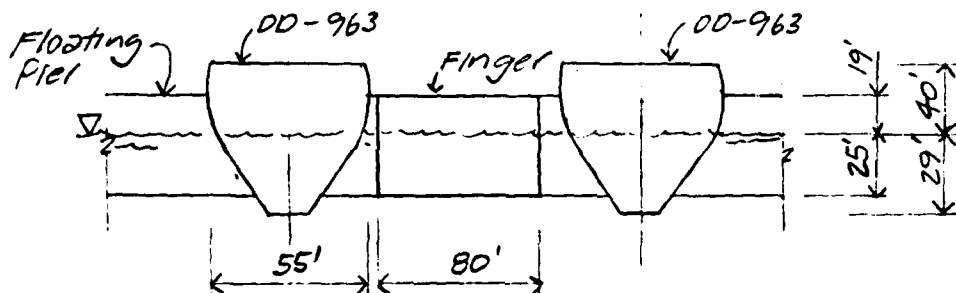
$$I_{ox} = [48447 + 3003 - 136.6 \times 14.7^2] \times 2$$

$$= 43864 \text{ F}^4$$

$$\sigma_t = 43864 / 15.3 = 2867 \text{ F}^3 > 2626 \text{ F}^3 \checkmark$$

Wind Loading to Pylons in the y-y direction

- Assume :
- a) sea state 2 : Wind velocity of 50 MPH in y-y direction @ 0° to longitudinal axis of ships.
 - b) All (12) ships are nested in the Pier
 - c) Ships nested in Leeward side of every Finger takes 50% of wind loads on ships at the windward side.
 - d) Only DD-963 nested at the pier
 - e) water does not absorbed energy



$$A_{DD} = 55 \times 40' = 2200 \phi \quad A_F = 80 \times 19' = 1520 \phi \quad A_{DD} = 2200 \phi \quad A_p = 19'(1000 - 6 \times 55' - 3 \times 80') = 8170 \phi$$

Total area exposed to wind:

$$A_T = 6(A_{DD}) + 3(A_F) + 6/2(A_{DD}) + A_p \\ = 6 \times 2200 + 3 \times 1520 + 3 \times 2200 + 8170 \\ = 32530 \phi \quad \text{say } 34000 \phi, \text{ to include higher portions of vessels}$$

From DM 26 : $F_W = C_{yw} \times 1/2 P_W V_W^2 A_s$

$$C_{yw} = 1.0, \text{ wind at } 0^\circ \text{ to ships} \\ P_W = 0.00237 \text{ * -sec}^2/\text{ft}^4 \text{ @ } 68^\circ\text{F} \\ V_W = 50 \text{ MPH} = 73 \text{ ft/sec @ } 40' \text{ above waterline} \\ F_W = 1.0 \times 1/2 \times 0.00237 \times 73^2 \times 34000 \phi \\ = 214 \text{ K total}$$

$$\text{Wind load/pylon} = 214 \text{ K} / 2 = \underline{107 \text{ K}}$$



Current loads in y direction :

Use same assumption as current forces in x-direction.
Current velocity : 3 knots at surface.

$$\text{Area}_{\text{DO-963}} = 29' \times 50' \\ = 1450 \text{ } \phi$$

$$\text{Area}_{\text{Finger}} = 25' \times 80' \\ = 2000 \text{ } \phi$$

$$\text{Area}_{\text{Spine}} = 25'(1000' - 6 \times 50' - 3 \times 80') \\ = 11500 \text{ } \phi$$

Total Area exposed to current :

$$\begin{aligned} A_{CT} &= 6(A_{DO}) + 6(0.5 A_{DO}) + 3(A_F) + A_{\text{Spine}} \\ &= 6 \times 1450 + 6 \times 0.5 \times 1450 + 3 \times 2000 + 11500 \\ &= 30550 \text{ } \phi \quad \text{say } 31000 \text{ } \phi \end{aligned}$$

New DM 26.6 (Pg. 26.6-3)

$$F_C = \sum C_{yc} (1/2) \rho C_D V_C^2 A_C$$

$$\begin{aligned} \text{When } \frac{W.D.}{T} &= \frac{40}{29} \quad \left(\frac{\text{Water depth}}{\text{Draft}} \right) \\ &= 1.4 \quad \text{Then } C_{yc} \approx 1.9 \end{aligned}$$

$$\rho_C = 1.9876 \text{ } \# \cdot \text{sec}^2 / \text{ft}^4 \text{ @ } 68^\circ \text{F}$$

$$V_{Cs} = \text{avg. current, f/s}$$

$$= (3 + 0.83) / 2$$

$$= 1.9 \text{ knots} = 3.22 \text{ f/s @ ships}$$

$$\overline{V}_{Cp} = (3 + 1.43) / 2$$

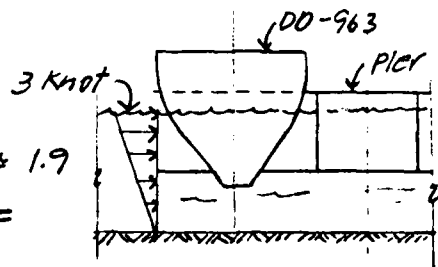
$$= 2.21 \text{ knots}$$

$$= 3.74 \text{ f/s @ Pier}$$

$$W.D./T = 40/25 = 1.6 \quad \text{Then } C_{yc \text{ pier}} = 1.7$$

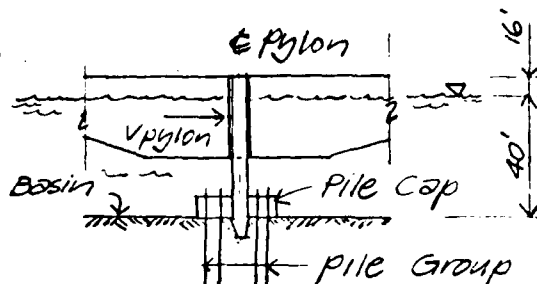
$$\begin{aligned} \text{Total Force}_C &= (1/2) (1.987) [1.9 \times 3.22^2 \times 13050 \text{ } \phi + 1.6 \times 3.74^2 \times 17500 \text{ } \phi] \\ &= 645 \text{ K} \end{aligned}$$

$$\text{Total Current load to each Pylon} = \sim 645 / 2 = \underline{\underline{323 \text{ K}}} \leftarrow$$



Total Environmental Load to Pylon in y-direction:

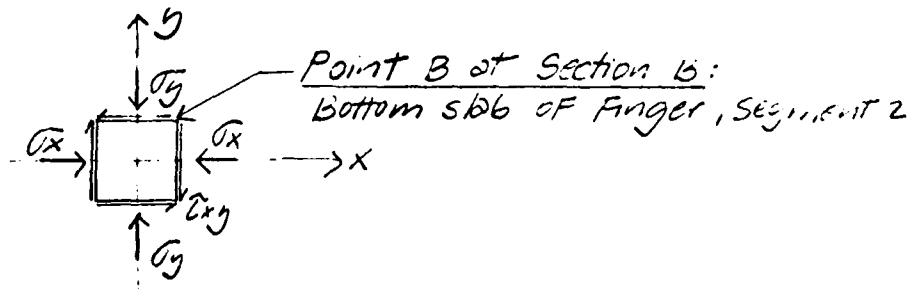
$$\begin{aligned} V_{pylon y} &= V_{wind y} + V_{current y} \\ &= 107 + 323 \\ &= \underline{430 k} \end{aligned}$$



Total Environmental Load to Pylon in x-x direction

$$\begin{aligned} V_{pylon x} &= V_{wind x} + V_{current x} \\ &= 658 k + 1313 k \\ &= \underline{1971 k} \quad \leftarrow \text{Governs} \end{aligned}$$

Primary stresses at Point B at Finger of Segment 2:



* Due to Environmental loads in y-y direction:

1) Wave Induced Stresses:

$$M_{max} = 595677 \text{ K-F (Wave Hogging)}$$

$$\sigma_{top,y} = 595677 \times 1000 / 2867 \times 144$$

$$= 1443 \text{ PSI (T)} \quad \leftarrow$$

$$\sigma_{bot,y} = 595677 \times 1000 / 2954 \times 144$$

$$= 1386 \text{ PSI (C)} \quad \leftarrow$$

2) Wind Induced Stresses:

Assume that wind and current loads acting on windward ships and pier are transferred through moorings located at the C.G.C. of Finger Pier. That is no torsion at the fingers due to these loads.

$$2(F_{W00}) = 2(1.0 \times 1/2 \times 0.00237 \times 73^2 \times 2200^{\#})$$

$$= 27.8 \text{ K}$$

$$(F_W)_{Avg} = 1.0 \times 1/2 \times 0.00237 \times 73^2 \times 1520$$

$$= 9.6 \text{ K}$$

$$\Sigma F_W = 27.8 \text{ K} + 9.6 \text{ K}$$

$$= 37.4 \text{ K}$$

$$\sigma_{W,y} = F_W / A_{F@B}$$

$$= 37.4 \times 1000 / 273^{\#} \times 144 = 1 \text{ PSI (Small)}$$

* Must check Section B for loads in x direction.

Cont' Primary stresses at Point B

3) Current y-y induced stresses:

$$2(F_{cy})_{DD} = 2[1.9 \times (1/2) \times 1.987 \times 3.22^2 \times 1450] \\ = 57K$$

$$(F_{cy})_{Fing.} = 1.6 \times 1/2 \times 1.987 \times 3.74^2 \times 2000^{\phi} \\ = 44.5K$$

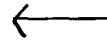
$$\Sigma F_{cy} @ B = 57 + 44.5 \\ = 101.5K$$

$$\text{Stress } \sigma_{cy} = 101.5 \times 1000 / 273 \times 144 \\ = \underline{\underline{3 \text{ PSI}}}$$

Total Primary stresses at Point B at Fingers due to environmental loads:

$$\Sigma \sigma_y = 1386 + 1 + 3 \\ = \underline{\underline{1390 \text{ PSI}}}$$

$$\Sigma \tau_{xy} = 0^{\text{PSI}} \text{ at slab edge}$$



Secondary stresses at Pt. B, Finger at Segment 2:

1) Buoyancy Bending

At Bottom Slab: Span = 20'-0"
t_{support} = 14"

2) Static Pressure:

D.L.:

Concrete: $14/12 \times .15 = 0.175 \text{ KSF}$

Mech. + Misc. = 0.05

Buoyancy: $13 \times .062 = -0.81$

Resultant: = -0.59 KSF

Slab section properties at Point B

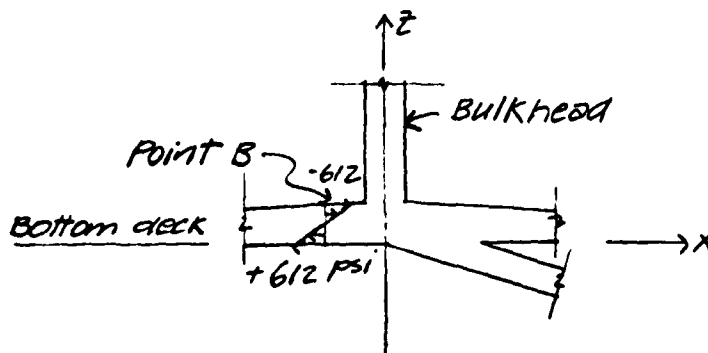
$I = 12 \times 14^3 / 12 = 2744 \text{ in}^4$

$S = 2744 \times 2 / 14 = 392 \text{ in}^3$

6) Moment: $L_n \approx 19'$, $L_1 = 40'$
 $L_1 / L_n > 2 \rightarrow \text{One-way slab}$

$M_{\max} = WL_n^2 / 11$
= $0.59 \times 19^2 / 11$
= 20 K-F

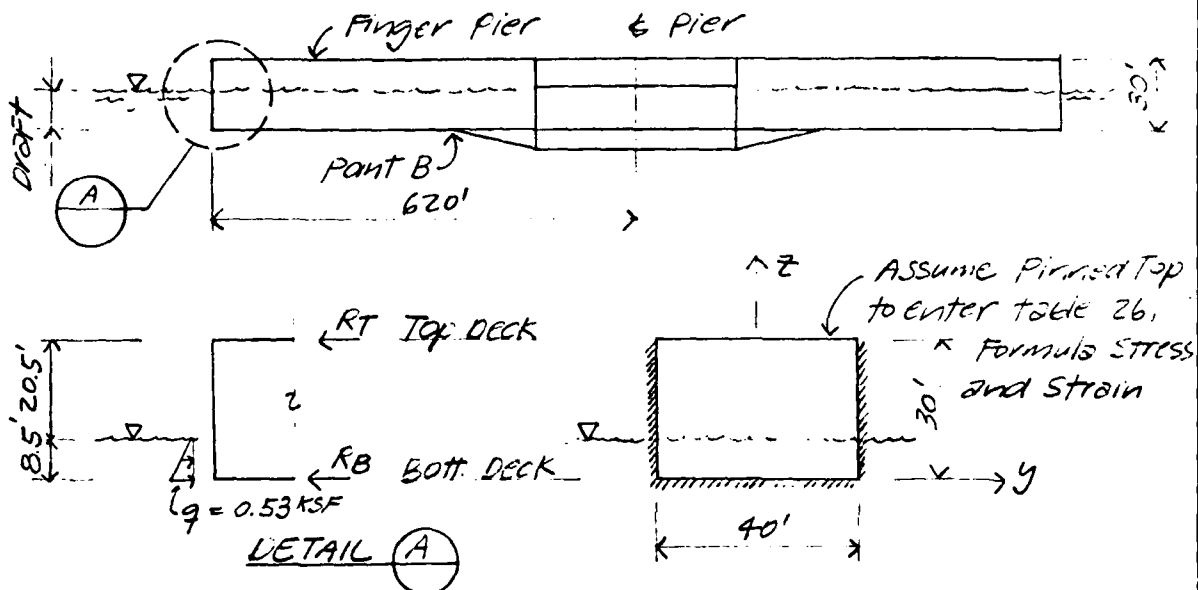
c) Stresses: $\sigma_{B,y} = 20 \times 1000 \times 12 / 392$
= $\pm 612 \text{ PSI}$



Cont' Buoyancy Secondary Stresses @ Pt. B

2) Buoyancy Axial Global Stresses:

a) Dead Load only : Draft $\approx 5'$



Case 9 ddd at $x=0, z=0$

$$R_B = \sigma_1 q_b \quad z/b = 1.33 \quad \bar{\sigma}_1 \approx 0.11 \text{ (extrapolating)}$$

$$R_B = 0.11 (0.53)(30)$$

$$= 1.83 \text{ K/F} \quad \text{or } 1.8 \text{ K}$$

Stresses at Point B :

$$\sigma_{x,y} = 1800 \text{ #}/12 \times 14$$

$$= 11 \text{ psi}$$

Dead Load
draft

b) Dead + Live Load + Tide : $12.5' + 2' = 14.5'$
Say 15'

$$\bar{\sigma}_1 = 0.20$$

$$q = 0.062 \times 15.0' = 0.93 \text{ KSF}$$

$$R_B = 0.20 \times 0.93 \times 30'$$

$$= 5.6 \text{ K/F} \quad \text{or } 5.6 \text{ K}$$

Full draft
+ Tide

stresses at Point B : $\sigma_{x,y} = 5600/12 \times 14 = 33 \text{ psi}$



STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca 94133

PROJECT: NOVA FICRS

ITEM: Floating Marina FICRS Sch. 2

DESIGN: Stress Summary @ Pt. B

DATE: 11/23/81

SHEET:

A-2.54

OF

REVISION:

Stress Summary at Point C due to Environmental loads
in the y direction

Loading	Stresses at Pt. B (psi)					
	Local		Global		Σ	
	σ_x	σ_y	σ_x	σ_y	σ_x	σ_y
Dead + Live + Buoyancy	—	+612	-33	-33	-33	+579 -645
Wave σ_y (Hog.)	—	—	—	-1386	—	-1386
Wind σ_y	—	—	—	—	—	—
Current σ_y	—	—	—	-3	—	-3
				Σ	-33	-2304

Psi

Prestressing in y direction

Partial Prestressing: Assume 80% of prestress to offset tensile stresses at bottom slab due to wave sagging.

$$\sigma_{\text{bottom @ B}} = +1386 \text{ psi}$$

$$\begin{aligned} \text{Prestress} &= -0.80 \times 1386 \text{ psi} \\ &= -1109 \text{ psi} \end{aligned}$$

$$\begin{aligned} \text{Total Prestressing Force } F, \\ F &= 1109 \times 137 \times 2 \times 144 / 1000 \\ &= 43749 \text{ K} \end{aligned}$$

Assume 270 Grade Prestressing Strands:

$$\begin{aligned} \text{Area'd} &= 43749 \text{ K} / 270 \times 0.70 \text{ ksi} \\ &= 231 \text{ in}^2 \\ &= 1.6 \phi \end{aligned}$$

Total % Steel in Cross-Area @ Point C :

$$\begin{aligned} \text{Assume mild steel} &= 1\% \\ \text{Prestressing} &= 1.6 / 2 \times 137 = 0.006 \\ &= 0.6\% \\ \Sigma &= 1.6\% \rightarrow 4.4 \phi \end{aligned}$$

Transformed Area OF Concrete @ Pt. B OF Finger :

$$\begin{aligned} f'_c &= 7000 \text{ psi}, E_c = 3.8 \times 10^6 \text{ psi} \\ A_T &= 2 \times 137 - 4.4 + 4.4 \times 29 / 3.8 \\ &= 303 \phi \quad \text{or} \quad 11\% \text{ increased area} \end{aligned}$$

$$\begin{aligned} \sigma_{\text{prestressing}} &= -0.80 \times 1386 / 1.11 \\ \sigma_y &= -999 \text{ psi} \end{aligned} \quad \leftarrow$$

$$\begin{aligned} \text{Wave Load}_y &= 1386 / 1.11 \\ \text{@ Bottom} &= \pm 1249 \text{ psi} \end{aligned} \quad \leftarrow$$

$$\begin{aligned} \text{Residual Tension} &= \text{Wave Sag}_y - \text{Prestress}_y \\ &= 1249 - 999 = 250 \text{ psi} \quad \text{Use mild steel} \end{aligned}$$

Final Compressive Stresses at Pt. B due to y-y loads:

$$\begin{aligned} \sigma_{x \max B} &= E\sigma_x + \text{Prestress}_x \\ &= -33 + 0 \text{ at Pt. B} \\ &= -33 \text{ psi} \end{aligned}$$

←

$$\begin{aligned} \sigma_{y \max B} &= E\sigma_y + \text{Prestress}_y \text{ at B} \\ &= -645 - 1249 - 3 - 999 \\ &= \underline{\underline{-2896 \text{ psi}}} \end{aligned}$$

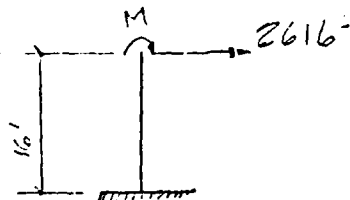
←

$$\begin{aligned} F_c \text{ required} &= \sigma_{\max} / 0.45 \\ &= 2896 / 0.45 \\ &= 6435 \text{ psi} \end{aligned}$$

Use Normalweight Concrete, $F_c = 7000 \text{ psi}$
at Fingers and Spine.

Pylon Design: Assume Pylon pinned @ the top.

$$\begin{aligned} \text{Load transverse loads} &= 1971 \times 1.05 \\ &= 2093 \text{ k} \end{aligned}$$



Ultimate Strength Design:

$$P_u = 1.25(2093) = 2616 \text{ k}$$

$$M_u = (2616)(116) = 41856 \text{ k-ft}$$

preliminary selection $\Rightarrow$ try 13'-0" O.D. / 11'-0" ϕ I.D.
Pylon w/ 2.5% steel $F_c = 6000 \text{ psi}$.

$$A_s = 1 \text{ in}^2 \text{ or } \sim 140 \# 9 \text{ bars}$$

distance to reinforcing steel
assume bars spaced equally

the output from FLOW-1DC gives the distance from the
compression face to each pair of reinforcing bars
The steel stress is also given. (see next page)

For the output shown, the depth of compression
block = 15.5 in.



STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca 94133

PROJECT: *North Pier*

ITEM: *Floating Marine Pylon Structure*

DESIGN: *Pylons*

DATE: *1/25/85 KM*

SHEET:

A-2.35

OF

REVISION:

'Cont' Pylon Design :

DIST. TO
COMP.
FACE

STRESS

DIST. TO
COMP. FACE

STRESS

4.03	+60.000
4.22	+60.000
4.52	+60.000
4.37	+60.000
5.55	+60.000
5.30	+60.000
7.13	+56.747
8.21	+52.431
9.37	+47.523
10.68	+42.034
12.12	+35.973
13.69	+29.354
15.39	+22.190
17.22	+14.495
19.17	+ 6.285
21.24	- 2.423
23.42	-11.613
25.71	-21.265
28.11	-31.361
30.61	-41.880
33.23	-52.800
35.89	-60.000
38.66	-60.000
41.51	-60.000
44.43	-60.000
47.42	-60.000
50.47	-60.000
53.57	-60.000
56.73	-60.000
59.93	-60.000
63.16	-60.000
66.43	-60.000
69.72	-60.000
73.02	-60.000
76.34	-60.000
79.65	-60.000

92.97	-60.000
95.27	-60.000
98.56	-60.000
92.30	-60.000
95.06	-60.000
99.25	-60.000
102.42	-60.000
105.52	-60.000
108.57	-60.000
111.56	-60.000
114.48	-60.000
117.33	-60.000
120.10	-60.000
122.79	-60.000
125.38	-60.000
127.88	-60.000
130.28	-60.000
132.57	-60.000
134.75	-60.000
136.82	-60.000
138.77	-60.000
140.60	-60.000
142.30	-60.000
143.87	-60.000
145.31	-60.000
146.62	-60.000
147.78	-60.000
148.81	-60.000
149.69	-60.000
150.43	-60.000
151.02	-60.000
151.47	-60.000
151.77	-60.000
151.91	-60.000



STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca 94133

PROJECT: NAVA FMS

ITEM: Pylon Mounting Sp. 2

DESIGN: Pylons

DATE: 11.5.78

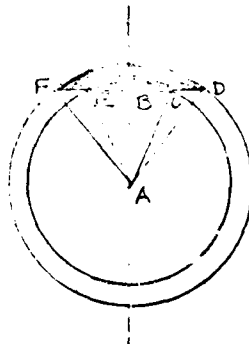
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A-239

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REVISION:

Pylon Design



$$AE = 62.5"$$

$$AC = 66"$$

$$AD = 78"$$

$$\angle EAD = \arccos \frac{62.5}{78} = 36.75^\circ = .641 \text{ rad}$$

$$\angle EAC = \arccos \frac{62.5}{66} = 18.74^\circ = .327 \text{ rad}$$

$$\angle CAD = 36.75 - 18.74 = 18.01^\circ = .314 \text{ rad}$$

$$\text{Area sector ADF} = (.641)(78)^2 = 3900 \text{ in}^2$$

$$\text{Area sector ACE} = (.327)(66)^2 = 1424.4 \text{ in}^2$$

$$\text{height of } \triangle ACD = (66) \sin 18.01 = 20.4 \text{ in}$$

$$\text{Area } \triangle ACD = (\frac{1}{2})(20.4)(78) = 795.6$$

$$\text{Area comp. block} = 3900 - 1424.4 - (795.6)(2) = 884.4$$

c.g. comp. block

$$= \frac{[(2)(78) \sin 26.75 (3900) - (2)(66) \sin 18.74 (1424.4) - (795.6)(2)(2/3)(62.5)]}{(3)(.641)} = 884.4$$

$$= 73.45"$$

$$C = (884.4)(.85)(6) = 4510$$

Resisting moment equals the sum of the moments about the compression face.

$$\phi M_N = (.9) [51503 - (4510)(4.55)/12] = 44811 \text{ K-FT.}$$

$$44811 > 41856 \checkmark$$

Preliminary calcs. indicate the wall thickness must be increased in

$$\phi v_c = (.85)(2) \sqrt{6000} = 131 \text{ psi}$$

$$\phi v_s = \frac{(2616)(2)}{(9952)(.85)} = .131 = .487$$

$$A_v / ft = \frac{(.487)(24)(12)}{60} = 2.34 \text{ in}^2 / \text{ft.}$$

$$\frac{2.34}{2 \text{ Sides}} = 1.17 \text{ in}^2 / \text{ft}$$

or # 6 @ 6"

increase the wall thickness in the effective shear area only

Pier Design

Piers must resist overturning moment + shear

assume a max. 1-in-6 batter

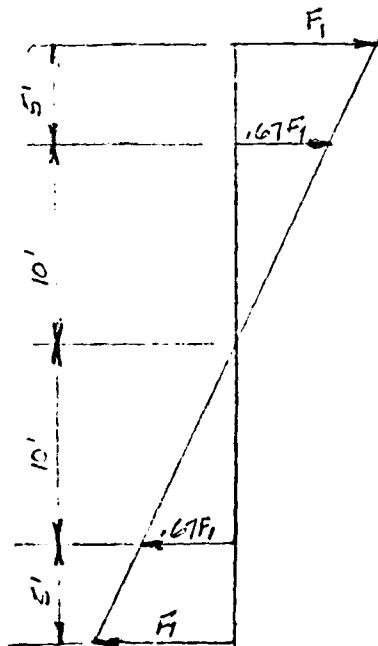
Shear

$$V_u = 2616 \text{ k}$$

Overturning

$$M_u = 20928 \text{ k} \cdot \text{ft.}$$

Since the force in the longitudinal direction is approx. 4 times that in the transverse direction, use a rectangular layout.



$$30 F_1 + (20)(.67 F_1) = 41852$$

$$F_1 = 964 \text{ k}$$



STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca 94133

PROJECT: NEW PIER

ITEM: Floating Marine Pier, S.W.

DESIGN: Pylon Foundation

DATE: 11/85 KM

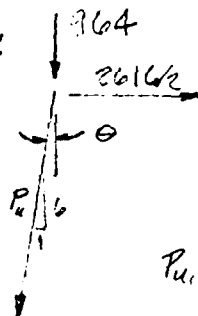
SHEET:

A-342

OF

REVISION:

File Design:



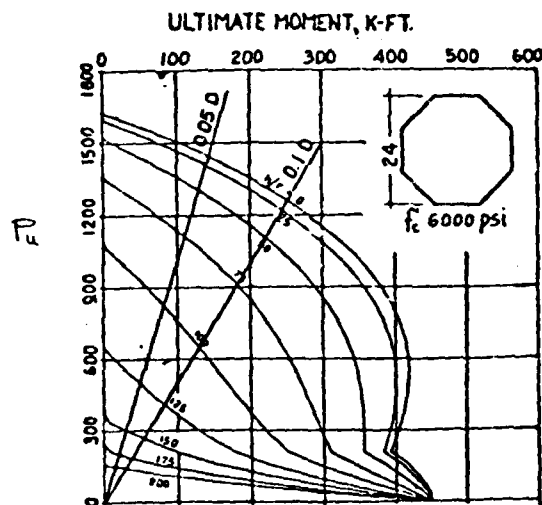
$$\theta = \arctan(1/6)$$

$$\theta = 9.46^\circ$$

$$P_{u1} = \frac{2616}{(2)(\sin 9.46)} + \frac{964}{\cos 9.46} = 8936^k$$

$$P_{u2} = \frac{2616}{(2)(\sin 9.46)} + \frac{(1.67)(964)}{\cos 9.46} = 8613^k$$

$$P_{u\text{allowable}} \approx 1400^k$$



$$n_1 = \frac{8936}{1400} \approx 7 \text{ piles}$$

$$n_2 = \frac{8613}{1400} = 7 \text{ piles}$$

assume allowable tension = 25% allowable compression

$$n + .25n = 14$$

$n = 12$ piles on each side of the pylon

For the transverse direction use 4 piles on each side since the load is approx. $1/3$.

Pylon/foundation interface

6000 psi concrete; minimum 11.5' diameter pylon @ the lower end; 1'-0" contact surface.

$$\text{allowable compressive stress} = (1.7)(.85)(6000) = 3570$$

$$\text{allowable force} = (3570)(12)(138) \approx 5912K$$

$$\text{moment arm} = \frac{41856}{5912} \approx 7.0'$$

w/ 1'-0" cover on the bottom and 1'-0" to contact surface T&B.

total depth of foundation =

$$\begin{array}{r} + 7 \\ + 1' \\ + 1 \\ + 1 \\ \hline 10'-0" \end{array}$$

wt. of pylon

assume 40'-0" of hollow pylon 13' O.D. / 11' I.D.
10'-0" solid 12.5"

$$\text{Volume} = \pi(6.5^2 - 5.5^2)(40) + (\pi)(6.25)^2(10) = 2735.2 \text{ ft}^3$$

$$\text{wt} = (2735.2)(.15) = 411K$$

or 205.5 TONS.

with increased wall thickness due to shear

$$\text{wt.} \approx 300 \text{ TONS.}$$



STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca 94133

PROJECT: NOVA PIERS
ITEM: Floating Marine Pier, Sec. 2
DESIGN: WEIGHT
DATE: 10 23 RM

SHEET:
A-2-44
OF
REVISION:

Weight of Structure : Scheme 2

$f_c = 7000$ psi Normal-weight concrete.

$\rho_c = 150$ PCF

Spine :	Area (F ²)	Length (F)	Volume (F ³)	WT (K)
Joint @ B.Slab	7' x 3.5'	1000	2.45×10^4	3675
Cross-sect.	836	1000	8.36×10^5	125400
Transv. Wall	1' x 40'	31' x 154'	1.91×10^5	28644
Transv. Beams	1.5' x 8'	24' x 154'	4.44×10^4	6653
Vert. Beams	1.5' x 8'	24' x 33'	9.5×10^3	1426
				<u>165798 K</u>
<u>Fingers :</u>				
Trench	30	6' x 540'	9.7×10^4	14580
Cross-sect.	274	6' x 360'	5.91×10^5	88650
X-X's	310	6' x 120'	2.23×10^5	33480
	380	6' x 60'	1.37×10^5	20520
Δ	2500	6' x 2.17'	3.26×10^4	4890
Transv. Walls	1' x 28'	6' x 8' x 76.7'	1.03×10^5	15450
	1' x 35'	6' x 2' x 76.7'	3.22×10^4	4830
	1.25' x 42'	6' x 2' x 100'	6.3×10^4	9450
Transv. Beams	1.3' x 3.75'	6' x 18' x 77'	4.05×10^4	6081
Vert. Beams	1.25' x 4.75'	6' x 18' x 33'	2.12×10^4	3174
Total Concrete wt.			Σ	366903 K $\times 1.05$ ↓ misc.
Topping (2" asphalt)			$2/12 \times 418200 \times 0.085$	= 5924 K
Mechanical C. Top Deck			0.100×562200	= 56220 K
E Spine Int.				
Total Dead Load				= 447392 K ←
<u>Live load :</u>				
@ 400 psf @ Main Deck			0.400×418200	= 167280
@ 150 psf @ Inter. Deck			0.150×144200	= 21630
@ 200 psf @ Bot. Deck			0.200×418200	= 83640
Total Live Load				= 272550 K ←

DRAFT :

Dead load draft :

a) Spine pier : $\frac{\text{Wt. in air}}{\text{draft} \times \text{Surface Area}} \times \text{Fresh}$

Surface area includes Spine + Area Finger attached to Spine
S.A. = $144000 + 31500 \times 2$
= 207000 #

Wt. air = Spine wt + $\frac{1}{2}$ Finger wt. + Topp. + Mech.
= $[165798 + 0.33 \times 201000] 1.05 + 2933 + 35100$
= 254107

DRAFT (normal wt.) = $254107 / 0.062 \times 207000 = 20'$ ← Normal

DRAFT (Light wt.) = $(254107 \times .12/.15) / 0.062 \times 207000 = 16'$ ← Light wt.

b) Finger Draft : 360' x 90'

DRAFT (normal wt.) = $\frac{[2/3 \times 201000 + 2991 + 21120]}{0.062 \times 211200} = 12'$ ← Normal

DRAFT (Light wt.) = $12' \times .12/.15 = 9.6'$ ← Light wt.

Must ballast spine pier to match Fingers when joining the segments in water

	Spine (Ft) Freeboard	Finger (Ft) Freeboard	Δ (Ft) Freeboard	Ballast Spine (psf)
Normal wt. Concrete	(44 - 20) = 24	(30 - 12) = 18	6	372
Light wt. Concrete	28	20.4	7.6	471
Normal wt. Sp. + Light wt. Fg.	24	20.4	3.6	223

c) Total Lighship draft :

Normal wt. = $447392 / 0.062 \times 418200 = 17.3'$ ← Normal wt.

Light wt. = $17.3 \times .12/.15 = 14'$ ← Light wt.

Cont' Draft:

$$\text{Live Load Draft} = \frac{272550 \text{ K}}{0.062 \times 419200 \text{ ft}} = 10.5' \quad \leftarrow \text{Live L. Draft}$$

Total drafts =

$$DL_{\text{normal}} + LL = 17.3 + 10.5 = 27.8' \quad \leftarrow \text{Total d. normal}$$

$$DL_{\text{light}} + LL = 14 + 10.5 = 24.5' \quad \leftarrow \text{Total d. light wt}$$

Vertical Center of Gravity H under Full Live Load:

(Normal wt. concrete only)							
	D.L. (K)	$\bar{y}_{\text{top}}$ (F)	(DL)($\bar{y}$) (K-F)		LL (K)	$\bar{y}_{\text{top}}$ (F)	(LL)($\bar{y}$) (K-F)
Spine				Spine			
Joint	3675	42	154350	Int. Slab	21630	16	346080
Cross-Sect.	162123	21.8	3534281	Bot. Slab	2884	40	115360
+ Wall + Bm.							
Mech.	14420	16	230720				

Fingers

Trench	14580	3.5	51030	Bot. Slab	54840	32	1760973
Cross-Sect.	88650	15.3	1356345				
	33480	18.8	629424				
	25410	21.8	553938				
Walls + Beams	38985	17	662745				
			7.17 x 10 ⁶				2.2 x 10 ⁶

$$\Sigma DL(\bar{y}) + LL(\bar{y}) = 7.17 \times 10^6 + 2.2 \times 10^6 = 9.39 \times 10^6 \text{ K-F}$$

$$H = (\Sigma DL(\bar{y}) + LL(\bar{y})) / \Sigma(DL + LL)$$

$$= 9.39 \times 10^6 / (366903 + 5924 + 56220 + 272550)$$

$$= 13.4' \text{ From the top}$$

Center of Buoyancy

$$\bar{KB} = (44' - 13.4') / 2 = 15.3' \text{ above bottom slab}$$



STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133

PROJECT: Navy Piers

ITEM: Floating Marine Pier, Sch. 2

DESIGN: Period X-X

DATE: 10/83 KM

SHEET:

A-2.47

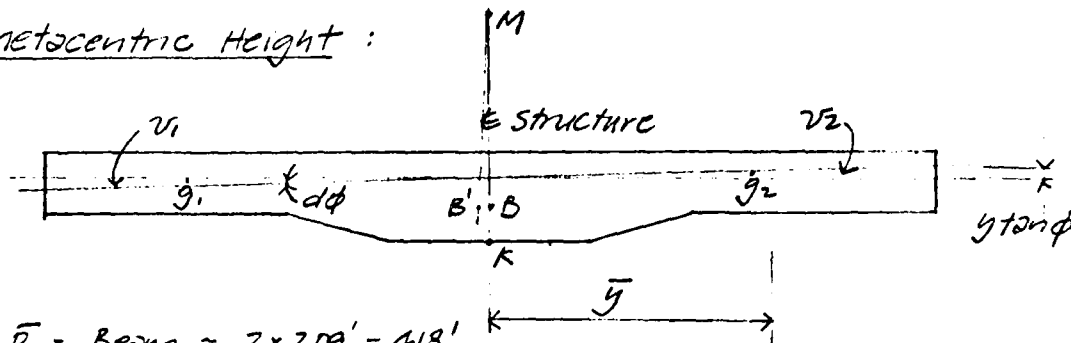
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Block Coefficient :

$$C_b = 35 \Delta / L \times B \times d = 35 (701597) / 418200 \times 27.8 \times 2.24 = 0.94$$

Metacentric Height :



$$\bar{B} = \text{Beam} = 2 \times 209' = 418'$$

$$\bar{y} \approx (418200 / 1000') \times 1/2 = 209'$$

$$\bar{BM} = \bar{BB}_1 / \tan \phi = v \times \bar{y}_1 \bar{y}_2 / \Delta \tan \phi$$

Δ : Volume of displacement
= 35 Δ

For small ϕ :

$$v = 1/2 \bar{y}^2 \tan \phi \times L$$

$$\Delta = L \times B \times d \times C_b = 418200 \times 27.8' \times 0.94 = 1.09 \times 10^7 \text{ F}^3$$

$$\bar{y}_1 \bar{y}_2 = 4/3 \bar{y} = 4/3 \times 209' = 278'$$

$$\bar{KB} = 15.3'$$

$$\bar{KM} = \bar{KB} + \bar{BM}$$

$$\bar{GM} = \bar{KM} - \bar{KG}$$

$$\bar{KG} = H = 30.6'$$

$$\phi = 10^\circ \rightarrow v = 1/2 (209')^2 \tan 10^\circ \times 1000' = 3.85 \times 10^6$$

$$\bar{BM} = 3.85 \times 10^6 \times 278 / 1.09 \times 10^7 \times \tan 10^\circ = 557'$$

$$\bar{KM} = 15.3 + 557 = 572'$$

$$\bar{GM} = 572' - 30.6 = 541.4'$$

$$T_d = C \times \bar{B} / \sqrt{\bar{GM}} = 0.52 \times 418' / \sqrt{541} = 9.3 \text{ sec} \leftarrow$$

Assume cost in-place concrete at \$900/C.Y. (REF. 1)
This cost includes: concrete, Normal weight
Steel (Rebar + Tendons)
piles
Ramp
Fenders
Construction - Flood Basin Method
Tow and Connection
MISC. @ 10%

$$\text{Total C.Y. concrete} = (366903 \times 1.05) / (0.15 \text{ Ft}) \times 27 \\ = 95123 \text{ C.Y.}$$

$$\text{Total Concrete Cost (not including utilities)} \\ = 95123 \times 900 \\ = \$85.6 \text{ M}$$

$$\text{Utility Cost by proportion From REF. 1 :} \\ \begin{array}{r} \$10 \text{ M} - 168000 \Phi \\ \times \quad - 418000 \Phi \\ \hline \times = \$25 \text{ M} \end{array}$$

$$\begin{array}{l} \text{Total Cost of Pier} = 85.6 + 25 \\ \text{(N.W.C.)} = 110.6 \end{array} \quad \text{Say } \$110 \text{ M} \leftarrow \text{Normal wt. conc. Pier}$$

$$\text{or} \quad \frac{110 \times 10^6}{(418 \times 2 + 144) \times 10^3} \\ = \$113 / \text{S.F.} \quad \leftarrow$$

IF use Lightweight Concrete

$$\begin{array}{l} \text{Total Cost Pier} = \$85 \text{ M} \times \frac{1050}{900} \text{ \$/C.Y.} + \$25 \text{ M} \\ \text{(L.W.C.)} \\ = \$129 \text{ M} \end{array} \quad \leftarrow \text{Light wt. C.}$$

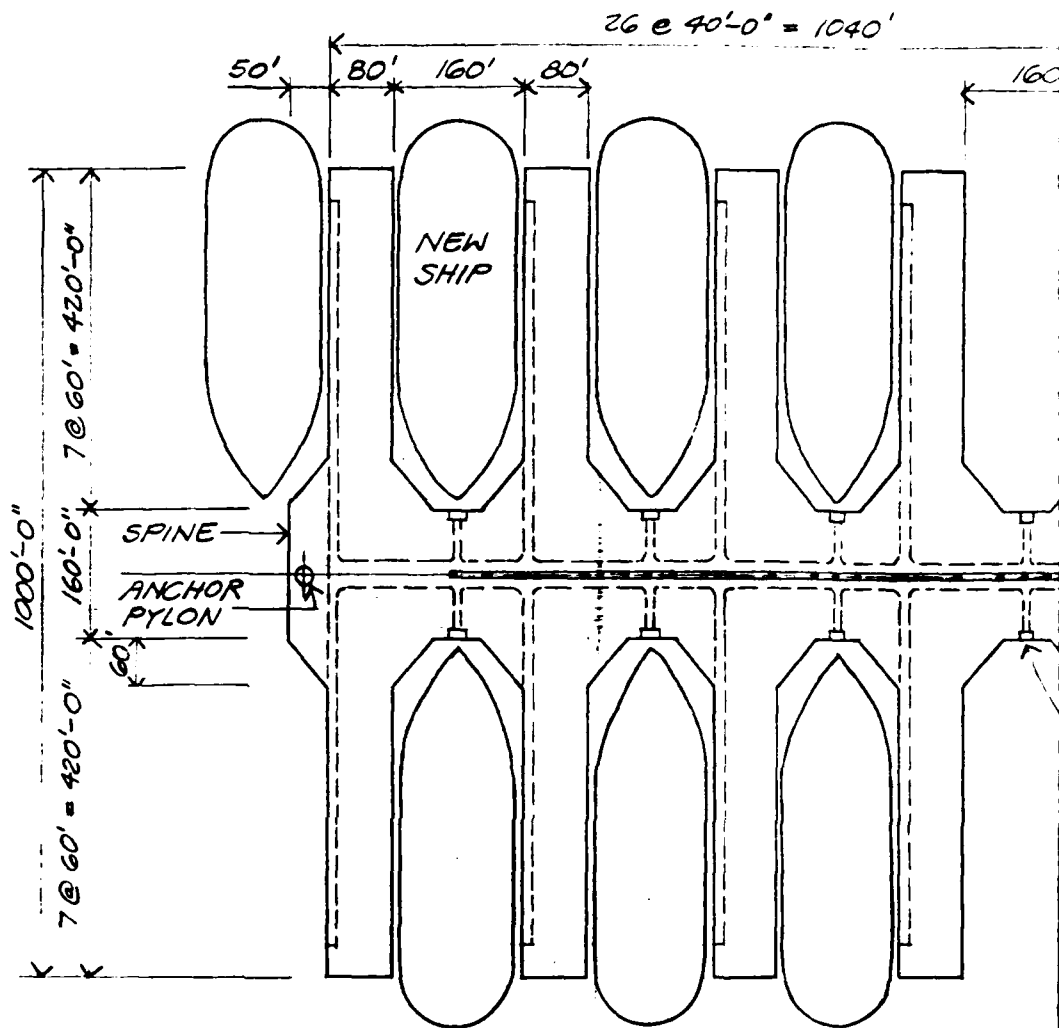
$$\text{or} \quad \$127 / \text{S.F.} \quad \leftarrow$$

(REF. 1) By proportion From Conceptual design of Navy Floating Pier, June 1982 by T.Y. Lin International.

DRAFTING

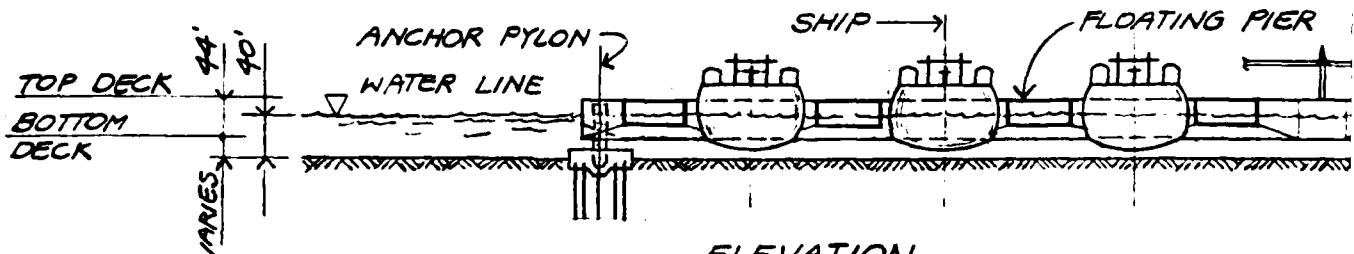
DESIGN

PROJECT NO



PLAN

1" = 200'-0"



ELEVATION

1" = 200'-0"

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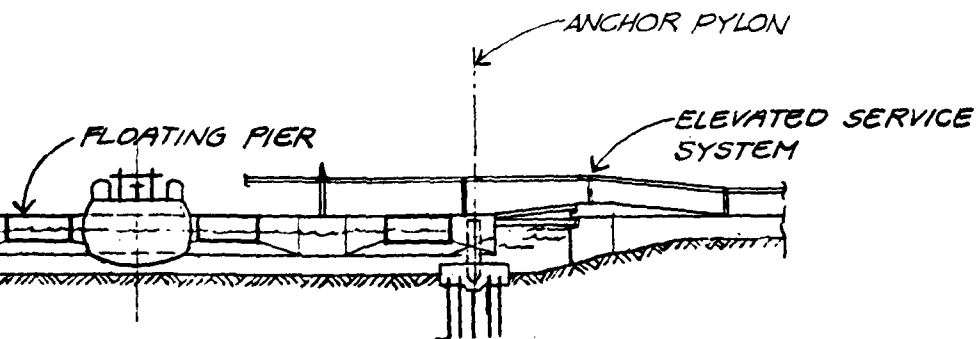
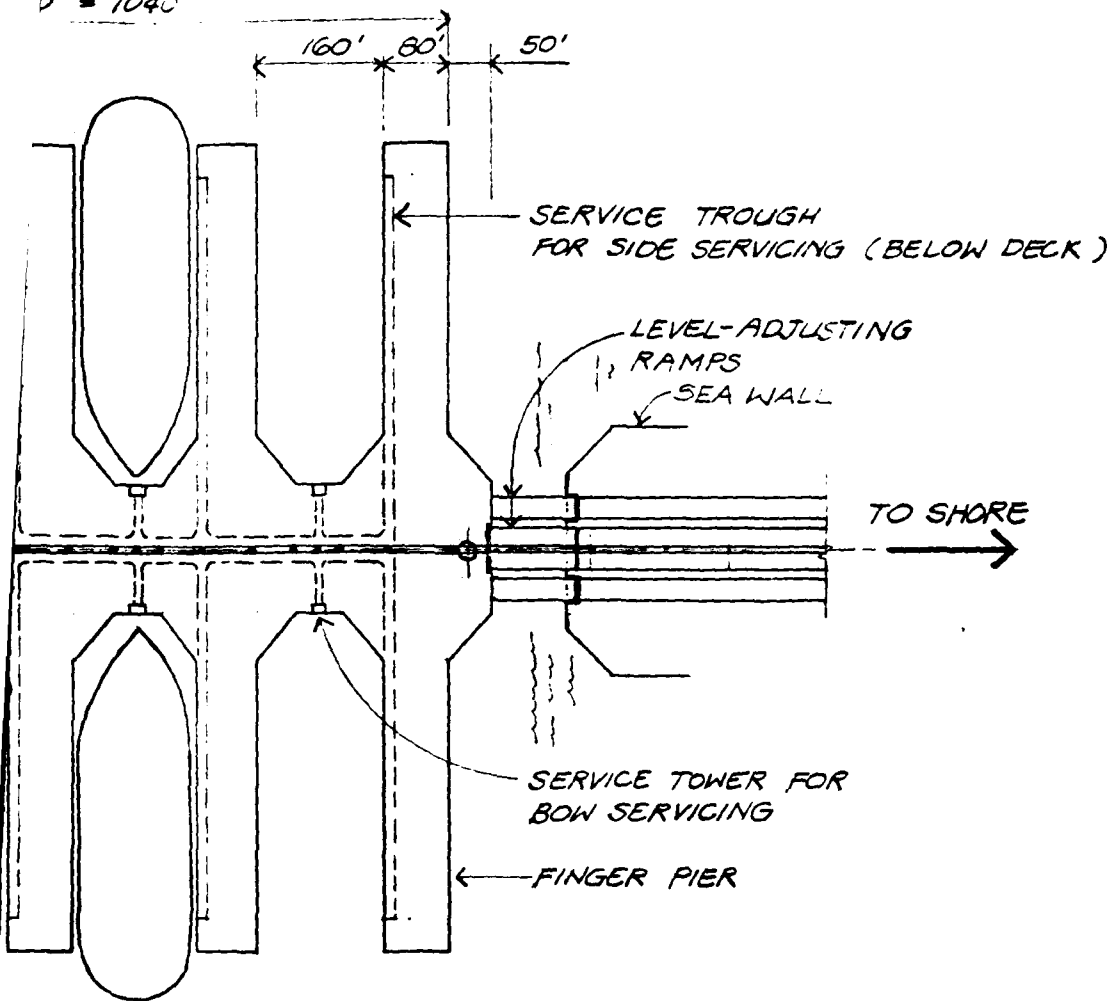
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Feb

D° = 1040'



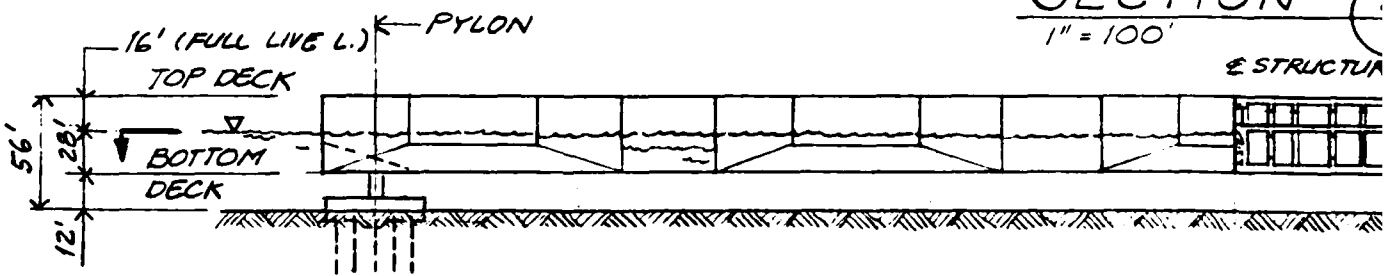
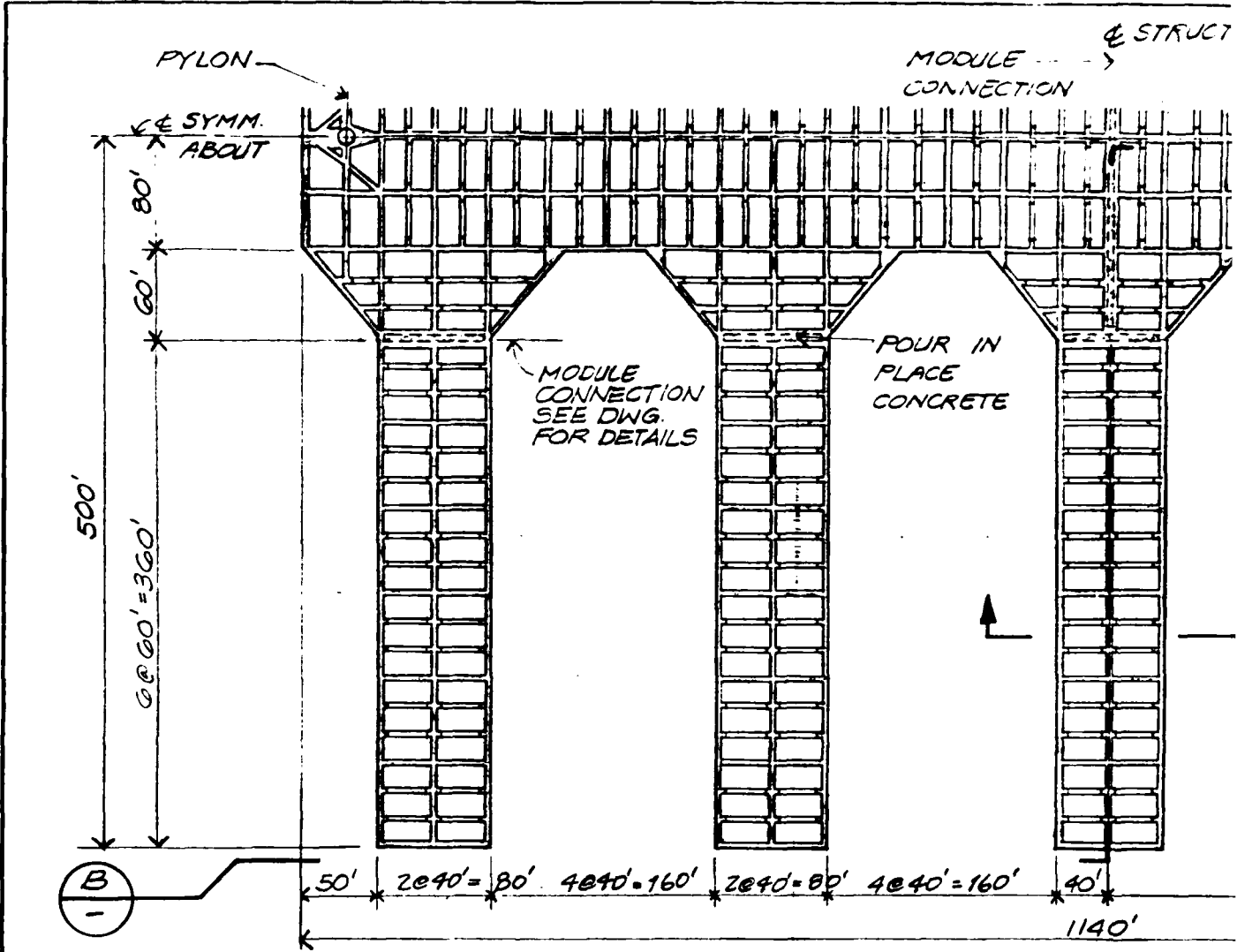
NO	REVISION	DATE

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	<i>FEB 68</i>	<i>RM</i>	PROJECT: <i>FLOATING MARINA PIER</i>	1

DRAFTING

DESIGN

PROJECT NO



SECTION (

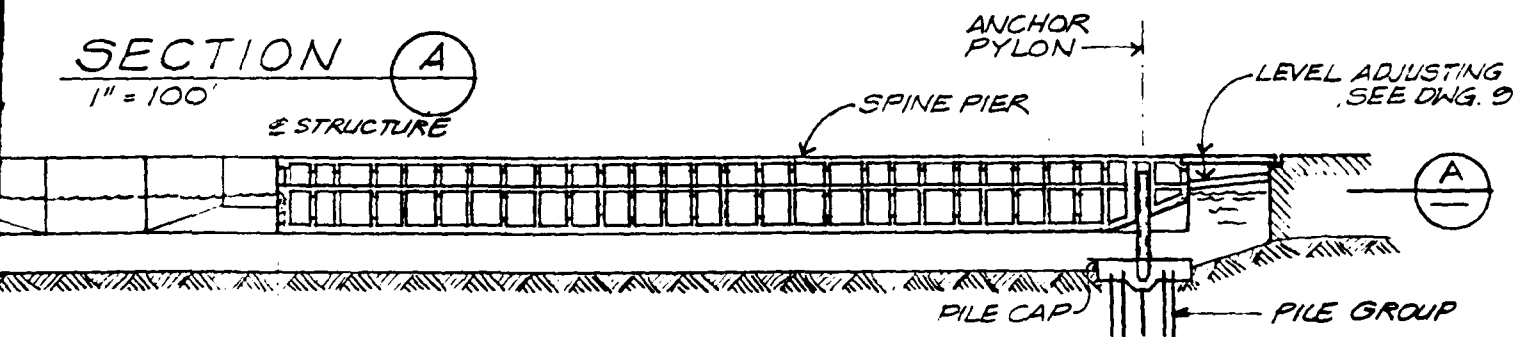
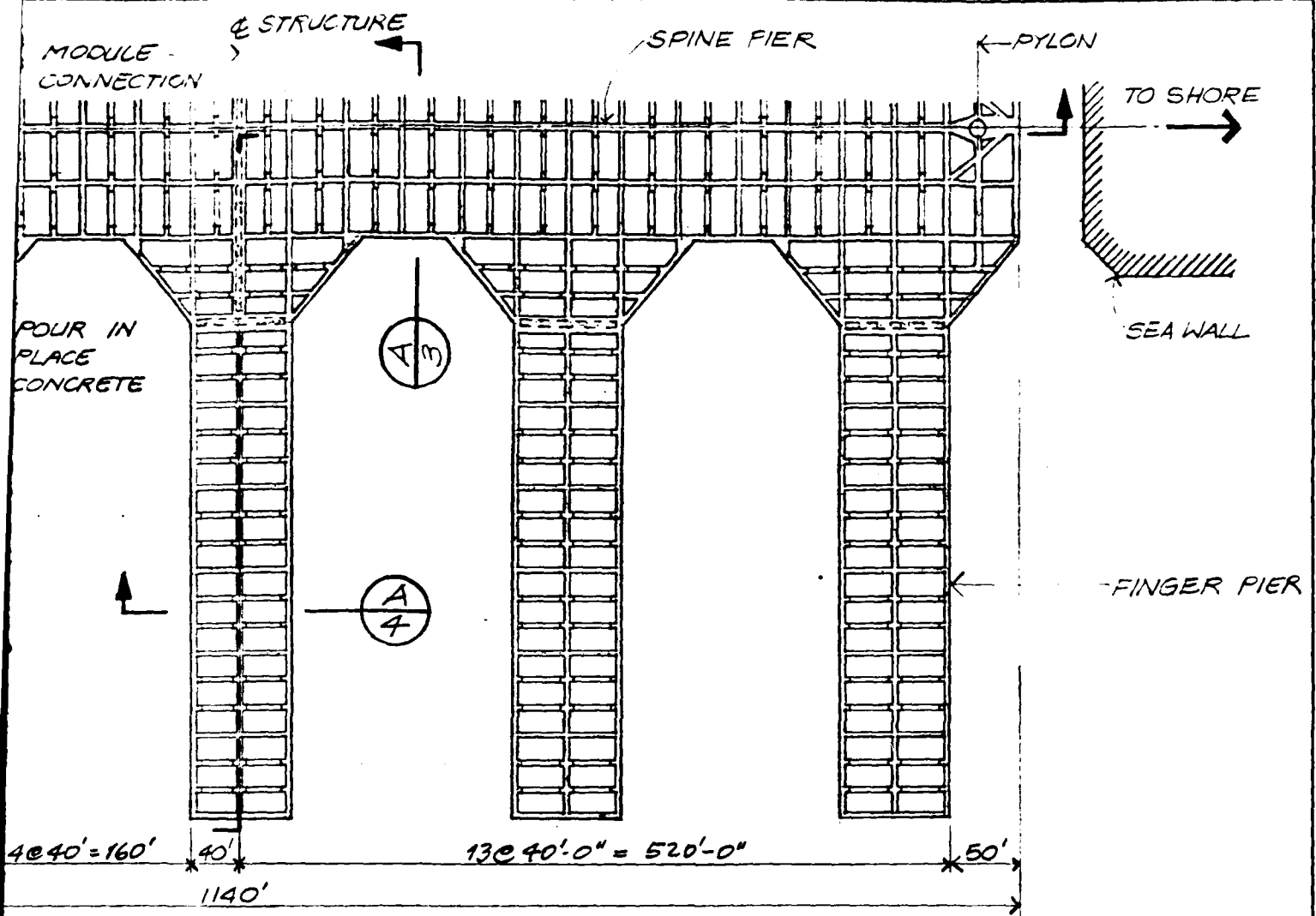
1" = 100'

SECTION (

1" = 100'

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 315 Bay St., San Francisco, Ca. 94133. Tel. (415) 982-1080

Issued For	Date



SECTION B

1" = 100'

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SHEET TITLE: LONGITUDINAL SECTIONS - SCHEME 1

PROJECT: FLOATING MARINA PIER

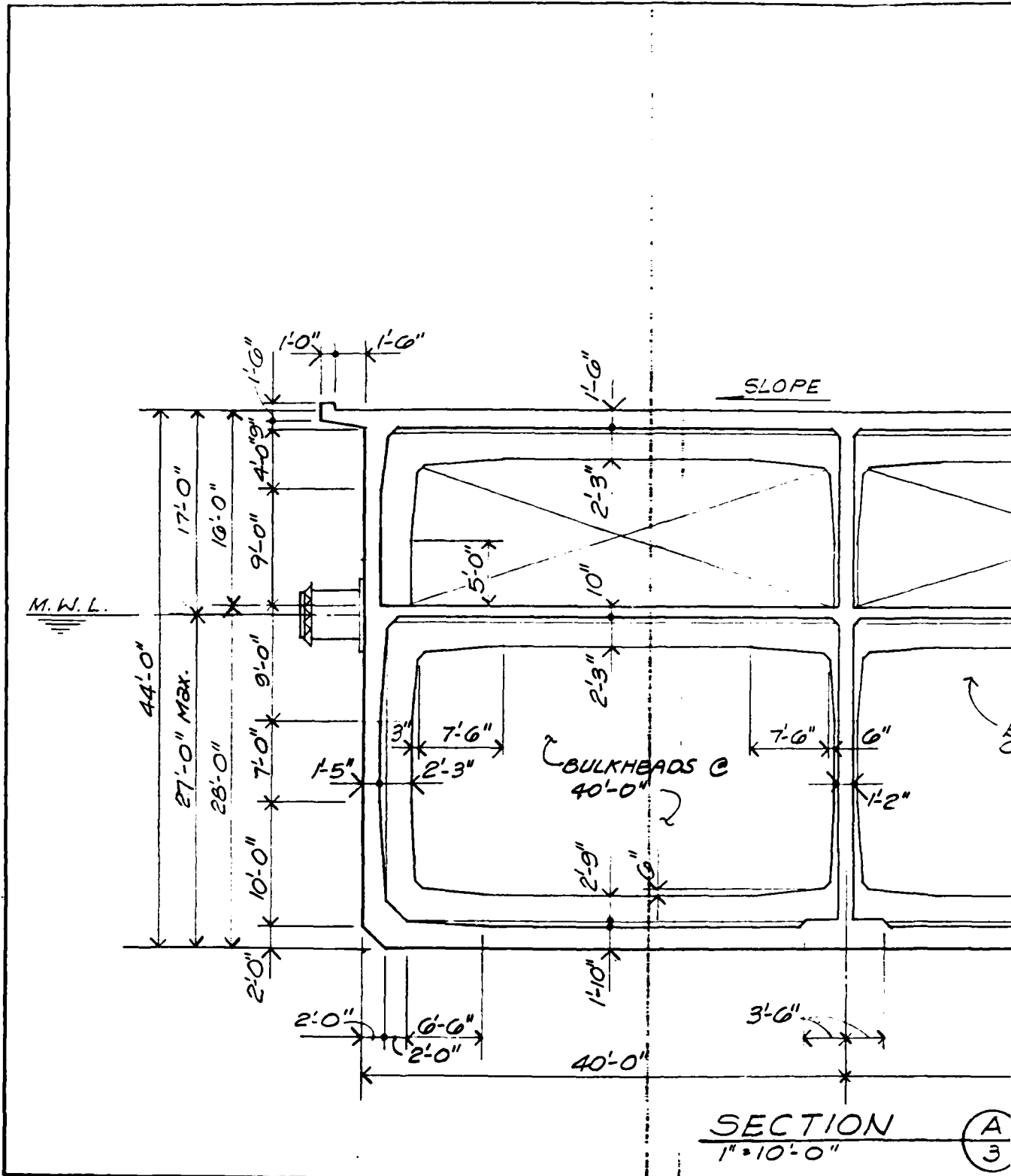
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DESIGN

PROJECT NO.

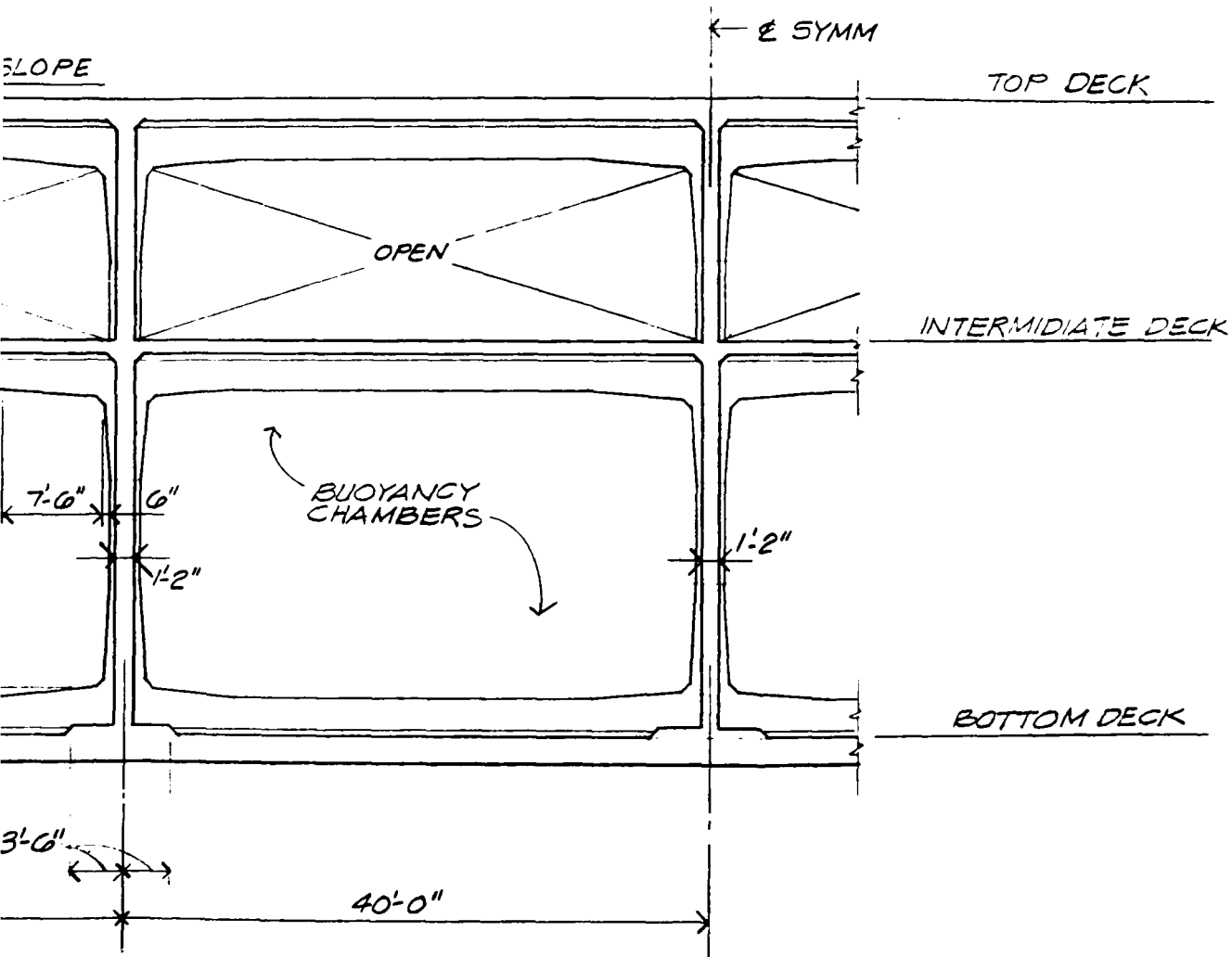


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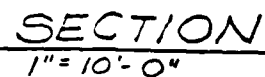


SECTION
10'-0"

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3

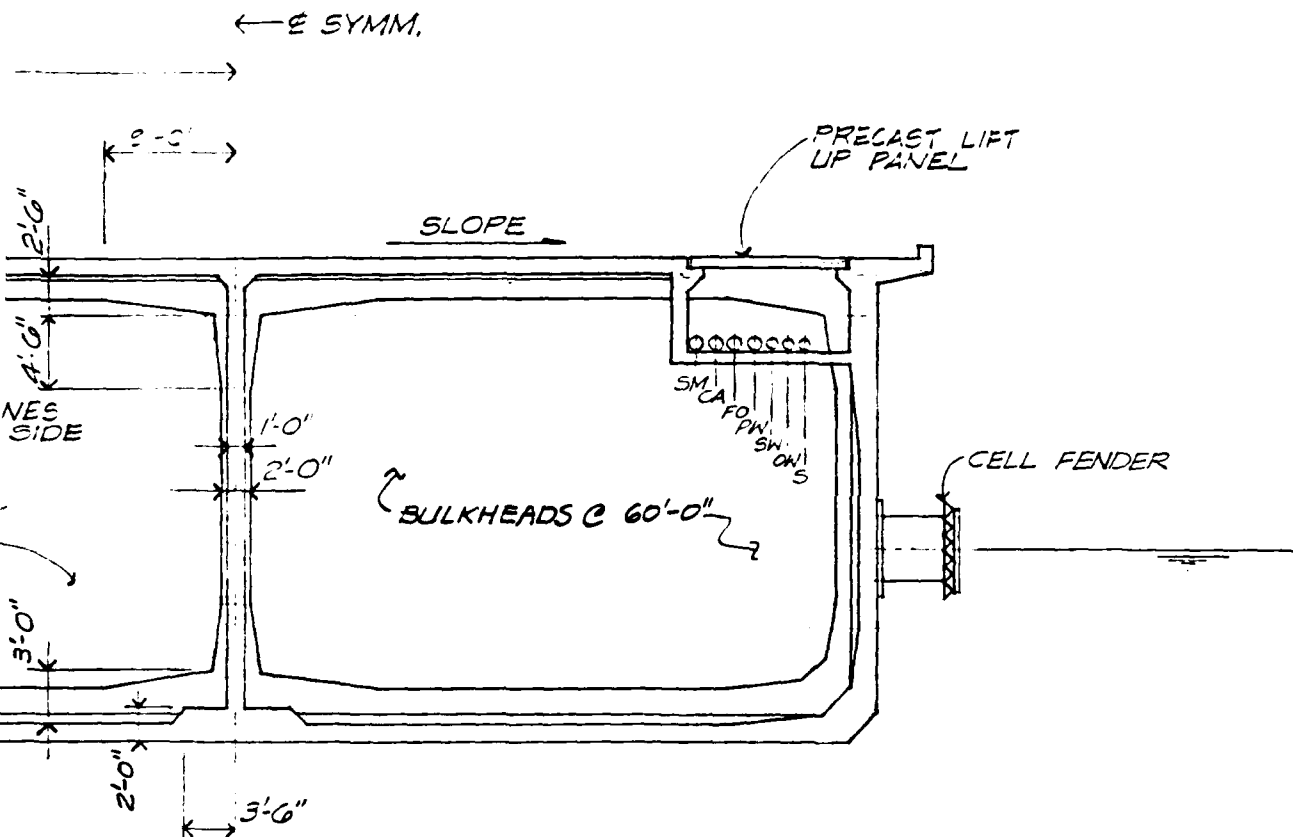
Issued For			Date	By	SHEET TITLE:		SHEET NO.	
					SPINE CROSS SECTION - SCHEME 1		23	
					PROJECT:			
					FLOATING MARINA PIER			

PROJECT N°



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315 Bay St., San Francisco, Ca. 94133. Tel. (415) 982-1000

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SECTION
1" = 10'-0"

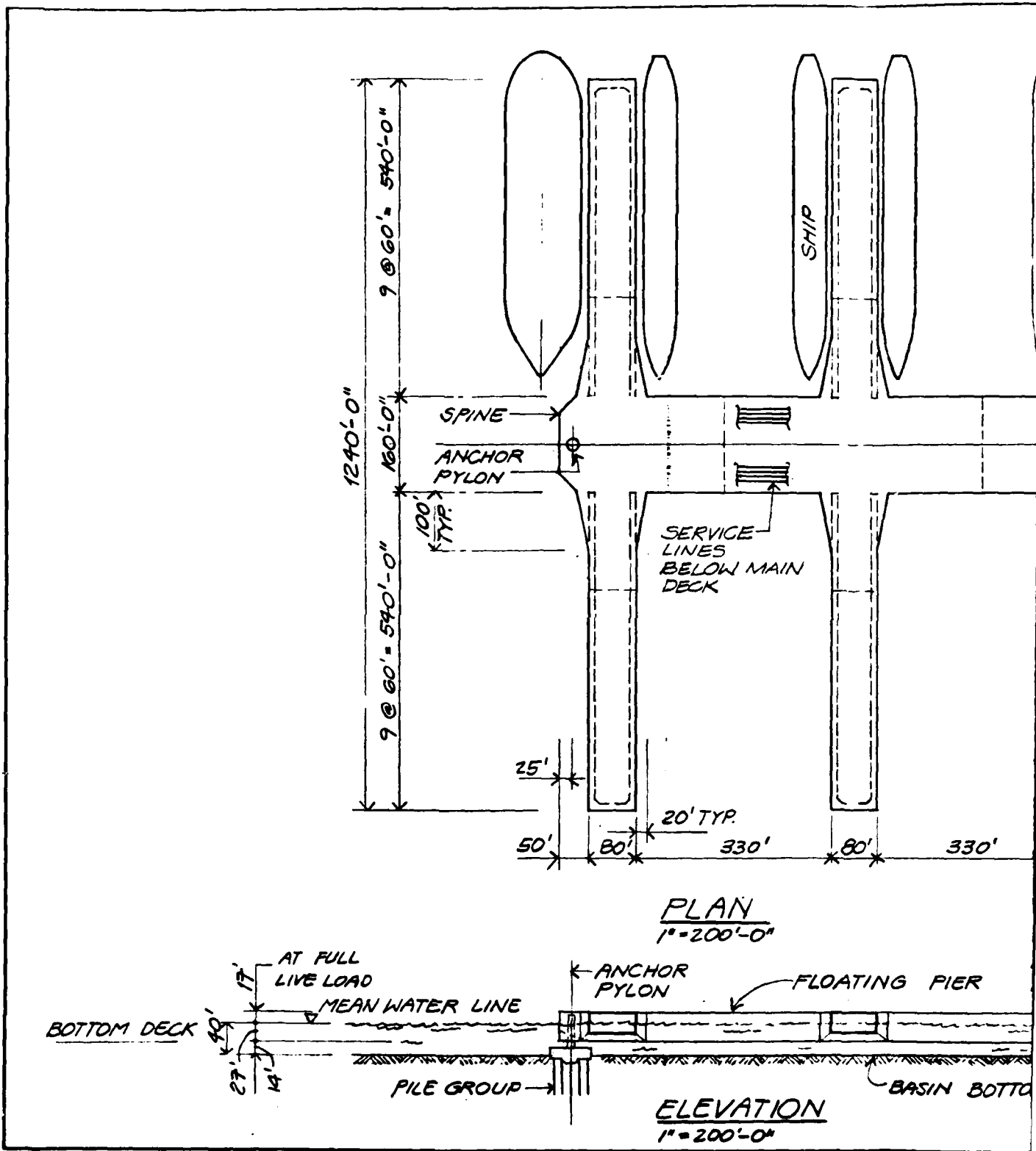
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Date			PROJECT: FLOATING MARINA PIER			42	
By							

DRAFTING

DESIGN

PROJECT NO



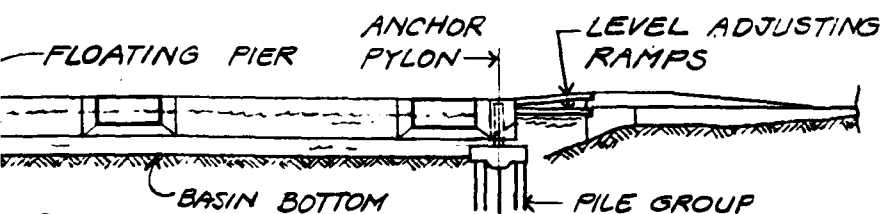
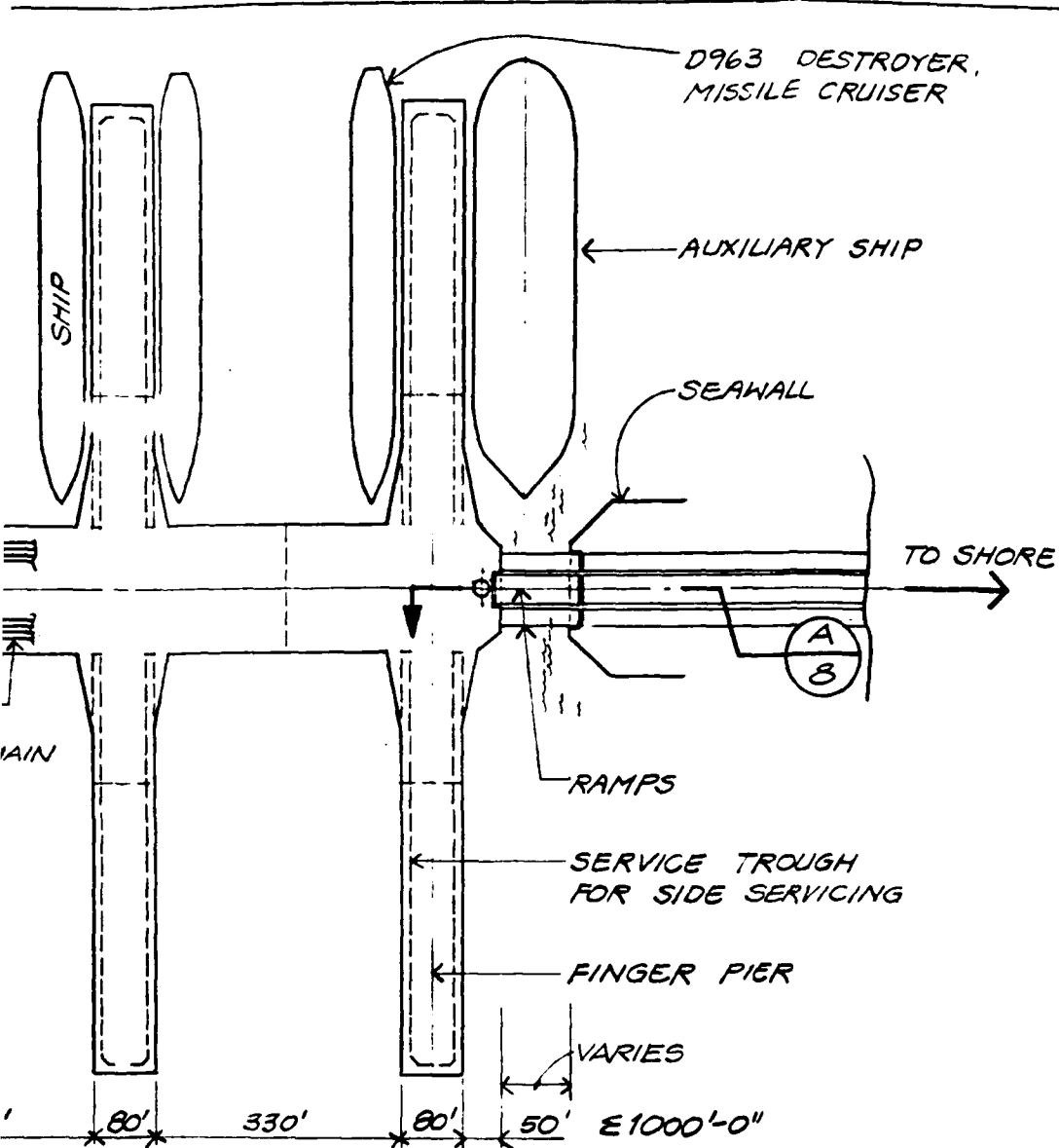
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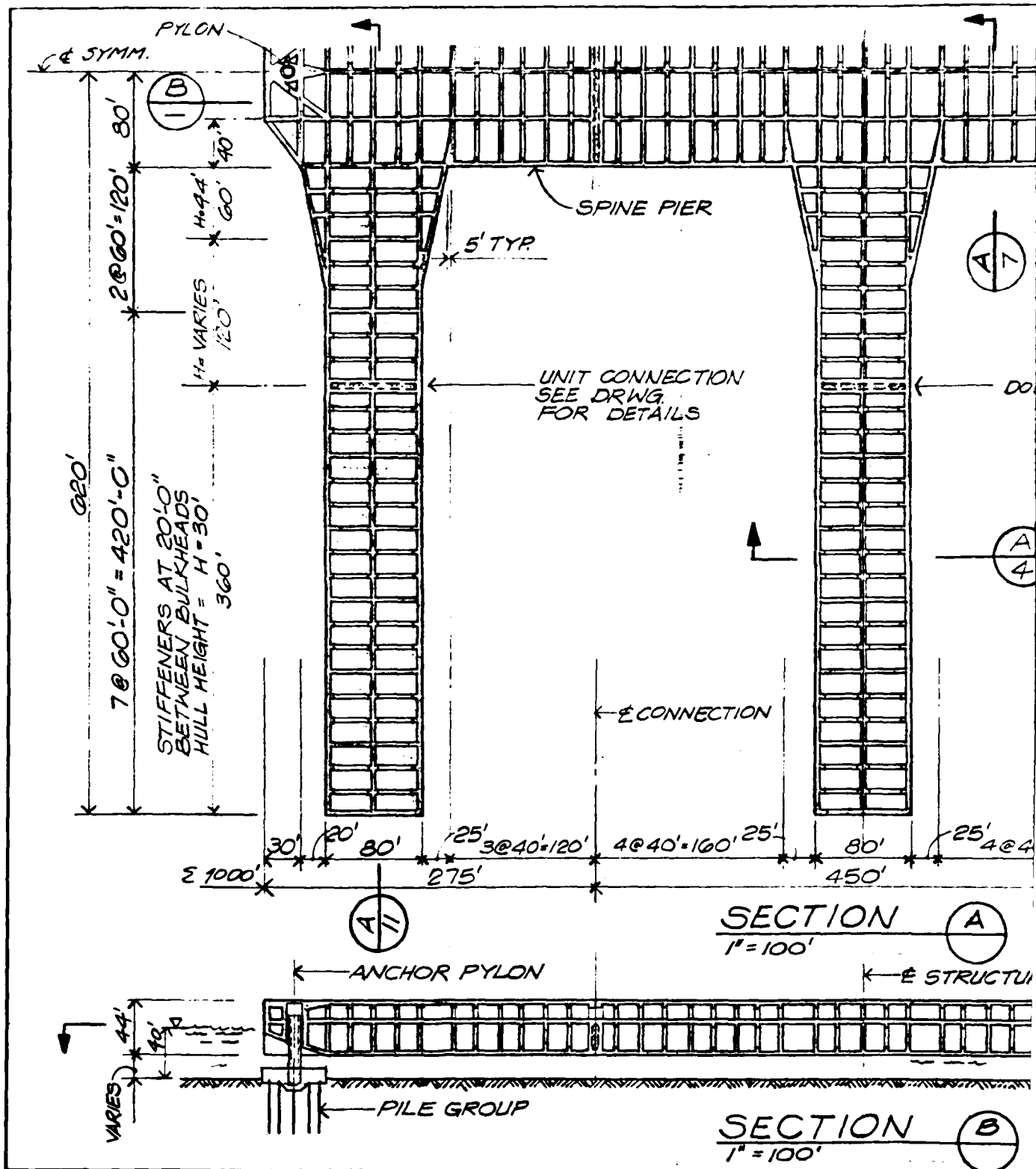
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			Nº		REVISION		DATE	
Issued For		Date	By	SHEET TITLE: PLAN AND ELEVATION-SCHEME 2				SHEET Nº
		FEB. 83	RM	PROJECT: FLOATING MARINA PIER				25

PROJECT NO. _____

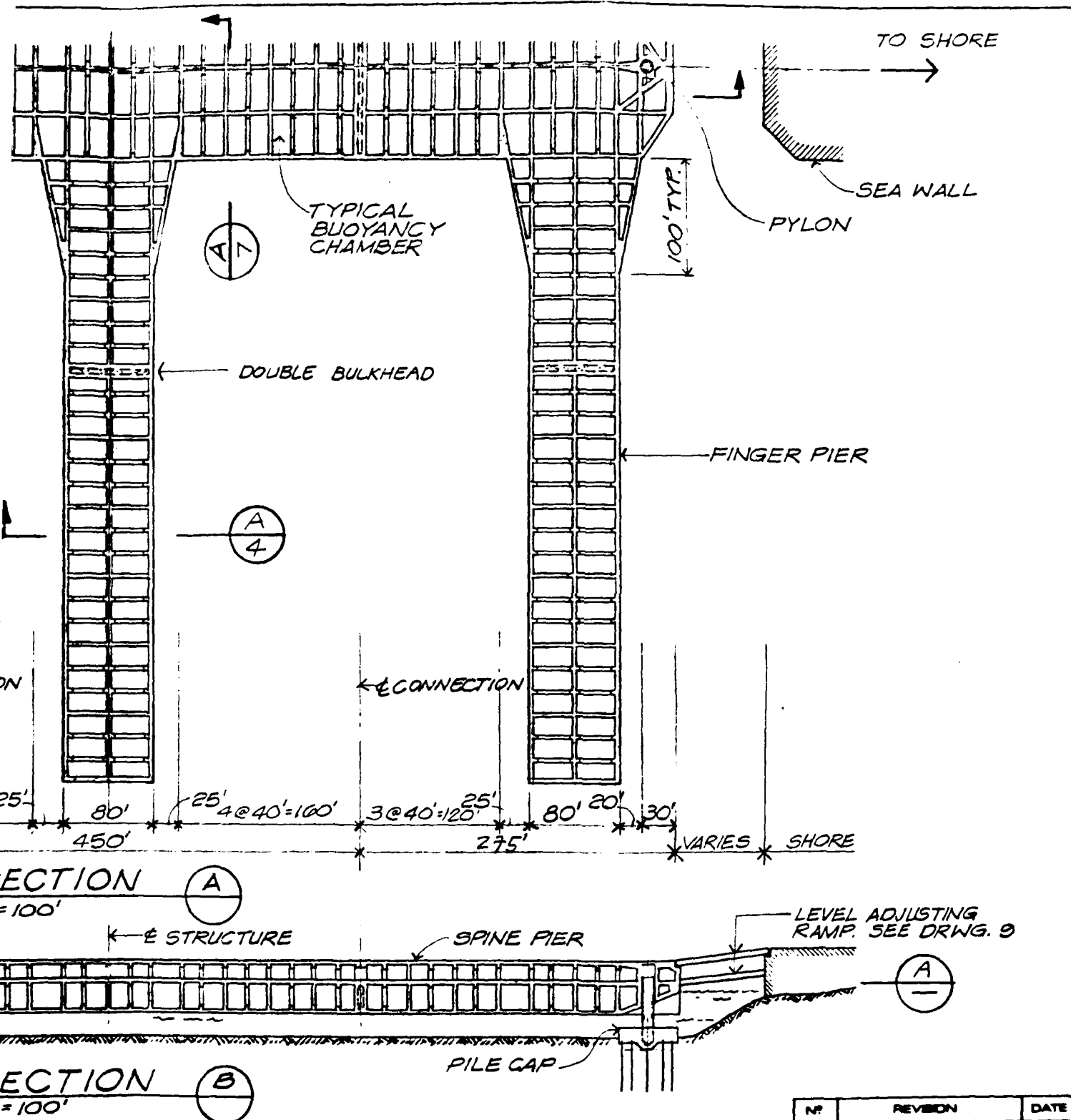
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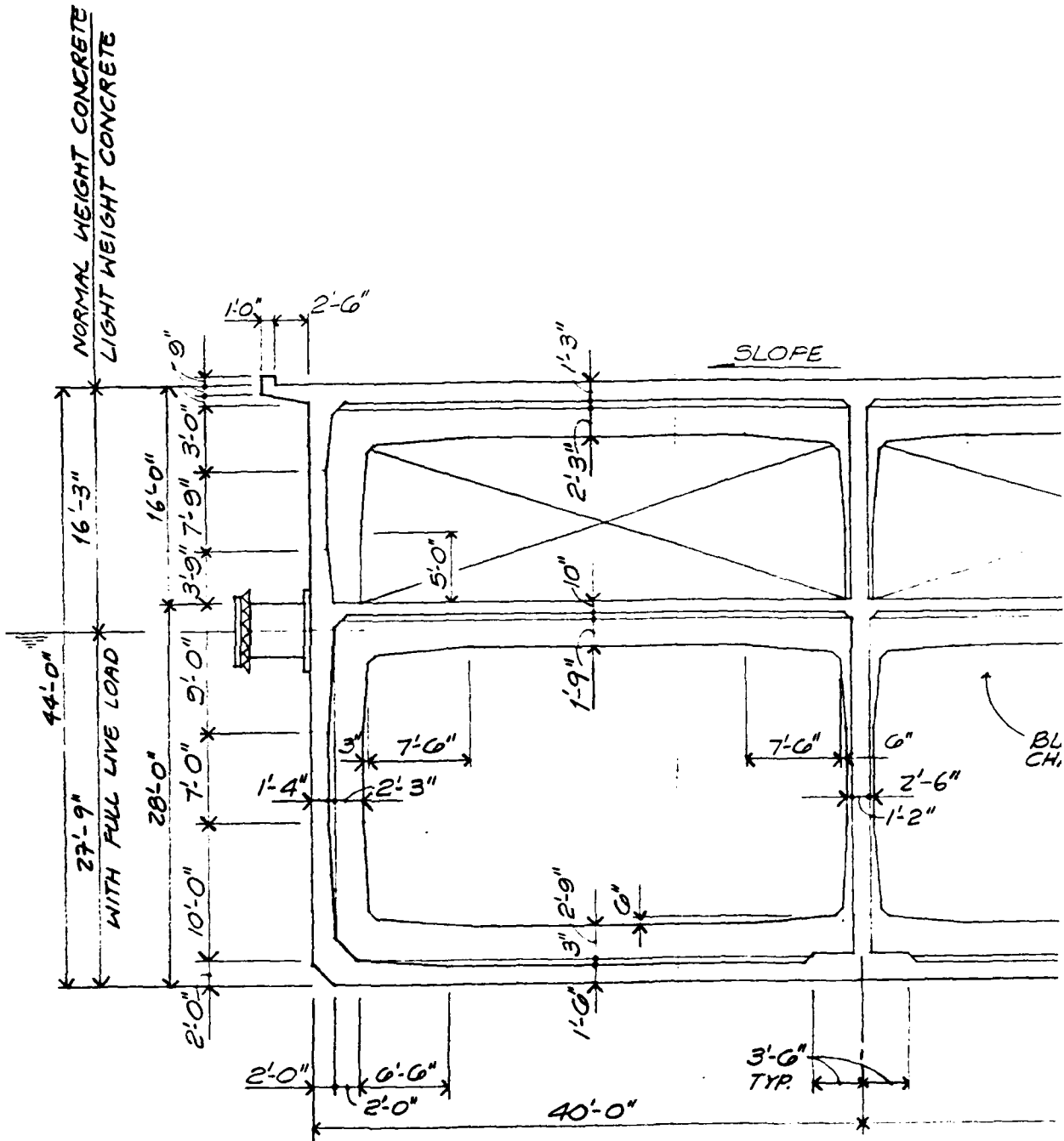


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					PROJECT: FLOATING MARINA PIER						

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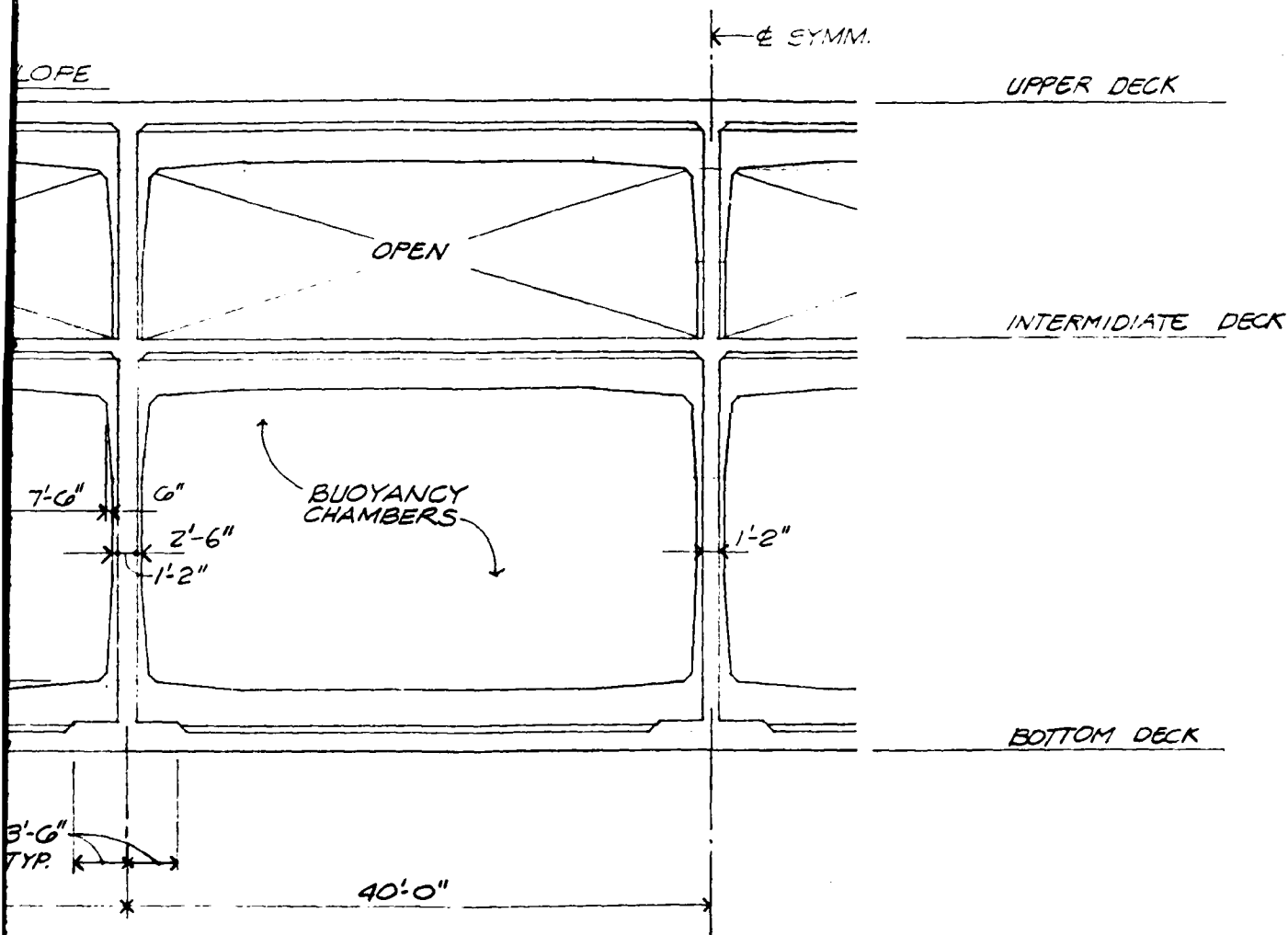
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SECTION
10'-0"

(A)
7

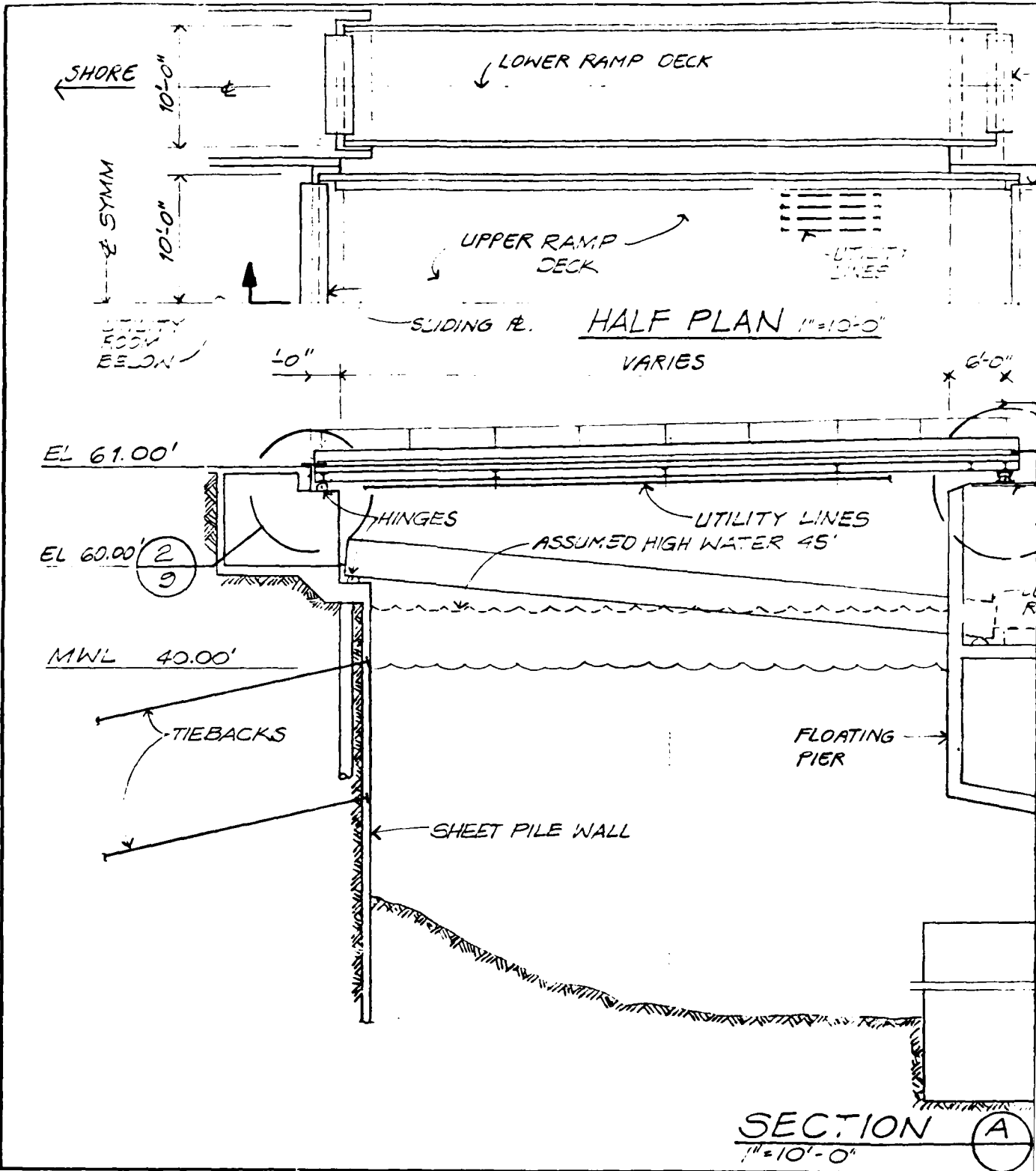
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									PROJECT:								
									FLOATING MARINA PIER								

SHEET NO.
27

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DESIGN

PROJECT NO

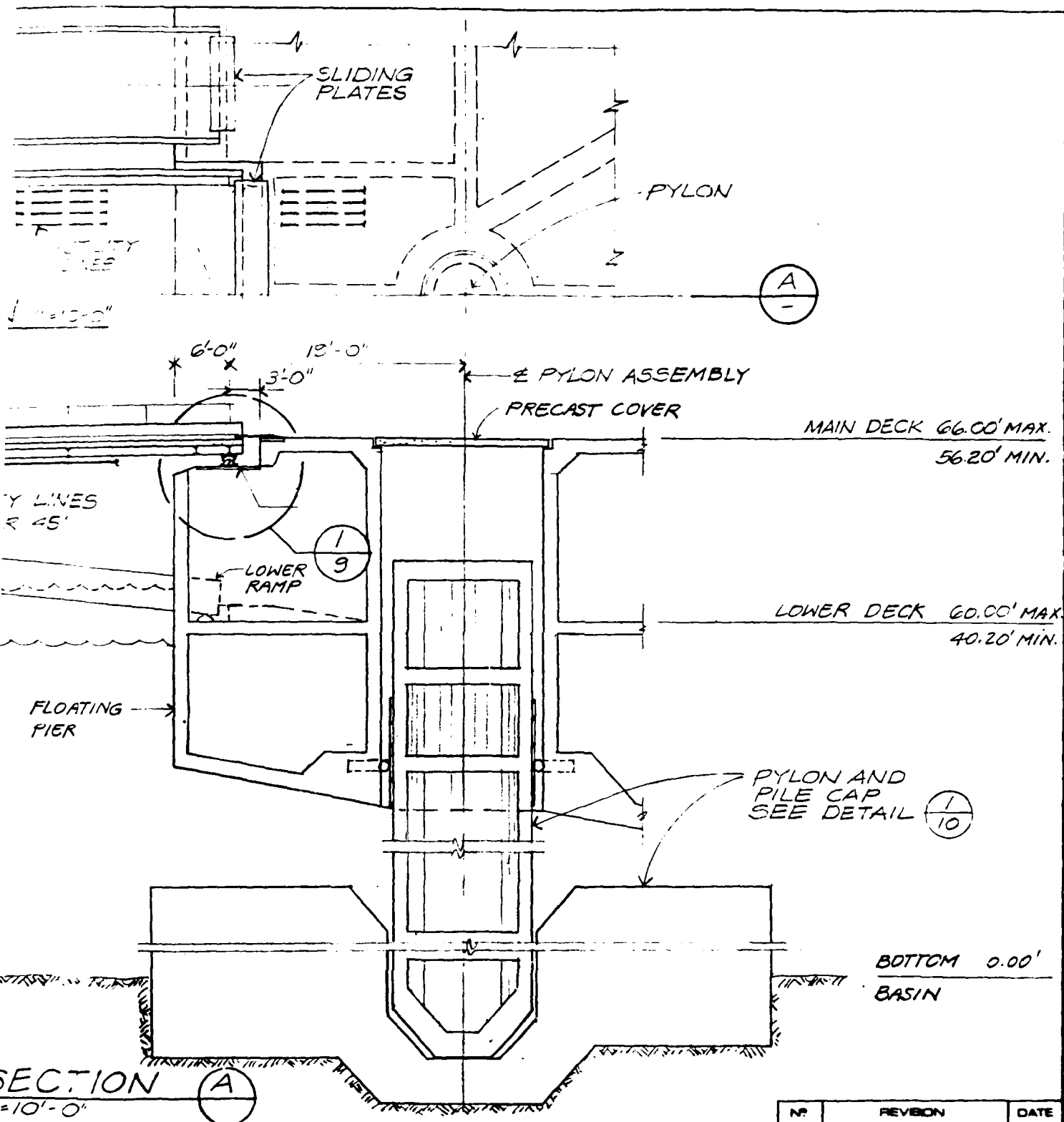


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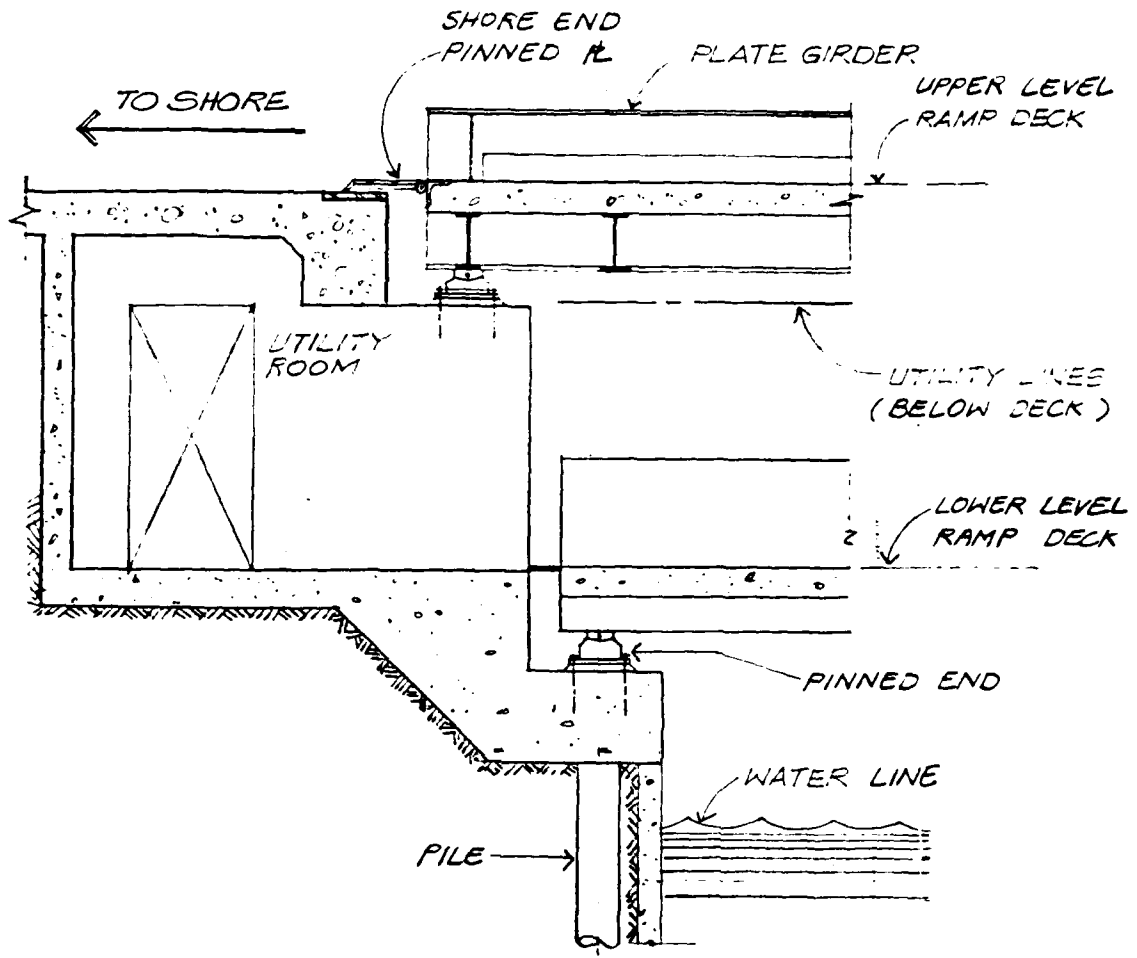
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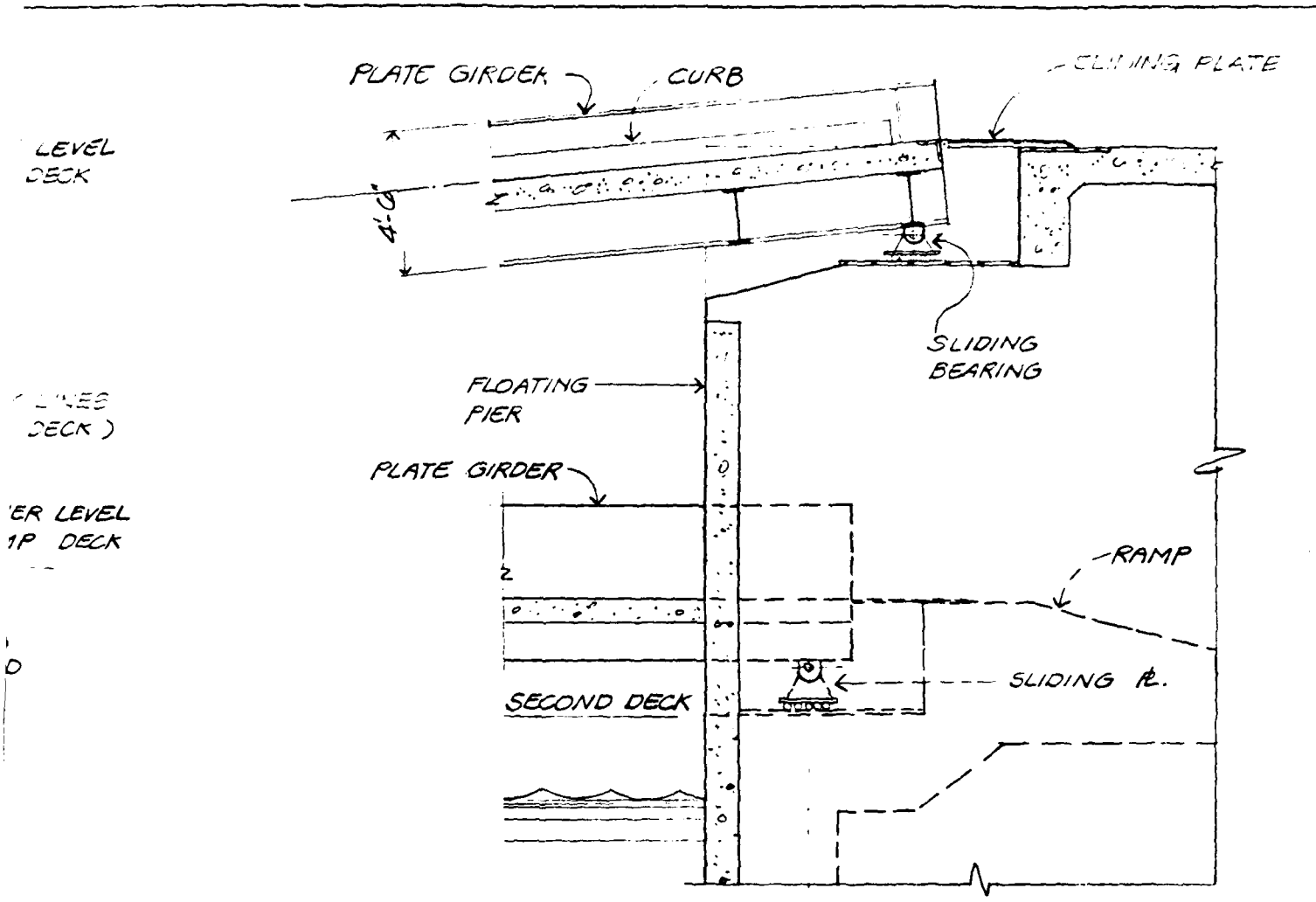


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			RAMP PLAN AND SECTION			28
			PROJECT:			
			FLOATING MARINA PIER			

PROJECT NO. _____
DESIGN _____
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DETAIL
 $\frac{1}{4}'' = 1' - 0''$ (2)



DETAIL

$\frac{1}{4}'' = 1'-0''$

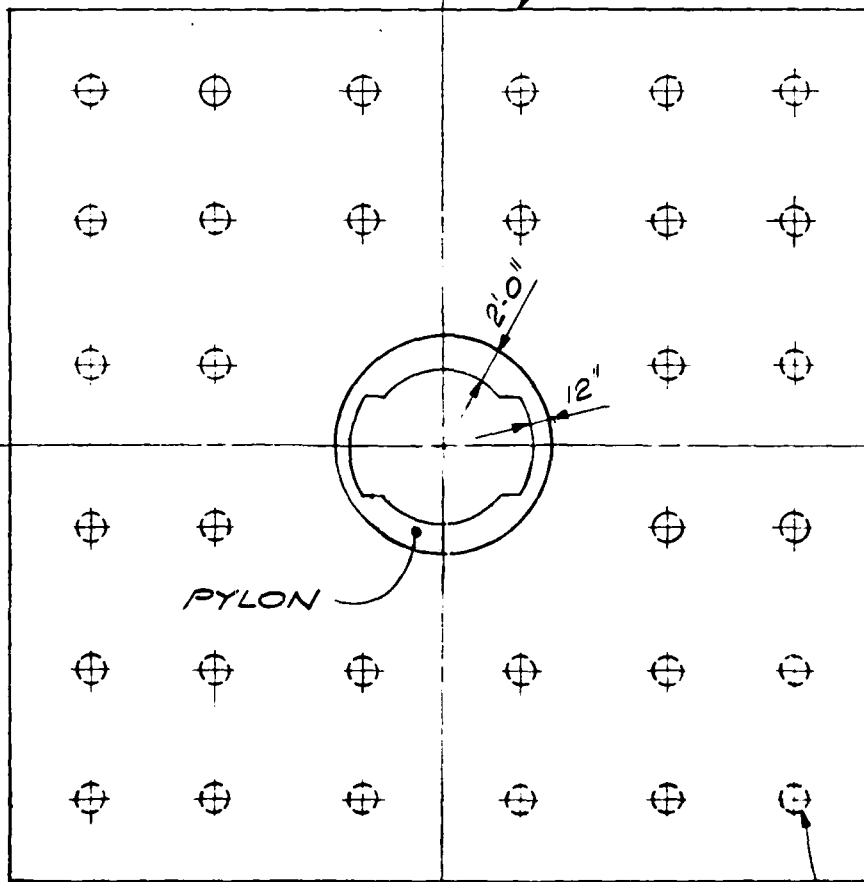
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			NO.	REVISION	DATE
Issued For	Date	By	SHEET TITLE: RAMP CONNECTION DETAILS		
			PROJECT: FLOATING MARINA PIER		
			SHEET NO. 29		

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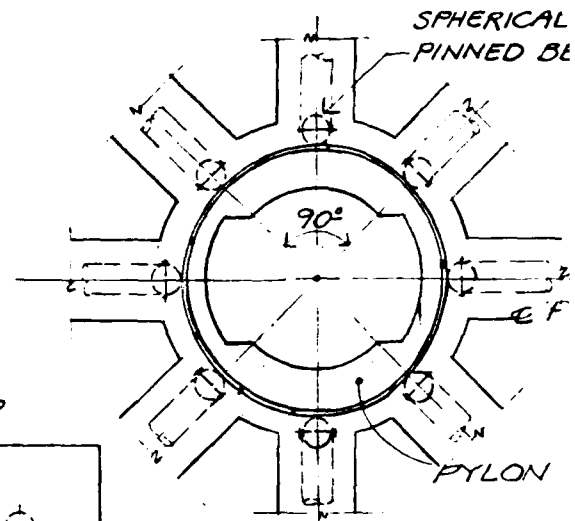
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PROJECT NO



PILECAP PLAN
1" = 10'-0"

3



PLAN SECTION

LINE OF PYLON

GROUT WITH -
TREMIE CONC.



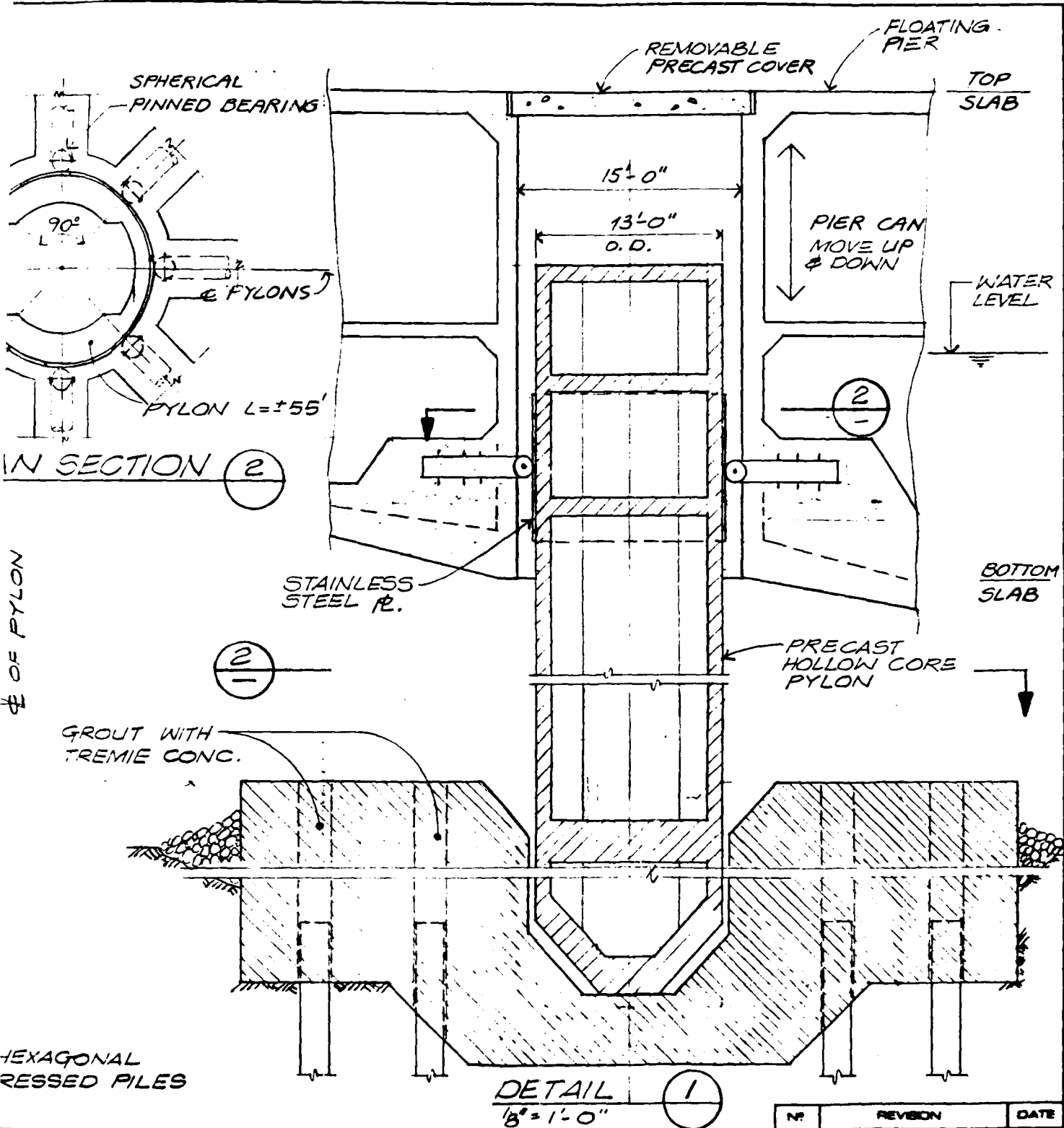
24" ϕ HEXAGONAL
PRESTRESSED PILES

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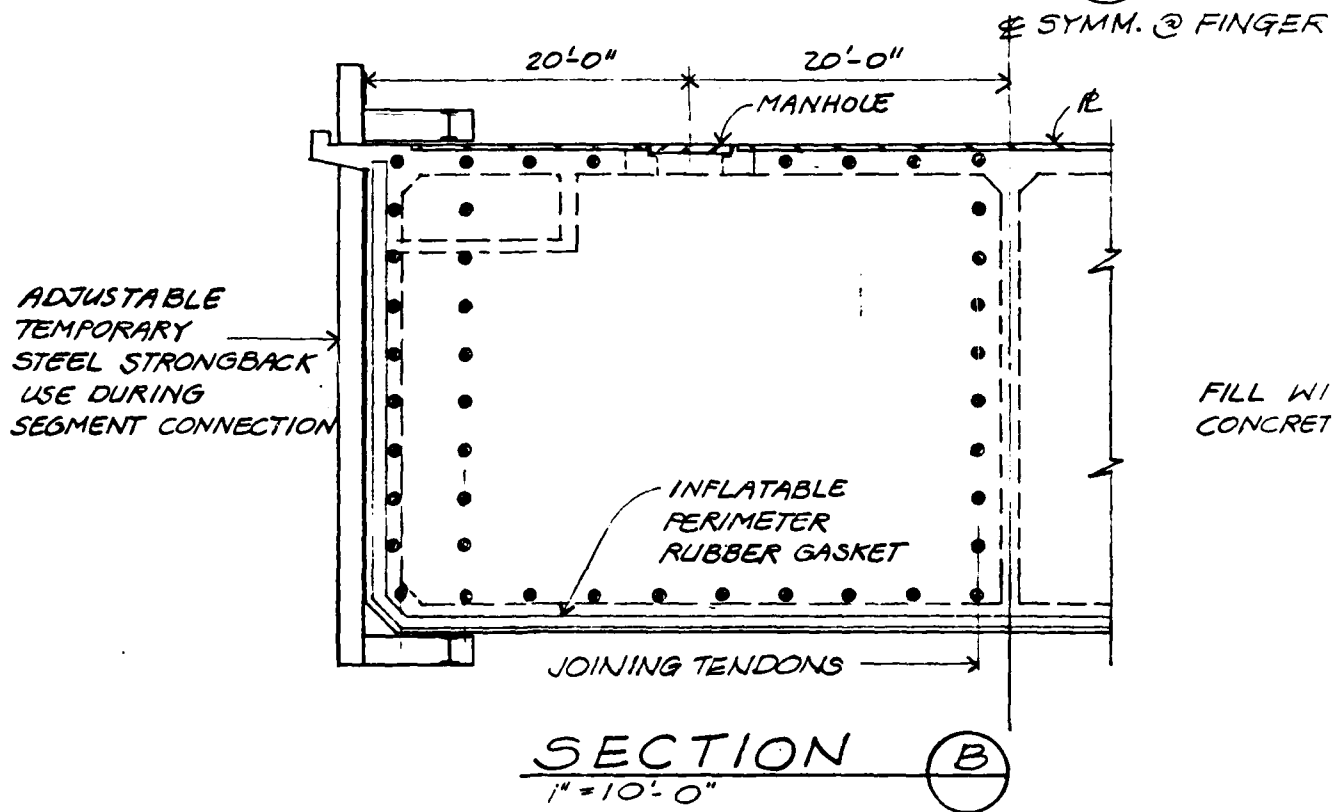
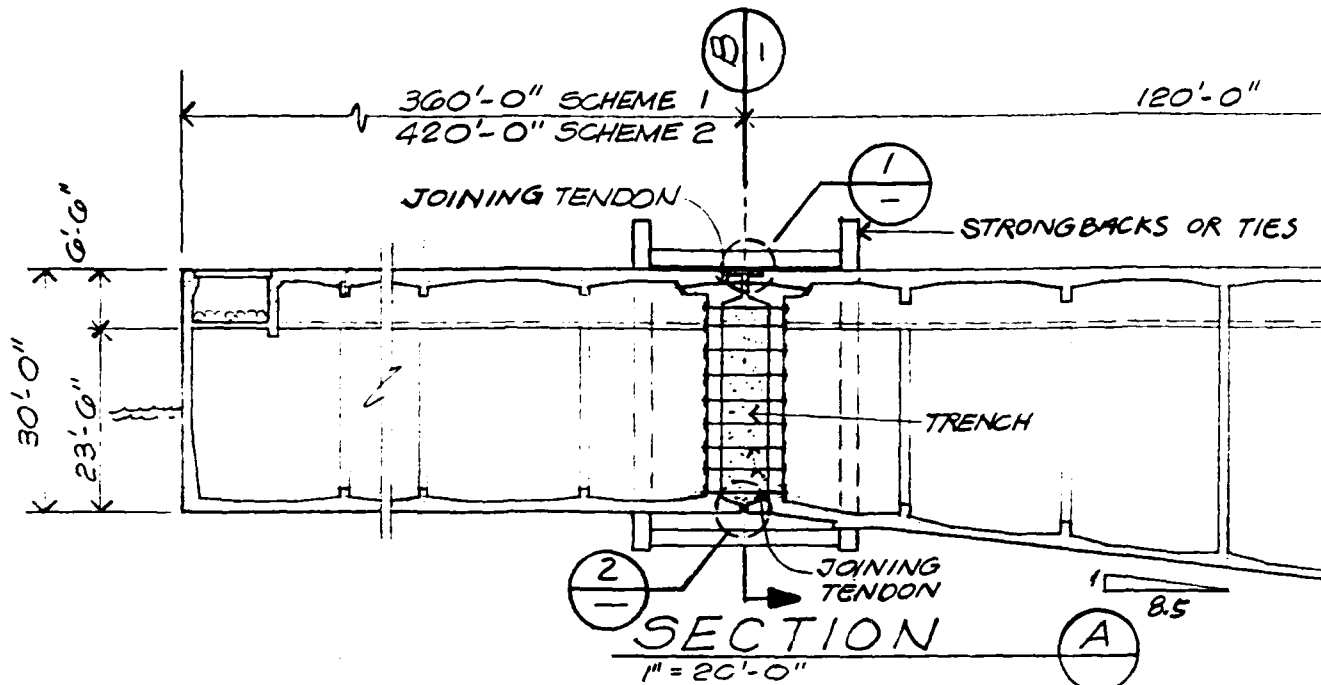
SHEET TITLE:	PYLON DETAILS
PROJECT:	FLOATING MARINA PIER

SHEET NO.	10
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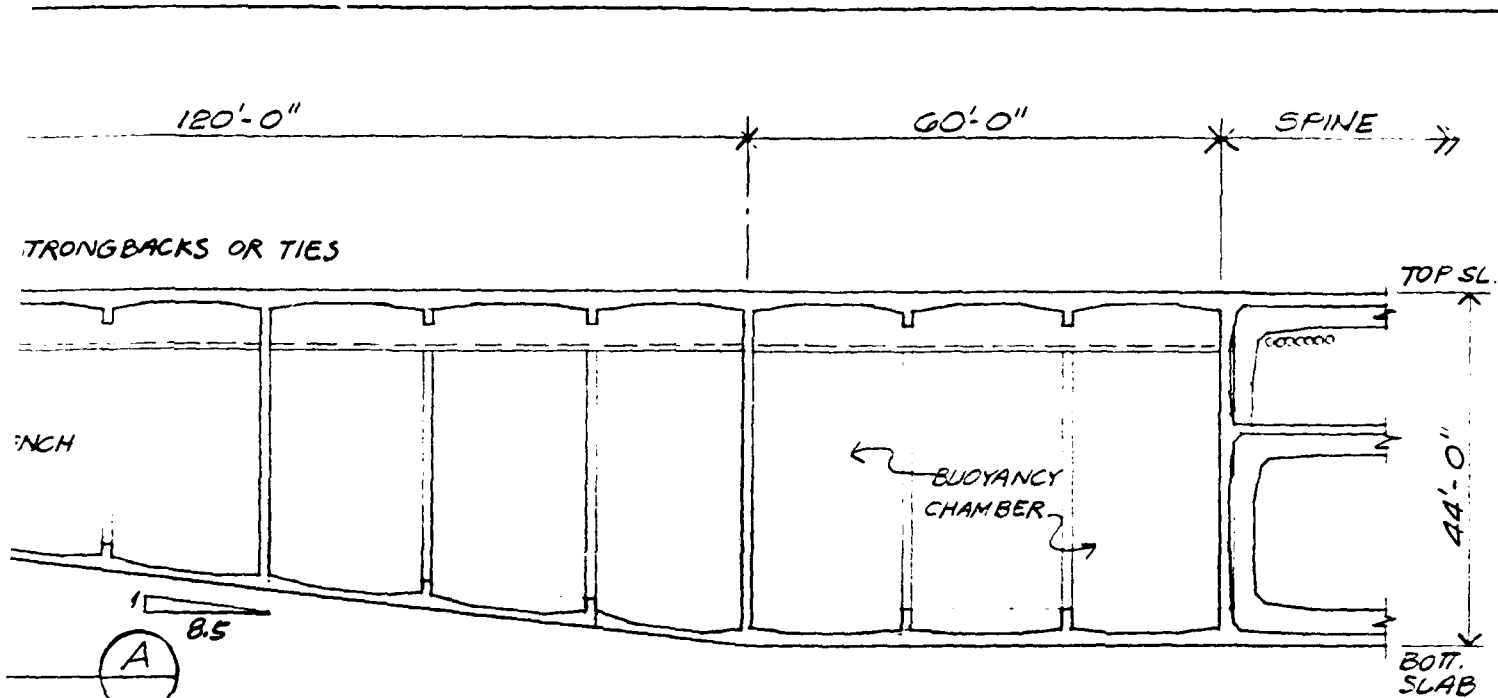


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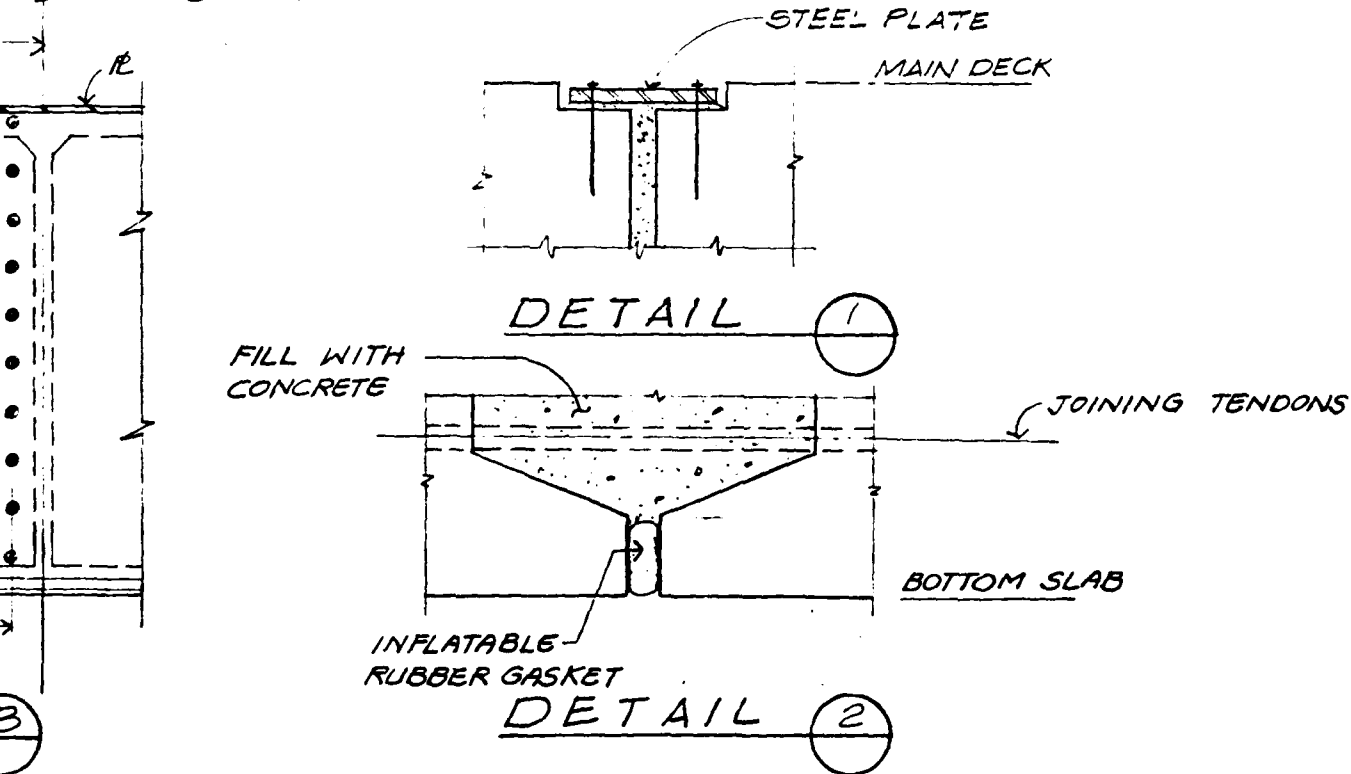
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SYMM. @ FINGER



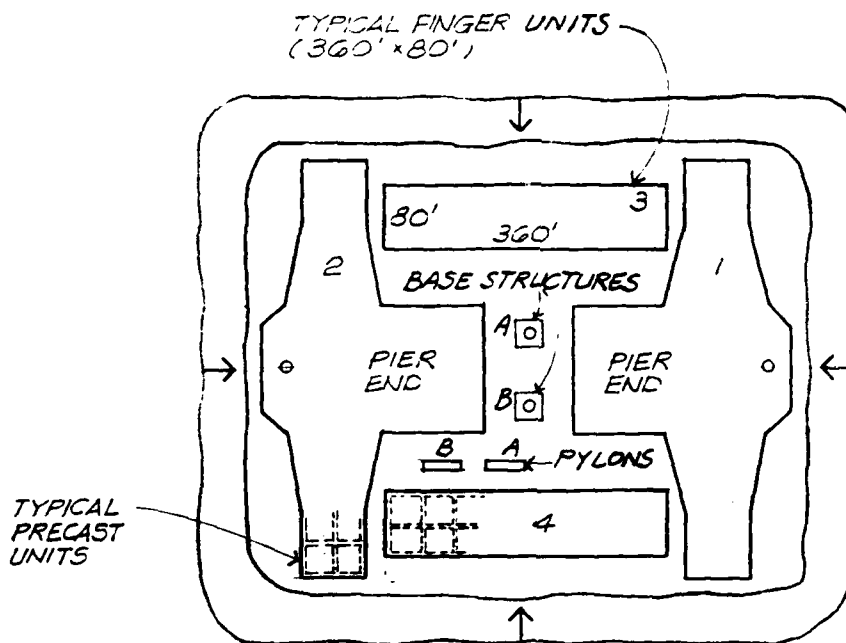
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Issued For	Date	By	SHEET TITLE:	SHEET NO.
			MODULE CONNECTION	211
			PROJECT:	FLOATING MARINA PIER

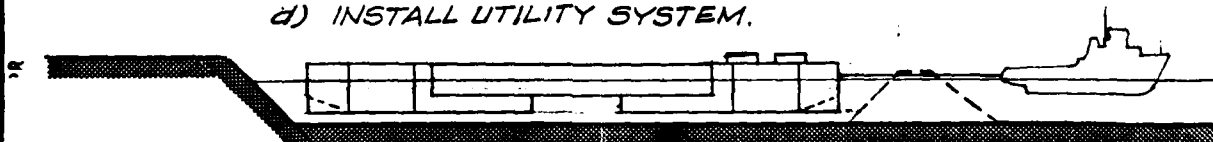
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PROJECT NO.

TYPICAL FINGER
UNITS 360' x 80'1. CONSTRUCTION IN FLOOD BASIN

- a) BUILD ONE BASIN 700' x 580'.
- b) JOIN PRECAST UNITS OF PIER ENDS (1, 2) AND FINGERS 3 AND 4 BY GROUTING AND POSTTENSIONING.
- c) FABRICATE BASE STRUCTURES AND PYLON A & B
- d) INSTALL UTILITY SYSTEM.

2 TOW TO ASSEMBLY SITE

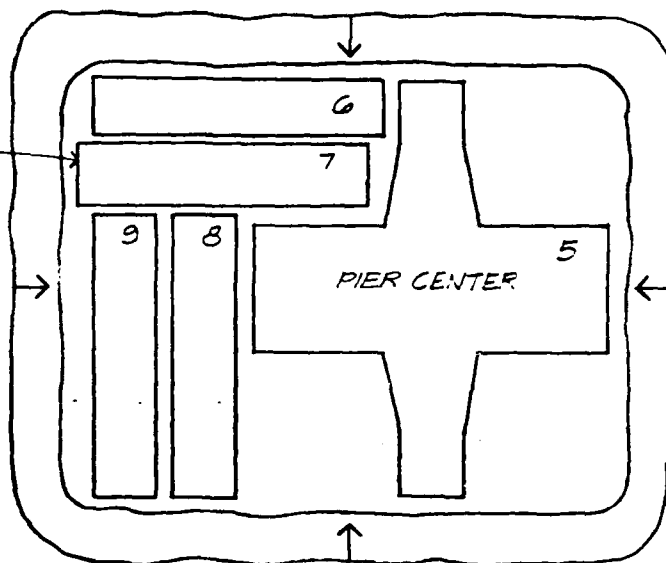
- a) REMOVE DYKE TO FLOAT UNITS. FLOAT PYLONS ON PIER END 1
- b) TOW TO FINAL SITE.

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TYPICAL FINGER
UNITS (360' x 80')



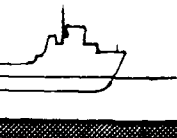
ASIN

WATER LEVEL



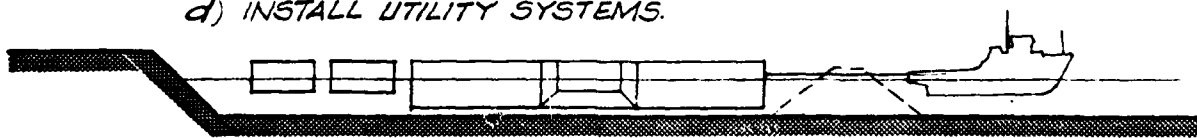
ND FINGERS 3

B



3. CONSTRUCTION IN FLOOD BASIN

- a) REBUILD DYKE (USE SAME BASIN)
- b) PUMP WATER OUT.
- c) JOIN PRECAST UNITS OF PIER CENTER (5) AND FINGER UNITS 6, 7, 8 AND 9 BY GROUTING AND POSTENSIONING.
- d) INSTALL UTILITY SYSTEMS.



4 TOW TO ASSEMBLY SITE

ON PIER END 1

- a) REPEAT STEP 2.

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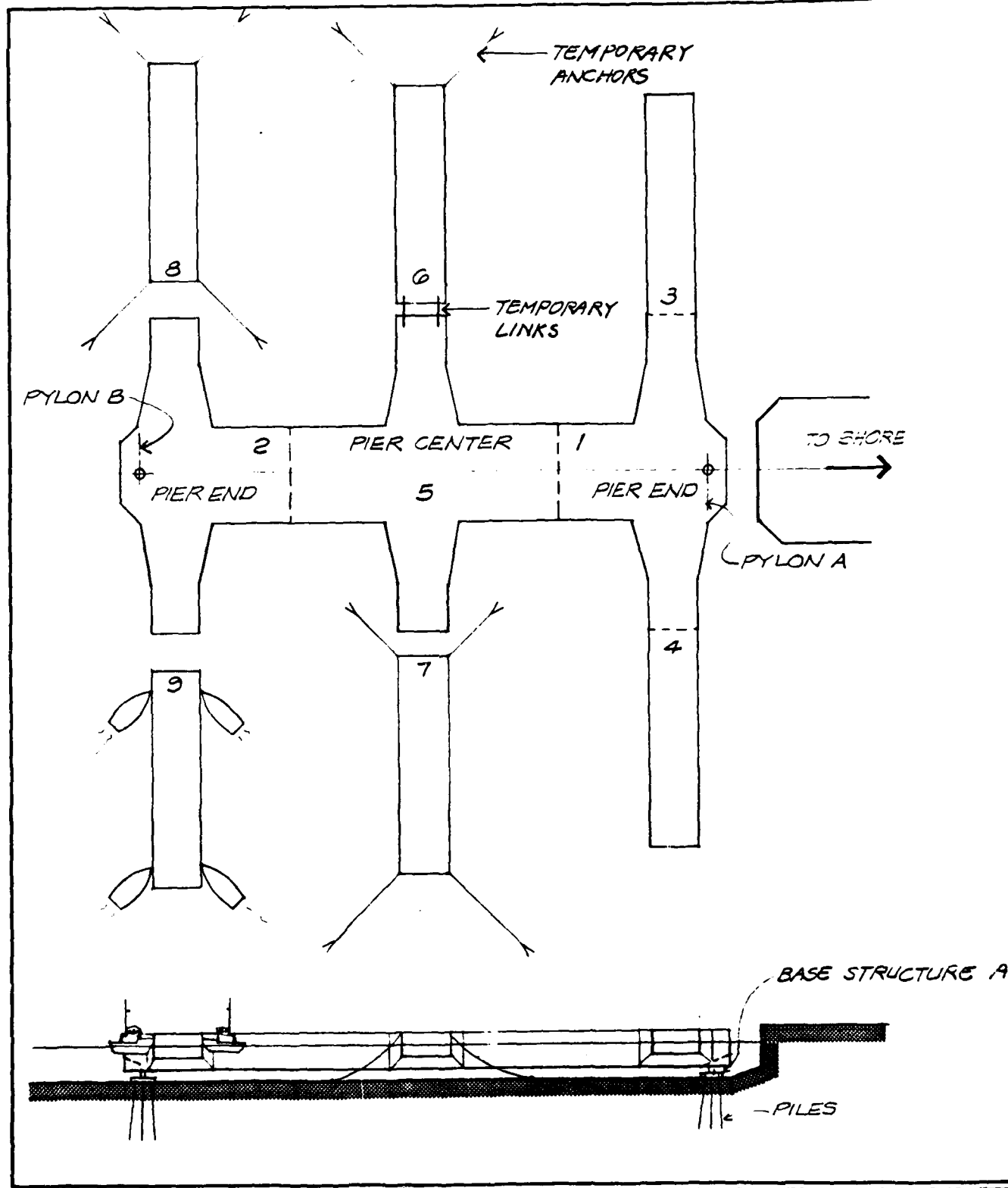
SHEET TITLE	CONSTRUCTION METHOD - 1
PROJECT:	FLOATING MARINA PIER

SHEET NO
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PROJECT NO



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5 INSTALL FLOATING MARINA PIER

- a) SINK BASE ST. - A ON PREPARED BED, DRIVE PILES THROUGH OPENINGS AND GROUT PILE SLEEVES
- b) POSITION PIER END 1 AT FINAL LOCATION AND INSTALL BY INSTALLING PYLON A
- c) POSITION PIER CENTER 5 AND CONNECT TO PIER END 1 BY POSTENSIONING AND GROUTING.
- d) POSITION PIER END 2 AND CONNECT TO PIER CENTER 5
- e) SINK BASE ST. - B , DRIVE PILES AND GROUT PILE SLEEVES
- f) INSTALL PYLON B
- g) POSITION FINGER "3" IN FINAL LOCATION AND JOIN TO SPINE BY POSTENSIONING AND GROUTING.
- h) REPEAT STEP g) FOR FINGERS 4, 6, 7, 8 AND 9.
- i) INSTALL RAMP AND CONNECT UTILITY SYSTEM BETWEEN PIER UNITS AND SHORE.

TO SHORE



V A

STRUCTURE A

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			PROJECT: FLOATING MARINA PIER		
			SHEET NO. 13		

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