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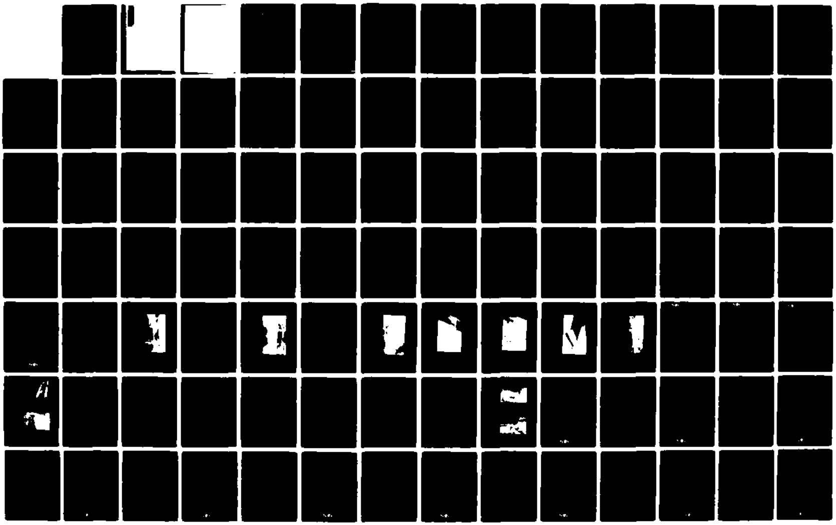

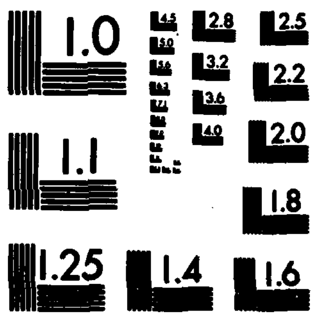

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An innovative concept for a floating pier to serve Navy surface combatants has been developed. The prestressed concrete pier is 1,200 feet long and 75 feet wide and offers a number of advantages over conventional pile supported piers. These advantages include:

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(a) A constant deck elevation with respect to berthed ships which results in decreased need to tend utility and mooring lines.

(b) A full interior deck which doubles the available length of ship-to-pier interface.

(c) A clear top deck with all utility lines located under the deck and accessible from the lower interior deck.

(d) A modern cell-type fender system.

In addition, the floating pier has significant merit when used to replace an existing deteriorated pier. The floating pier can be constructed in modules offsite while the old pier is demolished, the modules then floated into position, and the construction completed at the original pier site. Using the floating pier approach, the Navy would have an operational pier at least 12 months sooner than would be the case with a fixed pier. The initial cost for a floating pier has been estimated to be about 14% higher than that for a comparable pile supported pier.

CONCEPTUAL DESIGN OF FLOATING NAVY PIER

OUTLINE | **Page**

1. INTRODUCTION

This Report was prepared in fulfillment of Navy Contract No. N62474-81-C-9404 for Engineering Services for Navy Pier Design Concepts. The scope of the contract included the consideration of new and innovative design concepts for Navy piers and pier components that meet the needs of specific Navy ships. Primary effort was centered on designing a pier that will better serve Navy ships by improving berthing and refit functions.

Architect-engineer firms typically design Navy piers according to Navy standards. This procedure is followed to assure that the appropriate facilities are provided. However, the practice also discourages consideration of innovative design. This contract permitted the contractor freedom to pursue new design ideas, and therefore, represented a departure from normal practice. This freedom was nevertheless constrained by practicality. The results of this report should be applicable to final pier designs in the near-time frame starting in 1984.

The design concepts were to be generic, i.e., applicable in various harbor locations. Since the timing for the contract had coincided with the planning for a new pier at the Charleston Naval Base, the contract specified that the Charleston site be used to provide site data. Although some of the new ideas developed during this study could have possible application to the new Charleston pier, it was considered of greater value to direct this report to the generic features of the conceptual pier rather than to a site-specific pier design.

The new ideas that have been developed herein are presented on many levels, some detailed and some conceptual. The main new concept relates to a floating pier structural system. A preliminary design was developed to establish the feasibility of this concept for extreme loading conditions. For this purpose, the pier design was carried out far enough under the given conditions to demonstrate feasibility and practicality of the design. The preliminary design was sufficiently detailed to obtain accurate material quantity take-offs for estimating costs. Other new ideas that related to the anchoring systems, fendering system, utility systems, and construction methods were presented in a less detailed manner, but to the degree where feasibility was apparent. These ideas are presented with their rationale and supporting information.

2. OVERVIEW OF DESIGN CONCEPT

A floating pier having two decks, as shown in Figure 1, was conceived as the appropriate structure to serve combatant ships in a manner that is in many ways superior to that of fixed piers. The classes of ships to be berthed at the pier are listed in Table 1. The pontoon segment of the pier was sized at 75 ft. wide by 18 ft. deep in cross-section and fabricated of prestressed concrete. Double-wall construction was used for safety against collision damage, and three buoyancy cells across the section were provided for damage stability. Longitudinally, bulkhead walls were located every 40 ft. The overall length of the pier structure was 1200 ft. with a 50 ft. gap between the pier and shore, which was spanned by ramps. The pier was designed to be constructed as two 600-ft. long units. This length permitted off-site construction and subsequent tow to the final site. The two units would be joined rigidly by post-tensioning techniques and installed on-site by driving vertical piles through wet wells located down the centerline of the pier. The piles anchor the pier from horizontal movement resulting from berthing loads and extreme environmental loads of a combined 90 mph wind and 6 knot current. The pier is free to move vertically with tidal variations.

The roof of the pontoon section functions as the lower deck and has a freeboard of 5 ft. This deck provides space for small vehicle traffic, parking, general storage of equipment and material, and utility service equipment such as transformers, pipelines, trash containers, and salt water pumps.

The main deck was 65 ft. wide and located 20 ft. above waterline. This deck provides space for operation of large equipment, such as truck cranes of up to 90-ton capacity, semi-trailer trucks, and delivery trucks. The functions performed on the main decks are cargo handling and refit operations The main deck was designed to handle concentrated loads from crane outriggers without cribbing. It also was designed to be clear of most obstacles; however, electrical service mounds were located on the main deck for operational reasons.

Utility services to completely support berthed ships were provided by electrical and pipeline outlets spaced according to the needs of the ships. The utility pipes were hung under the main deck with clear access for maintenance from the lower deck. Outlets tee from the main pipes to a service walkway located 6 feet down from the main deck along the pier side. Hoses, stored on the walkway, would be connected to the outlets and fed to the ships without cluttering the main deck, except for the electrical cables.

Wooden fender piles and log camels have been eliminated. Modern cell fenders of the buckling cylinder type were selected.

A novel construction method was developed which provides versatility in constructing floating piers at any harbor location. The method uses a floating form; hence, land-based construction sites are not required.

3. ADVANTAGES AND DISADVANTAGES

3.1 Advantages

Both the Naval fleet and shore establishment benefit from the floating pier concept. The following advantages itemize several of the benefits which are unattainable by conventional, fixed pier structures.

3.1.1 Save Downtime of Piers Being Replaced

The Navy not only builds new piers, but also replaces old piers, many of which were built 40 years ago during WW II. Replacing piers can be more costly and time consuming than building new piers and, from a fleet support standpoint, is less desirable because an operational pier has to be taken out of service. The downtime for the operational pier is on the order of 18 months because additional time to the schedule is needed for the old pier to be removed before the new pier can be built. Replacing the old pier with a floating pier can reduce the downtime by some 12 months. The reason for the time savings is that the floating pier can be built at an off-site location prior to demolition of the old pier. Only after the floating pier has been built, including outfitting with utility systems, is the old pier demolished. The new pier is towed to the site, probably as two units, and the units are joined together. The pier is anchored and the utility systems connected to land. These operations can be accomplished within a 6 month period. The shore establishment gains from the short down-time for the pier replacement, and the fleet gains from improved readiness.

3.1.2 Advantages of Offsite Construction

The shore facility benefits from the off-site construction of the pier because major construction operations are conducted away from the base. Congestion is reduced. A typical Naval base is not a convenient location to build a pier because of lack of space for shore staging areas and because of base security restrictions on traffic flow for workers and material delivery.

Several construction methods are available to build floating piers, thus competitive bidding is promoted. Conventional construction methods for fixed piers provide limited alternatives. For floating piers the construction methods of using a dry dock, launching way, flood basin or construction barge can be used. Also, a novel construction method has been conceived, which permits the pier to be built in a floating mode. This construction approach uses a floating form which allows for incremental casting of 40-ft. long segments of the pier. Pier units of various lengths can be cast, for example 400 or 600 ft. long or, if appropriate, even a continuous 1200-ft. long unit can be cast. This single unit would probably be built at the final construction site, where towing would not be required. Once the floating form has been built, it is available for subsequent construction projects and it can be towed to other harbor sites which do not have existing flood basins or dry dock facilities.

3.1.3 The Advantages As Navy Piers

The floating pier provides a structure well suited to the berthing and servicing needs of combatant ships. Thus:

(a) The pier structure rides the tide along with the berthed ship. This means the mooring lines, brows, hoses, and electrical cables connecting the pier with the ship will vary little in suspended length. Pinching of hoses or cables between the ship and pier should not occur. Mooring lines can be taut and do not need to be tended as the tide changes.

(b) The floating pier is a natural structure for having two deck elevations. The roof of the pontoon section is located near the waterline and forms a natural lower working deck. An elevated main deck can be built to match ship deck elevations. Pier functions can be separated between the lower and the main decks; hence, a relatively narrow pier can provide a large, usable deck area. In addition, the most valuable deck space is along the perimeter of the pier next to the ship. A two-deck arrangement has twice the perimeter space of a single deck pier; this is a highly significant feature. To illustrate, a two-deck pier of 65-ft. width may have far greater servicing capability than a single deck pier of 130-ft. width.

(c) Utility pipelines can be located under the main deck. Full access to the pipelines is provided from the lower deck. Considerable space is available for expansion of the utility systems.

(d) A modern cell type fender system can be used. The fender system can be designed to contact the ship hull at the waterline because the pier and ships move together with the tide. The system can eliminate the typical wooden pile and camel fender system, which is a high maintenance item.

(e) By eliminating fender piles of any type, accidental damage to some ship components, such as propeller and sonar dome, would be prevented. The floating pier uses guide piles located along the centerline of the piers.

3.1.4 Adaptability to Different Site Conditions

The floating pier is adaptable to various site conditions. For typical sites where tidal variation, water depth, and soil strength are within normal range, piles can be driven to restrain horizontal movement of the pier. For those sites where the tidal variation or water depth is large or where soil conditions are unsuitable for piling, mooring chain and anchors or stake piles can be used to restrain the pier.

3.1.5 Better Earthquake Resistance

The floating pier can better survive a major earthquake than a fixed pier. If the floating pier is anchored on location by mooring chains, the structure is de-coupled from ground motion, and no damage would occur. If guide piles anchor the structure against horizontal forces, major ground motion could buckle the piles. However, the pier would still be floating and operational. The damaged piles would provide some horizontal restraint until auxilary mooring lines could be installed. Earthquake damage may not incapacitate a floating pier, as it could a pile-supported pier.

3.1.6 Water as Energy Absorber

A floating pier, that moves horizontally during ship impact, uses the water environment to absorb a substantial portion of the ship berthing energy. Such movement displaces large volumes of water, which dissipates energy. Displacement of water from between the ship's hull and the underwater portion of the pier also absorbs energy.

3.1.7 Mobility of Floating Pier

The floating pier can be relocated within the harbor or to distant sites. As new designs of Naval vessels evolve, this mobility will enable the Naval Base to respond to the changing requirements of the Fleet. Present fixed piers prohibit this flexibility, therefore precluding consideration of this feature. Once this capability exists, obsolete piers at prime locations can be moved to less important sites, allowing new modern piers to be installed in their places. Relocation of piers can become a regular feature of future Navy ports.

During times of national emergencies, piers may be required to rapidly upgrade advance bases. The response time of relocating an existing pier to a new site would be considerably less than that of building a new pier on site.

3.2 Disadvantages

Disadvantages exist with any concept, and it is important to acknowledge and consider the shortcomings of floating piers in their assessment. The following disadvantages are noted:

3.2.1. Higher Initial Cost

The floating pier will have a higher initial cost than that of a fixed pier for the following reasons:

(a) Larger quantities of material are required to fabricate the pontoon sections which support the main deck of a floating pier than are required to support the deck for a fixed pier.

(b) For a two-deck pier, as proposed herein, more deck area may be provided than actually required. A single deck fixed pier may meet working deck area requirements at a width of say 120 feet. The two deck concept will provide a width of 75 + 65 = 140 feet, whether required or not.

(c) Poor quality construction work will have more severe consequences for the floating pier than that of the fixed pier. Additional quality control procedures and inspection will be required beyond those services usually specified.

3.2.2 Require More Inspection

During the floating pier's operational life, it will require more inspection than a fixed pier. The buoyancy chambers will require periodic inspection for leakage, and the anchoring system will require cathodic protection, maintenance, and inspection.

3.2.3. Interface Problem Between Pier and Shore

The pier-shore interface could present the following problems:

(a) Level-adjusting ramps will be required to span the separation between shore and pier. The slopes have to be kept gentle, say below 1 to 10, for some equipment.

(b) The utility pipes must span the interface and accommodate vertical movement from tides and horizontal movement from jerking forces and environmental loads. Inspection and periodic replacement of flexible hose sections for the utility pipes will be required.

(c) The ramps must accommodate horizontal movement, both laterally and longitudinally, during an earthquake or ship collision.

3.2.4 Pier Movement

The floating pier moves in response to static and dynamic loads. This motion is likely to be slow, but still must be allowed for in all operations on the pier.

3.2.5 Perceived Disadvantages

There are two other considerations that are normally perceived as disadvantages, but in actuality are not. They are:

1. The floating pier is vulnerable to accidental sinking. The pier has been provided with a great deal of flotation redundancy from the close compartmentation of the pontoon. E.g., the flooding of any two adjacent cells will result in the loss of freeboard at the accident location of about 1 foot. The reserve buoyancy of the structure is tremendous so as to make the probability of sinking the pier very small.

2. In comparison to the hundreds of piles used for a fixed pier, the guide pile anchoring system for the floating pier may appear inadequate to restrain the structure. Piling for the fixed pier has to support all the design vertical and horizontal loads, while the guide piles for the floating pier only resist the horizontal loads. Horizontal loads on a floating pier located in a harbor environment are adequately resisted by approximately one-tenth the number of piles required for a fixed pier.

4. RELATED STRUCTURES

The State of Washington has constructed three floating bridges, two across Lake Washington and one across Hood Canal. The first Lake Washington Bridge was built in 1940 of reinforced concrete, and has a length of about 1.6 miles, while the second Lake Washington Bridge (Figure 2) was built in 1962 of prestressed concrete and has a length of about 1.4 miles. Both bridges were built in a similar manner using pontoon sections of 360 ft. long, 60 ft. wide, and 15 ft. deep (Figure 3). The exterior wall and bottom slab thicknesses are 9 inches and the interior wall thicknesses are 6 inches. The pontoon sections were joined together rigidly by bolts for the first bridge and post-tensioning rods for the second bridge. Dead-weight concrete anchors and mooring chain permanently anchor the structures on location in water depths to 220 ft. The level of Lake Washington fluctuates a maximum of three feet.

The Hood Canal Floating Bridge was built in 1957, and has a total length of 1.3 miles. The pontoon sections are prestressed concrete and have dimensions of 360 ft. long, 50 ft. wide and 18 ft. deep (Figure 4). The exterior wall and bottom slab thicknesses are 9 inches and the interior walls are 6 inches. An elevated roadway of 28 ft. wide is located 20 ft. above the waterline.

The bridge is interrupted at mid-span by a 600-ft. draw span to permit passage of vessels as large as aircraft carriers. Because of the break at midspan, the bridge can be viewed as two finger piers, each in approximate length of 3000 ft., extending from opposite shores.

The bridge is permanently moored in place by 42 concrete-filled dead weight anchor blocks and wire rope. The environmental conditions at the site are tidal range of 18 ft., tidal current velocity of 3 knots, maximum wind speed of 92 knots and wind generated waves of 5 ft. in height and 35 ft. in length. The bridge is exposed to seawater.

In 1979, a severe 100-year storm damaged the Hood Canal Bridge and caused a major section to sink. The failure was produced by a combination of dynamic response, movement of the anchors and flooding of the pontoons. Flooding occurred because the hatch covers were not tied down.

The replacement bridge is presently under construction. Prestressed concrete will again be used. The pontoon sections are wider for improved dynamic response behavior and have dimensions of 360 ft. long, 60 ft. wide and 18 ft. deep (Figure 5). The exterior walls and bottom slabs are 10 inches thick and the interior walls 8 in. The prestress level in the concrete is on the order of 1400 psi. The concrete is being manufactured with superplasticizers and a water to cement ratio of 0.33. The design 28 day compressive strength is 6500 psi. Concrete cover to reinforcing steel in walls exposed to seawater is 2 in., and to post-tensioning ducts is 2.5 in. All reinforcing steel is epoxy coated.

The pontoons are being built in a flood basin which has sufficient size to accommodate five pontoons at one time (Figure 6). Precast concrete elements are used: channel shaped exterior walls (Figure 7) and I-shaped interior walls (Figure 8). The top and bottom slab sections are cast-in-place concrete. The elevated roadway is fabricated after the pontoon sections are removed from the flood basin.

Another significant floating structure is presently under construction. A floating container terminal of prestressed concrete is being built for the Port of Valdez, Alaska (Figure 9). The terminal is a wharf type facility of size 700 ft. long, 100 ft. wide and 30 ft. deep, and is being built in two sections, each 350 feet long. Construction is in a dry dock, one pontoon section at a time, in Puget Sound, Washington. The sections will be towed to Valdez, Alaska; joined together rigidly by post-tensioning, and anchored by dead-weight anchors and mooring chains. Poor soil conditions and a tidal variation of 22 ft. exist at the site. Once installed, the terminal will carry a 40-ton container crane to unload ships of up to 650 ft. in length and 50,000 tons in displacement.

The pontoon sections are compartmentalized into four cells across the width, each 25 ft. wide (Figure 10). Bulkheads are spaced at 80 ft. The exterior wall, interior wall and bottom slab thicknesses are 12 in., and the level of prestress is approximately 900 psi. Normal-weight concrete of 28 day compression strength of 7000 psi is being used. A water/cement ratio of less than or equal to 0.44 and air-entrainment of 3 to 5 percent for freeze/thaw resistance is being used. The reinforcing steel is not epoxy coated.

Motion studies of the terminal (Reference 1) exposed to normal (annual return) waves of 5 ft. in height and 75 ft. in length and extreme waves of 10 ft. in height and 122 ft. in length showed the structure was stable for both cases. The roll period was calculated at 6 seconds and the resulting roll angle a maximum of 0.3 degrees. Crane operations would be limited to normal event sea states and thus permit a standard container crane to be used.

The methods of joining the pontoon sections is different from that of the floating bridges. The floating bridge pontoons were joined with a gap of only about 2 in., which space was dewatered and filled with grout prior to final post-tensioning. The terminal pontoons are joined with a gap of about 4 ft. so that men can work in the space to clean the concrete surfaces and connect post-tensioning ducts. Formwork is placed to cast concrete walls prior to final post-tensioning.

The Navy has had its own experiences with floating concrete structures. In 1943 the Bureau of Yards and Docks built 13 reinforced concrete floating repair docks each of 280 ton lift capacity. Each dock was 390 ft. long, 80 ft. wide and had an overall height of 40 ft., which included 14 ft. high bottom pontoons and 26 ft. high wing walls. Five of the repair docks were towed to advance bases in the Pacific Ocean. Several of these structures are still in existence and on loan to foreign countries.

In the early 1950's the Naval Air Command had a program to develop seaplanes. Offshore floating docks of precast, prestressed lightweight concrete were designed. Plans were approved for stations at Honolulu, Alameda, and San Diego, but a floating dock was built only at Honolulu. The dock structure was about 500 feet long and 100 feet wide and was built in three sections.

In 1977, the Navy built a concrete landing dock for the USS Arizona Memorial in Pearl Harbor. The dock is about 120 ft. long and 20 ft. wide. Cruise ships of about 100-ton displacement dock and unload as many as 750,000 tourists yearly.

Other floating concrete structures are ocean-going ships, which were built during the two world wars. During WW I, the U.S. Shipping Board Emergency Fleet Corporation Program built 12 concrete ships. The ships ranged in length from 260 to 434 ft., with beams from 43 to 54 ft. and depths from 26 to 36 ft. The hulls were of reinforced concrete and had thicknesses ranging from four to six inches. The average compressive strength of the concrete was 4,000 psi at 28 days. Lightweight concrete from crushed, expanded shale was used in eleven of the twelve ships. This was the first use of a manufactured lightweight aggregate. In addition to the concrete ships, a total of 32 concrete barges were built.

During WW II the U.S. Maritime Commission built 104 concrete vessels. Twenty four of the vessels were self-propelled dry cargo ships, each 350 ft. long by 54 ft. wide by 34 ft. deep. The remainder were barges which were designed for towing. They ranged from 265 to 366 ft. in length, 48 to 56 ft. in width and 18 to 38 ft. in depth. The concrete in all these vessels incorporated lightweight aggregate. The average water to cement ratio ranged from 0.45 to 0.5 and the concrete compressive strength at 28 days was 5000 psi. Hull wall thickness for the concrete vessels was 6.5 in.

The experience gained from the war time concrete vessels comprised the background knowledge that led to the fabrication of the floating concrete bridges in the State of Washington.

There have been other recent examples of commercial development of concrete floating vessels. Of interest are two concrete barge structures. The larger was a liquid petroleum gas terminal for permanent mooring in the Java Sea, Indonesia. This structure has a barge-shaped hull 460 ft. long by 136 ft. wide by 57 ft. deep, and displaces 68,000 tons when fully loaded. The structure was fabricated from precast concrete elements that were post-tensioned together. The thickness of the shell elements was 12 in. A single point mooring system permanently anchors the structure.

In 1981, a smaller concrete barge was built in Singapore and towed to Mexico. The barge was built as an offshore plant that would process dredged material from a phosphate deposit. The prestressed concrete hull has dimensions of 260 ft long, 110 ft. wide and 24 ft. deep. A new design feature for the barge was the use of an interior honeycombed structural system made up of circular walls, instead of the conventional rectangular bulkhead walls. The design life for the structure is 80 years.

5. DESIGN CONDITIONS

5.1 Site Conditions

The site conditions at the Charleston Naval Base are unusual. The base is located on a convex bend in the Cooper River. The river flows rapidly in an easterly direction along the north-side of the base, then south-easterly around the bend. Only the riverfront on the north side is used for ship berthing. The riverfront on the east side is separated from the navigation channel by a wide stretch of shallow shoal.

The new proposed pier is to be located between two existing piers, Piers L and M. Distance between the piers will be tighter than desired, so it was important that the width of the new pier be as narrow as possible. The tight pier spacing coupled with strong currents in the river will make for difficult berthing operations.

Some specific site conditions are as follows:

(a) The normal tide range is approximately 8 ft. and maximum tide range is approximately 10 ft.

(b) The average normal current condition in the channel is 4 knots, and the storm current condition about 6 knots.

(c) Storm wind direction usually starts out of the north or northwest, and changes direction clockwise to come out of the southwest. For major storms the wind can come from the south.

(d) Some locations along the waterfront have weak soil conditions. A soft marine clay overlies a marl formation at about 50 ft. below waterline. Reportedly certain locations along the coast have the marl formation at greater depths.

(e) Pier length is limited to 1,250 ft. by the shipping channel right-of-way.

5.2 Environmental Loads

The extreme environmental loading condition at Charleston, S.C., is from combined wind and current. For this preliminary study, hurricane winds of 90 mph were assumed to act concurrently with storm currents of 6 knots. The current profile was assumed as linear from 6 knots at the end of the pier to 0 knots at the shore. It was also assumed that the 6 knots existed at the surface and reduced linearly to 0 knots at the river bottom of 45 ft. depth. This current profile was considered more severe than would be encountered at the site because of existing piers J, K, and L located upstream from the proposed site. The piers and berthed ships would dissipate some of the current forces.

For maximum conditions, the hurricane wind and storm currents were assumed to act broadside to the ships and pier. The worst loading condition would be encountered when berthed ships occupied the entire length of the pier. This was represented by berthing a Destroyer Tender (AD) and Destroyer (DD) on the north side, which essentially covered the full length of the pier. On the south side of the pier, the berthing of two of the larger combatant ships, either Guided Missile Cruisers (CG) or Destroyers (DD) was assumed. Nested ships were not assumed for this loading condition, because with warnings of high winds the nested ships would put to sea.

The design prodecure for wind loading follows that of NAVFAC's Design Manual DM 26. The second ship on the lee side of the pier was assumed to take 50% of the load of the primary ship. The resulting forces from wind acting broadside to the ships is summarized in Figure 11. The calculations are given in Appendix A.

The wind forces acting longitudinally along the pier were calculated to be approximately 1/6th that of the lateral wind forces. This load was minor compared to that of the lateral load, and since the pier is also much stronger in the longitudinal direction, the longitudinal loading effects were not considered further in this investigation.

The current loads were estimated by initially using three methods: an approximation method, the new DM 26.6 method which is presently under NAVFAC review for future publication, and the existing DM 26 method. The approximation method and the new DM 26.6 method produced similar results, whereas the existing DM 26 produced current loads on the order of 50% lower than the other methods (Appendix A). The results from the new DM 26.6 were used and are shown in Figure 12.

Figure 13 shows the wind and current loads combined. To determine the horizontal load on each pile bent, the worst loading case was used, i.e., locating the AD on the far end of the pier. The resulting horizontal load on the pile bents was 125 kips per bent (Appendix A).

Although earthquake loading conditions were not analyzed in this report, available information indicated that as a rule of thumb for the Charleston area, the combined wind and current loads usually exceeded the load from design earthquake conditions. This is not a substitute for an earthquake analysis, which would be required in a full design effort. The rule-of-thumb assessment does place the magnitude of seismic effects in perspective. In a later section on pile analysis, it will be shown that the guide piles can deflect up to 6 in. horizontally and remain within allowable stresses. Considering that the amplitude of oscillation and ground motion of an earthquake in a Seismic Zone 3, where Charleston is located, is not likely to exceed 6 in., the large deflection capacity of the piles should protect the pier. For major ground motion or for resonant conditions, the steel piles would eventually yield and buckle. Even in this condition, the piles have residual strength to keep the pier on station until it is secured by auxilary mooring lines. In short, the floating pier has greater potential for remaining in operation because it is floating, as compared to fixed piers whose load carrying capacity depends on the structural integrity of the vertical piles.

The maximum horizontal load imposed on the pier during extreme environmental loading conditions was calculated by static methods to be 125 kips/pile-bent. This load can be resisted adequately by a pile anchoring system which will be discussed later. Dynamic loading conditions have not been analyzed, but are not considered critical in Charleston.

6. STRUCTURAL SYSTEMS

6.1 Pier Configuration

The total length of the pier could not exceed 1,250 ft. because of a channel right-of-way. This length pier was marginally adequate for berthing two CG's or DD's at 564 ft. long each along one side of the pier. If an AD were berthed on one side then the remaining space was only adequate for a Fast Frigate (FF) or Guided Missile Fast Frigate (FFG). Therefore, the floating pier was designed for the maximum length of 1,250 ft.; a 1,200 ft. long structure with a ramp of 50 ft. to span between the pier and shore.

The preliminary design is conducted for two 600-ft. long pier units. The decision to use two pier units was to demonstrate that the pier could be built off-site in multiple units and joined together at the site. The joint design would be similar whether 2,3, or 4 units were used to assemble the 1,200 ft. length.

The cross-section of the pier, shown in Drawing 1, is a two-deck configuration. The pontoon section has dimensions of 75-ft. wide by 18 ft. deep. The exterior wall and bottom slab are haunched sections with a minimum thickness of 9 in. The interior wall thickness is 10 in. A double exterior wall is provided because of the probability of a ship collision. The remaining cross-section is divided into three buoyancy chambers with a width of 21 ft. each. Bulkheads are spaced at 40 ft. This arrangement compartmentalizes the total pier into 90 buoyancy chambers, thereby providing high damage stability.

The roof of the pontoons serves as the lower deck. Because of longitudinal bending moment considerations from "hogging" and "sagging" the thickness of the lower deck is equal to that of the bottom slab at 9 in. This deck is adequate to resist a uniform liveload of 600 psf, and the more critical wheel loading condition from a 20-ton fork-lift truck. The lower deck has a freeboard of 7.3 ft. for a zero liveload condition. The operational freeboard is 5 ft. when the entire main deck area is subjected to a liveload of 160 psf. For the lower deck to be swamped (zero freeboard), the liveload on the entire main deck would need to be 510 psf. If the middle third of the structure were loaded at 600 psf, the pier would have zero freeboard at midlength and the material stresses would still be in the elastic region.

The main deck is located 15 feet above the lower deck and is 65 ft wide. A 5-ft. set-back on each side of the pier is provided. The thickness of the main deck is 18 in. and it is post-tensioned longitudinally to a level of about 660 psi. The controlling liveload condition for this deck was the concentrated load from an outrigger of a 90-ton truck crane.

The maximum outrigger load was 187 kips. The design concept was to allow the outrigger to be placed at any location on the main deck. An alternative design concept was to provide specific corridor sections for the outriggers. The dead-weight of the main deck would be minimized with this approach but field personnel report that in practice the outriggers would not be placed only on the thickened corridors. Hence, this alternative is not recommended.

Openings in the main deck are provided for pile installation. The openings are covered with a metal grating instead of cast-in-place concrete, for the specific purpose of avoiding inadvertent placement of an outrigger on the cover. A solid cover may be deceptive insofar as the appearance is one of strength, whereas metal grating does not appear strong enough to support an outrigger.

The main deck is essentially clear of obstacles, although certain locations have obstructions such as electrical mounds and access openings to the lower deck for trash, stairs and cargo transfer. These items occupy less than 3% of the deck area. The access openings are located at less active areas of the pier, but the electrical mounds are located at busy areas of the berthed ships. The size of the access openings is 12 ft. wide and 15 ft. long and on each side of the openings are two clear traffic lanes of 24.5 ft. width.

On the lower deck, columns divide the deck into three longitudinal bays, each having a clear width of 19.5 ft. The center bay is interrupted by pile bents every 40 ft.; however usuable center bay space is about 20,000 $ft.^2$. Up to 15,000 $ft.^2$ is already designated for specific purposes, such as trash containers, workshops, classrooms (if required), transformer stations, and a salt water pump station. The remaining unused space of 5,000 $ft.^2$ is available for small vehicle parking, material and equipment storage, and expansion of the utility systems.

The two outside bays serve as traffic lanes and perimeter space for equipment that is actively servicing the ships. One-way traffic lanes of 12 ft. wide are provided near the inside column line. Between the traffic lane and the edge of the pontoon is a space about 12 ft. wide for locating portable contractor equipment next to the berthed ships. Head room along the perimeter is limited to about 9 ft. because of the service walkway. This means that only smaller equipment can be used on the lower deck.

Combining the width of the lower and main decks gives a total pier width of 140 ft. This amounts to a pier surface area of approximately 3.8 acres. The actual narrow width of only 75 ft. is a desirable feature, especially for piers at a location such as Charleston. In comparison, a large single deck pier has a width of 120 ft. This floating pier provides 15% more deck space while occupying 60% less water space.

Engineering calculations to support the preliminary design are presented in Appendix B. The intent of the analysis was to establish feasibility for the floating pier in withstanding significant loads and to obtain accurate material quantity takeoffs for estimating cost.

The results of the analysis are the structural drawings and details, as shown in Drawings 2 through 7.

The pontoon was designed with a uniform prestress level of 750 psi compression in the longitudinal, transverse, and vertical directions of the pier. Prestress level across the joint was also 750 psi. Details of the prestress system are shown in Drawings 2, 4, 5, and 6. In order to accommodate a floating construction approach where the exterior surfaces of the pontoon are not accessible, the prestressed system has been designed so that workmen can perform the post-tensioning tasks from topside and inside the pontoon.

6.2 Anchoring System

Three methods are available for anchoring the pier to withstand horizontal loads: vertical piles, batter piles, and mooring chain (Drawing 11). Each pile bent must resist the horizontal design load of 125 kips. For the Charleston site it is proposed that vertical piles be used for the floating pier. Vertical piles have the advantages of simple installation and large displacement under load. They act in bending only while batter piles act in bending and axial compression or tension. Batter piles are more efficient in steel utilization, so they are lighter in weight than vertical piles in direct comparison. However, this advantage is offset somewhat by their rigidity, which may be translated into a heavier fendering system. The batter piles will also require more field installation work. Design calculations for both vertical and batter pile systems are given in Appendix C. Mooring chains with dead-weight concrete anchors or stake piles can be used for difficult site conditions due to poor soils, deep water or large tidal variations.

For the vertical piles, a total of 58 piles of 48 in. diameter by 1 in. wall thickness are required. The piles will be filled with sand to increase their local buckling resistance and provide corrosion resistance to the interior surface. The exterior of the piles will be epoxy coated before the piles are installed. Cathodic protection will be used for the underwater portion of the piles. A quarter inch thickness was alloted for corrosion over the 40 year design life. This sacrificial thickness equates to a corrosion rate of 6 mils per year. The protection systems should hold the corrosion rate to less than 3 mils per year.

For the vertical piles the restraining system at the pier-pile interface is steel angle sections bolted to the pontoon that rub on steel strips welded to the piles. The angle sections have replaceable teflon pads for the contact face and neoprene blocks to absorb impact forces. The bolts anchoring the angle section to the pontoon are designed to fail in shear during overload situations. It is preferable for the bolts to shear than for the piles to buckle. Hence, in an earthquake condition if the bolts shear, an additional 6 in. of lateral displacement is permitted.

If batter piles are used, 58 piles of 36 in. diameter by 1 in. wall thickness are required. The piles are joined together at the top by a shear plate.

I-beam sections welded to the non-vertical piles provide the vertical rubbing surface. The batter piles are filled with mass concrete to increase their dead-weight and assist in uplift resistance. Good soil conditions are required for batter piles because of the uplift forces.

Should damage occur to piles from any source, a convenient system is provided for pile replacement. The top deck has openings directly above the piles. The openings are covered by metal gratings. The gratings can be removed to provide construction access to the piles.

One of the advantages of locating the piles down the center of the pier is the avoidance of accidental contact between the piles and ship's sonar dome and other components, which occasionally occurs in conventional piers with wooden fender piles. The expense of repairing damage to a sonar dome is high because the ship must be dry docked.

6.3 Fendering System

Traditionally, the Navy has used wooden piles with log camels for the fendering system. The system has low first cost but high life-cycle costs.

During the course of this study, it was observed that Naval vessels have numerous protrusions from the side of the hulls except in the region near the waterline. It would appear by keeping only the waterline region clear of protrusions, the ship designers had intended for the ships to be fendered by log camels. At other locations on the hull, both above and below the waterline, protrusions have been welded to the hull. Figures 14 and 15 show two examples of hull protrusions. Other common protrusions are pad-eyes, located above the waterline and air maskings bands directly below the waterline. These protrusions would preclude the use of most modern fendering systems on fixed piers.

Unless a fleet-wide effort is made to remove the above waterline protrusions, it is difficult to foresee soft-type, modern fendering systems being used at Naval bases. The protrusions will eventually tear the soft fenders. Another disadvantage of soft fenders is that the hull plating is loaded and not the ship's frame system; thus, the plating deflects inward between the framing locations.

The floating pier permits modern fendering systems to be used because the pier and the ships move together with tidal variations. Cell fenders of the buckling cylinder type can be mounted on the pier and always contact the ship in the waterline region. The contact surface between the fender and the ship needs to be narrow and long, to simulate that of log camels. A fendering system that meets these criteria uses multiple small diameter cell fenders that are joined together to function as a unit. For the purpose of this report, the energy absorption of the pier has been assumed to be zero in the initial contact period. Each fender unit must therefore be capable of absorbing the total energy from a berthing ship. Calculations of the berthing energy are given in Appendix D. It shows that three cell fenders of 3 ft. diameter each would have sufficient capacity to absorb the ship berthing energy. A frame on the outboard face of the cells causes the individual cells to work together and also provides a hard face to the fendering system. Drawings 3 and 4 show this arrangement. In order to keep the contact area narrow, the frame has a width of 2.5 ft. The length of 15 ft. was required to limit the contact pressure against the ship hull to 45 psi. Clear span distance between fendering units is 25 ft. This distance is sufficiently small so that the bow of a ship should not catch on the side of the fender unit.

An estimate of lateral movement of the pier during a berthing operation was made by assuming that the load from the berthing ship is applied slowly enough so that the pier can respond. The berthing force will go partly into the pile system and partly into displacing water from the backside of the pier by lateral movement of the pier. When the fender cells have stopped the ship, the cells are pushing against the pier with approximately 350 kips. This force will be absorbed by approximately 1/2 in. of movement by the pier. The concentrated load of the fender on the concrete wall can be withstood because the fenders are located at bulkhead sections.

6.4 Naval Architectural Considerations

6.4.1 Wave Height

The maximum wave height for the pontoons was calculated by using two methods. The first was the longitudinal bending moment method for Trocoidal waves and the second was according to the American Bureau of Shipping (ABS) Rules. Once a longitudinal bending moment was calculated for a Trochoidal wave of some assumed height, the allowable wave height could be calculated by proportioning. This was possible because the allowable bending moment for the pontoon section was known. Calculations for both methods are shown in Appendix E. The Trochoidal wave method and the ABS method gave similar results for pontoon units of 600 ft. in length. The allowable wave height was 5.4 ft. for the assumed wave length of 600 ft. The design procedures specify that the wave length be equal to the length of the vessel.

A pier unit built in protected water should not encounter a wave height of more than 5.4 feet. It is also unlikely that a 5.4 ft. wave will be as long as 600 ft.

For pier units intended for ocean towing, it would be necessary, depending on the severity of tow conditions, to decrease the length of the pier to increase its allowable bending moment, and therefore, increase the allowable wave height. For a pier unit of length 400 ft., the wave height was calculated as 12.2 ft.; and for a pier unit of length 300 ft., the wave height was 21.7 ft.

The design for the 300 ft. pier unit was changed slightly from that of the other lengths in order for the bottom slab to be able to withstand a hydrostatic head of 22 ft. at the bottom slab. The post-tensioning ducts were more closely spaced, at 18 in. instead of 21 in.

6.4.2 Roll Period

The roll period for a pier of 1200 ft. in length was calculated as 6.8 sec. Most barge-shaped vessels have a period in the range of 7 to 8 sec. In order for the pier to roll with noticeable amplitude, waves of 6.8 sec. period would have to hit the pier broadside or at least over a major length of the pier. This is most unlikely because of ships berthed at the pier and the presence of other nearby piers. The problem is complex, and requires further study. From the experience of the Hood Canal Bridge, excessive rolling motion does not appear to be a problem; however, this structure is moored with cables at a prescribed tension.

6.4.3 Damage Stability

Damage stability calculations were carried out for the 1200 ft. long pier for two cases of flooding of the buoyancy chambers. One case had two adjacent cells at the end of the pier flooded, and another case had two adjacent cells on the side of the pier flooded. The maximum change in freeboard at the end was a list of 1.2 ft. and trim of C.9 ft. In this damaged condition, the pier would be able to function to its full capacity. Repairs could be made without interrupting the operational function of the pier.

Should flooding occur in one buoyancy cell, the local freeboard change would be about 4-5 in.

6.4.4 Heeling from Major Loads

Heeling from two major loading conditions was analyzed. The first condition was that of two large combatant ships moored on the lee side of the pier during high wind and current condition. The breasting lines are tied off to the bollards on the main deck, generating an overturning moment in the pier. If the mooring force was assumed as half of the maximum environmental load from Figure 12, than a maximum condition will be obtained. For a load of 1365 kips, the resulting heel angle was 0.763°, and the freeboard change was 6.0 in.

The second loading condition was that of a 90-ton crane on the edge of the main deck, making a maximum lift. The load at the edge of the main deck was estimated at 320 kips, which produced a heel angle of 0.268° and freeboard change of 2.1 in.

6.5 Materials

6.5.1 Normalweight Concrete

The key to concrete durability is to obtain a pore size in the cement paste below a critical diameter of about 0.1 micrometers (Ref. 2). Pores of this size and smaller restrict movement of water molecules to the point that the concrete is essentially watertight. Without movement of water within the concrete, deterioration cannot occur from sulfate attack or from corrosion of reinforcing steel.

A concrete mix design which uses a minimum cement content of 700 pounds per cubic yard and a maximum water-to-cement ratio of 0.4 will assure low permeability. A mix of this design can be difficult to place because of its low slump. The use of superplasticizers is recommended to improve workability, and to avoid the temptation to add water to the concrete by workers in the field. Superplasticizers produce high slumps with water-to-cement ratios as low as 0.33. The present Hood Canal Bridge construction uses this approach. Extra attention is required, however, because superplasticized concrete loses its high slump within 30 minutes after mixing.

6.5.2 Lightweight Concrete

Lightweight concrete is recommended for the floating pier for two important reasons: 1) to reduce the draft of the structure, which can be important during the construction phase, and 2) to improve the durability of concrete exposed to a marine environment. Past experience has shown that the performance of lightweight concrete in a marine environment is excellent. The WW I lightweight concrete ship USS "Selma" is a case in point. The ship was scuttled in 1922 in tidal waters off Galveston, Texas. When examined 31 and 60 years later, the concrete and the rebars were found to be in excellent condition (Ref. 3) in spite of a cover no greater than 5/8-in.

Lightweight concrete can have a compressive strength of 5000 psi and greater, using a cement content of over 700 pounds per cubic yard, and presaturated lightweight aggregates. This mix will have a higher slump than normalweight concrete for a water cement ratio of 0.4. The purpose of pre-soaking the aggregates is to assure that water is present for a high state of cement hydration. As cement paste hydrates, it expands in volume and fills much of the void volume originally occupied by free water. For a water-to-cement ratio of 0.4, the hydrated cement can fill enough volume to bring the average pore size down to 0.1 micrometers.

Good quality lighweight concrete has the same watertightness as that exhibited by quality normalweight concrete. Haynes has demonstrated this (Ref. 4) on concrete spheres with wall thicknesses of 3 in. subjected to external pressure heads of up to 4,500 ft. Pressure tests on the lightweight aggregate alone showed that the pore volume was quickly filled with seawater; hence, the watertightness of the lightweight concrete was obtained by the cement paste surrounding the aggregates. Watertightness provides protection to the reinforcing steel and prevents corrosion because sufficient quantities of chloride ions cannot work their way into the concrete to depassivate the high pH environment of the cement surrounding the steel reinforcing bars.

There is another feature of lightweight concrete that makes it superior to normalweight concrete. Lightweight aggregates are manufactured from expanded shale or clay and therefore contain pozzolanic materials. These pozzolans combine with the chemical compounds of the hydrated cement to form an interlocking bond at the interface of the aggregate to cement paste (Ref. 5). Under extreme loading conditions, lightweight concrete therefore responds more as a homogenous material than does normalweight concrete. The modulus of elasticity of lightweight aggregate is close to that of the cement paste, and the bond between the two materials is strong. For normalweight concrete, the bond between the cement paste and the high modulus aggregate is the location of micro-crack development because of the dissimilarity in properties between the two materials. With micro-cracking, the watertightness of normalweight concrete is reduced.

The use of lightweight concrete is recommended for general harbor structures in addition to floating piers because of its superior durability features.

The disadvantages of lightweight concrete are that its impact and abrasion resistance and shear strength are typically less than that of normalweight concrete. The cost for lightweight concrete is also about 50% greater than that for normalweight concrete (about $70 per cubic yard as compared to $45 per cubic yard). However, less lightweight concrete is used in the pontoon sections because the required buoyancy is provided by a barge-shaped hull of smaller dimensions. With less concrete used, less prestressing steel is required, so the higher material costs are off-set by savings from using less materials.

The fatigue behavior of lightweight concrete is similar to that of normalweight concrete and is not considered to be of concern for the floating pier. A guideline on fatigue design is to maintain the stresses in the concrete below 50% of ultimate strength. This criterion is easily met for the floating pier.

7. UTILITY SYSTEMS

7.1 Ship Data Analysis

The ship data contained in NCEL publication Ship Requirement Data and Pier Design Criteria, 1981, were used to determine the utility requirements for the 5 classes of ships listed in Table 1.

The detailed ship utility data were used to locate outlets at the required locations along the pier for connection to the ship's utility system. The method used was to plot the utility outlets on scale drawings of the ship's cross-section and plan view. The plan views were cut out and superimposed on a scale drawing of the pier. Initial effort in overlaying the plan views showed that the utility valves grouped themselves into clusters. An alignment system between the ships and the pier was designated by having the ship's first perpendicular, i.e., where the bow breaks the waterline, align with assigned berthing location guide marks on the pier.

Four berthing location guide marks were required on each side of the pier to accommodate all of the vessels. This is shown in Drawing 8. The AD can be berthed either near the shore or near the far end of the pier and the remaining berth space is adequate for a Fast Frigate (FF) or Guided Missile Fast Frigate (FFG). If an AD is not berthed, any combination of combatant ships can be berthed.

Once the berthing location guidemarks were determined, the individual utility systems for the different ships could be plotted and analyzed. These data are shown in Figures 16 thru 19. For example, consider Figure 16, which shows the electrical and telephone system data. The solid line represents the length of the longest ship, an AD. The data plotted on the solid line were for the case of all ships heading toward the left. The data plotted below the solid line were for the case of all ships heading toward the right. Above the solid line is a summary of the electrical mound locations on the pier.

For some ships, the data for certain utility systems were not available. This situation is noted on the figures. For example, in Figure 16 the telephone data for the FFG were missing. It was unlikely that a major error was introduced by the omission of some data because in general the outlet locations for the combatant ships were well grouped.

A summary of the utility outlet locations on the pier is shown in Figures 20 through 22. The solid line for these figures represents the entire length of the pier. For the potable water and saltwater systems (Fig. 20) additional outlet locations were incorporated to support fire fighting; these additional locations are noted.

Table 2 summarizes the utility outlet data and shows that the mean horizontal distance for utility hoses to run between the pier and combatant ships is on the order of 25 ft. The electrical system is an exception; for certain instances, electrical cables must run horizontally on the order of 200 ft. to reach distant connection points for the AD. For comparison purposes, normal practice has utility outlets spaced at 150 ft. This means that horizontal runs between the pier and the ship will be a maximum of 75 ft. Table 2 shows a comparison of the number of outlet valves required for a conventional utility layout versus the tailored layout. It is observed that approximately the same number of outlet valves are required. The difference in the layout methods is the horizontal distance that hoses must run. The tailored layout method saves handling and storing extra lengths of hose that are presently required using the conventional layout method. It is recommended that the tailored layout method be used.

7.2 Utility Layout

The utility layout for the pier is shown in Drawings 8 through 10. Drawing 8 shows an overall plan of the pier with a schematic of a bypass system for the utility pipes and a plan and elevation of Pier Unit 1. Drawing 9 shows Pier Unit 2 and Drawing 10 shows miscellaneous sections. The various utility systems are discussed below:

7.2.1 Electrical

The maximum demand for power at each berthing location is controlled by an AD requiring sixteen 400 amp receptacles, and a nested CG with ten 400 amp receptacles. The total required is, therefore 26 receptacles. The present design provides for 32 receptacles at each berthing location for an oversupply of 23%. Past records show that, in general, the power requirements for ships has doubled every ten years (Ref. 6). To plan for future expansion, space is reserved for duplicating the transformer stations on each pier unit. A high-voltage connection at each berth is also provided for portable transformer stations.

Electrical power is supplied to the pier by four 12.5kv high voltage lines. Two power lines run down each side of the pier and provide redundancy to each transformer station. As shown in Drawing 8, the transformer stations are located in a center buoyancy cell and the switching gear are located directly above in a center bay. The transformers are located on a raised floor and can be installed and removed through a removable floor panel. Sufficient space is available to house the transformer and switching gear in this arrangement.

Another transformer station is located in a center bay section adjacent to the switching gear for supplying grounded contractor power. Grounded power was incorporated into the design for Pier 2, San Diego Naval Base, for safety purposes.

Electrical receptacles are located in an electrical mound on the main deck. This is shown in Drawing 10, Section G. In each electrical mound, 32 Viking receptacles are provided along with telephone connections, which include the T.V. and data lines, two fire boxes, contractor power outlets for 120 volts, 240 volts, and 480 volts, and a high voltage connection for a portable transformer.

The decision to locate the electrical mounds on the main deck was for operating reasons. The electrical receptacles on the ships are located at high elevations. Handling the electrical cables, either manually or mechanically, will be relatively easy because the cables are accessible and space exists for proper lay-out. A certain amount of main deck space will unfortunately be occupied by the cables spread out in a single layer. The important point is that maneuvering room is available for this necessary operation. To avoid build-up of heat from electrical resistance, the cables cannot be rolled or stacked while in use.

Although the cables use main deck space, they are the only cables or hoses located on the main deck, so the images of conventional pier decks with heaps of electrical cables intermingled with other hoses, including water hoses (Figure 23) do not apply. A neat layout of short runs of cables should be visualized. Long runs are required only for supplying the AD with power. Figure 24 shows a neat cable layout with a long run at Pier 2 in San Diego.

An alternative location for the Viking receptacles is on the lower deck. This has the advantage of keeping the main deck clear, particularly in the prime operating location at amidship. However, having the receptacles on the lower deck would require considerably more man-handling of electrical cables. Also, those cables would be in the same vicinity as the steam and water pipes. Operationally, it does not appear advantageous to have the receptacles on the lower deck.

Another alternative is to have the electrical mound on the main deck but located 10 feet from the edge of the pier with the receptacles facing the ship. This would provide a corridor for the horizontal cable runs. The disadvantage is that the electrical mounds will restrict the movement of deck equipment, and prevent the equipment from getting close to the ship hull. This arrangement is not recommended.

7.2.2 Telephone

Two telephone trunk lines of 50 channels each are provided on the pier. This system contains cable television and data link lines. The outlet locations are at the electrical mounds.

7.2.3 Steam, Potable Water, Fuel Oil, and Compressed Air

These services all require pipelines under pressure. The utility pipes shown in Drawing 10, are located on the underside of the main deck. Each pipe is about 11 feet overhead and accessible for ease in repair or maintenance by working from the lower deck. Also, space exists for expansion of systems. The pipelines originate from shore, where two pipes for each service are provided for redundancy. Drawing 7 shows the pipeline interface from shore to pier. Section A of Drawing 7 shows the pier-side connection where a metal reinforced hose is used. This type of flexible connection allows free movement of the pipes in all directions.

The steam system will require an alternative design to that of the flexible hose arrangement. The steam line cannot have a gooseneck hanging down because it collects condensed water. For the steam line, a gooseneck arrangement with elevation higher than that of the pipeline is required.

Valves are located on each side of the flexible connections for ease in replacing sections when required. Connection movement is slow and the cycles few. Also, the hose is not exposed to sunlight so external deterioration will be slow. Hence, the life of the flexible hose connection is estimated at 5 years.

On the shoreside end, Drawing 3 shows a room beneath the ramp which provides a location for connecting the utility pipes to the underground pipes with a swivel joint.

On the pier, the utility pipes tee off the trunk line at the required locations and terminate at the service walkway. Typically two hose connections are provided at each outlet. The outlet valves remain under the main deck to protect them from damage from dropped objects, yet they are easily accessible for hose connection hook-up. The walkways extend along the entire length of the pier and under the main deck, leaving considerable space available for storage of hose sections.

Fuel oil spillage is contained by high toe-boards on the service walkway. Drainage ducts lead from the walkway to the double wall buoyancy cells, where spillage can be collected. After a spill the buoyancy cells are pumped dry.

The steam pipe is sized at 10 inches to allow for increased demand by the ships. With insulation, the pipe diameter will be about 20 inches. The potable water pipe and fuel oil pipe are sized at 8 inches diameter and the compressed air pipe is sized at 6 inches diameter.

7.2.4 Salt Water

The salt water system is contained entirely within the pier. A pumping station where saltwater is drawn from a wet well is located at the end of the pier. Three pumps are provided with one pump designated as a back-up. Power for the pumps is supplied from a nearby transformer station. A back-up diesel motor and generator system is also provided to assure that power for the pump will be available during a fire.

Extra salt water pipe outlets have been provided for fire fighting purposes. The outlets are spaced at no greater than 240 feet apart. At the end of the pier, four valves are provided for fire boat hook-up.

Shipboard personnel use saltwater in fighting fires onboard ships, but shore based firemen prefer to use potable water. For this reason, extra potable water outlets have been located on the pier for fire fighting.

The salt water pumps can be installed and removed as one unit through a removable deck panel on the main deck (Drawing 10 section D).

7.2.5 Sewage and Oily Waste

The sewage and oily waste pipes are gravity flow, discharging into holding tanks. The holding tanks are separate for each utility and located near the midpoint of the pier. Center buoyancy cells are used as holding tanks and each has a capacity of 50,000 gallons. This capacity is at least three times that currently provided at existing piers which use a collection system. A pump removes the waste from the holding tank and under low pressure pumps the waste to shore. This method of waste removal and storage is recommended over that of a force main system where water under pressure is used to flush the sewage lines. The changes in elevation of the pier due to tidal variations does not permit a force main system to be used.

For the gravity flow sewage system the recommendations of DM 5.8 should be followed. A 4-in. diameter pipe system is provided to throttle the discharge velocity of waste being pumped from ships. Ships have sewage pumps with capacity far in excess of pierside sewer system designs. The 4-in. pipe manifold system discharges into an 8-in. diameter gravity flow pipe that leads to the holding tank.

Venting the sewage holding tank is not recommended because the vent would be at working deck elevations. The recommended procedure is to remove the sewage by pumping it to shore within 3 hours of placement.

Although an oily waste disposal system is provided on the pier, it is possible that ship oil-water separation for coastal zones may be authorized in the near future. If authorization occurs, the oily waste disposal system will not be necessary on the pier.

7.2.6 Bollards

Bollards of 70,000 pound horizontal holding capacity are provided at intermittent locations along the length of the pier both on the main and lower decks as shown in Figure 22 and Drawings 9 and 10. The main deck bollards are the principal mooring system for the ships.

Spacing of the bollards is closer to the bow and stern of the vessels and further apart in the mid-ship section. Drawing 10, Section D shows the mooring line slopes from the ship to the bollards. The set-back of the main deck provides a good force component in the mooring line perpendicular to the pier. This set-back provides the same function as stand-off camels provide on fixed piers.

On the lower deck, bollards are required for only a few AD mooring lines. Extra bollards are provided for the general purpose use of mooring various vessels that the pier has not been designed for, such as small craft and barges.

Bollards of only one size capacity have been provided. San Diego Pier No. 2 also uses only one size of bollard. The background reasoning is that it is safe for small size ships to moor themselves to the large capacity bollards, but unsafe for large ships to erroneously moor themselves to small capacity bollards.

7.2.7 Brows

Brow locations on the pier must align with the quarterdeck locations on the combatant ships and the exit door on the AD. Figure 19 shows the quarterdeck locations for the various ship decks and the designated locations on the pier which should remain clear for the brows. Figure 22 shows the relationship of the brows to the electrical mound locations. A small overlap occurs in some instances but not enough to cause operational problems. In Drawing 10, Section D, the slope of the brows between the ships and the main deck is shown. A main deck elevation of 20 feet above the waterline is approximately halfway between the quarterdeck elevation for the DD and CG and the quarterdeck of the FF and FFG. The personnel exit door on the AD is at 22 feet above the waterline.

7.2.8 Trash System

Four trash wells are provided on the main deck so trash may be disposed of by dropping it into dumpsters located on the lower deck. Two dumpsters of size 8 ft. by 8 ft. by 12 ft. are provided under each trash well. In the present concept the dumpsters need to be specially built with wheels for the floating pier. It is envisioned that the dumpsters will be rolled into a traffic lane and joined together so that a vehicle can tow them off the pier for emptying by a conventional garbage truck. This unloading sequence is not as convenient as the conventional system. However, considerable advantage is realized in keeping the dumpsters off the main deck. A more sophisticated trash system can be envisioned where typical Navy dumpsters are located on a platform just under the main level deck. The platform is supported by hydraulic jacks that raise the dumpsters to the main deck level for emptying by a conventional garbage truck.

7.2.9 Stair and Cargo Wells

Four relatively large openings of size 12 ft. by 21 ft. are provided on the main deck to allow for stairs and an open area for cargo transfer. The clear opening for transferring material and equipment from the lower to the main deck is 12 ft. by 15 ft. This opening also provides a trash well for contractors. Typically contractors have large dumpsters of the size 8 ft. by 8 ft. by 20 ft. which occupy a considerable amount of deck space.

7.2.10 Workshops, Storage Rooms, and Classrooms

Space is available in the center bay of the lower deck for enclosing rooms for various purposes. On some piers, classrooms may be highly desirable for training purposes. On other piers, workshops or secure storage areas may be preferred. These rooms can be inexpensively built because only walls are required. The major construction items, the floor and the roof, already exist.

7.2.11 Ramps

A ramp system is provided with a 20-foot wide roadway to the main deck for the large load handling equipment and trucks. Smaller ramps having a width of 15 ft. are provided as access to the lower decks. The ramp arrangement was designed to have a maximum slope of 1 to 10 for the extreme high and low water levels. To accommodate the grade limitations, an elevated roadway on the shore side was required with a maximum elevation of 12 ft. above ground level. This elevated road provides a cellular abutment room for the utility pipes to connect to the underground pipes.

The ramps are pinned at the shore-side end and slide on the pier end. A horizontal sliding surface of 3 feet longitudinal movement is provided on the pier end to accommodate extreme displacement. The ramps have through-girders with an open-rib orthotropic deck. It is important that the ramps do not fall off the pier because of their critical function. The pier may survive a major earthquake but it can function only if the ramps are not disabled. At certain harbors, for example San Diego, shore side space is not available for an elevated approach roadway. In this case, an alternate design would provide longer ramps so that tidal variations do not produce excessive grades.

8. CONSTRUCTION METHODS

Three basic construction methods have been considered for building the floating pier: on-land construction, which includes using a flood basin, graving basin, dry dock, or launch ways; barge-mounted construction, which includes building pier sections on a barge; and the novel floating form method which has the pier built while afloat.

The following discussion on each of the construction methods envisages the situation wherein an existing pier is replaced. For this situation, as much construction work as possible would be conducted at an off-site location. The floating pier concept is not limited to this scenario, and much in this discussion would also apply in building a new pier at its final site.

8.1 Flood Basin Construction Method

Drawing 12 shows the construction sequence for the flood basin construction method. An on-land site needs to be excavated so the base of the site is below sea level by about 13 ft. A temporary dyke is constructed to keep the water out. In the dry flood basin, normal construction operations and equipment are used to build the pier units. Depending on the size of the on-land site, either two 600-ft. pier units or three 400-ft. long pier units would be built. The pontoon and main deck structure would be built using a precast concrete construction approach, similar to that used for the Hood Canal Bridge and the floating container terminal for Valdez. As much construction as possible would be completed in the flood basin; this includes completely outfitting the pier with the utility systems. At this stage, water is allowed into the flood basin to float the pier units. The dyke is then removed and the pier towed to its final site. At the final site the first pier unit is positioned, moored on location, and installed by driving piles. The second pier unit would be aligned with the first unit, joined together rigidly with the post-tensioning tendons and then installed by driving piles. The utility systems would be connected between the pier units and to shore. Minor construction items would complete the pier.

8.2 Barge Mounted Construction

For the barge-mounted construction approach, shown in Drawing 13, four pier units, each 300 ft. in length, are required. The length is dictated by the availability of construction barges. Numerous barges of size 100 ft. by 400 ft. are available whereas larger barges are few in number. One pontoon section at a time is built on the barge. More barges could be used to speed construction time. The barges are relatively expensive at rental rates of $5,000 to $6,000 per day and should be released as soon as possible to minimize cost. The pontoon sections are floated off the barges after the barges are submerged, and then towed to a location for deck construction. This location would probably be a commercial pier. The concrete work is completed and the utility systems installed. The pier units would be towed to the final construction site. The first unit is positioned and installed by driving piles. The second unit is joined to the first unit by post-tensioning tendons and installed. The third and fourth unit are installed in a similar manner. The utility systems are connected between the different pier units and to shore. Minor construction items complete the pier.

8.3 Floating Form Construction

The concept of floating form construction is to allow a pier unit to be supported by the water environment while incremental segments are built and added to the pier. This construction approach is similar to that of incremental cast-in-place contruction for cantilever box girder bridges. For the bridges, a form is moved forward in incremental steps of typically 20 ft. and the box-girder post-tensioned after every step. On water, the construction approach is simplified because gravity forces are supported by the water. To execute the floating concept, a form is required that will float at the same elevation as the pier and provide a dry construction well for building new segments. This type form is essentially a dry dock with wing walls on three sides; the fourth side has the pier extending out into the water.

A sketch of the floating form is shown in Drawing 14. Its overall dimensions are 140 ft. x 115 ft. x 30 ft. The front half of the form is the construction platform and has wing walls on three sides. The construction well area is sufficiently large to build a 40-ft. long pier segment. The back half of the form is provided to clamp the form onto the pier at two bulkhead locations. The clamping action is from flat jacks in both vertical and horizontal directions. The force of the clamps must be sufficient to prevent any relative movement between the form and the floating pier. Loads which may cause relative movement are from small waves of passing ships, swell within the harbor, hydrostatic pressure across the front end of the form, and the added weight of newly cast concrete. These forces are not overly large and can easily be designed for. A third clamping location is provided just behind the construction well. The concrete at this location has not developed full strength nor is a bulkhead available, so clamping forces are appropriately reduced.

A watertight seal must be made between the form and the pier. Several methods are possible. For example, a sliding wall with neoprene gasket material at the interface is jacked against the concrete. Another method is the use of inflatable elastomeric gaskets, or elastomeric flaps, called "J" seals. Two gaskets or seals, spaced about 1 ft. apart, would be used as backups. Water leaking past the first seal would be stopped at the second seal and then pumped out.

The floating form is provided with ballast chambers so that weight changes can be compensated. In particular, after concrete has been placed for a new pier segment, the added weight needs to be countered by pumping some water out of ballast chambers. Before the clamps are released, the form is ballasted to an elevation equal to that of the floating pier.

The construction sequence, shown in Drawing 14, consists of building the first pier segment in the construction well, and then moving this segment back by jacking against the front of the form to make room for the second pier segment.

The first segment requires an extra jacking step because a temporary bulkhead wall is required to keep seawater out of the construction well. This segment is initially jacked to the temporary bulkhead. The watertight seal system is installed. Then the bulkhead is removed and the segment jacked farther away from the construction well. The clamps are used to hold the first segment tight within the floating form.

The second pier segment is then cast against the first. These two segments are jacked backwards so that a third segment can be cast and the cycle is repeated. When segment one extends beyond the end of the floating form, the pier is anchored by piles or mooring chain, and the form is moved forward from that stage inward.

The sequence of construction for building a segment is as follows: On day one, prefabricated reinforcing mats for the bottom slab are laid out and tied together. On the same day, concrete is cast and allowed to cure overnight. On days two and three, prefabricated reinforcing mats for the longitudinal walls are set in place. The forms for the walls and roof deck are then slipped forward from the previous segment. Precast concrete bulkhead walls and wet well section are then placed between the previous segment and new segment. Prefabricated reinforcing mats for the roof slab are laid out and additional reinforced steel is installed to tie the walls and roof together. On day four, concrete is cast for the walls and the roof and allowed to cure. On day six, sufficient concrete strength has developed to permit post-tensioning.

This construction sequence produces one 40-ft. pier segment at the rate of one segment per week. At this rate, a 1200-ft. long pier can be built in some 30 weeks. Allowing for start-up time and occasional problems, the total construction time would be about 36 weeks, or nine months.

The floating form construction approach is adaptable to on-site as well as off-site construction. On-site construction consists of building the floating pier at its final location. Construction starts at the shore end and continues for the total 1200 ft. span. Shortly behind the floating form, the main deck is constructed. Once sufficient deck has been fabricated, installation of the utility systems can begin. By the time the floating form is finished, completion of the rest of the structure is not far behind.

8.4 Construction Time Periods

Table 3 shows a comparison of the time periods for construction of a fixed pier to that of a floating pier. The fixed pier, built with conventional methods, consists of precast, prestressed concrete piles and cast-in-place pile caps and deck slab. Three construction methods have been considered for the floating pier — flood basin, barge mounted, and floating form method — and each has its own construction time frame.

The time scale is set to represent the total scenario, including the replacement of an existing pier with a new pier. Adjustment can easily be made for the case of building a pier at a new site. Time zero is when the existing pier is taken out of operation. For the fixed pier, construction cannot begin until the existing pier is demolished. However, for the floating pier, the demolition work can be preceded by some nine months of construction work on the new pier itself. After the new pier is constructed, it will take approximately 6 months after the existing pier is taken out of operation for the new pier to be installed and in operation. The barge-mounted method would take longer than 6 months to place the pier back in operation because of the additional number of joints that are required for the smaller length pier units. The rapid turn-around time in placing the pier back in operation is contrasted to that of the fixed pier which requires 18 months. A savings of 12 months is realized by using the floating pier concept.

It is interesting to note that there appears to be little difference in construction time between the various construction methods proposed for the floating pier. From a construction standpoint, the important consideration is the availability of either a flood basin, submersible barge, or a floating form. If a flood basin is not available, the time for the construction of a flood basin, which would include getting permits and environmental impact reports, must be added to the total construction schedule. The availability of barges and their day-rate cost is the major consideration for the barge mounted method. There are additional limitations to the barge mounted method, as was mentioned earlier, that would make it the least desirable of the three methods. A floating form is now in the conceptual stage. It appears that the lead-time would perhaps equal the time required to build a new flood basin. All things being equal, the choice appears to be for the floating form approach rather than the flood basin approach, principally because there is a serious shortage of space around harbors to build flood basins. The floating form is also transportable and can be relocated to different harbor sites where it is needed.

As mentioned before, Table 3 can also be used to obtain construction times for the scenario of building a pier at a new site (not replacing an existing pier). Assuming that a flood basin or a floating form is presently available, then the construction time for these approaches is on the order of 15 months. The barge mounted method may require about one additional month. The conventional method for a fixed pier shows about 16 months; however, it is not uncommon for conventional construction time periods to extend up to 24 months.

9. COST ANALYSIS

9.1 Floating Pier Cost

Table 4 shows a cost breakdown for the floating pier. Material quantity take-offs from the preliminary pier design allowed for a reasonably accurate estimate on the in-place costs of the concrete, prestressed steel, reinforcing steel, piles, ramps and fenders. The cost of fabrication, which represents the cost of the floating form, flood basin or barges and the cost of mobilization, was estimated at $2 million. The difference in cost between the individual construction methods did not appear significant enough to justify itemized fabrication costs for each construction method. Cost for the towing and connection operation was estimated at $0.5 million for a pier built from two floating units. The miscellaneous cost estimate of $1.8 million includes such items as the shore-side work for ramp structures and outfitting the pier with the service walk-way, stairs, bollards and other such items. The estimated total structure cost was $18 million.

This report does not cover the detailed cost analysis of the utility systems. From past experience, however, a utility system in the magnitude of the one proposed would run somewhere between $8 million and $10 million. The latter amount has been used in the estimates. The grand total estimated cost for the pier was therefore $28 million.

9.2 Cost Comparison

The cost of the floating pier structure has been compared to that of a fixed pier structure. The unit costs for the floating pier are shown in Table 5. Three different unit cost figures have been derived. The first is the total cost of the structure divided by the total cubic yards of concrete. The resulting in-place unit cost for the concrete was $1,080 per cubic yard. The next two items dealt with the square footage costs. If the floating pier was considered as only having a 75-ft. width over the length of 1200 ft. then the unit cost would be $200 per ft.^{2}; however, if the width of both decks was used, then the cost would be divided by the width of 140 ft. and length of 1200 ft. for a unit cost of $107 per ft.^{2}.

These costs can be compared to those of the Hood Canal floating bridge, which is similar to the proposed pier, and to those of conventional fixed pier structures. For the Hood Canal bridge, the total low bid cost for a causeway section was $60 million. This cost included 9 pontoon sections and three specialty pontoon sections, and mobilization and anchorage costs. Other included costs, which were removed from the total, are: electrical work at $1 million, and removal of old pontoon sections at $2 million. The average cost per pier unit amounted to $4.75 million. Each of the normal pontoon sections contained a total of 4,010 cubic yards of concrete. The resulting in-place unit cost of concrete was $1,180 per cubic yard. The higher unit cost for the Hood Canal Bridge is probably due to the more sophisticated mooring system (a mooring line tension adjustment system) than that of the floating pier. Otherwise, the costs are comparable.

The unit cost of conventional fixed pier structures is usually stated on a square foot basis. Today's costs range from $100 to $120 per ft.^{2}. The higher cost is generally related to areas of poorer soil conditions. These estimates are substantiated by a recent report (Ref. 7) conducted for the Los Angeles Harbor Department which was completed in April 1981. This report stated that the cost for wharf type facilities was $100 per ft.^{2} (adjusted to 1982 dollars).

For comparison purposes for this report, a unit cost of $105 per ft.^{2} was considered an appropriate estimate for a finger pier. The floating pier at its full 140-ft. width has a unit cost of $107 per ft.^{2}, which was comparable to that of the fixed pier. However, at this stage in pier development it is unlikely that a fixed pier would be built at a 140-ft. width. The question arises as to determining the width of a fixed pier that would be comparable to the floating pier. Pier 2 in San Diego has a width of 120 ft. and represents the current state of the art for Navy piers; hence, this width was selected for comparison purposes.

The total cost of a fixed pier at 120-ft. wide by 1250 ft. long (the floating pier with ramp is 1250 ft. long) and at a unit cost of $105 per ft.^{2} is $15.75 million. This is the cost most comparable to that of the floating pier at $18 million. The floating pier, therefore, costs about 14% more than a fixed pier.

If first cost is the only criterion, then the choice would be for the fixed pier. However, the floating pier has certain features that make its life cycle costs less than those of a fixed pier.

9.3 Life Cycle Considerations

Table 6 shows life cycle considerations for both the fixed and the floating pier for a design life of 40 years. For the concrete structure itself, there should be practically no maintenance or replacement costs for either type structure.

The piles for a fixed pier would be precast prestressed concrete which should not require any maintenance or repair, except in case of accidents. Piles for the floating pier are steel, and will require maintenance. However, with a proper corrosion protection system for the piles, a 40-year life could be assumed. A common system is by cathodic protection for below water, and by fused epoxy coating for the splash zone. One-quarter inch of sacrifical thickness is also provided as a safety factor. Replacement due to accidental conditions has a lower probability for the floating pier because the location of the piles is along the center line of the pier. There is less chance for collision of ships or impact from a dredge cutter head. If damage does occur to the steel piles, they can be easily replaced. Openings are provided on the main deck for removal and installation of piles in the future. This would not be the case for the fixed pier structure. All in all, one would surmise that savings will result from the pile system used on the floating pier.

The ramps on the floating pier will require maintenance but should not require replacement. The cost for the maintenance is assumed to equal the savings from the pile system.

Fenders for the fixed pier are assumed to be wooden fender piles and log camels. This type of fendering system definitely requires maintenance and replacement during the life of the structure. Maintenance is constant, from small damage to major replacement from collision of ships. Depending upon the harbor location, wood boring mollusks can destroy wood fender piles within 7 to 10 years. However, field reports show that wooden fenders are replaced at such frequency from ship damage as to eclipse the problems due to mollusk damage. The life expectancy for a wooden fendering system is 5 years. Using 1982 dollars, the replacement cost for a fendering system is about $200/linear ft. or $500,000. The prorated yearly cost is $100,000 per year, per pier.

The cell fender system for the floating pier is a relatively new development. The fenders have been used in the field for only 15 years, and during this time period, no maintenance has been reported for their use. However, they can be damaged by berthing ships. Damage occurs when the bow of a ship strikes the side of a fender. For this to occur, the ship has to come toward the pier at an unusually sharp angle, which is possible but unlikely.

The cost of each cell fender unit is on the order of $20,000. The spacing for the cell units on the floating pier is closer together than on conventional piers, so the probability of a bow striking a fender cell is reduced. Assuming the damage rate is such that one cell unit per year per pier has to be replaced, the total maintenance cost is then on the order of $20,000 per year in 1982 dollars. The saving is $80,000 per year compared to the conventional system. Over a 40-year life, about $3.2 million (1982 dollars) is saved by using the cell-type fenders. This item alone appears to justify the higher initial cost for the floating pier structure.

The utility systems for both the fixed and the floating pier will also require maintenance and replacement. The difference between the two piers is the transition from pier to shore for the utilities on the floating pier. The flexible connections on the floating pier will require extra maintenance and periodic replacement. Assuming that the flexible hose connections have a life span of 5 years, and that the replacement cost is in the order of $50,000, the total replacement cost during the life of the structure is in the order of $400,000. This added cost should be offset by the savings from the use of a modern fendering system.

In summary, the floating pier is more economical than that of the fixed pier from a life-cycle standpoint. The higher first cost of $2.25 million and the added costs of $0.4 million for utility system maintenance is offset by the $3.2 million savings in fendering system costs. An additional savings for floating pier results from the rapid turn-around time for replacing an existing pier. The value of having an operational floating pier 12 months sooner than that of a fixed pier is difficult to define. If the Navy were to rent a comparable pier, which it cannot, the value would be approximately $100,000 per month or $1.2 million per year. In addition, during the 12 month period, four ships every three months, can be berthed for maintenance and refitting of equipment. Without the pier, these ships would remain in less than optimum operating conditions of military readiness.

10. FUTURE PIER STUDIES

The following are considered problem areas that require further developmental work during the design phase of the pier. They could possibly be resolved ahead of time and therefore make the floating pier concept more attractive to potential users.

10.1 Dynamic Behavior of Pier

The dynamic behavior of the floating pier needs to be studied for both normal and extreme environmental loading conditions. Operating conditions may be affected by the motion of the pier and inhibit certain functions; for example, truck cranes making heavy lifts to and from ships could experience excessive loads if the pier moved in response to waves in such a manner that the load was increased by an apparent added mass factor. The response of the pier to normal wave environments should be determined by analysis and model studies.

The response of the pier to extreme wave conditions from storm environments needs to be analyzed. Resonance created by certain wave environments should be determined, and the structural consequences evaluated. The damping effect of the long, torsionally stiff pier, with and without ships berthed, also needs to be determined.

Likewise, the response of the pier to berthing of ships must be better understood, e.g., the absorption of energy from a berthing ship by the piles and by water displacement.

Another topic is related to the dynamic behavior of the pier subjected to seismic loading conditions. Studies of simulated earthquake conditions should be made on a model to determine the behavior of the long narrow structure for the various types of anchoring systems.

10.2 Anchoring Systems

The different type anchoring systems need to be analyzed for different type soil conditions. Batter piles will be appropriate only for certain types of soils because of the uplift requirements. There will be certain cut-off depths for which both vertical and batter piles become too long and therefore uneconomical. The trade-offs between the pile and soil conditions need to be analyzed to find the optimum conditions for each pile system. The dynamic response of the pier will be influenced by the anchoring system.

10.3 Pier to Shore Transition

The utility pipe transition from pier to shore needs to be studied to determine the necessary hardware systems for the task. The magnitude of movement of the pier will be determined from the dynamic analysis study. These data are critical to the design of the flexible connection systems.

10.4 Floating Form Construction System

The floating form construction system needs to be advanced another stage, to the point where a decision can be made on whether to proceed with this alternative construction method. The advantages that can accrue from this construction approach appear to justify its full development. It will greatly enhance the capability of the Navy to carry out rapid construction at advance bases, and at harbors where on-land construction sites are not readily available.

11. SHIP MODIFICATIONS

At present, piers are designed to accommodate ships with little attention given during the ship design process to pier facilities or functions. The purpose of this section is to have pier designers propose ship design modifications so that an improved interface between ships and piers can result.

11.1 Navy Ships Without Protrusions

The sides of Naval ships above the waterline should be kept clear of protrusions. This modification would permit several types of modern fendering systems to be used on Navy piers. Wooden fendering systems would become obsolete and a major maintenance problem solved.

11.2 Lower Elevation for Electrical Connections

Electrical connections on the ships should be located on a lower deck elevation. This action will assist in minimizing the length of electrical cable running from the pier to the ship. Extra cable length imposes handling difficulties and power loss from resistance through long cables.

11.3 High Voltages

The ships should be designed to accommodate higher line voltages from the pier. The present power requirement of about 6000 amps at 480 volts for certain combatant ships justifies higher line voltages to reduce power costs. Safety is a separate question; however, aircraft carriers presently receive 4160 volts.

11.4 One Location for Utility Hook-Up

All utility hook-ups should be made at one location, preferably at the bow or stern. This action would considerably simplify service operation by reducing the number of utility outlet locations on the pier.

11.5 Side Hatches

General stores could be loaded onto the ships easily in containerized packages that are pushed into side hatches on the ships. Because the floating pier remains at the same elevation as the ship, side hatches could be located to receive material from the lower deck.

12. SUMMARY

The floating pier has a number of important advantages which make the concept highly attractive as an alternative structural system to that of fixed piers. The pier will improve services to Navy ships because the pier floats with the ships during tidal variations. This allows for a double deck configuration, and improved operation where main deck elevation is more in alignment with ship deck elevation. The floating pier also permits the replacement of the wooden fendering systems with modern, low maintenance systems.

The floating pier is of significant merit when used in replacing an existing deteriorated pier. The new floating pier is built off-site and outfitted with utilities before the existing pier is demolished. Once the existing pier is taken out of operation, the floating pier is quickly installed and in operation within six months. This procedure is compared to that for a fixed pier which requires 18 months construction time. Using the floating pier approach, the shore establishment has an operational pier 12 months sooner than a fixed pier.

First cost for a floating pier structure is about 14% higher than that for a fixed pier. However, after life-cycle costs are considered, in particular the saving from eliminating the wooden fender pile system, the floating pier shows an economic advantage.

An innovative development from the floating pier concept was a floating construction concept. This development allows construction within harbors which lack on-land facilities, such as flood basins or graving docks.

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TABLE 1

SUMMARY OF SHIP CLASSES

SHIP CLASS	OVERALL LENGTH (FT)	MAX. BEAM (FT)	MAX. DRAFT (FT)	DISPLACE-MENT (TONS)
CG-47, GUIDED MISSILE CRUISER	563	55	31	9,200
D-963, DESTROYER	563	55	29	7,800
FF-1052, FAST FRIGATE	440	47	25	4,100
FFG-7, GUIDED MISSILE FAST FRIGATE	445	47	24	3,700
AD-41, DESTROYER TENDER	643	85	26.5	22,800

TABLE 2

SUMMARY OF UTILITY OUTLETS

UTILITY ITEM	MAX. DISTANCE TO AN OUTLET (FT)		NUMBER OF OUTLET LOCATIONS FOR:	
	COMBATANT SHIPS	DESTROYER TENDER, AD	TAILORED LAY-OUT, THIS REPORT	CONVEN-TIONAL LAYOUT[a]
ELECTRICAL	80	200	4	4
TELEPHONE	80	20	4	4
STEAM	20	10	16	14
POTABLE WATER	20	40	22	14
SALT WATER	25	50	22[b]	14[b]
SEWAGE	30	40	18	14
FUEL OIL	25	70	14	14
OILY WASTE	25	40	18	14
COMPR. AIR	25	50	14	14

a MAX. DISTANCE TO AN OUTLET FOR CONVENTIONAL LAYOUT IS 75 FT., EXCEPT FOR ELECTRICAL WHICH IS ON THE ORDER OF 200 FT.

b DOES NOT INCLUDE OUTLET VALUES FOR FIRE BOATS.

TABLE 3

CONSTRUCTION TIME PERIODS

<u>CONSTRUCTION METHOD</u>

1. **FIXED PIER: PILE - SUPPORTED**
   - DEMOLISH EXISTING PIER
   - CONSTRUCT NEW PIER
   - INSTALL UTILITIES AND COMPLETE

2. **FLOATING PIER: FLOOD BASIN METHOD**
   - CONSTRUCT FLOOD BASIN
   - CONSTRUCT PIER UNITS AND UTILITIES
   - DEMOLISH EXISTING PIER
   - INSTALL PIER UNITS, CONNECT UTILITIES AND COMPLETE

3. **FLOATING PIER: BARGE - MOUNTED METHOD**
   - CONSTRUCT PIER UNITS & UTILITIES
   - DEMOLISH EXISITNG PIER
   - INSTALL PIER UNITS, CONNECT UTILITIES AND COMPLETE

4. **FLOATING PIER: FLOATING FORM METHOD**
   - CONSTRUCT FLOATING FORM
   - CONSTRUCT PIER UNITS & UTILITIES
   - DEMOLISH EXISTING PIER
   - INSTALL PIER UNITS, CONNECT UTILITIES AND COMPLETE

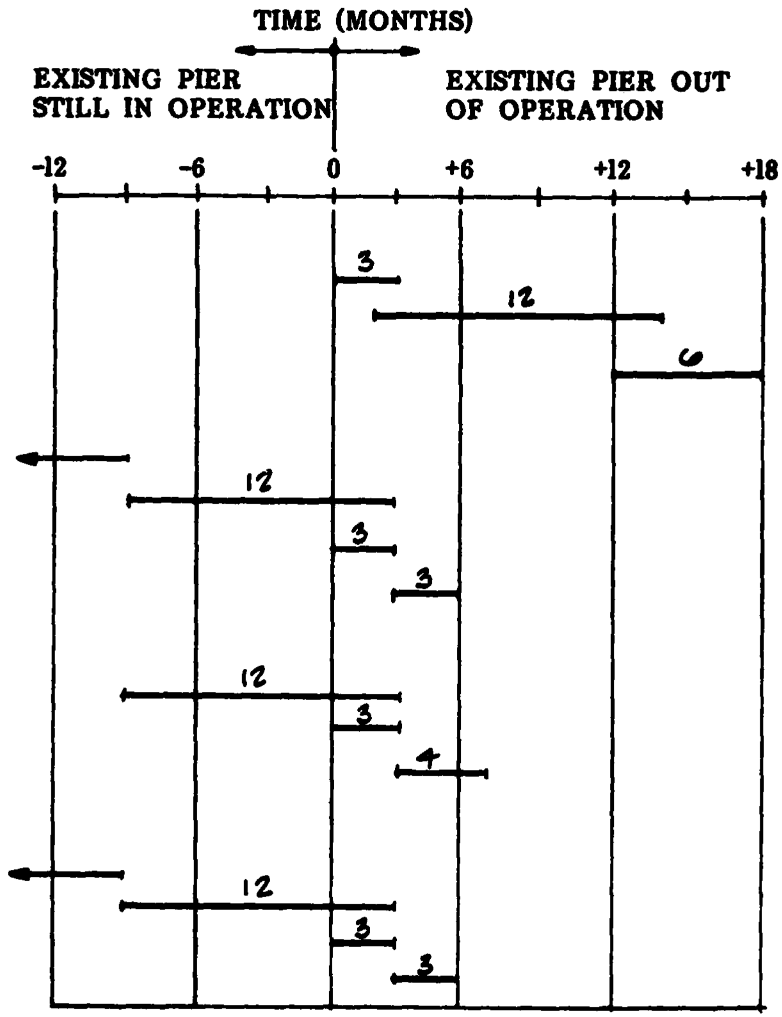


TABLE 4

FLOATING PIER COST ESTIMATE

ITEM	QUANTITY	UNIT PRICE	COST ($M.)
CONCRETE	16,600 C.Y.	$ 400/C.Y.	$ 6.64
PRESTRESS - STEEL	1,535,900 lb	$ 1.50/lb	$ 2.30
REBAR - STEEL	1,220,000 lb	$ 0.50/lb	$ 0.61
PILES	2,911,600 lb	$ 0.75/lb	$ 2.18
RAMPS	340,000 lb	$ 2.00/lb	$ 0.68
FENDERS	188 CELLS	$ 6,500/CELL	$ 1.22
FABRICATION METHOD	FLOATING FORM, FLOOD BASIN, OR BARGES		$ 2.00
TOW AND CONNECTION			$ 0.50
MISC. 10%			$ 1.80
	ESTIMATED TOTAL STRUCTURE COST =		$ 17.93
		SAY =	$ 18.00
	ESTIMATED UTILITY COST		$ 10.00
	GRAND TOTAL PIER COST		$ 28.00

TABLE 5

UNIT COST

(ESTIMATED COST OF STRUCTURE = $18.0 MILLION)		
ITEM	QUANTITY	UNIT COST
IN-PLACE CONCRETE	16,600 C.Y.	$ 1,080/C.Y.
ASSUMED SINGLE DECK WIDTH	75' x 1200'	$ 200/FT^2
DOUBLE DECK WIDTH	(75' + 65') x 1200'	$ 107/FT^2

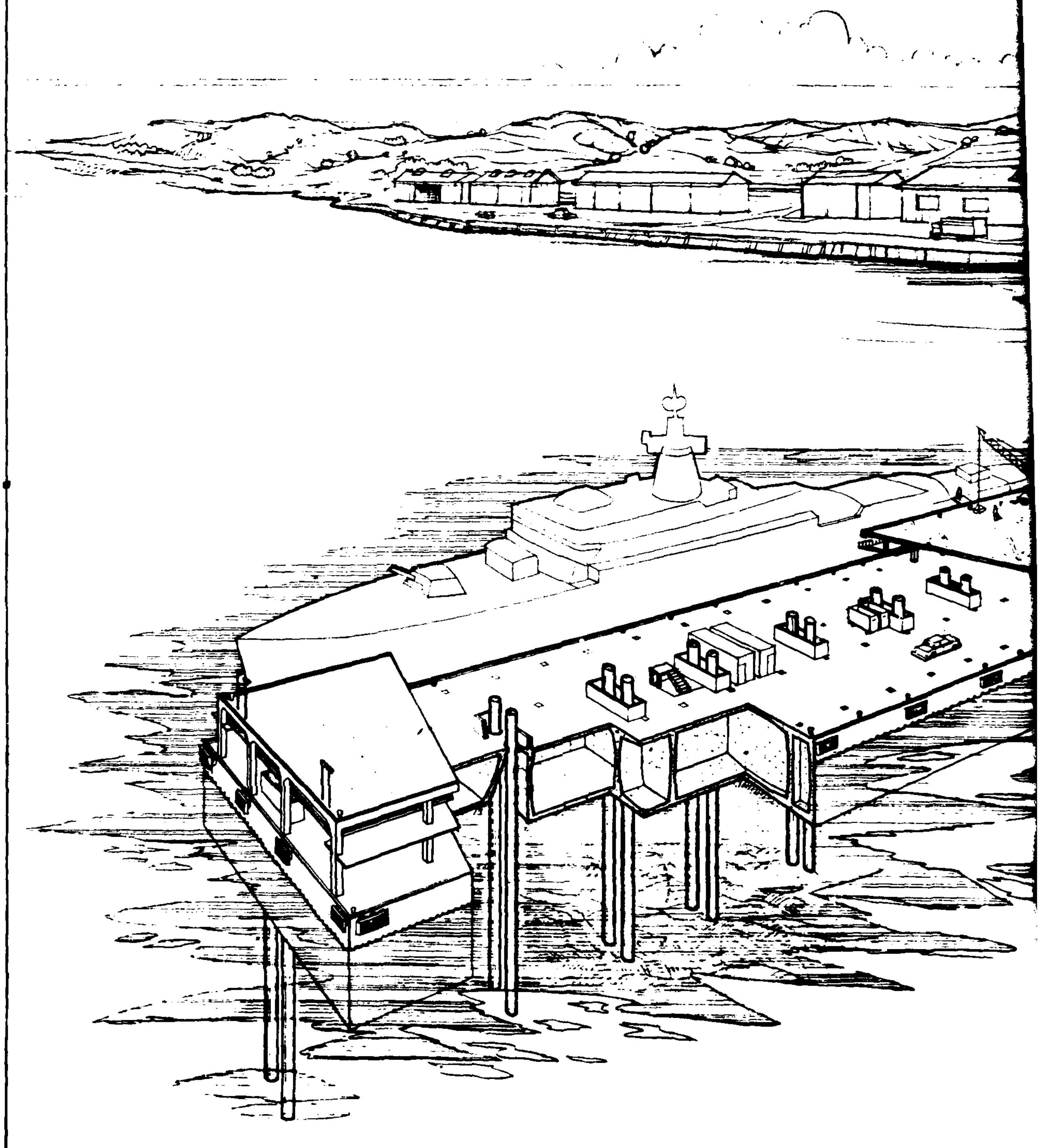
TABLE 6

LIFE CYCLE CONSIDERATION

ITEM	FIXED PIER MAINTENANCE	FIXED PIER REPLACEMENT	FLOATING PIER MAINTENANCE	FLOATING PIER REPLACEMENT
CONCRETE STRUCTURE	NO	NO	NO	NO
PILES	NO	ACCIDENT	YES	ACCIDENT
RAMPS	-	-	YES	NO
FENDERS	YES	YES	NO	YES *
UTILITIES	YES	YES	YES	YES

* ONLY 15 YEARS OF FIELD EXPERIENCE AVAILABLE.

PROJECT N° ____ DESIGN ____ DRAWN ____ CHECKED ____



T.Y. LIN INTERNATIONAL

STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133, Tel. (415) [illegible]

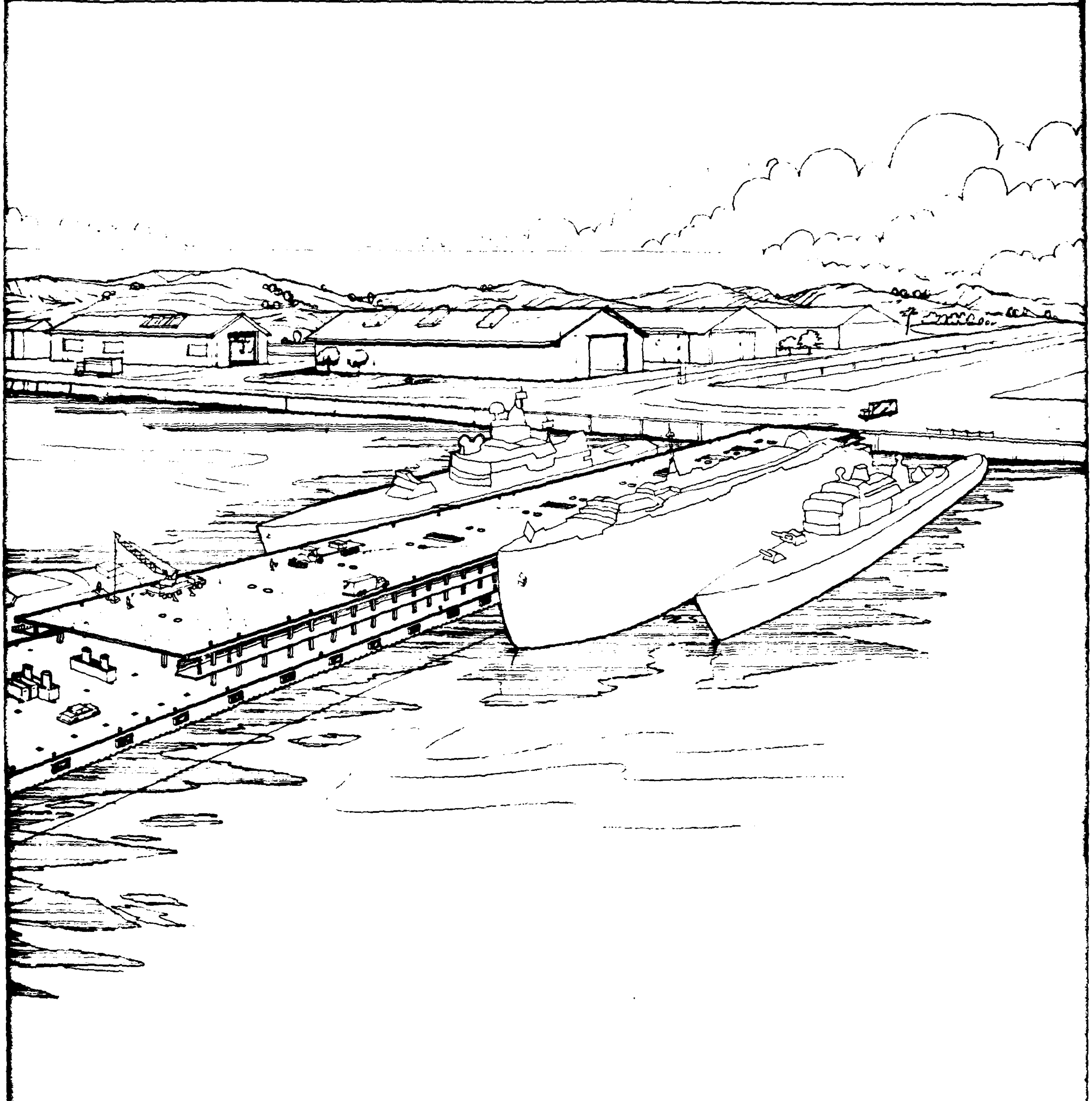


FIG. 1 RENDERING OF CONCEPT OF FLOATING NAVY PIER

NO	REVISION	DA

Issued For	Date	By

SHEET TITLE: **PERSPECTIVE**

PROJECT: **NAVY PIER DESIGN CONCEPTS**

SHEET No

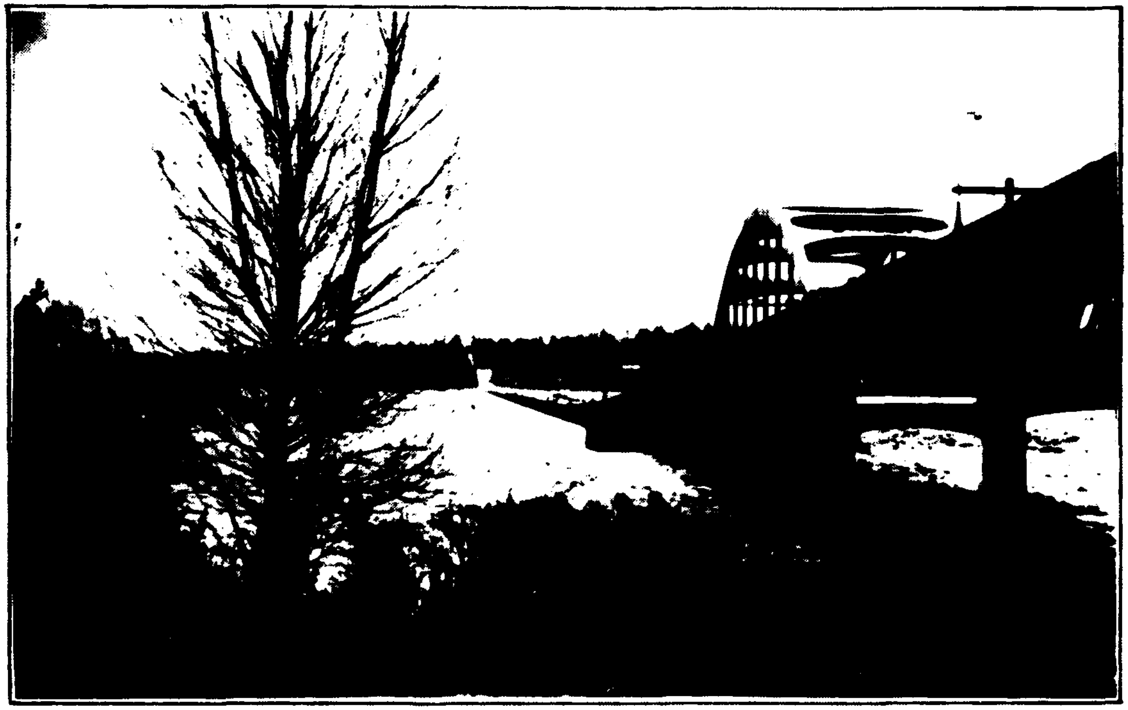
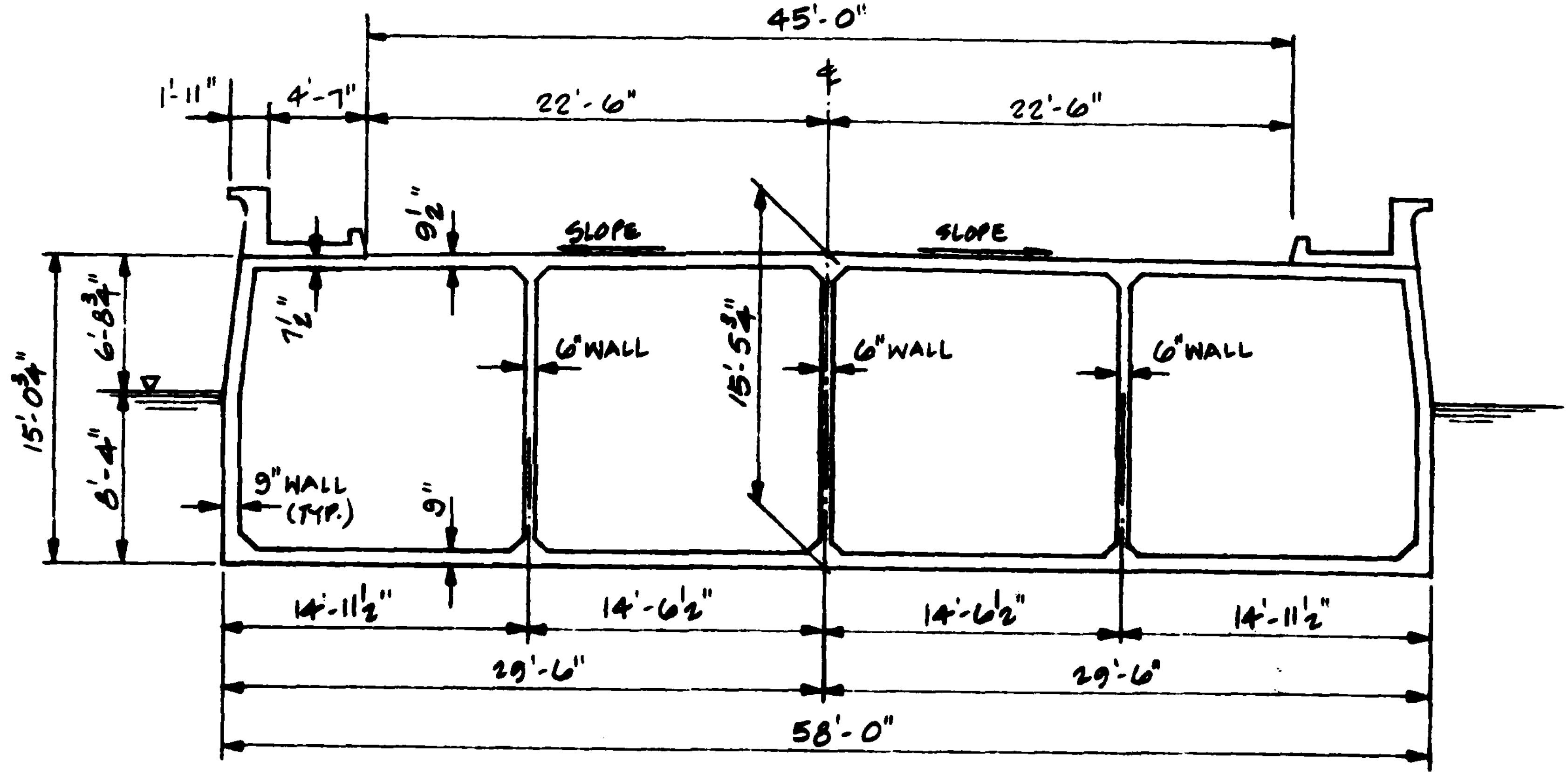


FIG. 2 SECOND LAKE WASHINGTON FLOATING BRIDGE BUILT IN 1962



CROSS SECTION

⅛" = 1'-0"

FIG. 3 CROSS-SECTION OF SECOND LAKE WASHINGTON FLOATING BRIDGE

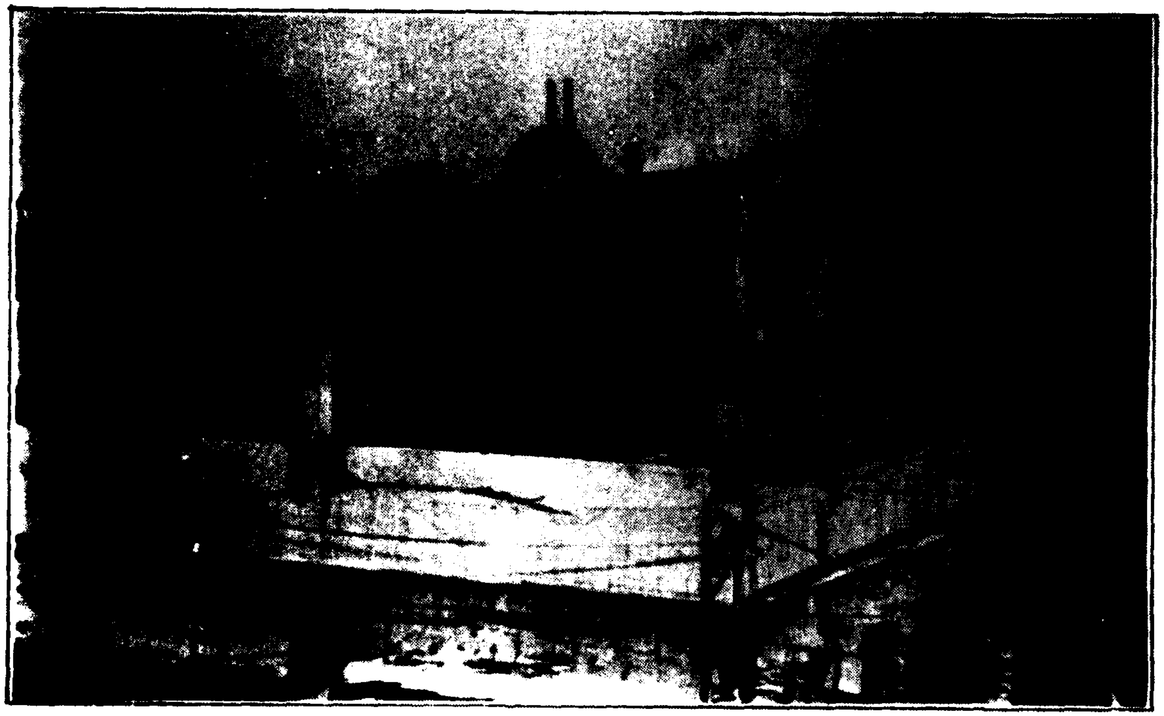


FIG. 4 SEGMENT OF ORIGINAL HOOD CANAL FLOATING BRIDGE BUILT IN 1957

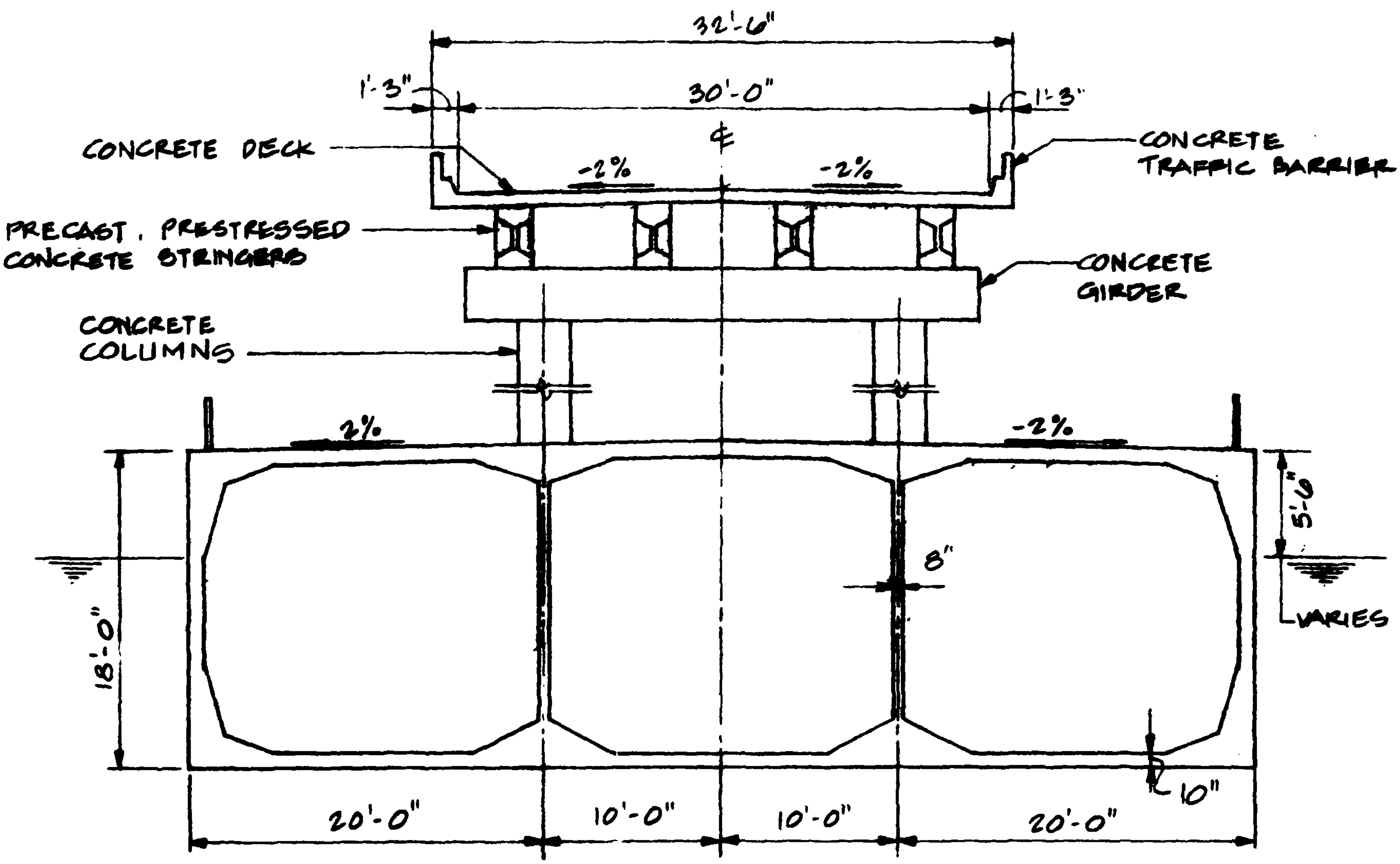


FIG. 5 CROSS-SECTION OF NEW HOOD CANAL FLOATING BRIDGE BEING BUILT IN 1982

FIG. 6 NEW HOOD CANAL FLOATING BRIDGE PONTOONS BEING CONSTRUCTED IN FLOOD BASIN

FIG. 7 PRECAST CHANNEL-SHAPED EXTERIOR WALL OF NEW HOOD CANAL FLOATING BRIDGE PONTOON

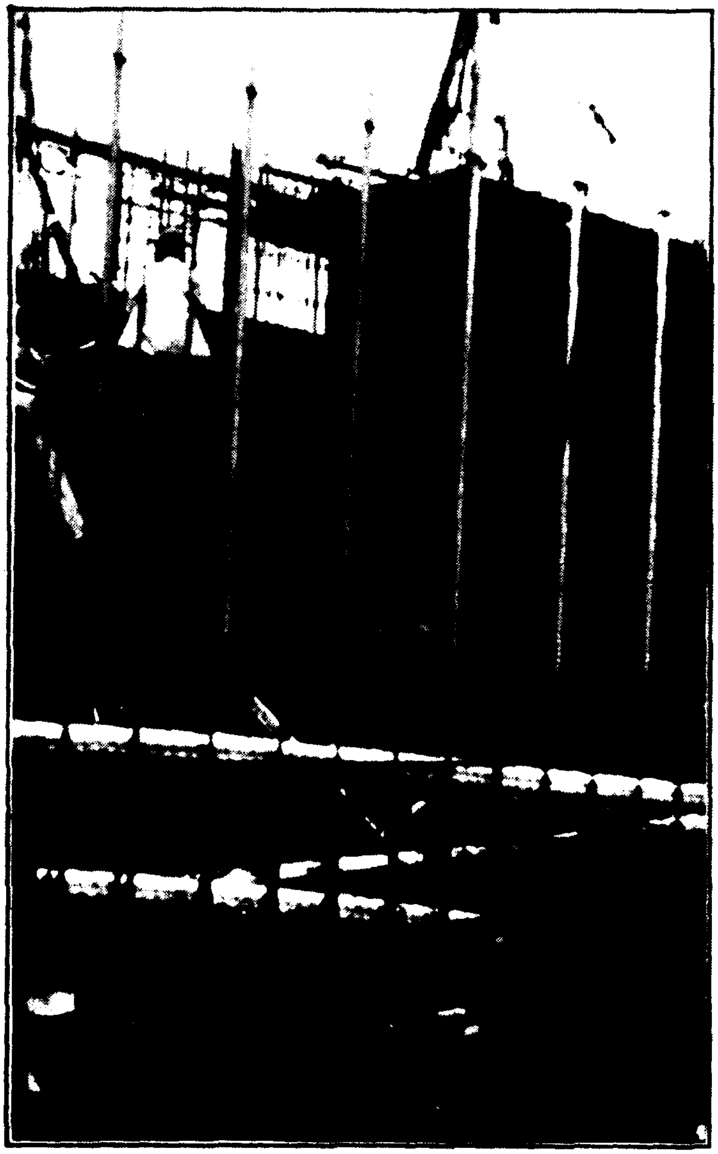


FIG. 8 PRECAST I - SHAPED INTERIOR WALL OF NEW HOOD CANAL FLOATING BRIDGE PONTOON

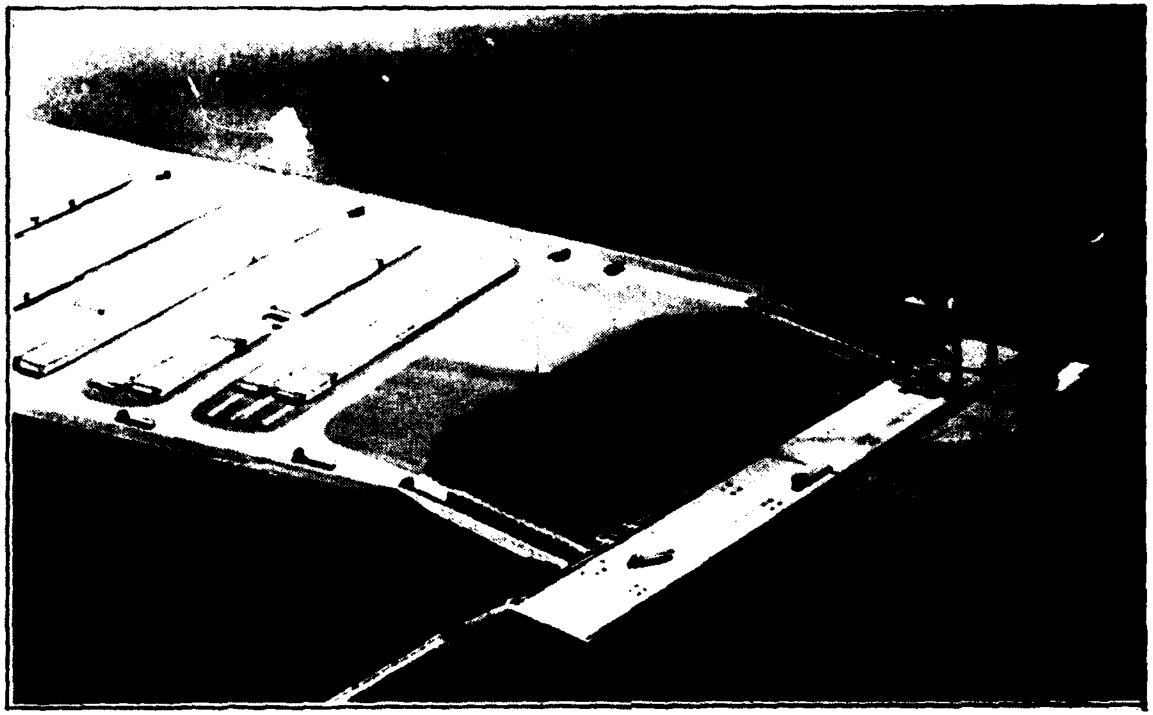


FIG. 9 RENDERING OF FLOATING CONTAINER TERMINAL FOR PORT OF VALDEZ, ALASKA

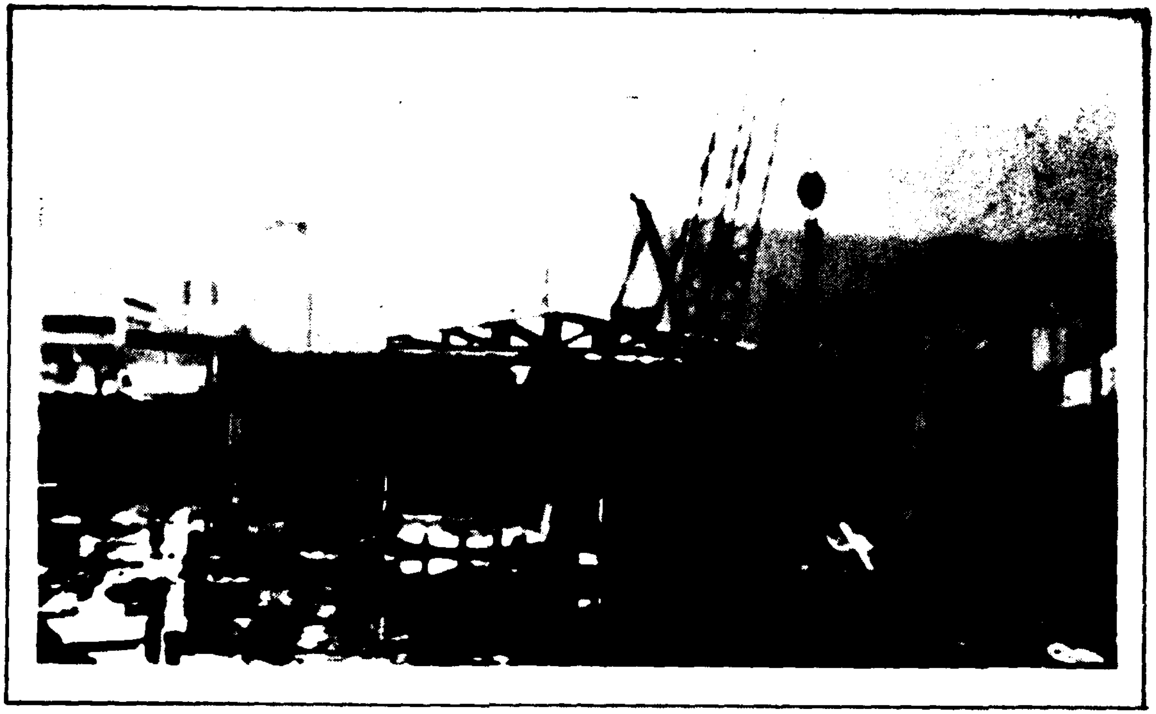


FIG. 10 FLOATING CONTAINER TERMINAL FOR VALDEZ UNDER CONSTRUCTION IN 1982 IN GRAVING DOCK LOCATED AT PUGET SOUND, WASHINGTON

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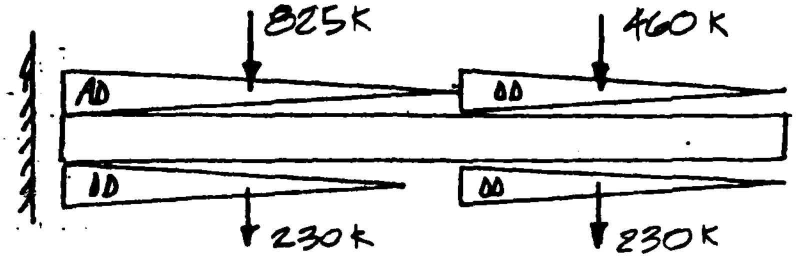
PROJECT:
ITEM: WIND LOADS
DESIGN:
DATE:
SHEET:
OF
REVISION:

SUMMARY WIND LOADING 90 MPH WINDS

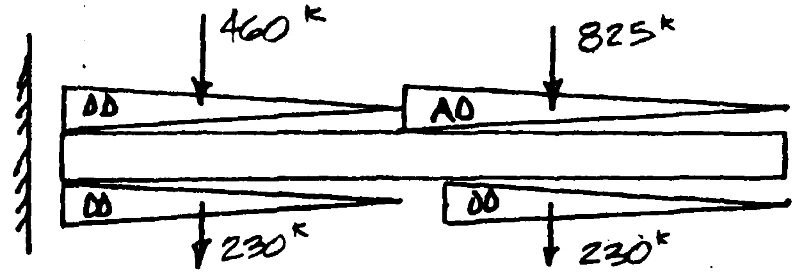
CASE I AD NEAR SHORE



NOTE: SECOND SHIP PICKS UP 50% OF LOAD.

TOTALS 1055K 690K

CASE II AD NEAR CHANNEL

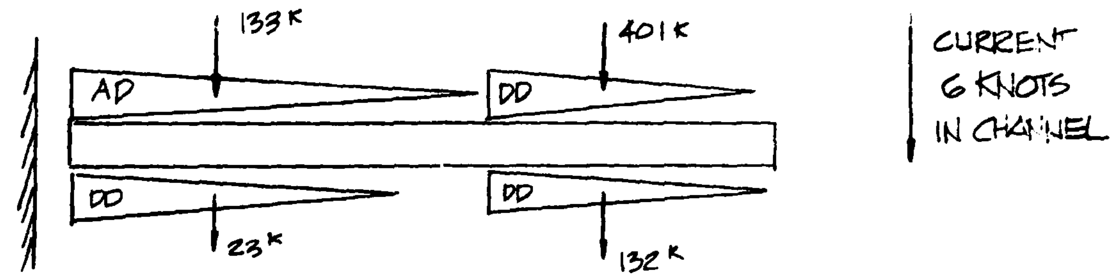


TOTALS 690K 1055K

FIG. 11 SUMMARY OF WIND LOADING

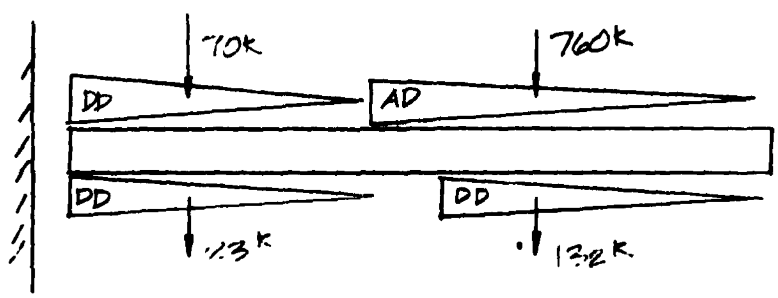
SUMMARY CURRENT LOADING

CASE I: AD NEAR SHORE



TOTALS 156 K 533 K

CASE II: AD NEAR CHANNEL



TOTALS 93 K 892 K

FIG. 12 SUMMARY OF CURRENT LOADING

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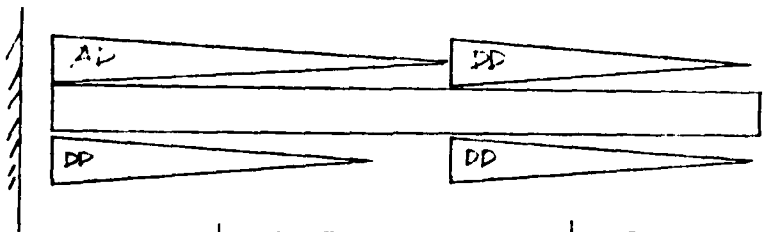
PROJECT:	SHEET:
ITEM: CURRENT & WIND LOADS	OF
DESIGN:	REVISION:
DATE:	

ENVIRONMENTAL LOADS COMBINED

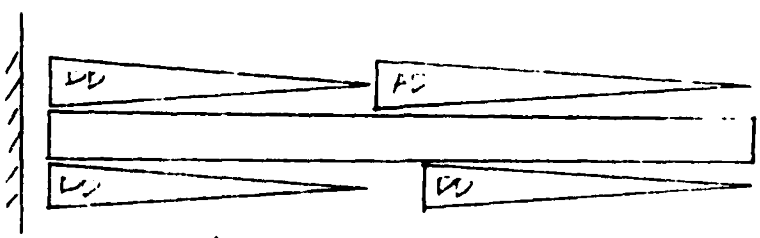
WIND = 90 MPH
CURRENT = 6 KNOTS IN CHANNEL

CASE I : AD NEAR SHORE



WIND	1055 K	690 K
CURRENT	156 K	533 K
TOTAL	1,211 K +	1,223 K = 2,434 K

CASE II : AD NEAR CHANNEL



WIND	690 K	1055 K
CURRENT	93 K	892 K
TOTAL	783 K +	1,947 K = 2,730 K

FIG. 13 SUMMARY OF WIND AND CURRENT LOADING COMBINED

FIG. 14 PROTRUSIONS FROM THE HULL OF A DD 963

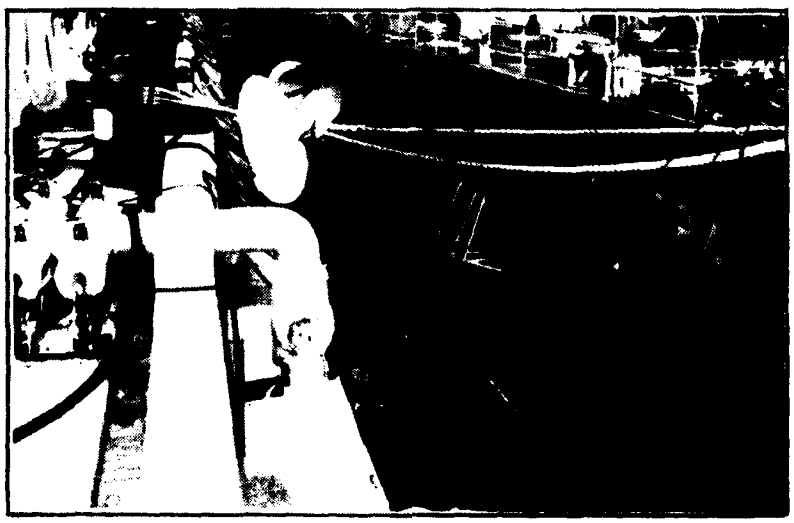


FIG. 15 PROPELLER GUARD PROTRUSION FROM THE HULL OF A DD 984

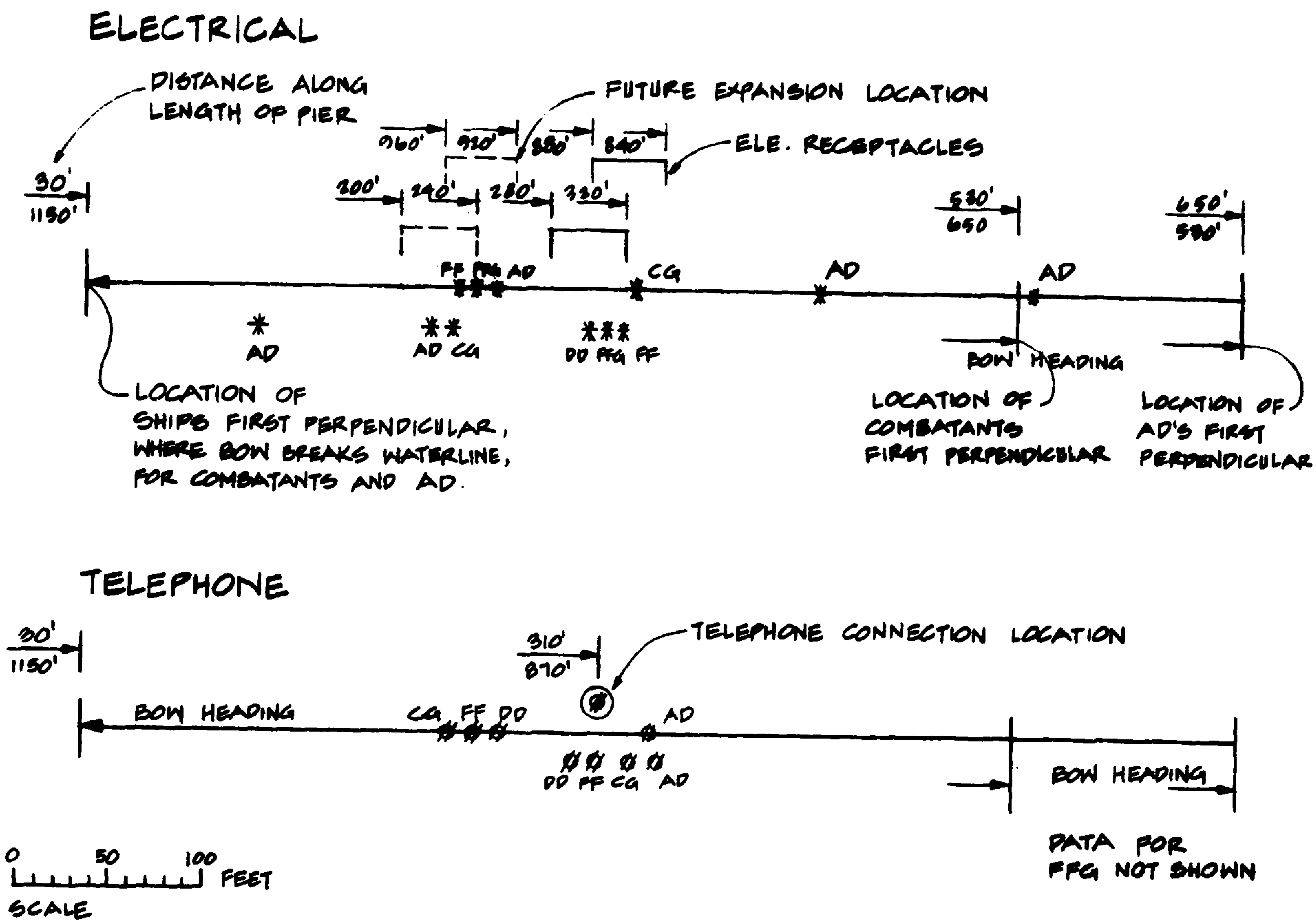


FIG. 16 SHIP UTILITY OUTLET LOCATIONS

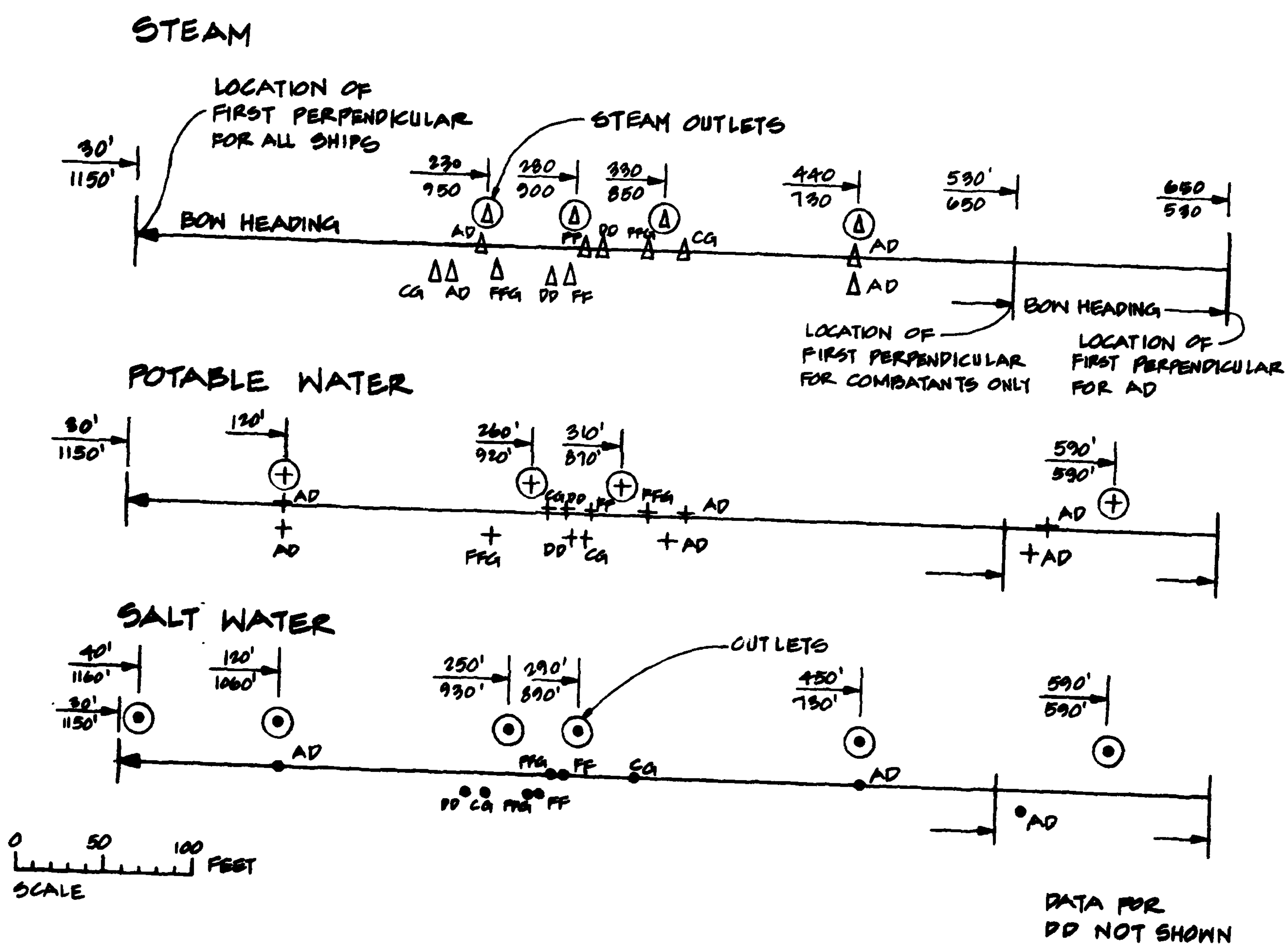


FIG. 17 SHIP UTILITY OUTLET LOCATIONS

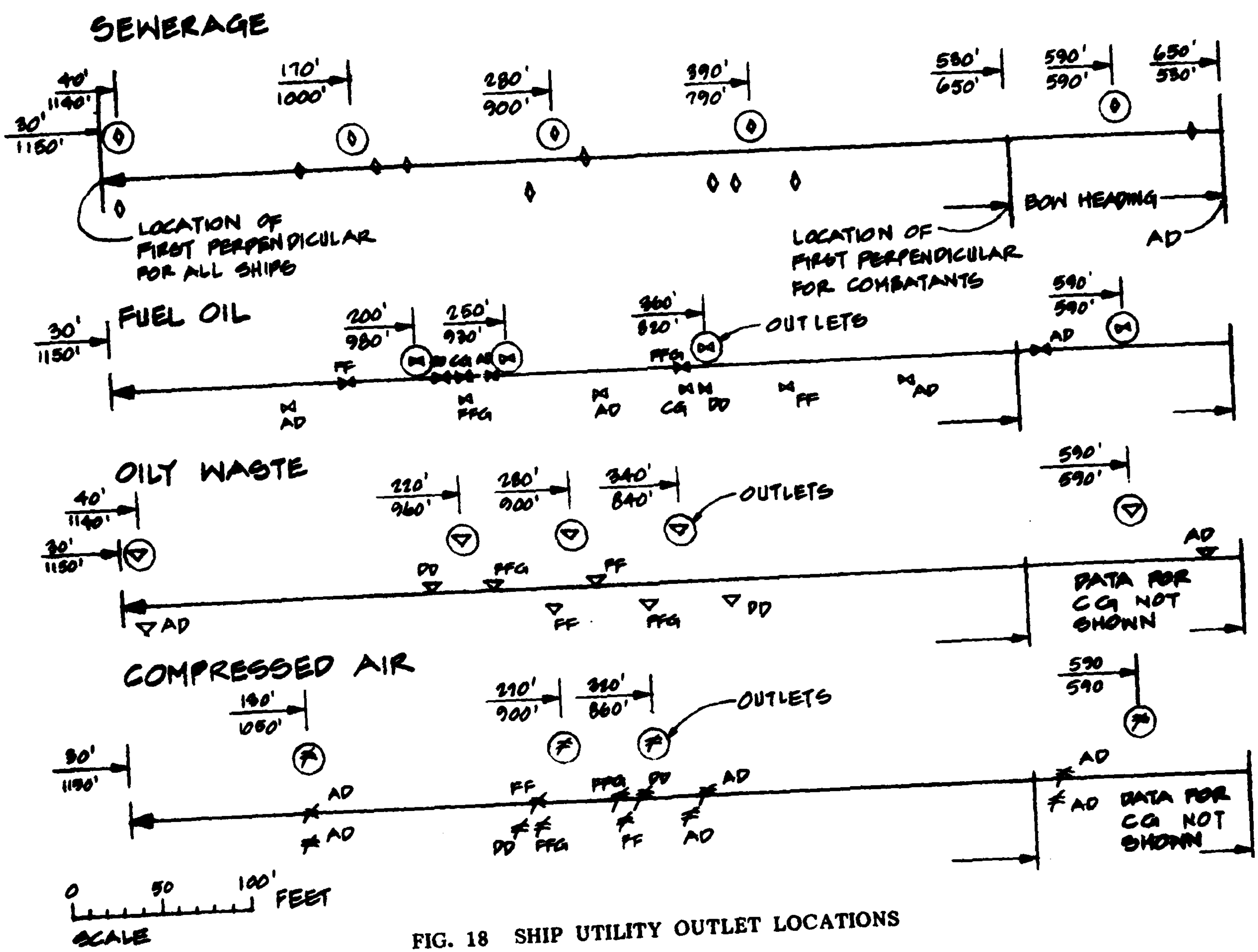


FIG. 18 SHIP UTILITY OUTLET LOCATIONS

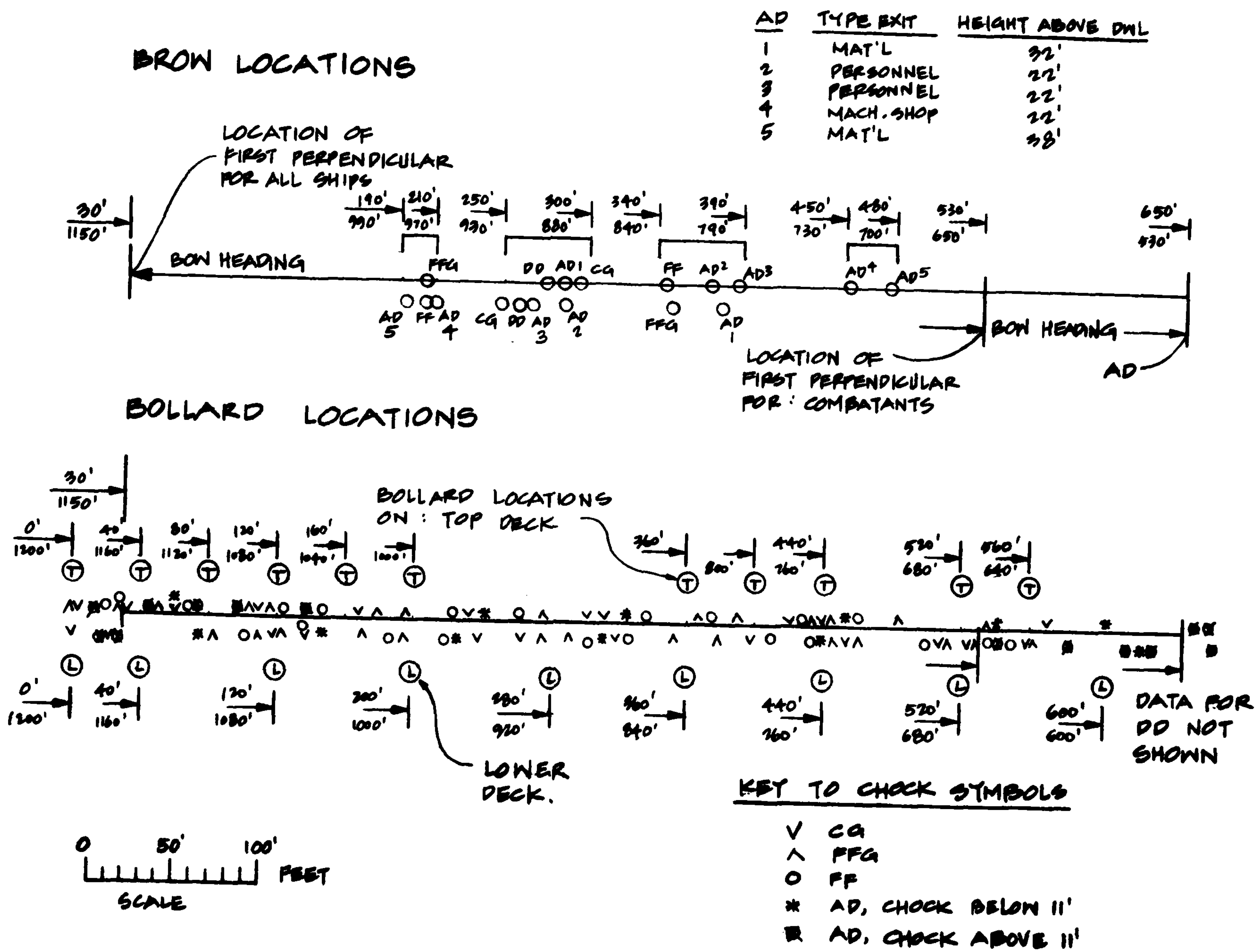


FIG. 19 SHIP BROW AND BOLLARD LOCATIONS

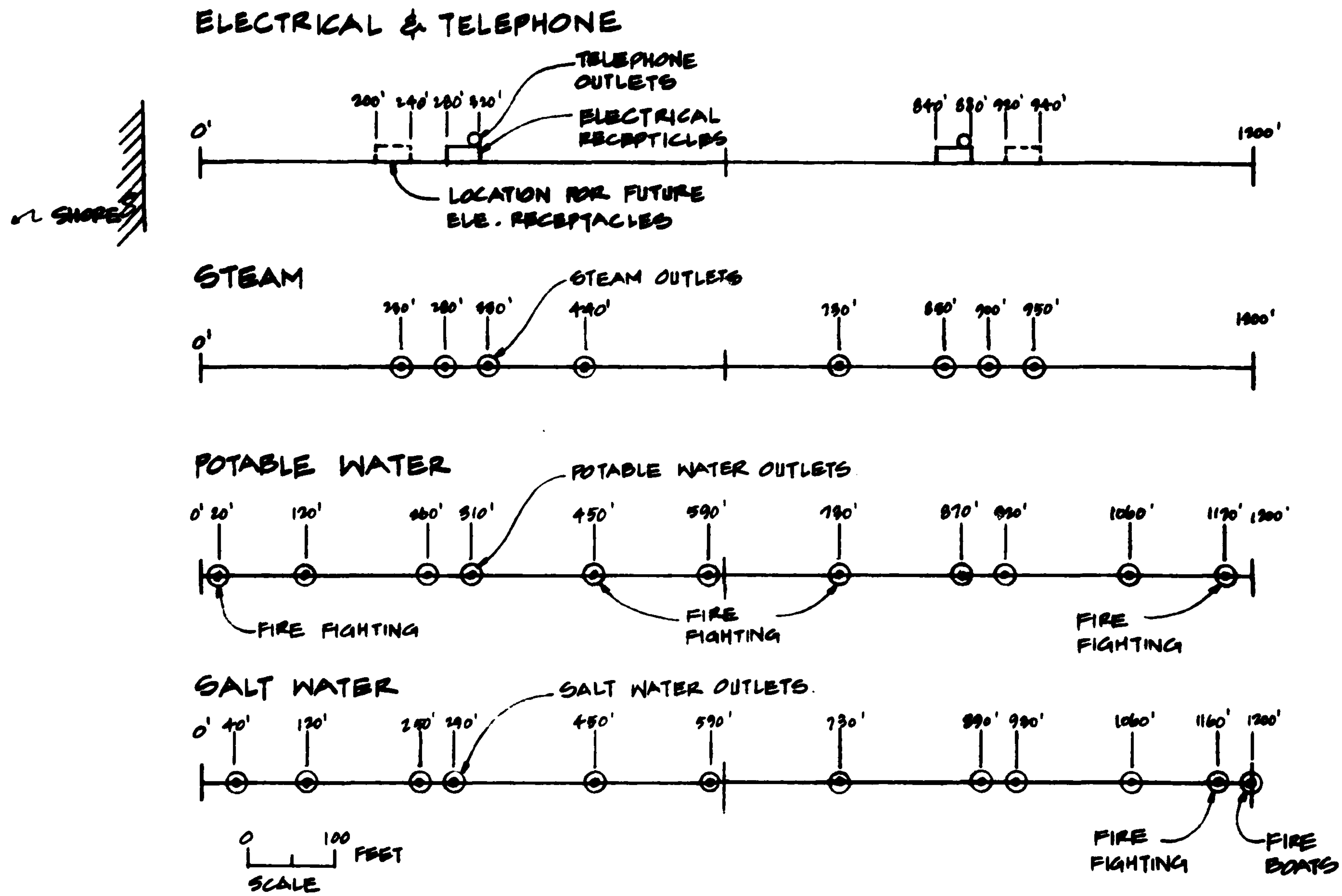


FIG. 20 PIER UTILITY OUTLET LOCATIONS

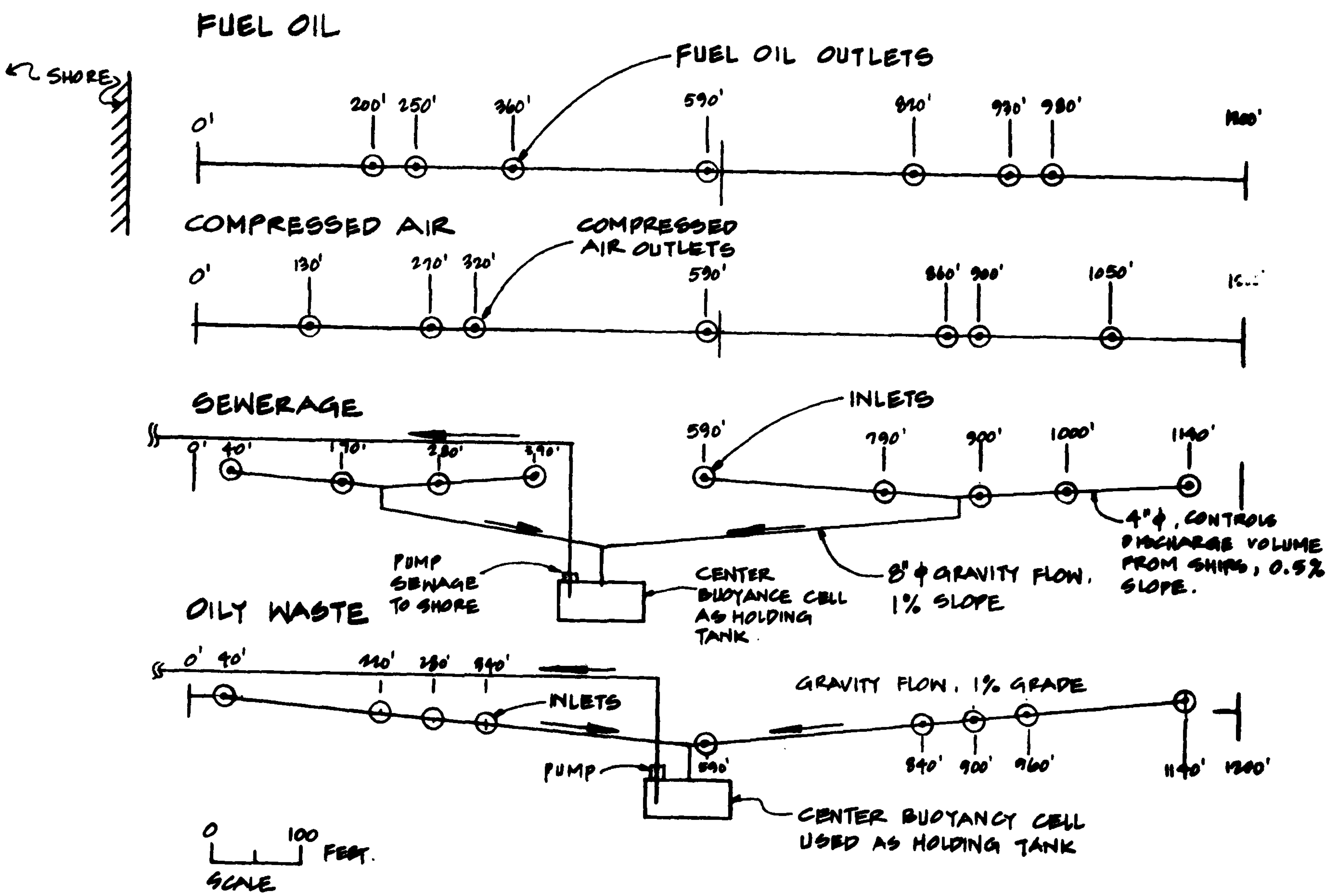


FIG. 21 PIER UTILITY OUTLET LOCATIONS

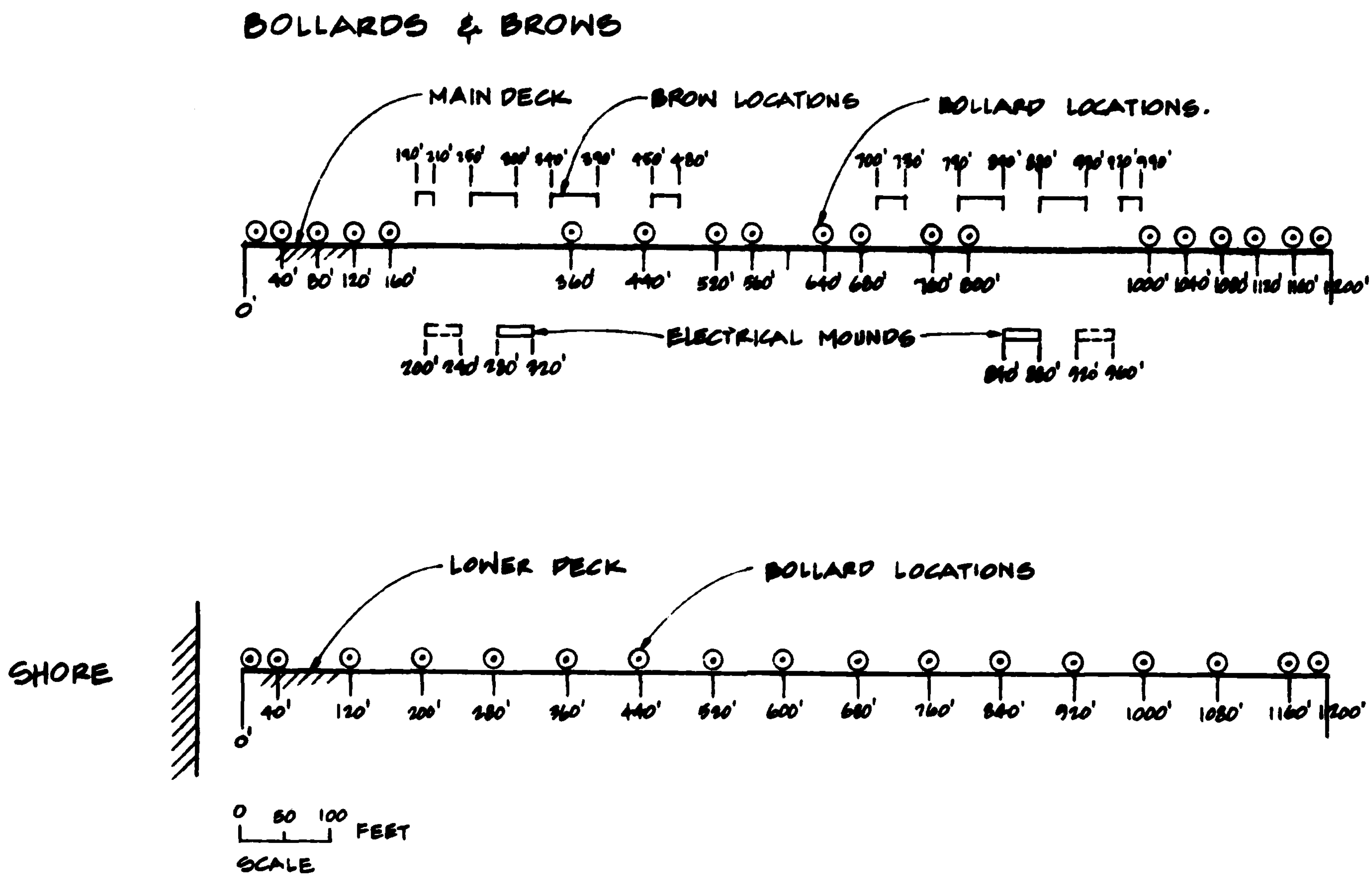


FIG. 22 PIER BOLLARD AND BROW LOCATIONS

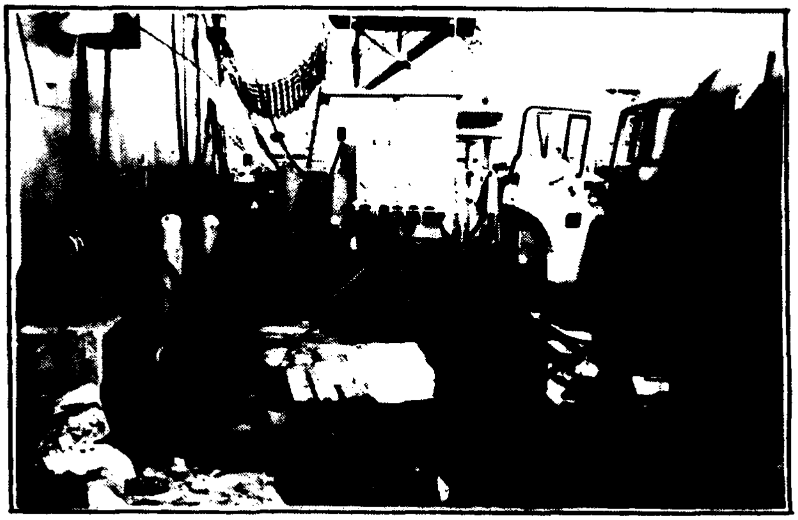


FIG. 23 JUMBLE OF ELECTRICAL CABLES MIXED WITH OTHER HOSES.

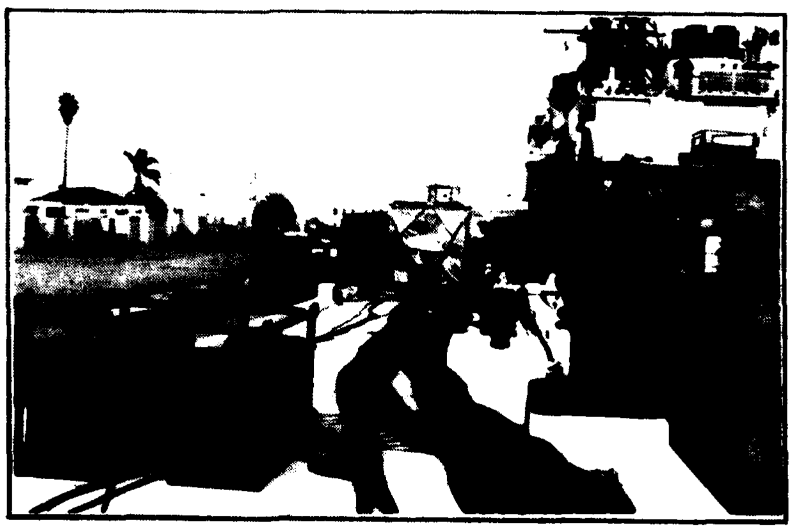
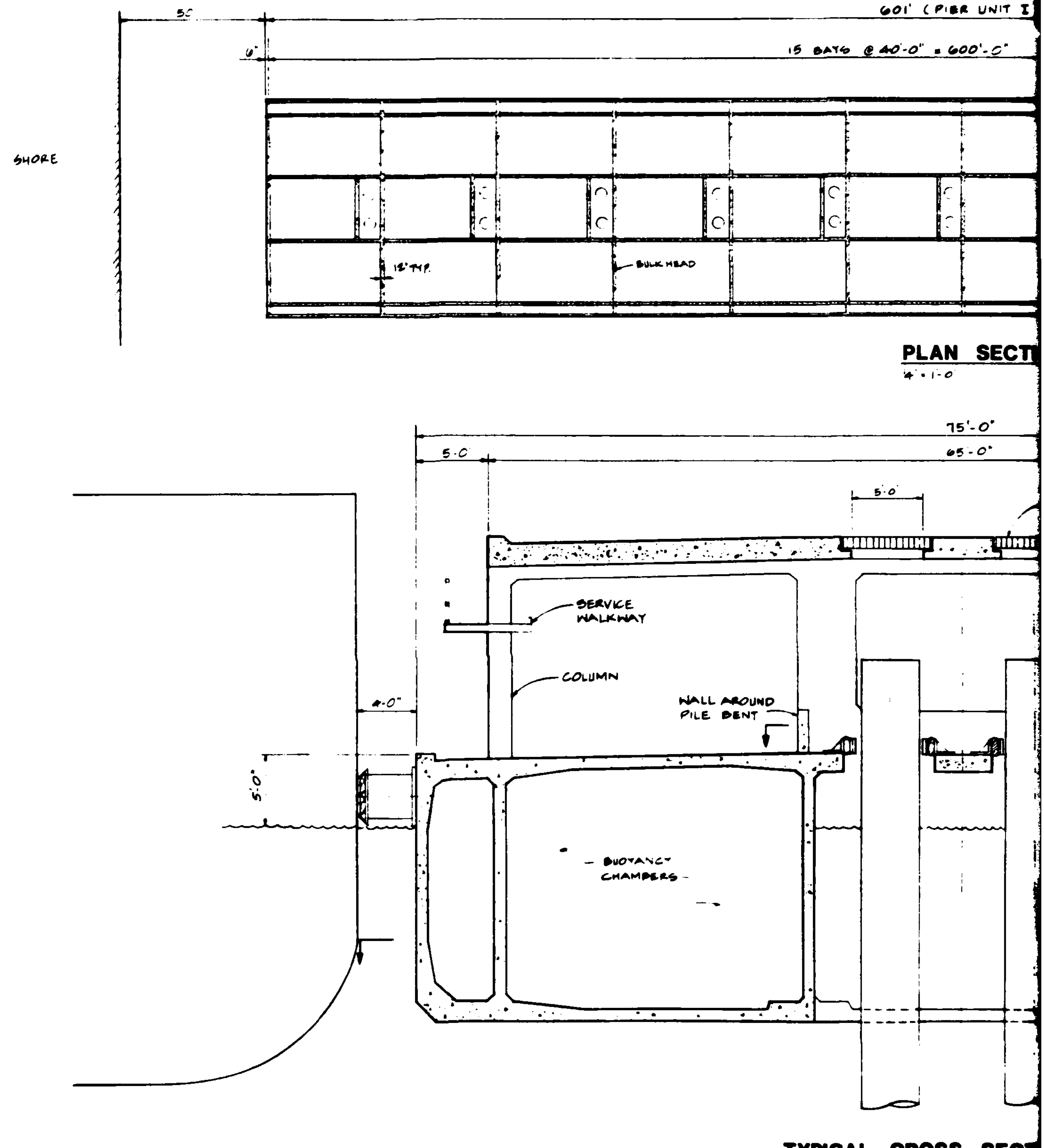


FIG. 24 NEAT CABLE LAYOUT AT PIER 2, SAN DIEGO



PLAN SECT

TYPICAL CROSS SECT

PROJECT
DESIGN
DRAWN
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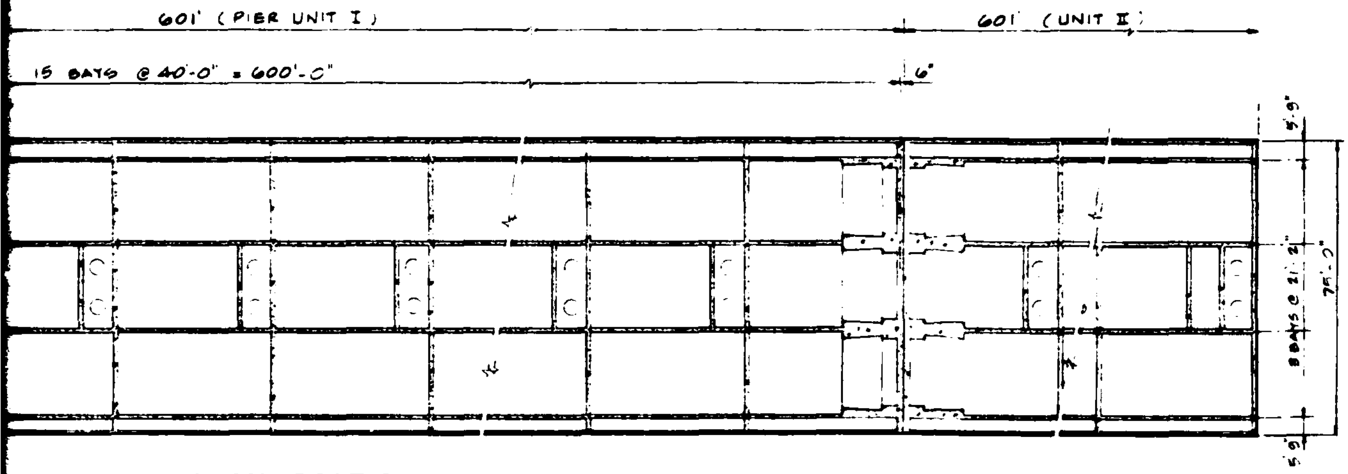
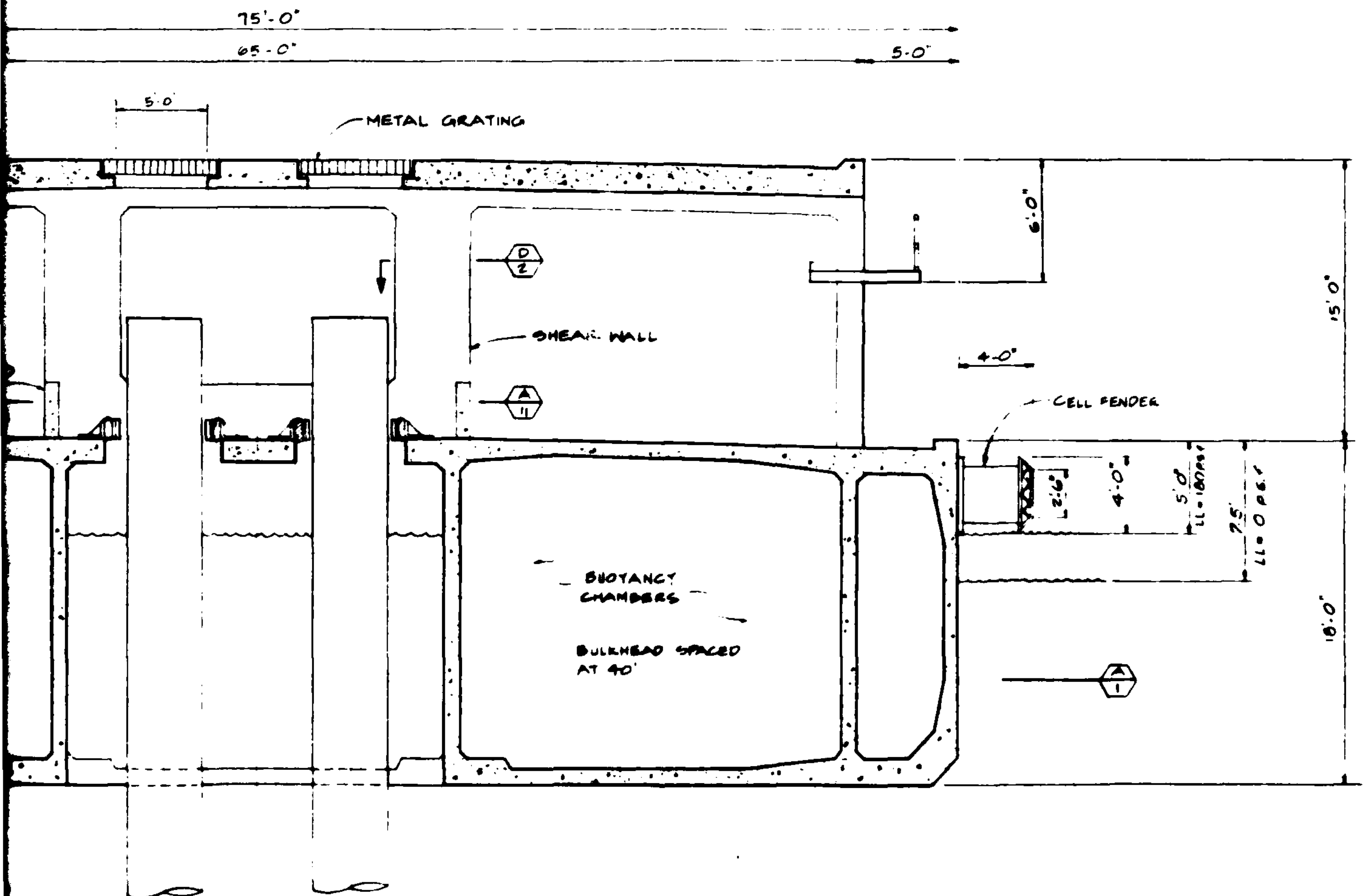
100%



PLAN SECTION



TYPICAL CROSS SECTION AT PILE BENT

NO	REVISION	DATE

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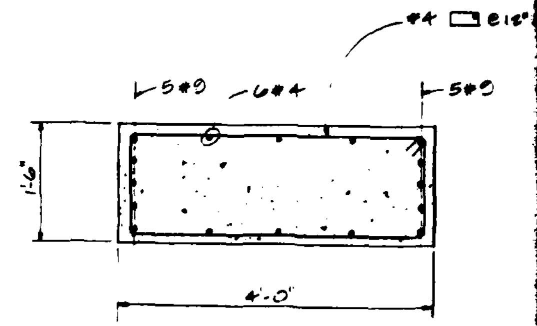
100%	APR 82	

SHEET TITLE
TYPICAL CROSS SECTION AT PILE

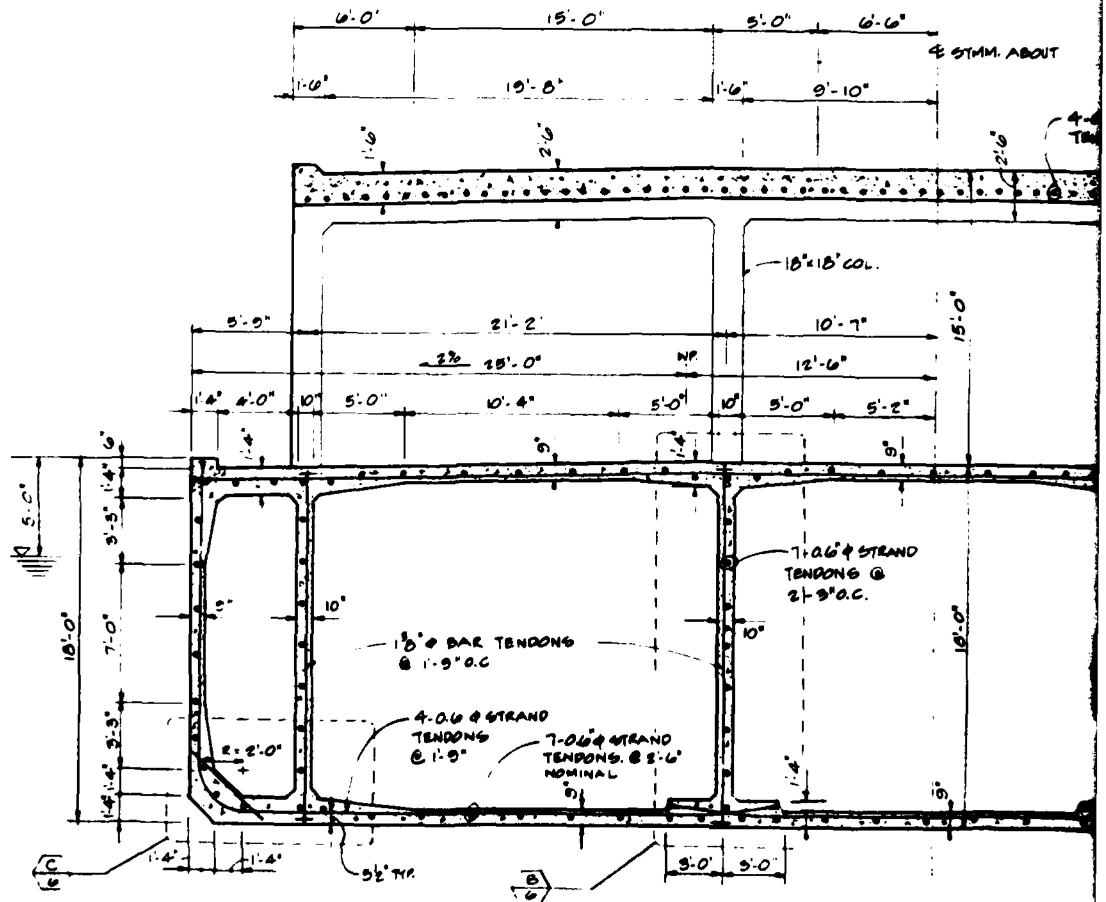
PROJECT
NAVY PIER DESIGN CONCEPTS

SHEET N° DRAWING
1

2



SHEAR WALL (D/2)
1" = 1'-0"



TYPICAL CROSS SECT
¼" = 1'-0"

DESIGN
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PROJECT N

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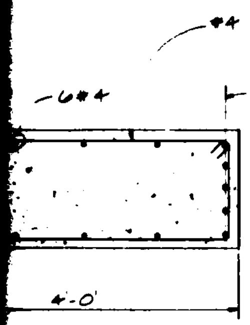
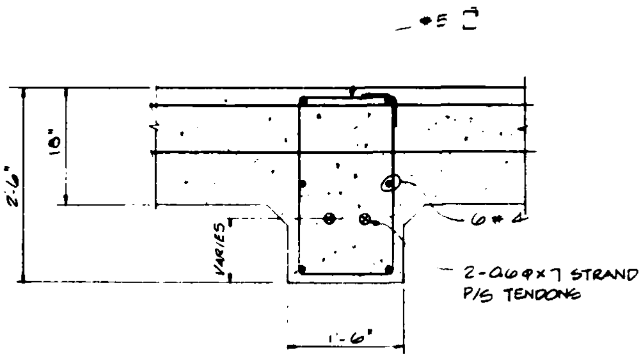
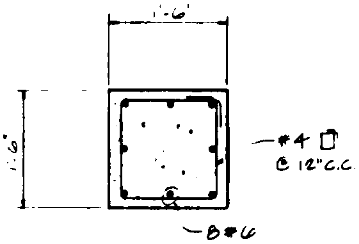


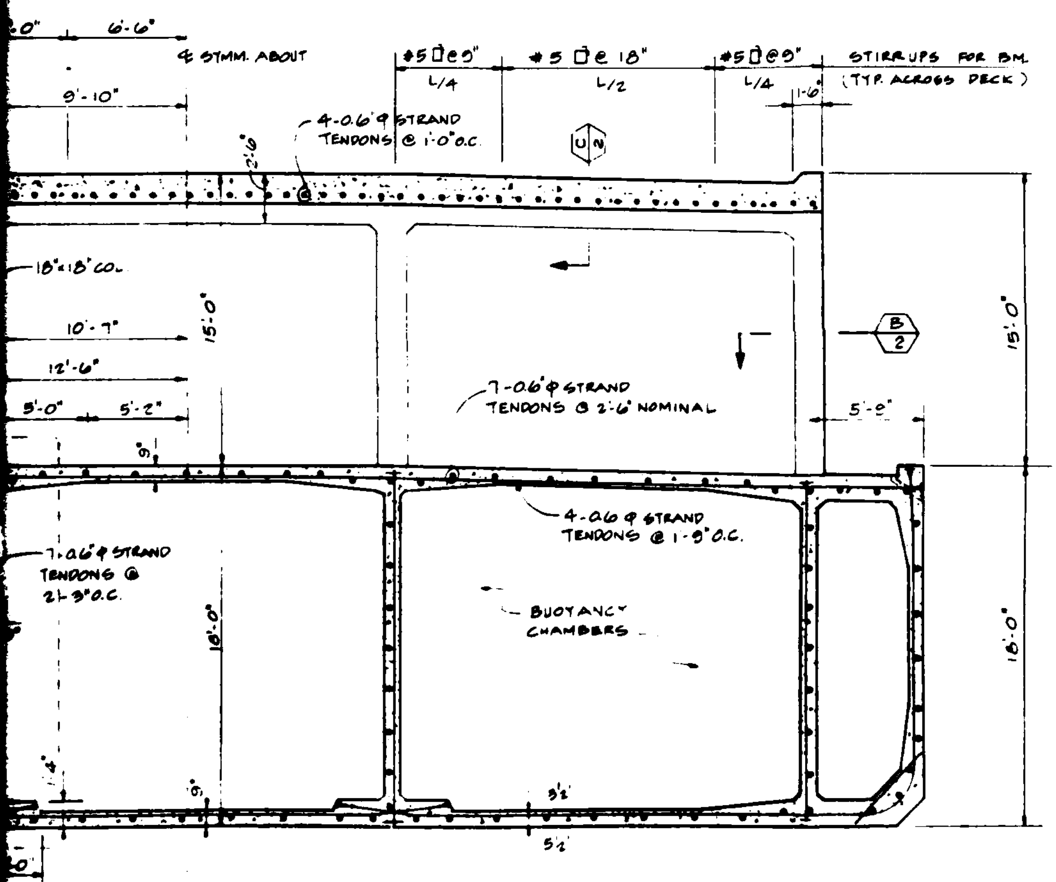
HEAR WALL (D/2)
=1'-0"



BEAM (C/2)
1"=1'-0"



COLUMN (B/2)
=1'-0'



NOTE :

REBARS NOT SHOWN

TYPICAL CROSS SECTION (A/2)
¼" = 1'-0"

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Issued For	Date	By
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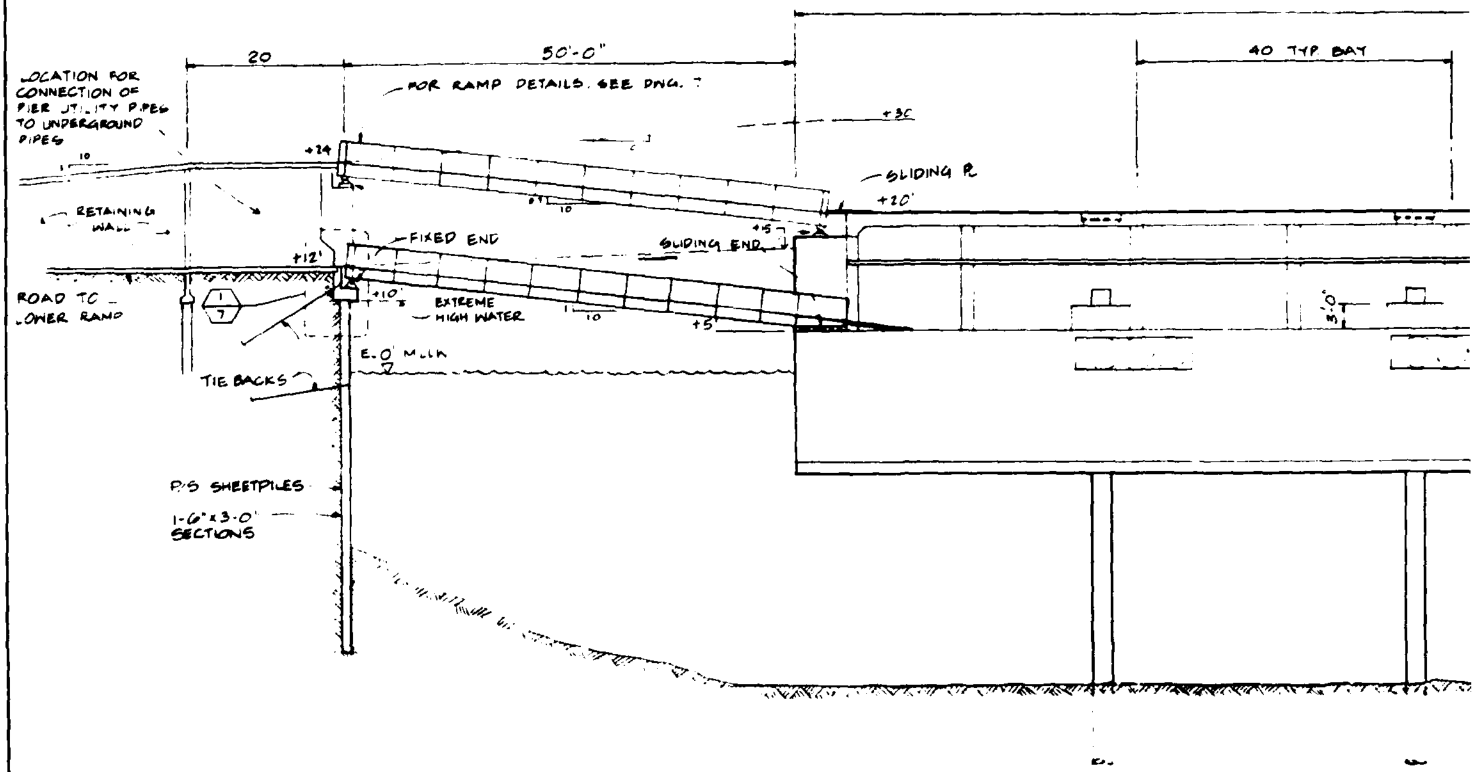
SHEET TITLE: TYPICAL PLAN & CROSS SECTION

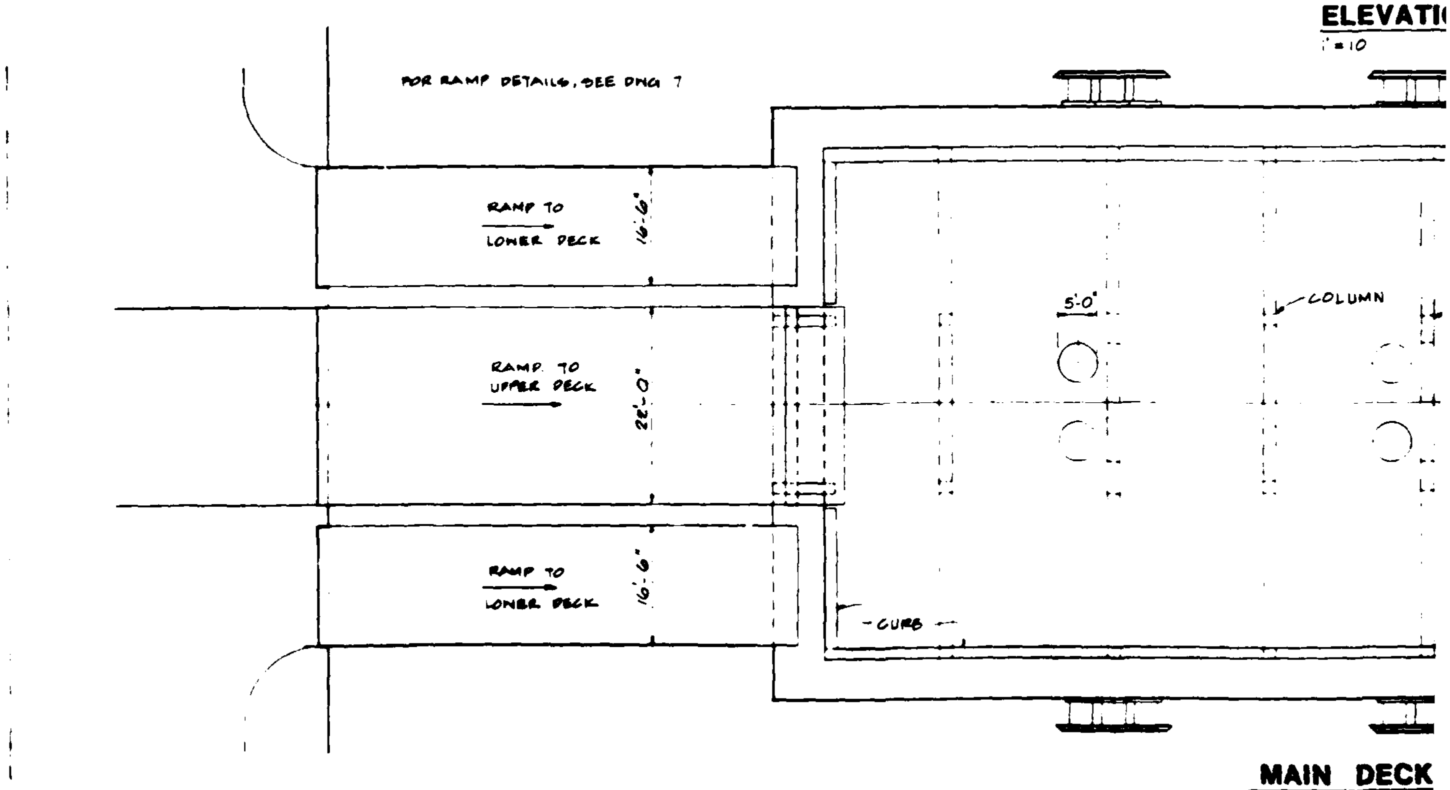
PROJECT: NAVY PIER DESIGN CONCEPTS

SHEET Nº DRAWING: 2



ELEVATI

1" = 10



MAIN DECK

1" = 10'

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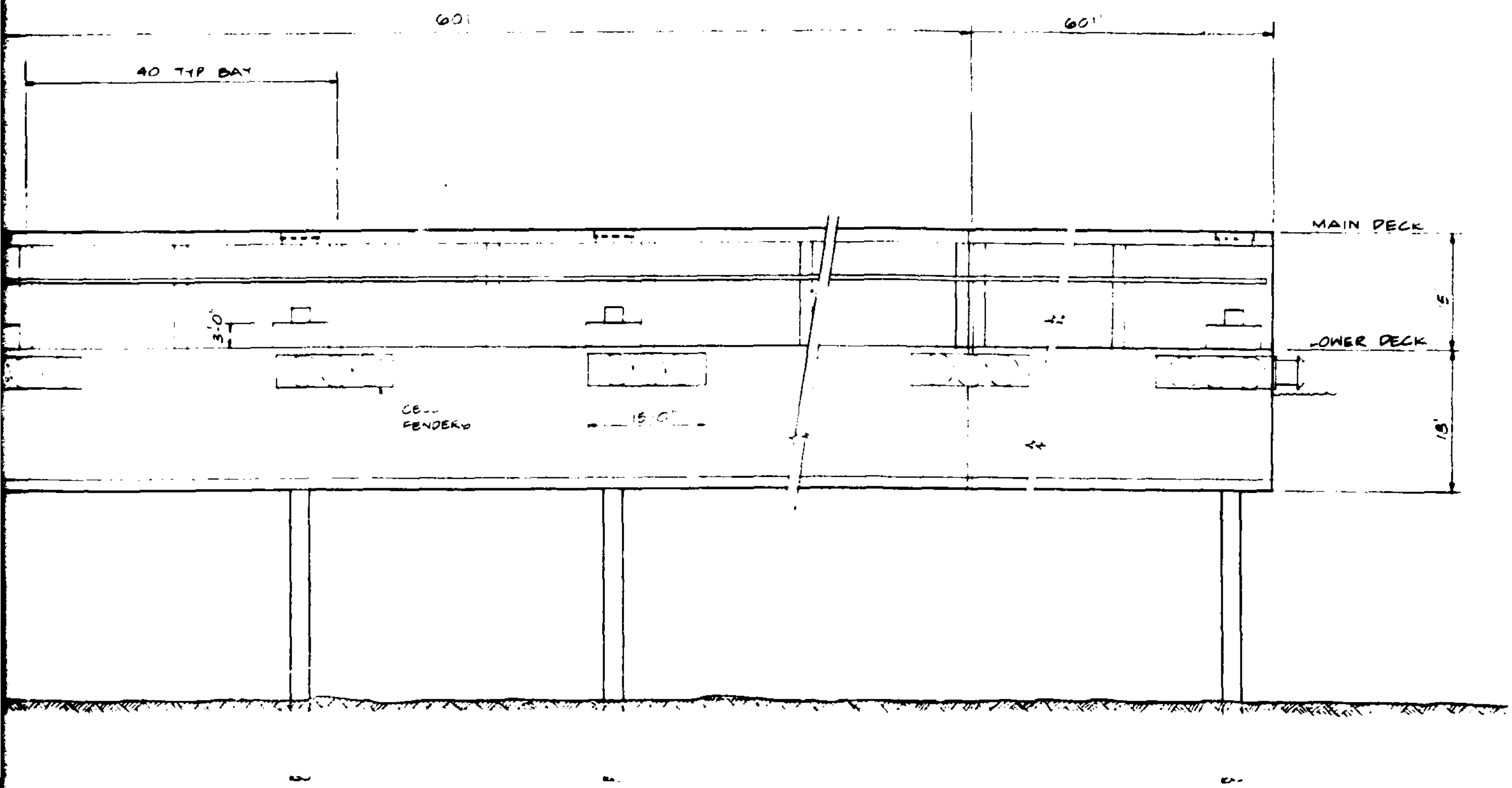
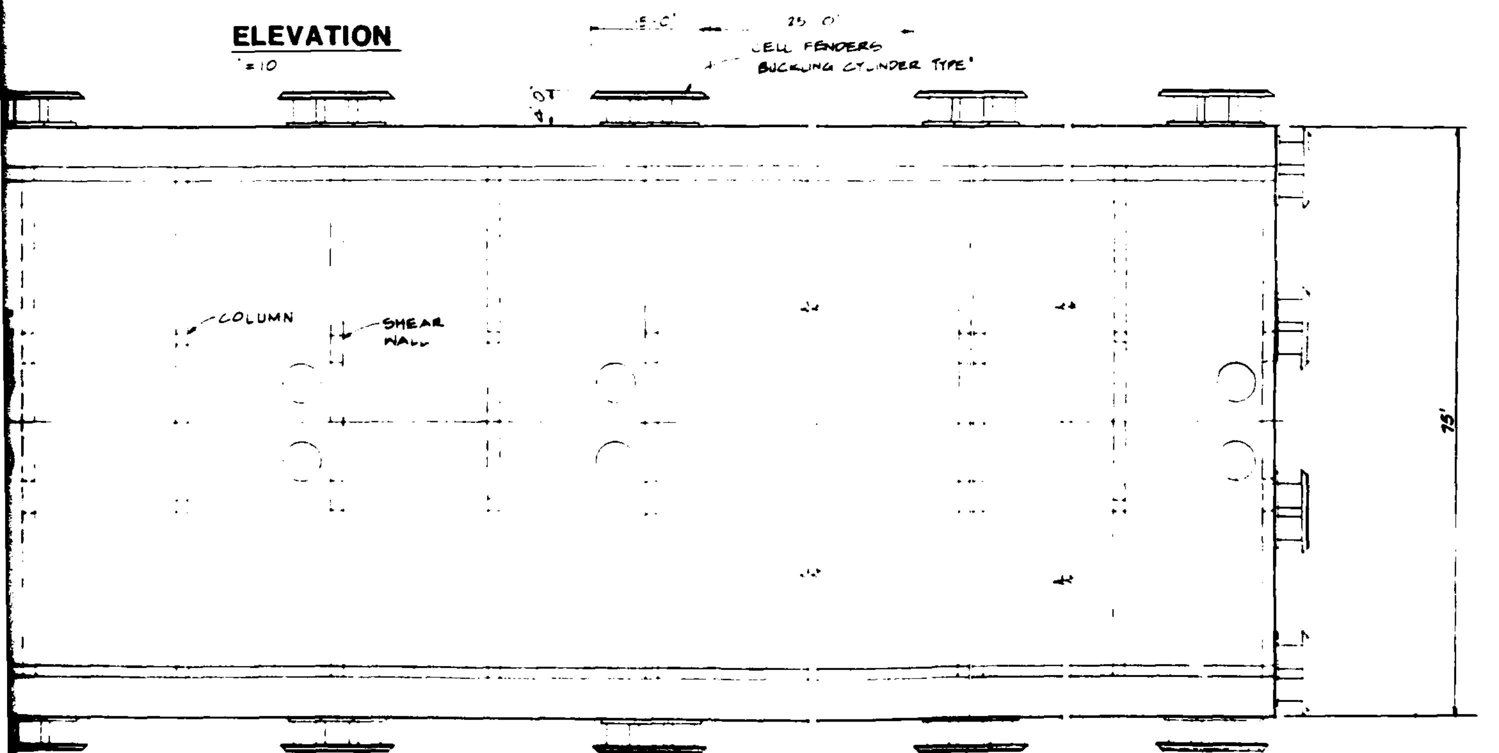
100 %



ELEVATION

1"=10'



MAIN DECK PLAN

1"=10'

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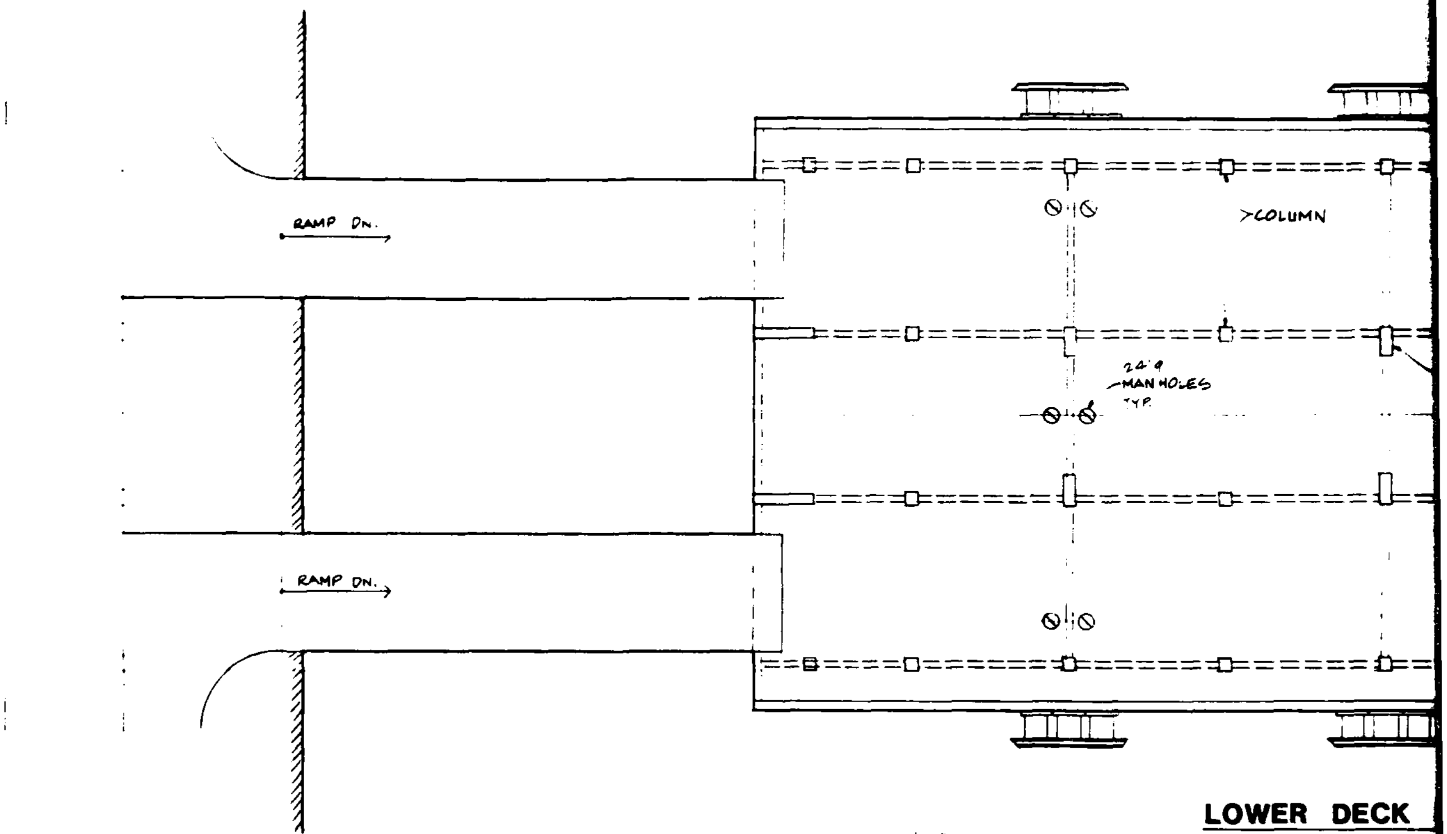
SHEET TITLE: ELEVATION & MAIN DECK PLAN

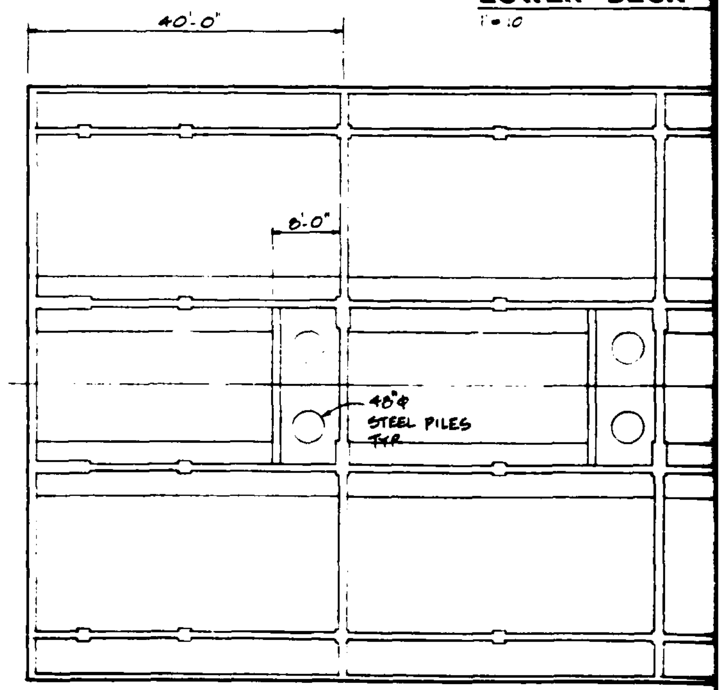
PROJECT: NAVY PIER DESIGN CONCEPTS

SHEET Nº DRAWING: 3



LOWER DECK

1" = 10'



PONTOON FLOOR

1" = 10'

PROJECT N
DESIGN
DRAWN
CHECKED

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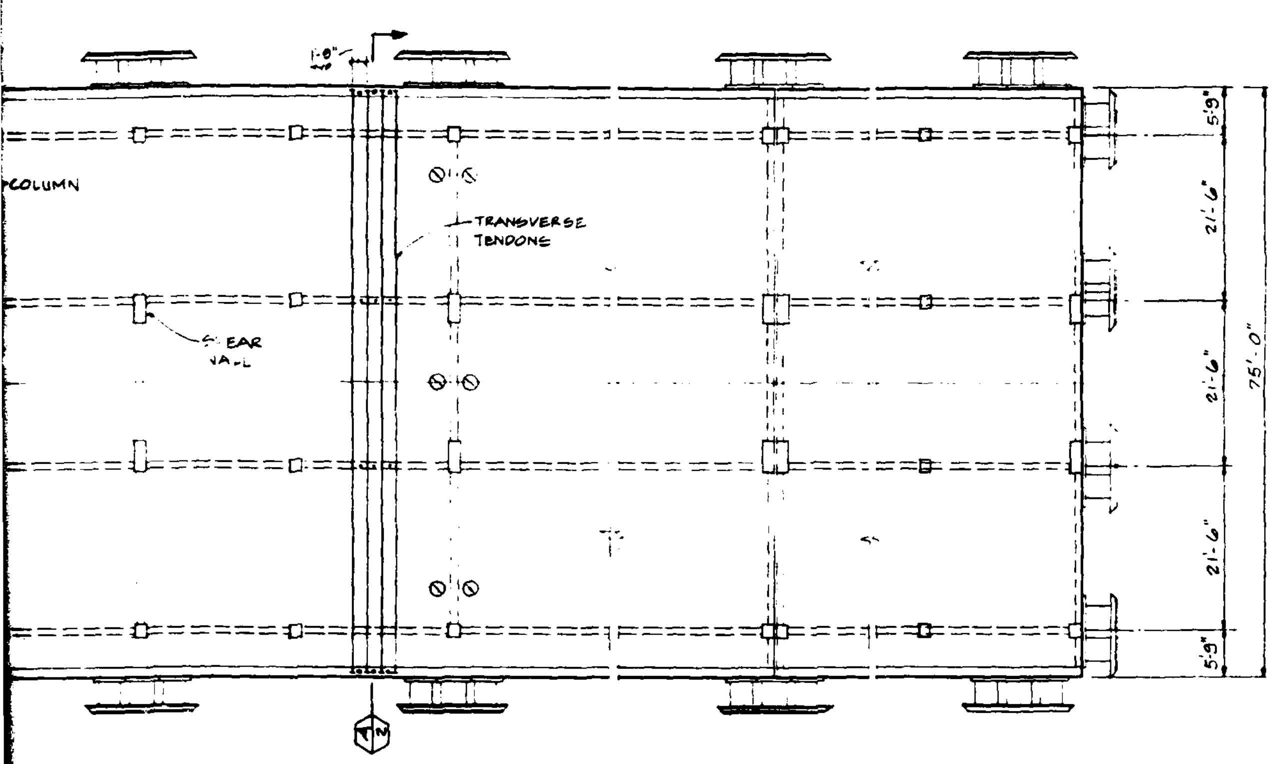
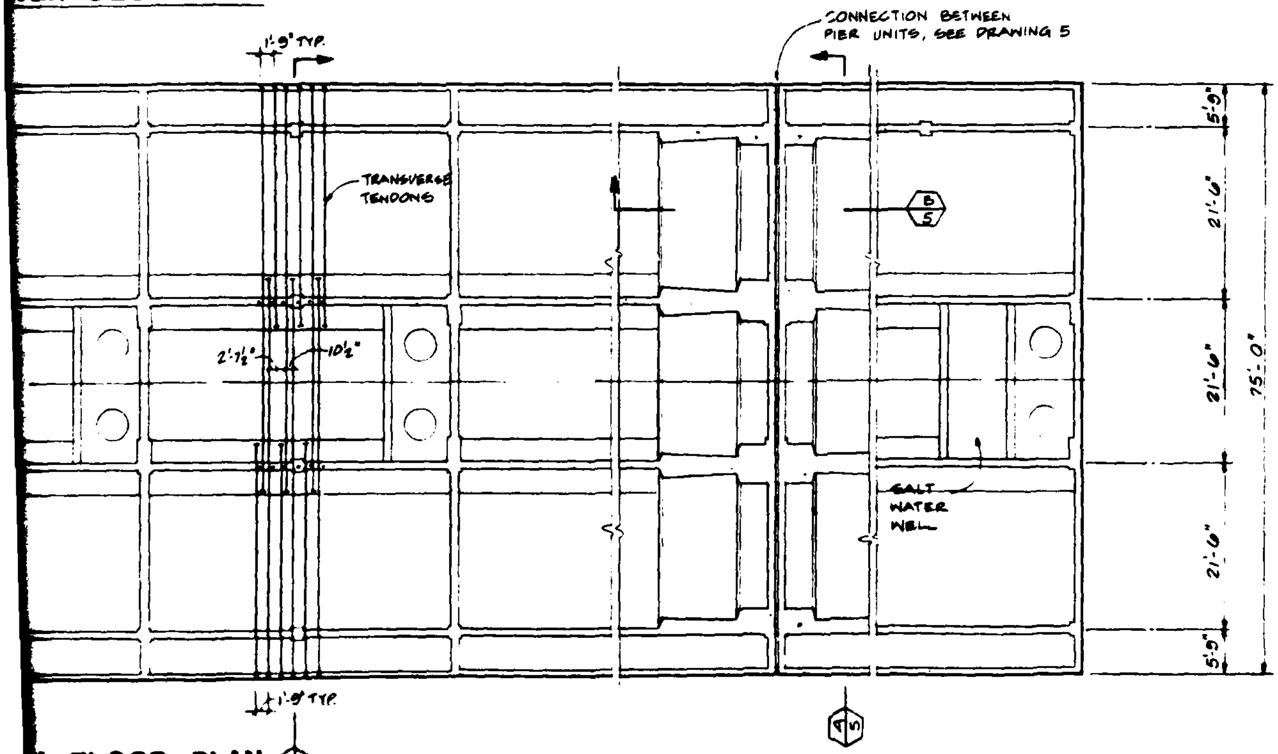


WER DECK PLAN



FLOOR PLAN

NO	REVISION	DATE

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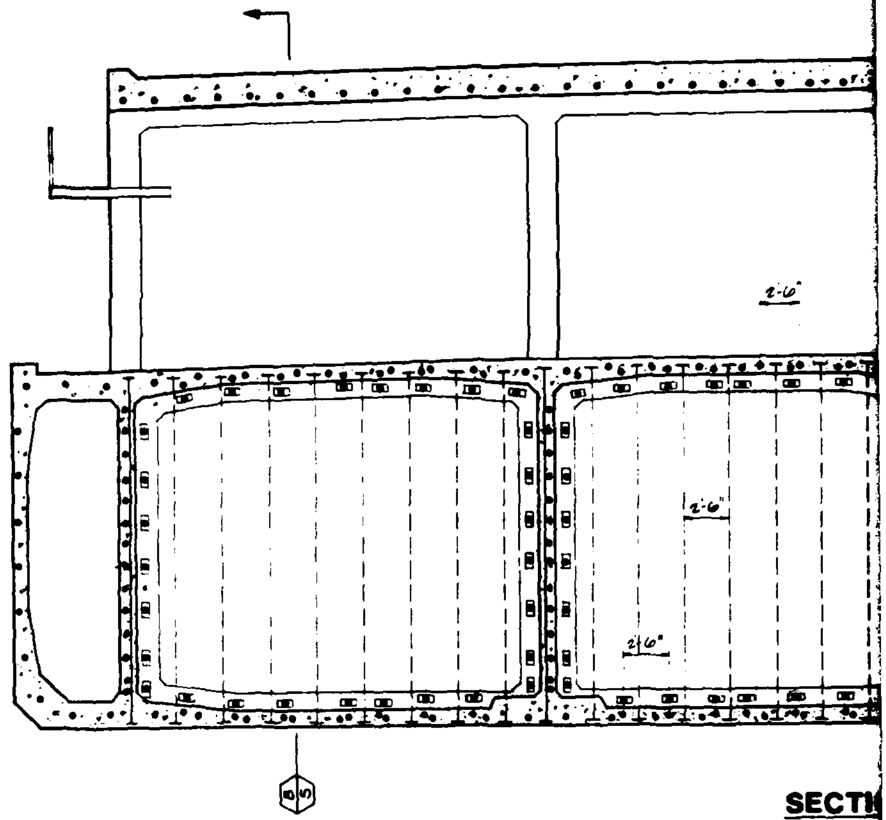
SHEET TITLE: LOWER DECK & PONTOON FLOOR PLAN

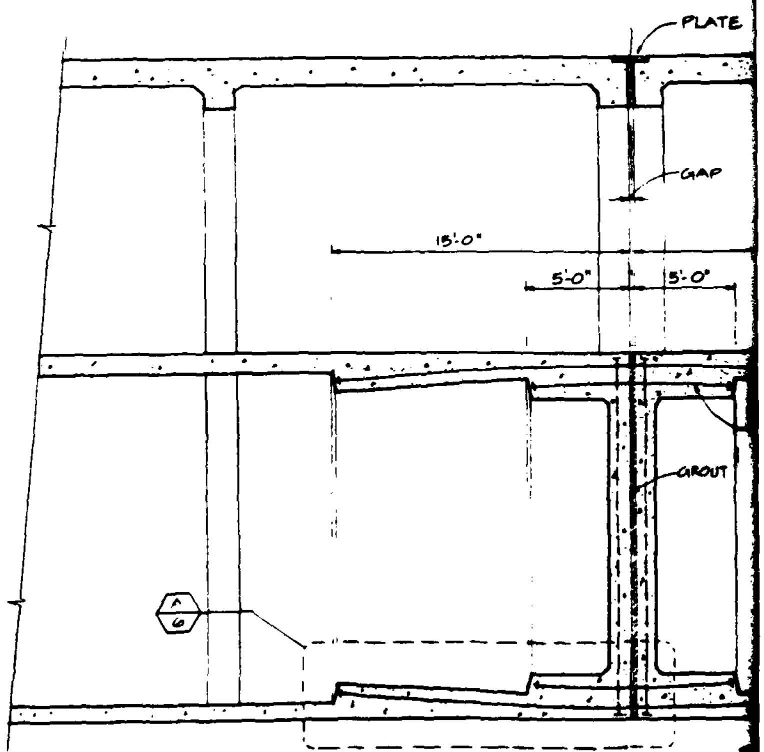
PROJECT: NAVY PIER DESIGN CONCEPTS

SHEET No DRAWING: 4



SECTI

1/4"=1'-0"



CONNECT

1/4"=1'-0"

DESIGN ____ DRAWN ____ CHECKED ____

PROJECT No

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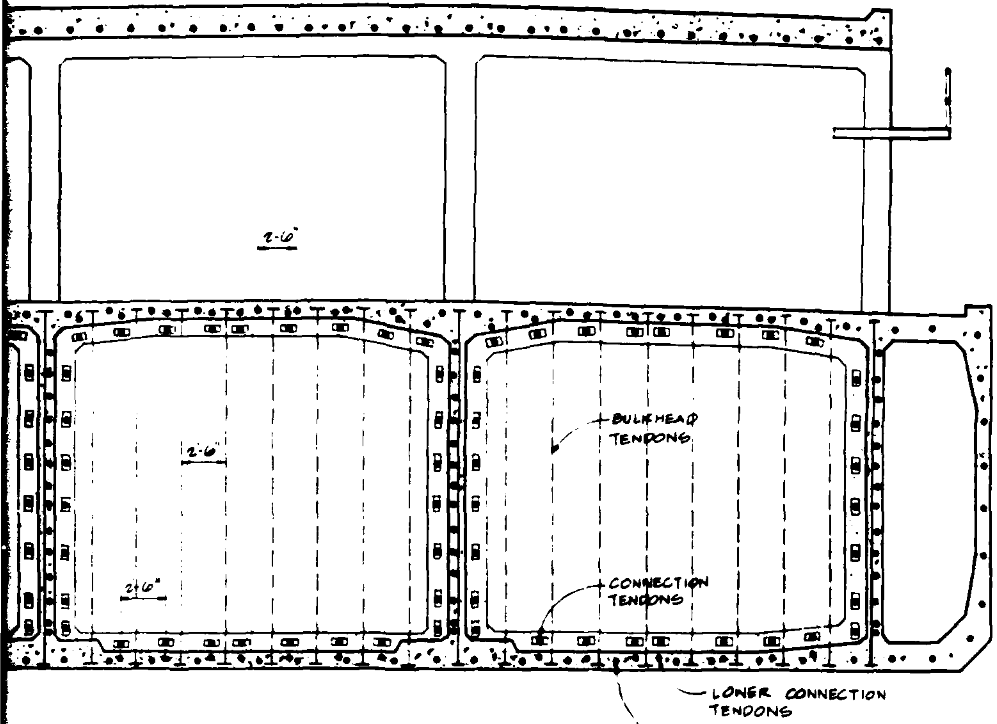
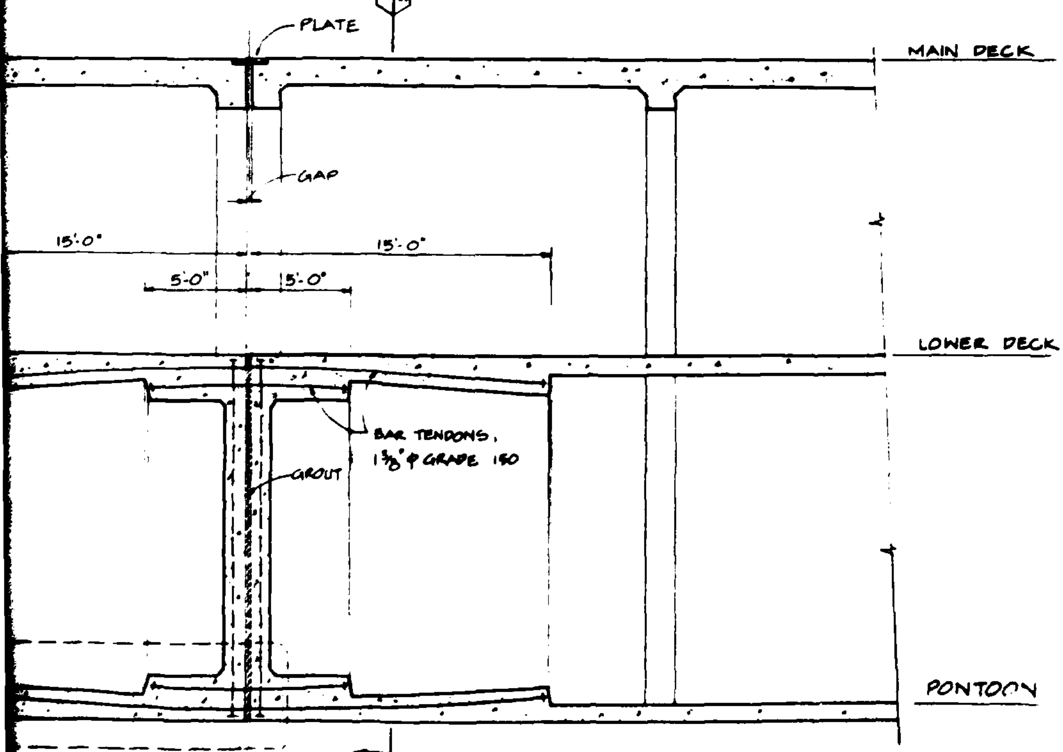


SECTION A/5
1/4" = 1'-0"



CONNECTION - SECTION B/5
1/4" = 1'-0

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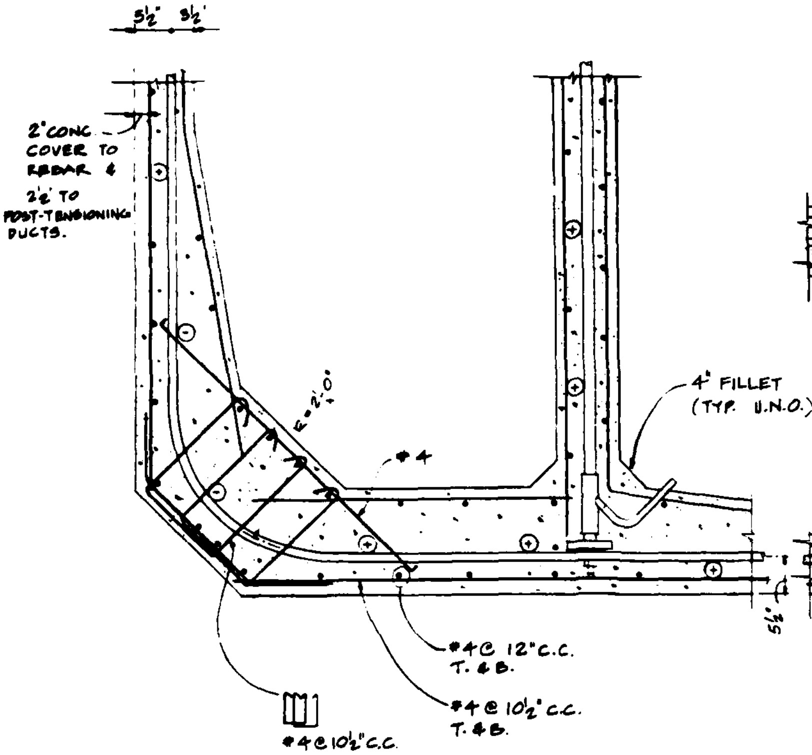
100%	APR. 82	

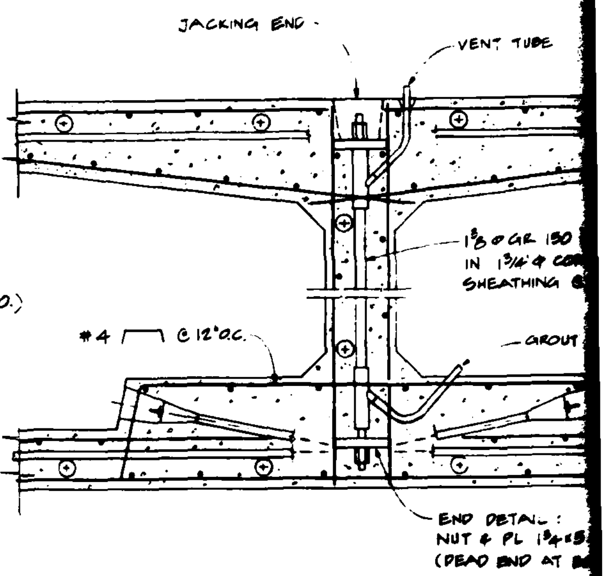
SHEET TITLE: **UNIT CONNECTION SECTIONS**

PROJECT: **NAVY PIER DESIGN CONCEPTS**

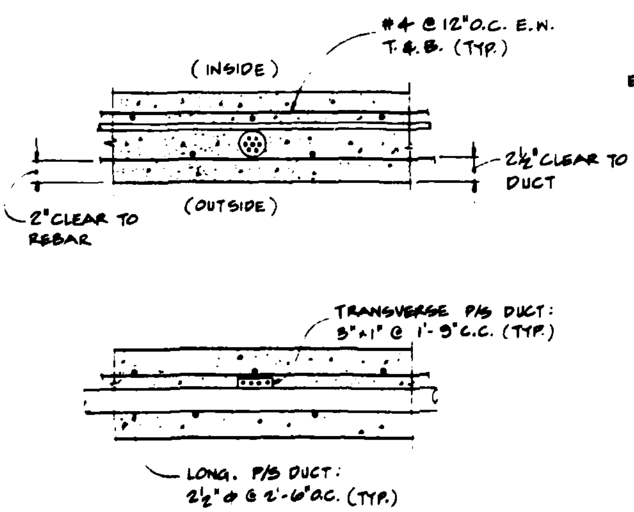
SHEET No DRAWING **5**



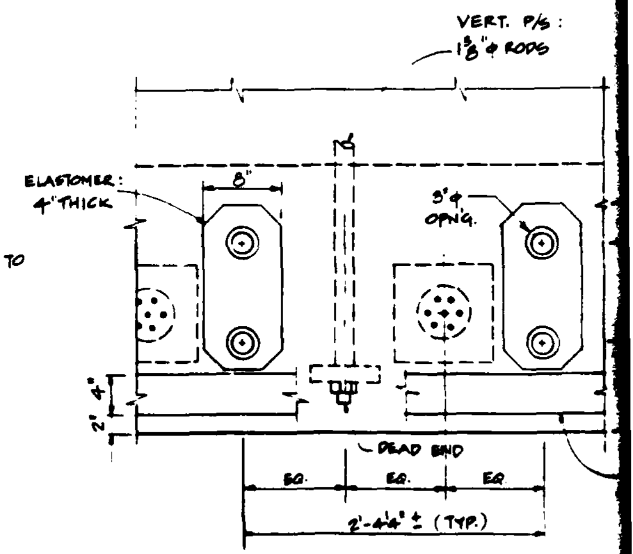
SECTION C/6
1" = 1'-0"



SECTION B/6
1" = 1'-0"



PONTOON SECTION E/6
1½" = 1'-0"
(TOP & BOTTOM SLAB & EXTERIOR LONGITUDINAL WALLS)



PARTIAL END VIEW AT JOINT
1½" = 1'-0"

PROJECT N°
DESIGN
DRAWN
CHECKED

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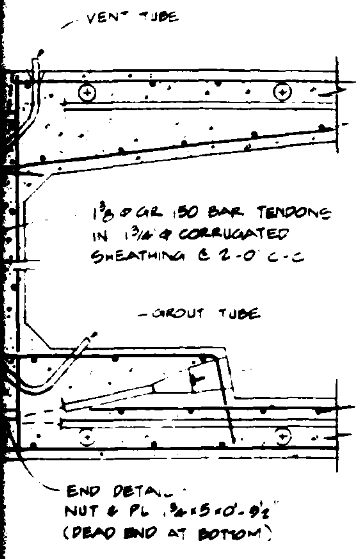
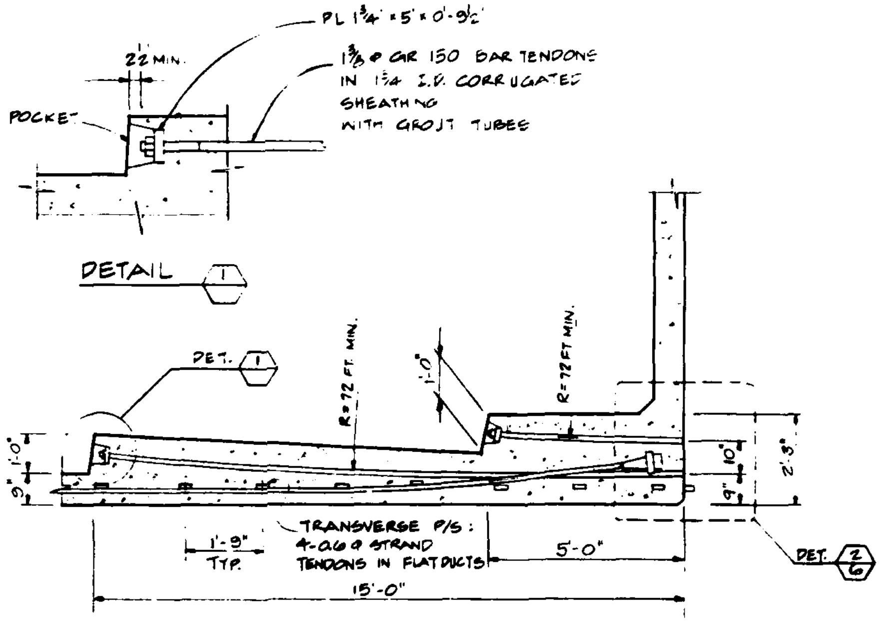
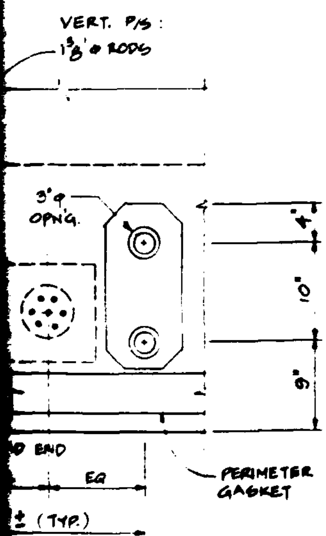
100%

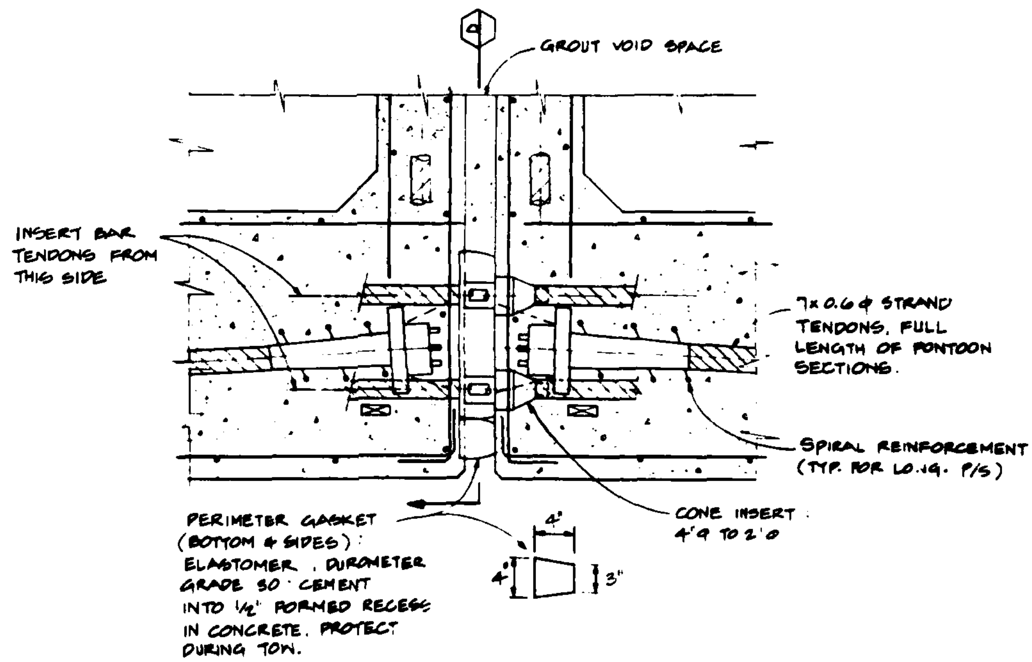




PARTIAL - END SECTION A/6
1/2" = 1'-0"



VIEW AT JOINT D/6



DETAIL 2/6
1 1/2" = 1'-0"

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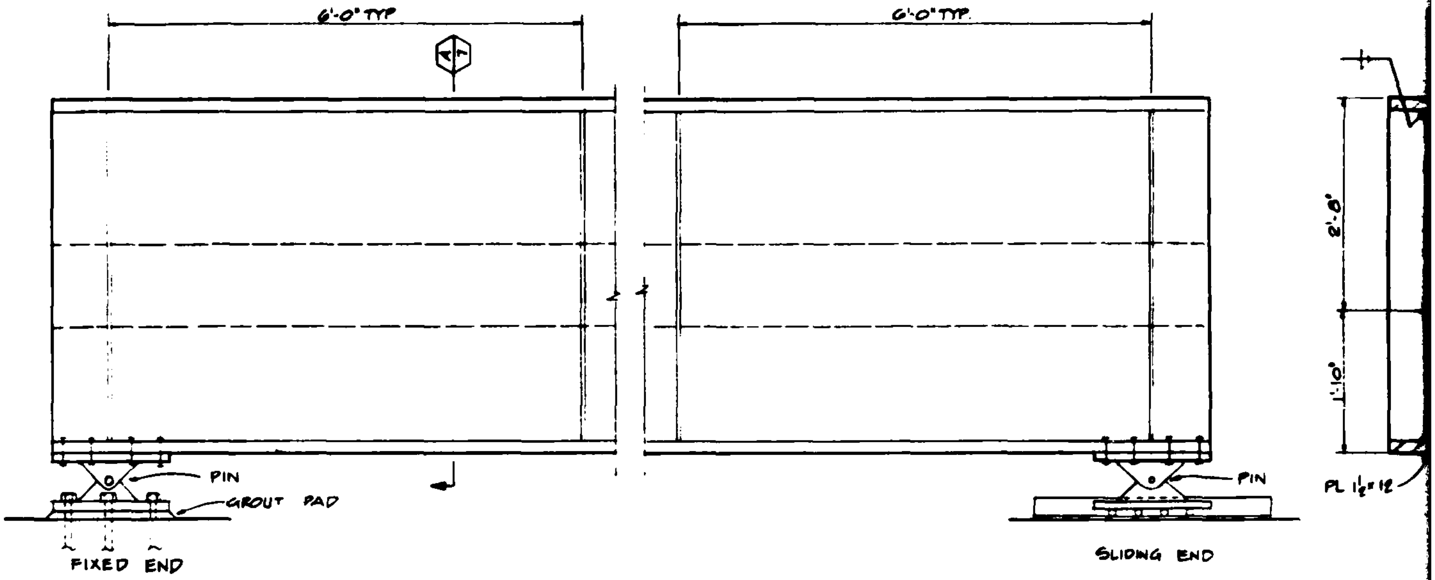
SHEET TITLE: **UNIT CONNECTION DETAILS**

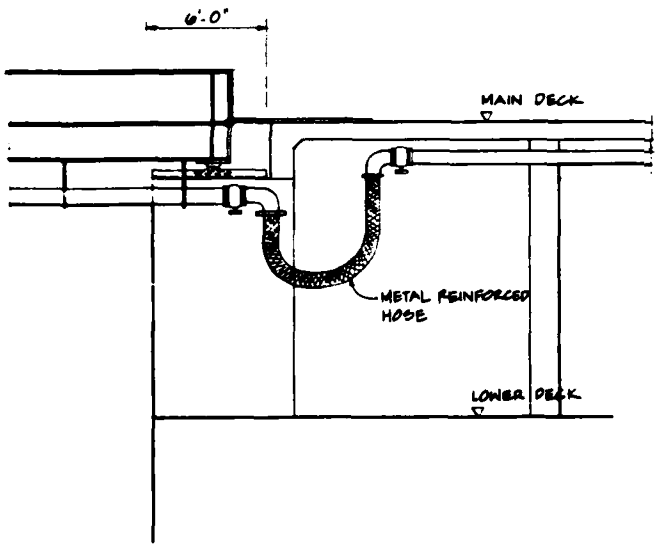
PROJECT: **NAVY PIER DESIGN CONCEPTS**

SHEET Nº DRAWING: **6**

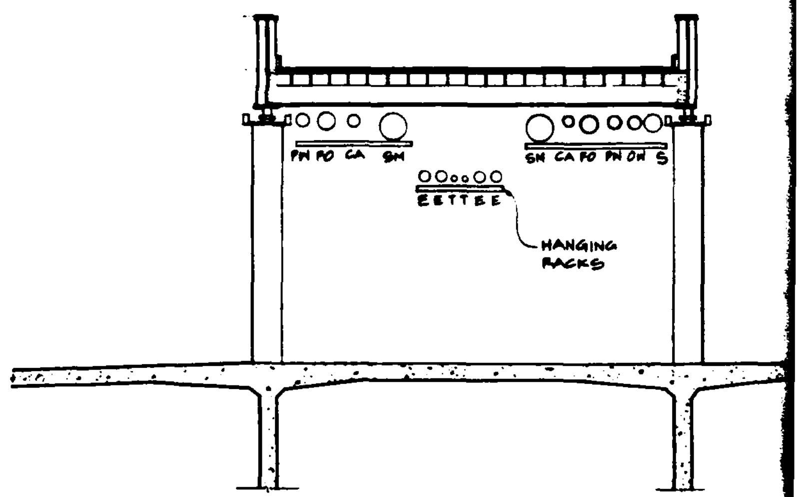
2



TYPICAL RAMP ELEVATION
1"=1'-0"



SECTION C/7
¼"=1'-0"

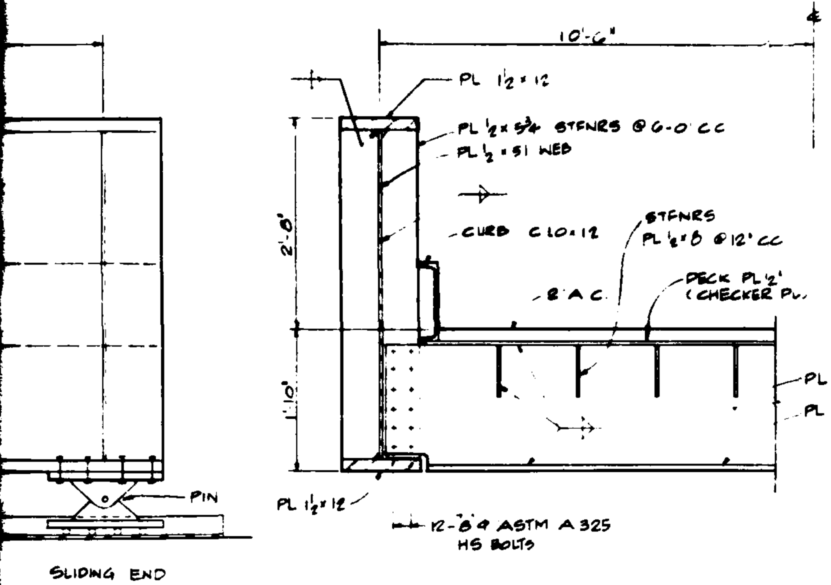


SECTION B/7
¼"=1'-0"

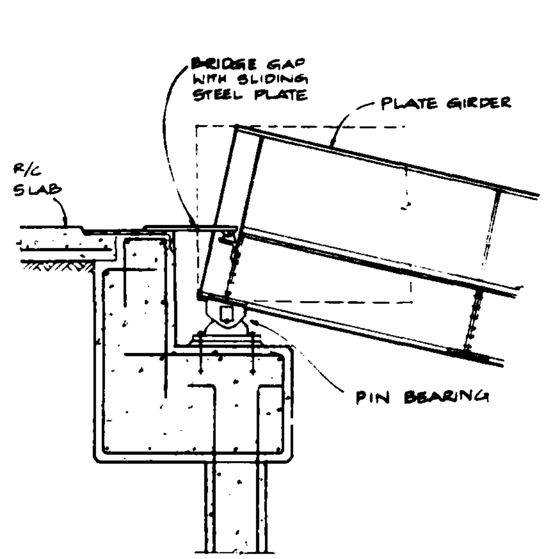
PROJECT N°
DESIGN
DRAWN
CHECKED

TY LIN INTERNATIONAL
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133 Tel. (415) 989-1050

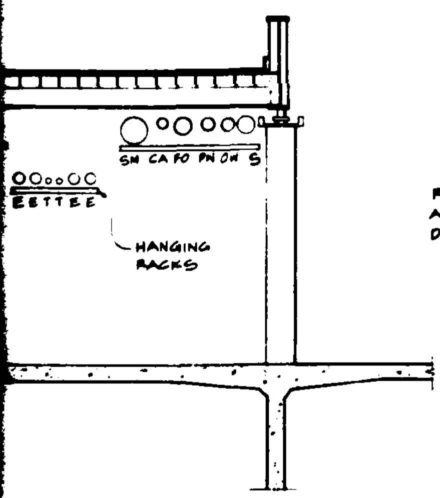
100%



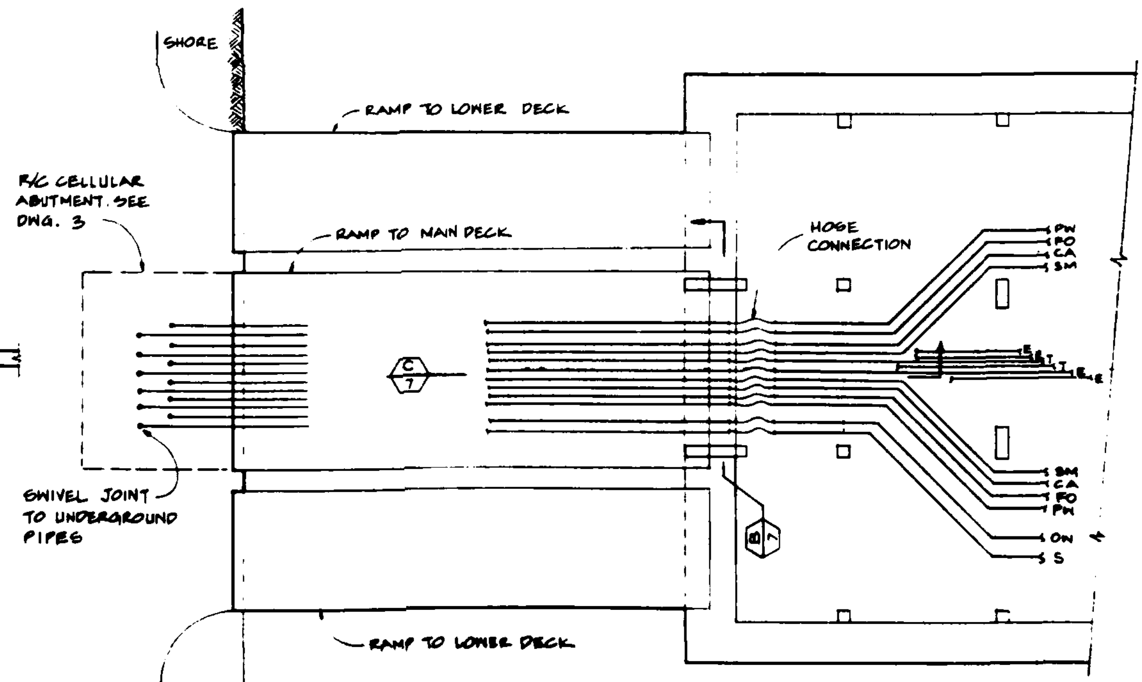
SECTION A/7
1" = 1'-0"
(ALL STEEL ASTM A36)



DETAIL 1/7
½" = 1'-0"



SECTION B/7
¼" = 1'-0"



UTILITY PIPE TRANSITION FROM PIER TO SHORE - PLAN
1" = 10'

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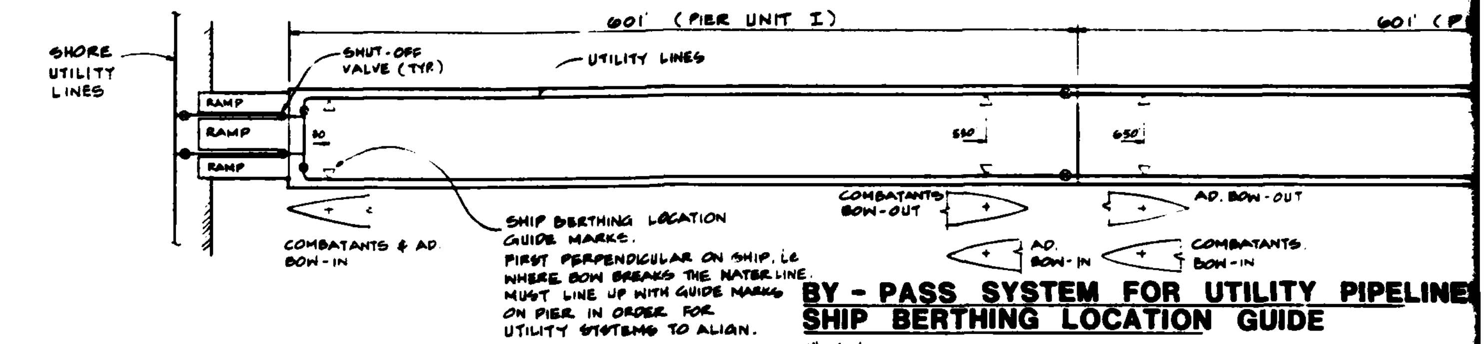
SHEET TITLE
RAMP & UTILITY LINE DETAILS

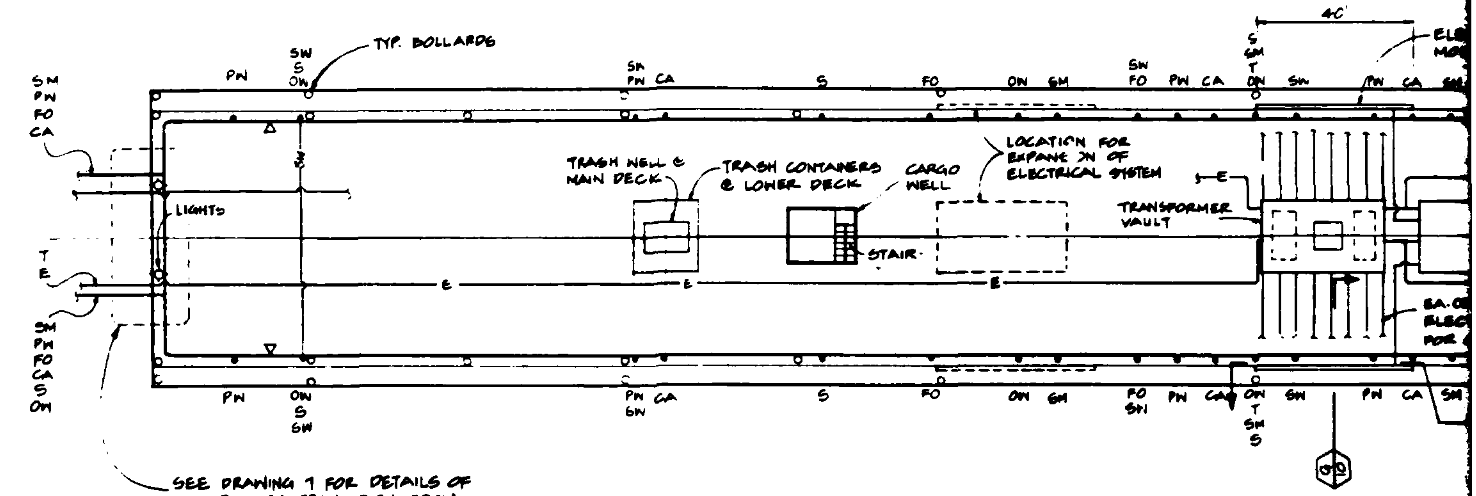
PROJECT
2
NAVY PIER DESIGN CONCEPTS

SHEET No
DRAWING
7



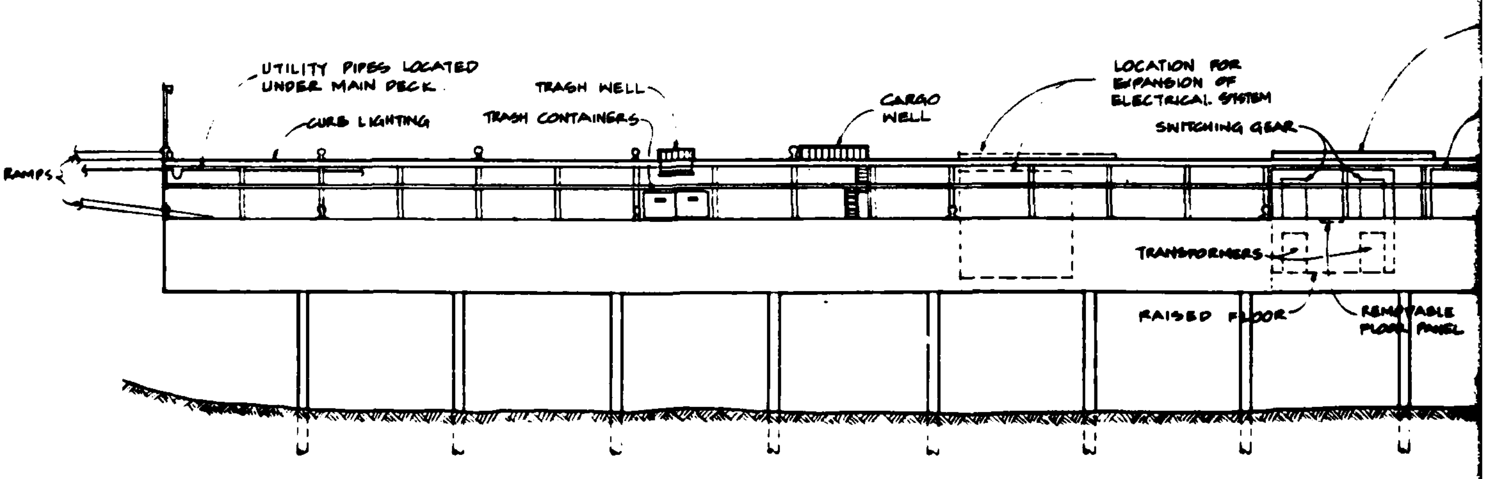
BY - PASS SYSTEM FOR UTILITY PIPELINE
SHIP BERTHING LOCATION GUIDE

1" = 60'



UTILITY AND ELECTRICAL PLAN (UNI

1" = 20'



UTILITY AND ELECTRICAL ELEVATIO

1" = 20'

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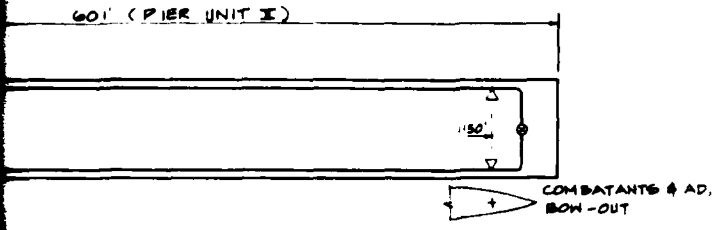
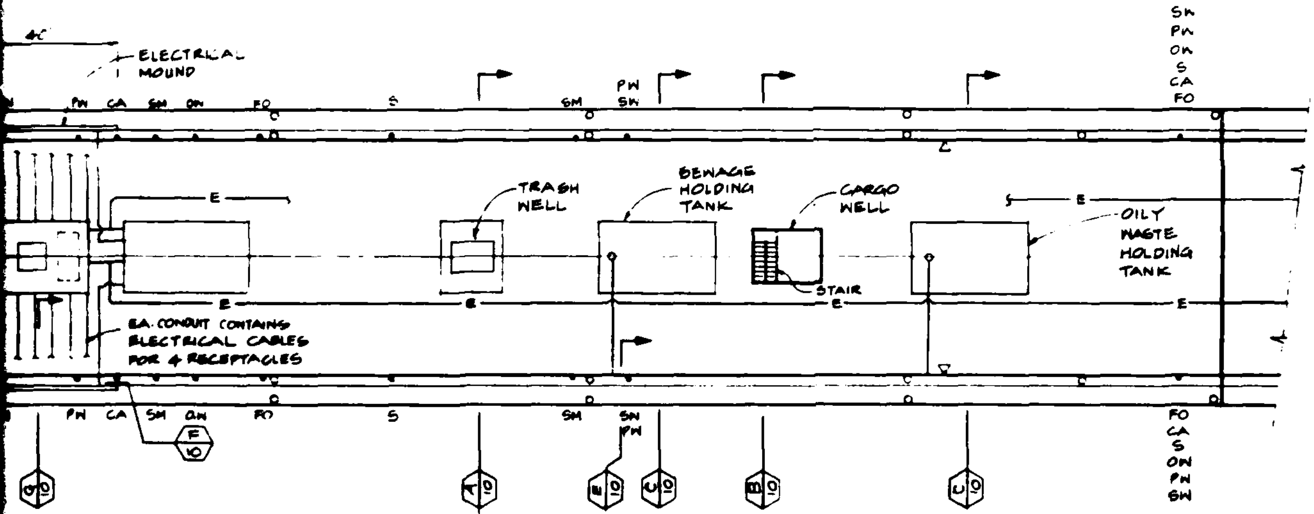


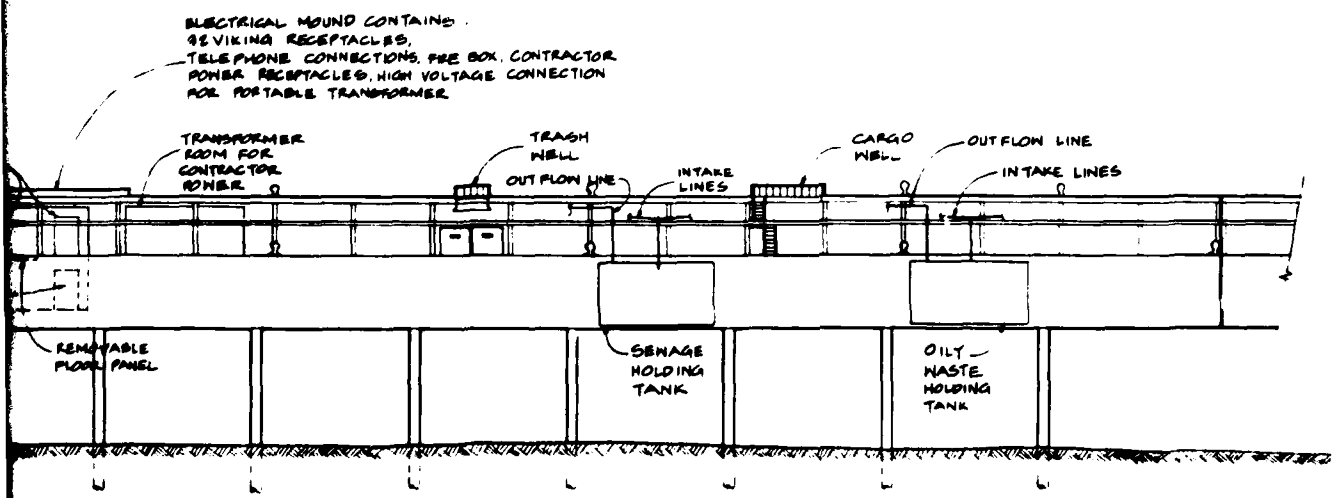
LEGEND

SM	STEAM
PW	POTABLE WATER
SW	SALT WATER
S	SEWAGE
FO	FUEL OIL
OW	OILY WASTE
CA	COMPRESSED AIR
T	TELEPHONE
•	VALVE
▽	SHIP BERTHING LOCATION GUIDE MARKS

PIPELINES &



LAN (UNIT I)



ELEVATION

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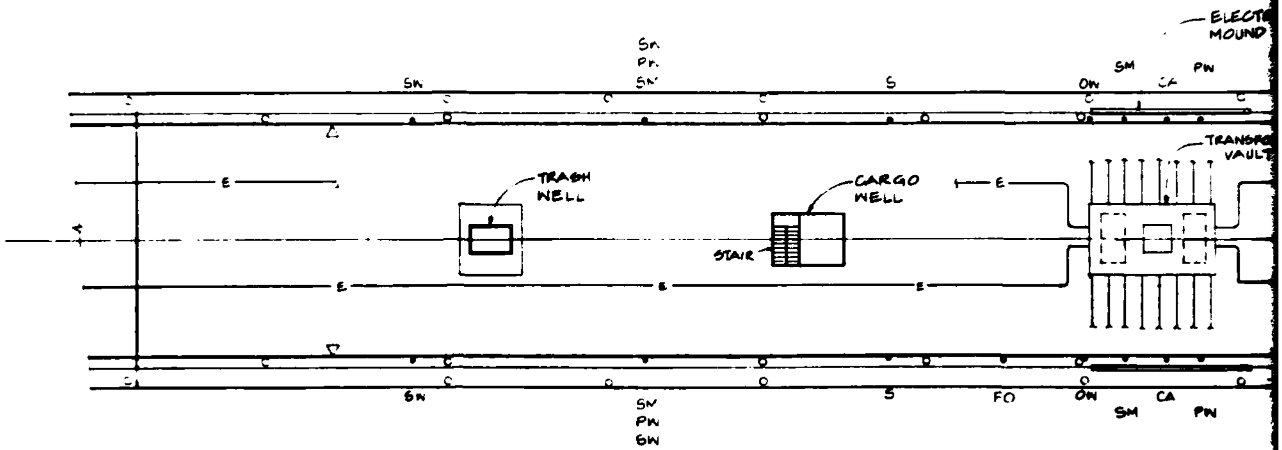
SHEET TITLE: **UTILITY SERVICES (PIER UNIT I)**

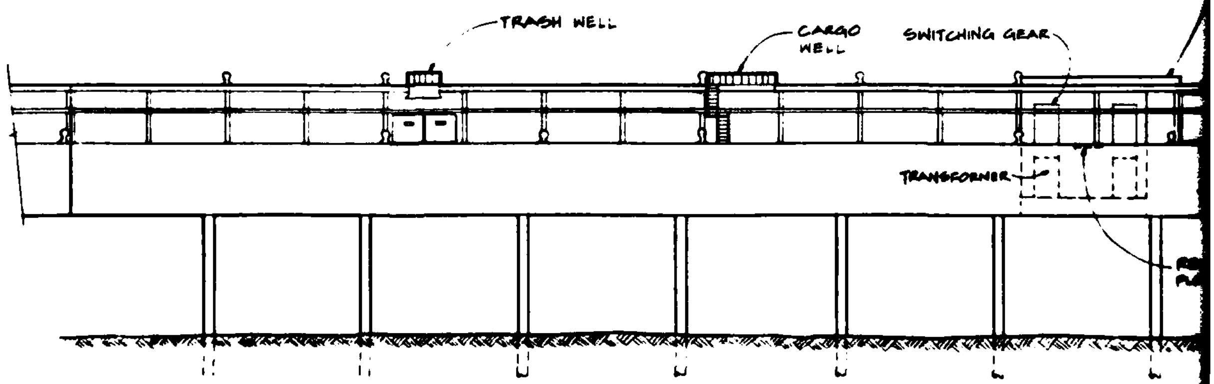
PROJECT: **NAVY PIER DESIGN CONCEPTS**

SHEET Nº DRAWING **8**



UTILITY AND EL

1" = 20'



UTILITY AND

1" = 40'

PROJECT N°

DESIGN

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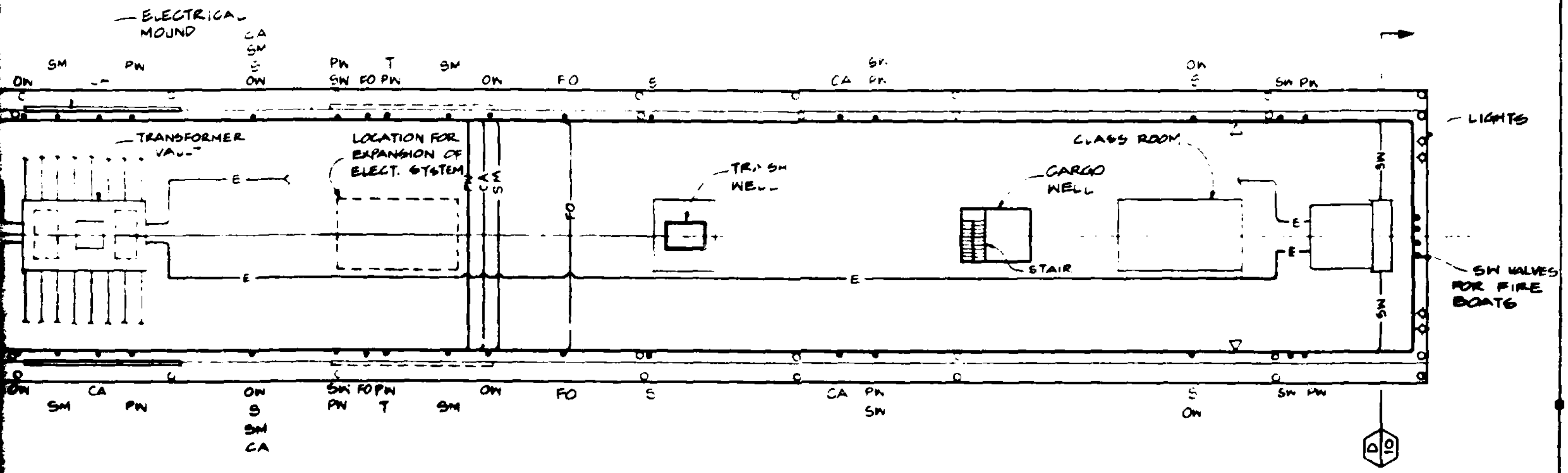
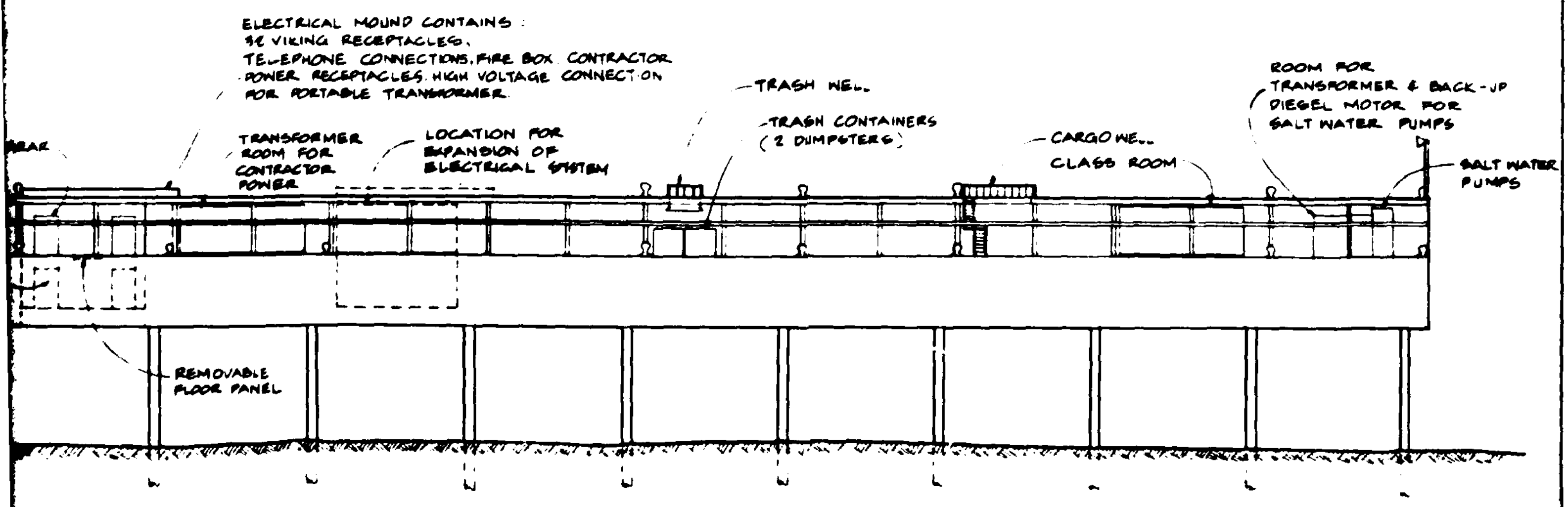


ITY AND ELECTRICAL PLAN (UNIT II)



ITY AND ELECTRICAL ELEVATION

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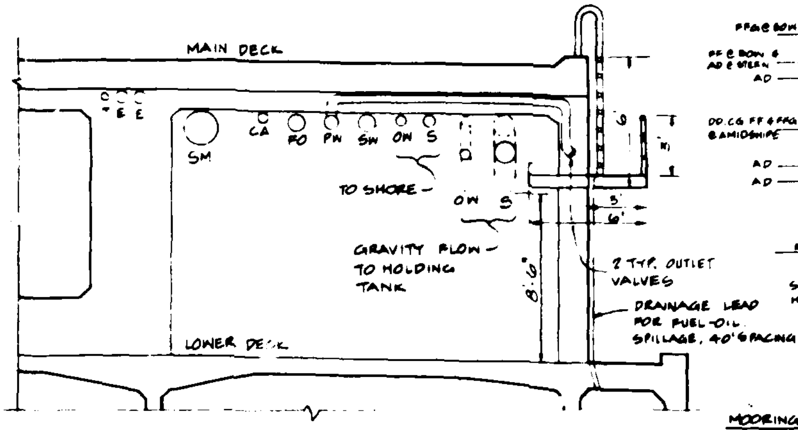
SHEET TITLE
UTILITY SERVICES (PIER UNIT II)

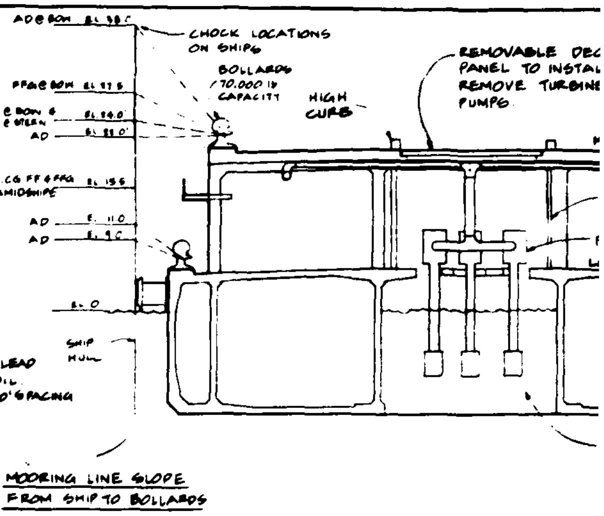
PROJECT
NAVY PIER DESIGN CONCEPTS

SHEET N° DRAWING
9



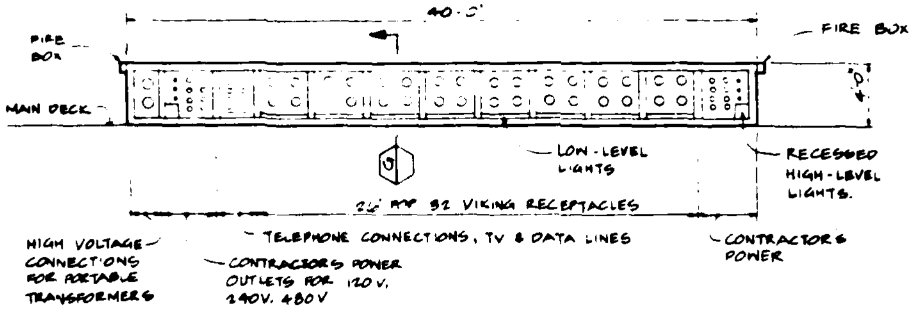
TYPICAL UTILITY PIPE LAYOUT (E)

1/4" = 1'-0"



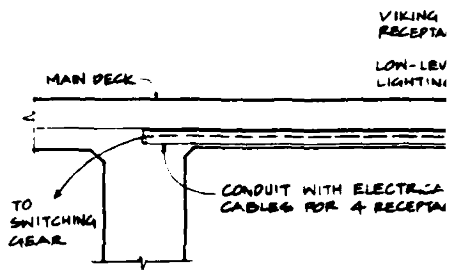
SALT WATER PUMP S

1" = 10



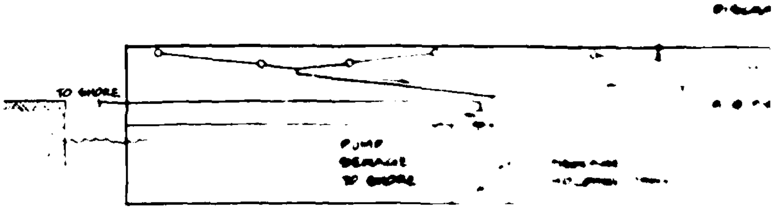
ELECTRICAL MOUND - ELEVATION (F/10)

1" = 5



ELECTRICAL MOUND -

1/4" = 1'-0"



SEWAGE FLOW DIAGRAM

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PROJECT N.

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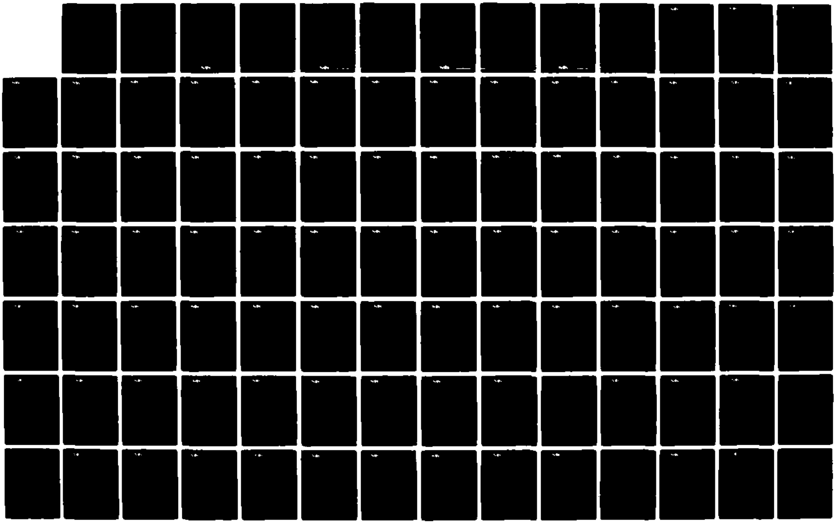
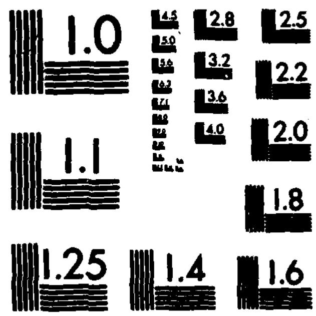
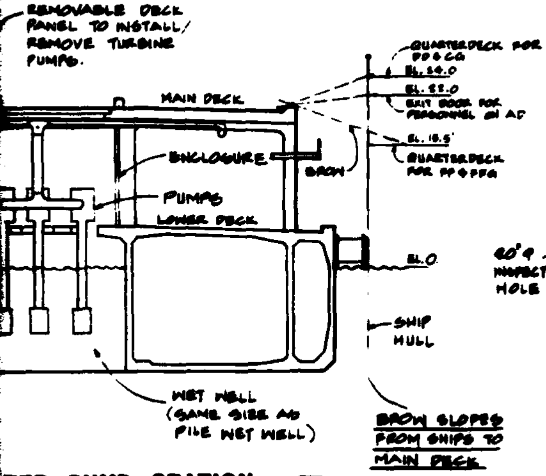
AD-A121 865 CONCEPTUAL DESIGN OF NAVY FLOATING PIER(U) LIN (T Y) INTERNATIONAL SAN FRANCISCO CA SEP 82 NCEL-CR-82-031 N62474-81-C-9404 2/3

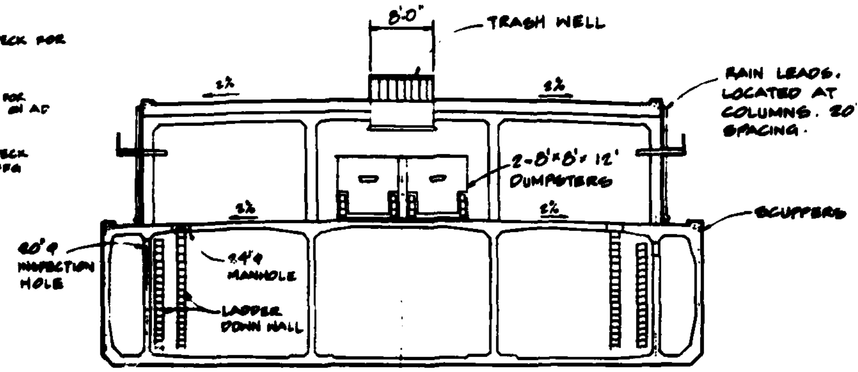
UNCLASSIFIED F/G 13/2 NL




1.0
1.1
1.25
1.4
1.6
1.8
2.0
2.2
2.5
2.8
3.2
3.6
4.0

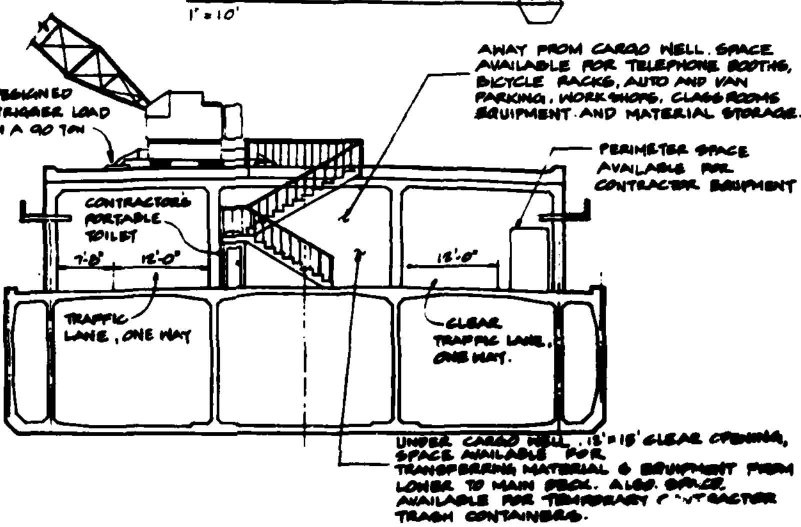


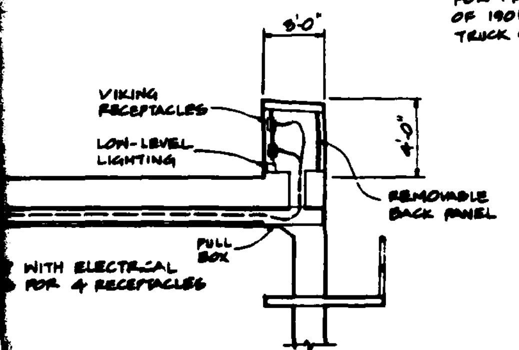
TER PUMP STATION (D)



DRAINAGE, MANHOLES AND TRASH WELL - SECTION (A)

1" = 10'

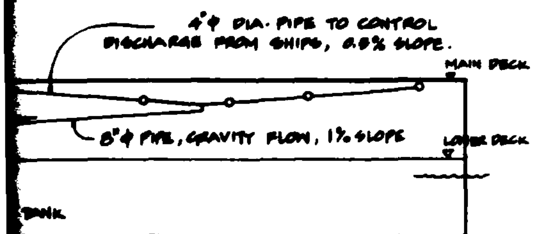


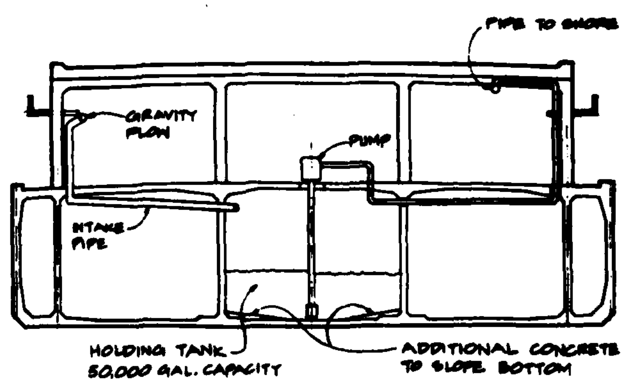


AL MOUND - SECTION (G)

STAIR & CARGO WELL - SECTION (B)

1" = 10'





AGRAM

HOLDING TANK FOR SEWAGE & OILY WASTE (C)

1" = 10'

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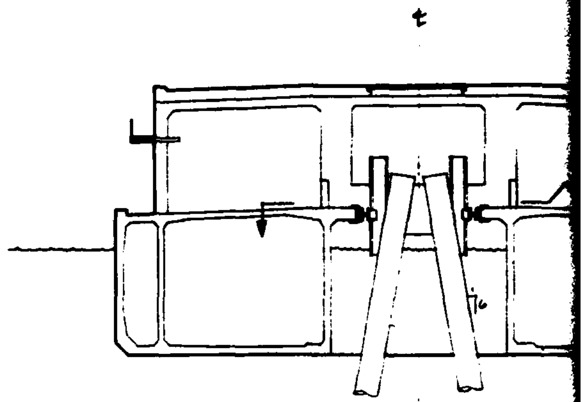
SHEET TITLE: **MISCELLANEOUS SECTIONS**

PROJECT: **NAVY PIER DESIGN CONCEPTS**

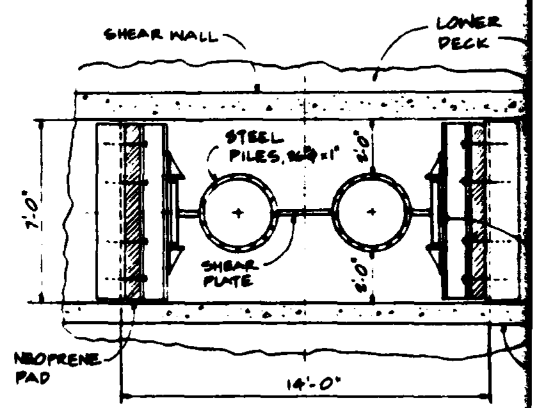
SHEET NO. DRAWING **10**

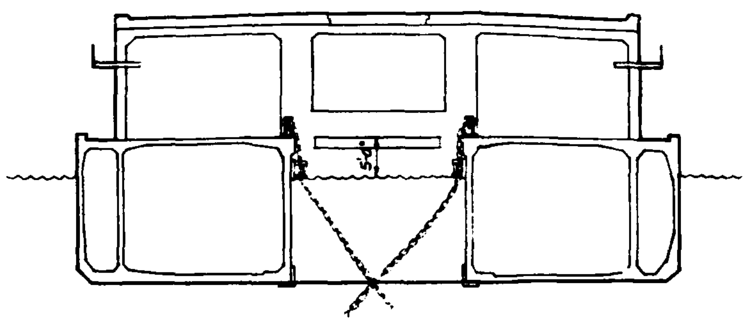


ALT. 2

BATTER PILES

1" = 10'





ALT. 3

MOORING CHAIN (C/11)

SECTION (B/11)

1" = 5'-0"

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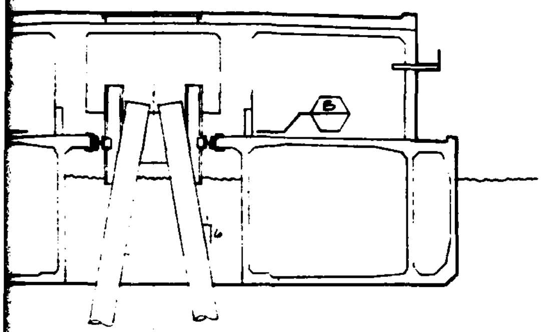
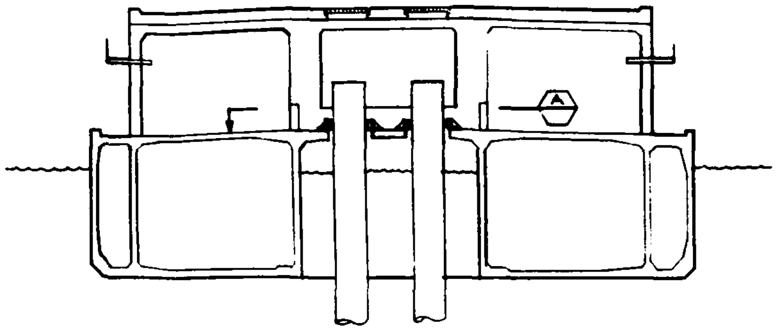


ALT. 2

BATTER PILES

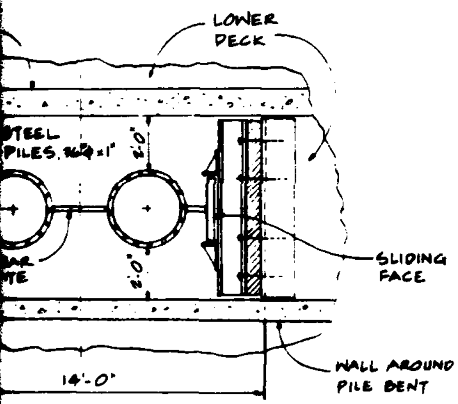
1" = 10'



ALT. 1

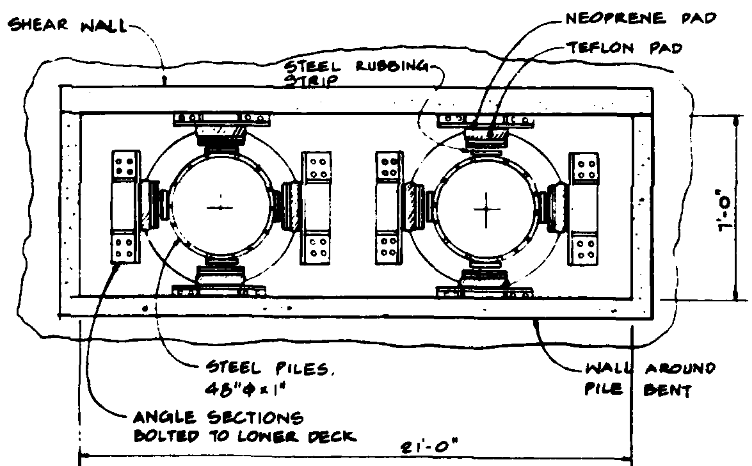
VERTICAL PILES

1"=10'



SECTION B / 11

1"=3'-0"



SECTION A / 11

1"=3'-0"

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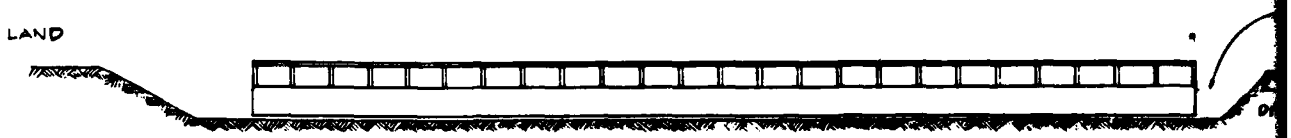
SHEET TITLE: **ANCHORING SYSTEMS**

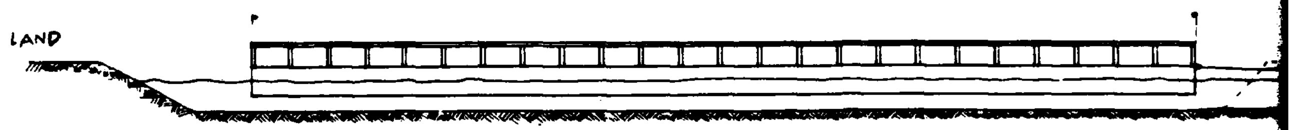
PROJECT: **NAVY PIER DESIGN CONCEPTS**

SHEET Nº DRAWING **11**



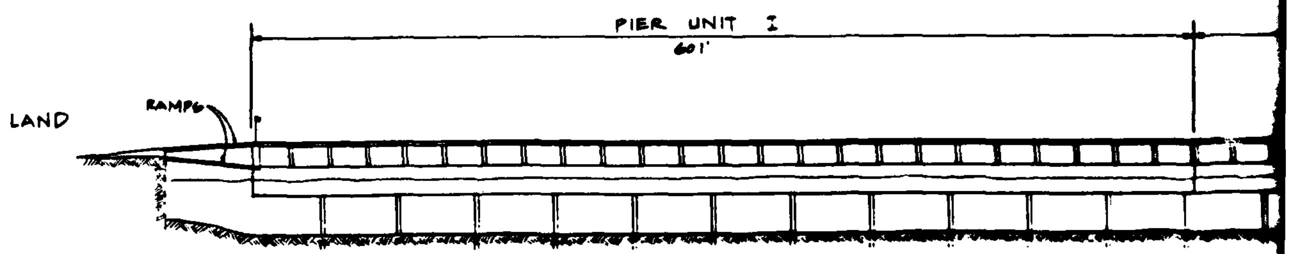
(1) CONSTRUCTION IN FLOOD BASIN (OR DRY DOCK)

a). BUILD TWO 601 FT. LONG PIER UNITS (OR THREE 401 FT LONG PIER UNITS)
b). INSTALL UTILITY SYSTEMS.



(2) TOW TO FINAL SITE

a). REMOVE DIKE TO FLOAT PIER UNITS.
b). TOW TO FINAL SITE



(3) INSTALL FLOATING PIER

a) POSITION PIER UNIT I AT FINAL LOCATION AND INSTALL BY DRIVING PILES
b). POSITION PIER UNIT II AND JOIN TO UNIT I BY POST-TENSIONING.
c). DRIVE PILES FOR UNIT II
d). CONNECT UTILITY SYSTEMS BETWEEN PIER UNITS & TO SHORE
e). COMPLETE CONSTRUCTION.

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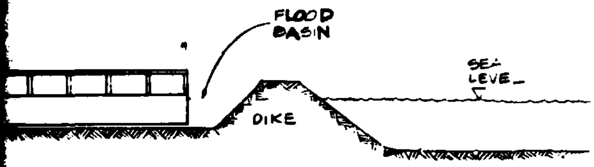
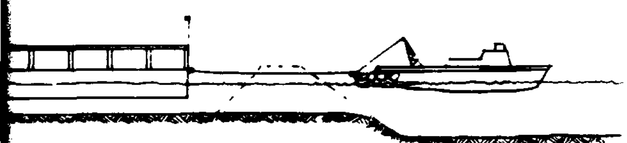
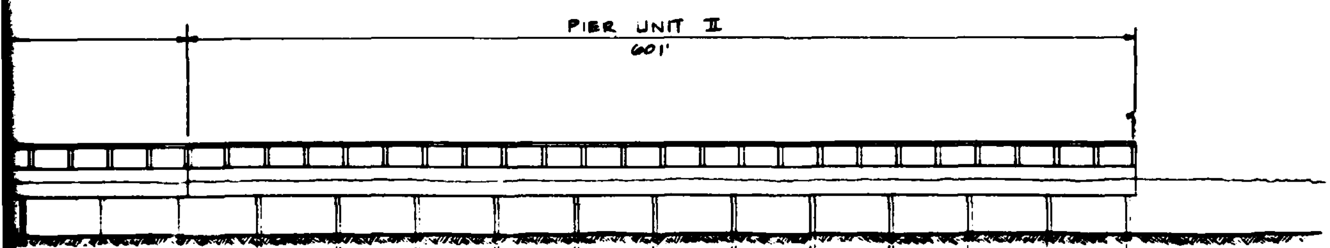


FT LONG PIER UNITS)





T DRIVING PILES
NING.

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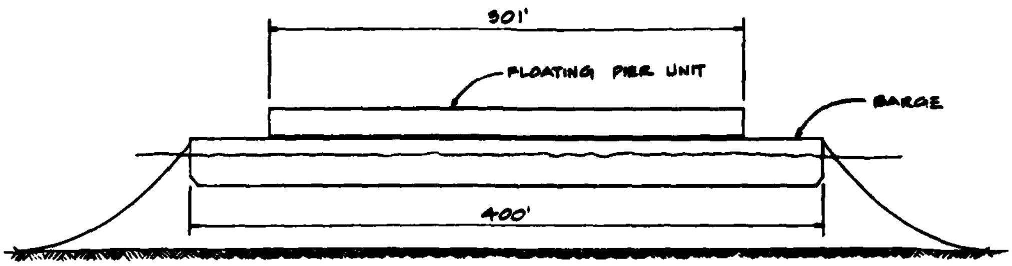
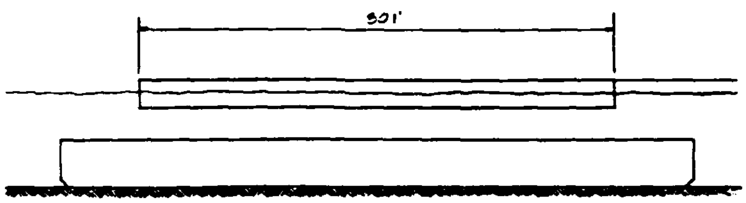
SHEET TITLE: **FLOOD BASIN CONSTRUCTION METHOD**

PROJECT: 2 **NAVY PIER DESIGN CONCEPTS**

SHEET Nº DRAWING **12**

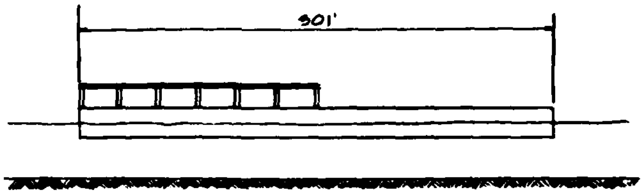


① CONSTRUCT 301 FT FLOATING PIER UNIT ON BARGE



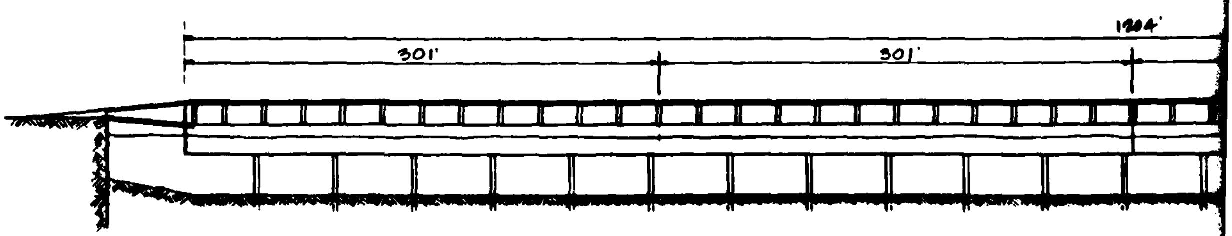
② FLOAT PIER UNIT

a). SINK THE BARGE
b). MOVE PIER UNIT AWAY FROM BARGE
c). REPEAT CONSTRUCTION CYCLE TO BUILD OTHER PIER UNITS (TOTAL OF 4 REQUIRED)



③ COMPLETE CONSTRUCTION OF PIER UNIT

a). CONSTRUCT MAIN DECK
b). INSTALL UTILITY SYSTEMS.



④ INSTALL FLOATING PIER

a). TOW FLOATING PIER UNITS TO FINAL SITE
b). POSITION FIRST PIER UNIT AND INSTALL BY DRIVING PILES
c). POSITION SECOND PIER UNIT AND JOIN TOGETHER BY POST-TENSIONING, THEN DRIVE PILES
d). REPEAT CYCLE UNTIL TOTAL LENGTH IS COMPLETE
e). CONNECT UTILITY SYSTEMS BETWEEN PIER UNITS & TO SHORE
f) COMPLETE CONSTRUCTION

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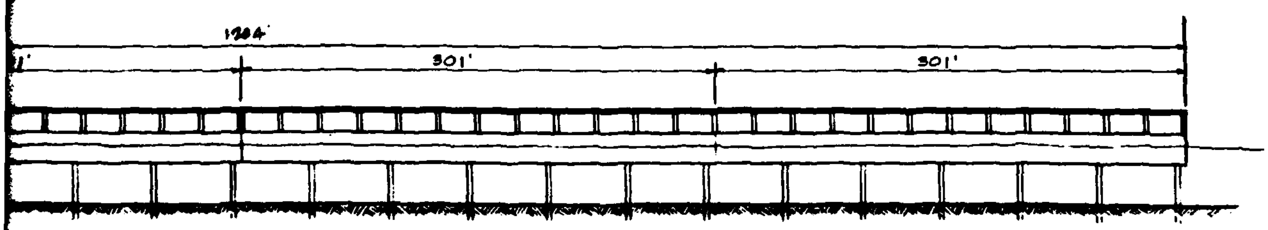
(TOTAL OF 4 REQUIRED)



TENSIONING. THEN DRIVE PILES.

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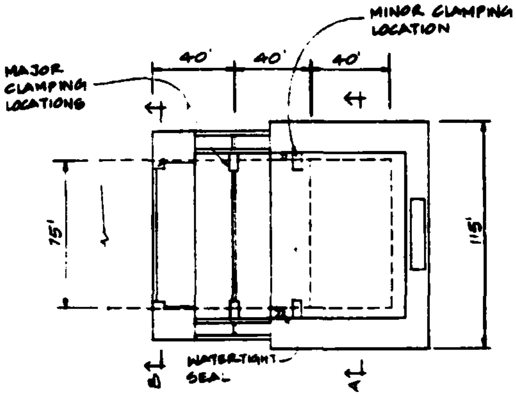
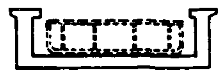
SHEET TITLE: BARGE MOUNTED CONSTRUCTION METHOD

PROJECT: NAVY PIER DESIGN CONCEPTS

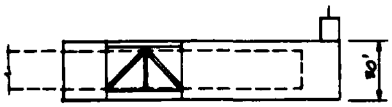
SHEET Nº DRAWING: 13



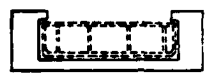
FLOATING FORM - PLAN



SECTION A-A



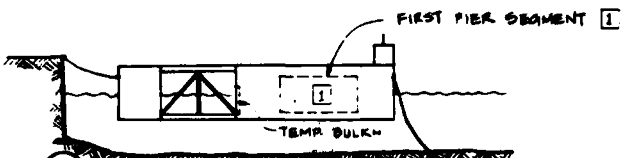
ELEVATION



SECTION B-B

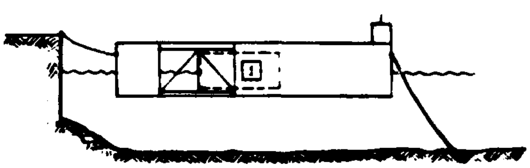
① CONSTRUCT FLOATING FORM

a). CONSTRUCT STEEL FLOATING FORM
b). ANCHOR FORM AT CONSTRUCTION SITE USING MOORING LINES



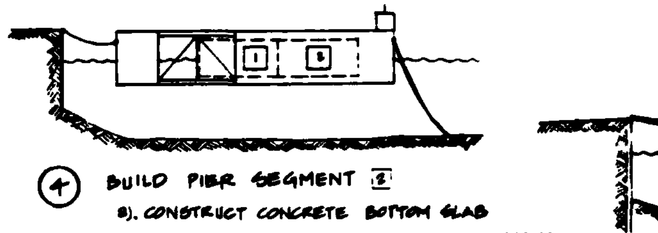
② BUILD PIER SEGMENT 1

a). PROVIDE FLOATING FORM WITH TEMPORARY BULKHEAD TO KEEP WATER OUT OF CONSTRUCTION WELL
b). CONSTRUCT CONCRETE BOTTOM SLAB
c). CONSTRUCT CONCRETE PIER WALLS AND TOP SLAB
d) PRESTRESS WALLS AND DECKS.



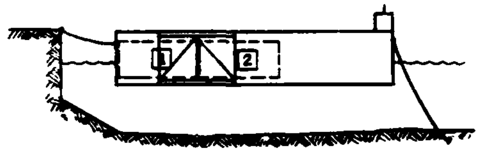
③ SHIFT PIER SEGMENT 1

a) JACK SEGMENT 1 TO TEMPORARY BULKHEAD.
MAKE WATERTIGHT SEAL BETWEEN SEGMENT 1 AND FORM. USING
. TWIN "J" SEALS. JACK SEGMENT 1 FURTHER OUT OF CONSTRUCTION WELL
b). CLAMP FORM TO SEGMENT 1 BY USING FLAT JACKS



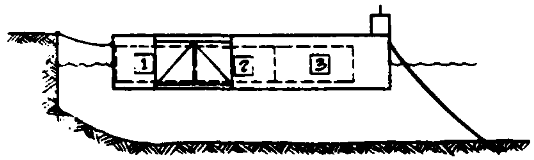
④ BUILD PIER SEGMENT 2

a). CONSTRUCT CONCRETE BOTTOM SLAB
b). SET REBAR FOR WALLS & PULL LONGITUDINAL WALL & TOP SLAB FORMS FROM 1 to 2
c). USE PRECAST BULKHEAD WALLS AT JUNCTURE 1 - 2
d). SET REBAR FOR TOP SLAB & PLACE CONCRETE
e). PRESTRESS WALLS AND DECKS.

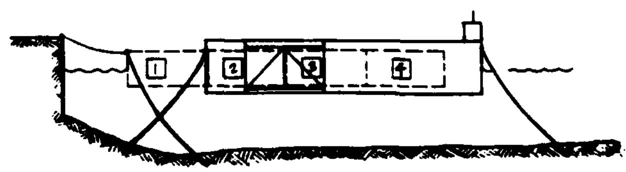


⑤ SHIFT PIER SEGMENT 1 & 2

a). JACK SEGMENT 1 & 2 OUT OF CONSTRUCTION WELL
b). CLAMP FORM TO SEGMENT 1 & 2



⑥ BUILD PIER SEGMENT 3



⑦ BUILD PIER SEGMENT 4

a). JACK FORM FORWARD
b). ANCHOR SEGMENT 1 USING MOORING LINES OR
c). CLAMP FORM TO SEGMENT 2 & 3
d). BUILD SEGMENT 4

DESIGN DRAWN CHECKED PROJECT No

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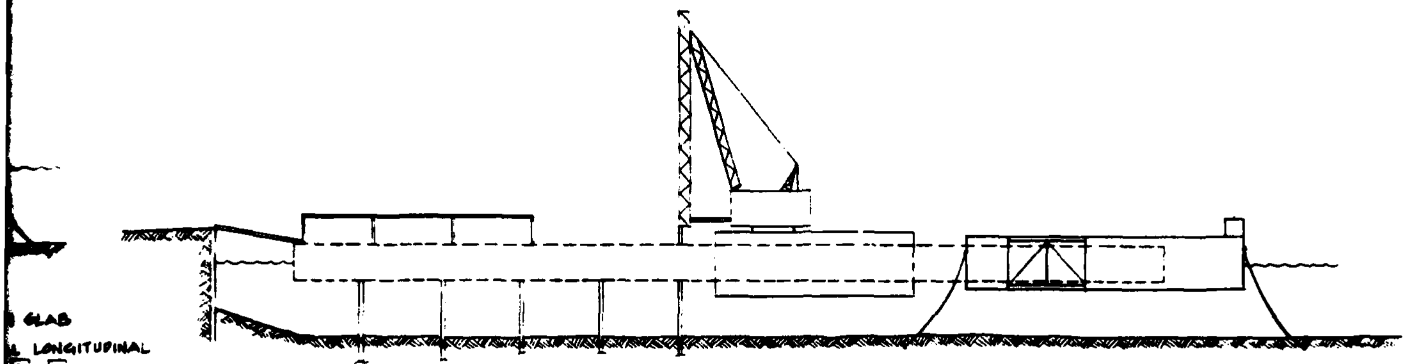


SLAB
LONGITUDINAL
1 to 2
AT JUNCTURE 1 - 2
PLACE CONCRETE
...ED.

(8) ALTERNATE I :

CONSTRUCTION AT FINAL PIER SITE

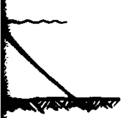
a). REPEAT CONSTRUCTION STEP (7) FOR TOTAL 1200 FT LENGTH
b). DRIVE PILES BEHIND FLOATING FORM
c). CONSTRUCT MAIN DECK LEVEL
d). INSTALL UTILITY SYSTEMS
e). COMPLETE CONSTRUCTION

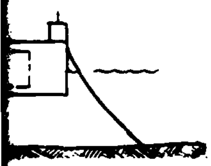
2
CONSTRUCTION WELL
1 + 2

(8) ALTERNATE II :

CONSTRUCTION AT OFF-SITE LOCATION

a). REPEAT CONSTRUCTION STEPS (2) - (7) TO BUILD TWO 600 FT. PIER UNITS.
b). BUILD MAIN DECK LEVEL & INSTALL UTILITY SYSTEMS.
c). TOW PIER UNITS TO FINAL SITE.
d). INSTALL PIER UNIT I BY DRIVING PILES.
e). JOIN PIER UNIT II BY POST-TENSIONING & DRIVE PILES.
f). CONNECT UTILITY SYSTEMS BETWEEN PIER UNITS & TO SHORE
g). COMPLETE CONSTRUCTION.





...ING MOORING LINES OR PILES.
... 2 + 3

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100%	APR. 85	

SHEET TITLE: FLOATING FORM CONSTRUCTION METHOD

PROJECT: NAVY PIER DESIGN CONCEPTS

SHEET Nº DRAWING: 14

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PROJECT:
ITEM: ENVIRONMENTAL LOADS
DESIGN: WIND LOADS
DATE: H.H.

SHEET: A-1
OF
REVISION:

APPENDIX A : ENVIRONMENTAL LOAD CALCULATIONS

WIND LOADING

DM 26

$$F_w = C_{yw}\ 1/2\ \rho_w V_w^2 A_s$$

WIND AT 90° TO SHIP, $C_{yw} = 1.0$

$\rho_w = 0.00237 \frac{LB-SEC^2}{FT^4}$ @ 68 °F

V = VELOCITY, ft/sec AT 33' ABOVE WATERLINE

USE HURRICANE VELOCITY, V = 90 MPH = 132 ft/sec

A_s = SIDE PROJECTED AREA

FOR AD-41

SIDE AREA (60' HIGH)(640' LONG) = 38,400 ft²
USE A_s = 40,000 ft²

$(F_w)_{AD} = 1.0(\frac{1}{2})(0.00237)(132\ ft/sec)^2(40,000)$

= 825 KIPS

FOR DD-963

DM 26.6 GIVES DATA ON SMALLER DD'S

DD-692 L = 377' h = 22' A_s = 10,200 ft.²
DD-931 L = 418' h = 31' A_s = 13,000 ft.²

EXTRAPOLATING
DD-963 L = 564 h = 39.6 USE A_s = 22,300 ft²

$(F_w)_{DD} = 1.0(\frac{1}{2})(0.00237)(132)^2(22,300)$ = 460 KIPS

NOTE: NESTED SECOND SHIP PICKS UP 50% OF LOAD

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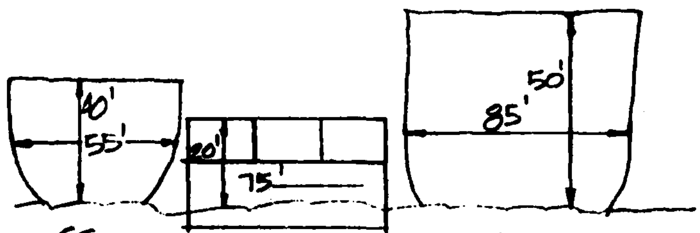
LONGITUDINAL WIND LOAD



CG PIER AP

WIND = 90 MPH
132 fps

A = 55 (40) = 2200 ft² A: 20 (75) = 1500 ft² A = 50(85) = 4250 ft²

A_T = 7950 ft²
SAY 10,000 ft² TO INCLUDE HIGHER PORTIONS OF VESSELS.

$F = C_{yw} \frac{1}{2} P_w V_w^2 A$

$= (1.0)\frac{1}{2}(.00237)(132\ fps)^2 \frac{(10,000)}{1000\ LB/K}$

= 206 KIPS

ASSUME 2ND SHIP PICKS UP 50% OF WIND LOAD
OR 103 KIPS

TOTAL LONGITUDINAL WIND LOAD = 206 + 103 = 309 KIPS

LOAD ON EACH PILE BENT = $\frac{309^K}{29\ BENTS}$ = 10.6 KIPS

LOAD ON EACH PILE = $\frac{10.6}{2}$ = 5.3 KIPS OR APPROX.
1/10TH OF LATERAL WIND LOAD

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ITEM: CURRENT LOADS	SHEET A-3
DESIGN:	OF
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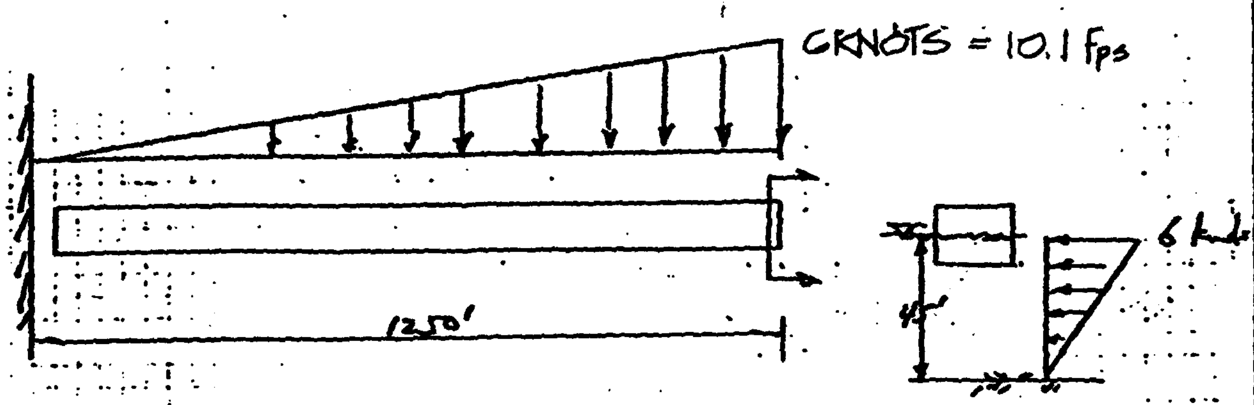
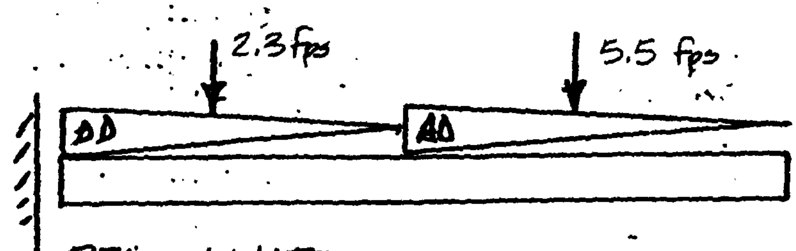
CURRENT LOADS

ASSUME: 6 KNOT IN CHANNEL

CURRENT PROFILE



WORST CASE



BELOW WATER

DD L = 530'
DRAFT = 18'

AD L = 620'
DRAFT = 24'

DETERMINE CURRENT FORCES BY THREE METHODS

1. APPROX. METHOD
2. NEW DM 26.6 (UNDER NAVY REVIEWS FOR FUTURE PUBLICATION
3. EXISTING DM 26

FOR DD-963

APPROX. METHOD

$F_c = K A_s V_c^2$

$K = 1.5$ FOR FLAT SURFACES USE
1.0 FOR CURVED SURFACES

$A_s = (530')(18') = 9540 \text{ ft}^2$

$F_c = 1.5\,(9540)(V_c^2) = 14{,}300\, V^2$

V_c	F_c
2.3 fps	76 KIPS
5.5 fps	433 KIPS

NEW DM 26.6 (Pg 26.6-3)

$F_c = C_{yc}\,\frac{1}{2}\rho_c V_c^2 (LWL)\, T$

C_{yc} FROM Fig 4; WHEN $\frac{WD}{T} = \frac{45'}{18'}$ WATER DEPTH / DRAFT

$= 2.5$

THEN $C_{yc} = 1.4$

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FOR $V = 3.2$ KNOTS

$(F_c)_1 = 80$ TONS$(2.24) = 179$K

ADJUST TO DD-963

$(F_c)_2 = 179 \dfrac{7800}{3400} = 411$K

ADJUST TO WD = 45 FT

$(F_c)_2 = 0.5\,(411\text{K}) = 206$ K

SUMMARY OF RESULTS

METHOD	CURRENT FORCE ON ONE SHIP FOR: $V =$ 1.4 KTS OR 2.3 FPS	$V =$ 3.2 KTS OR 5.5 FPS	
APPROX.	76	433	
NEW DM 26.6	70	401	USE
EXISTING DM26.6	47	206	

USE NEW DM 26.6 FOR CONSERVATIVE ESTIMATE.

FOR TWO SHIP NESTED, DATA FROM EXISTING DM 26.6 SHOWS THAT SECOND SHIP PICKS UP ABOUT 1/3RD OF CURRENT FORCE ON FIRST SHIP.

RESULTS FOR TWO DD-963

	$V = 2.3$ fps	$V = 5.5$ fps
NEW DM 26.6 TOTAL	$F = 1.33(70) = 93$K	$F = 1.33(401) = 533$ K

INCREMENT FOR 2ND SHIP $F_{2ND} = 23^k$ $= 132^k$

CURRENT LOAD ON TENDER, AD-41

USE NEW DM 26.6 METHOD

$F_c = C_{yc} \frac{1}{2} \rho_c V_c^2 (LWL) T$

C_{yc} FROM FIG 4.

WHEN $\frac{WD}{T} = \frac{45'}{24'} = 1.875$

THEN $C_{yc} = 1.7$

$LWL = 620'$
$T = 24'$

$F_c = 1.7 (\frac{1}{2})(1.9876)(V_c^2)(620)(24) =$
$= 25,140\ V_c^2$

FOR $V_c = 2.3$ fps

$F_c = 25,140(2.3)^2 = \underline{133\text{K}}$

FOR $V = 5.5$ fps

$F_c = 25,140(5.5)^2 = \underline{760\text{ K}}$

ONE SHIP ONLY FOR AD

HORIZONTAL LOADS AT EACH PILE BENT

$$\text{NO. BENTS} = \frac{1{,}200 \text{ LENGTH}}{40' \text{ SPACING}} - 1 \text{ @ CENTER} = 29 \text{ BENTS}$$

ASSUME UNIFORM DISTRIBUTION LOAD INTO EACH BENT

CASE I : AD NEAR SHORE

$$\frac{\text{TOTAL LOAD}}{\text{NO. BENTS}} = \frac{2434^{K}}{29} = 84^{K}/\text{BENT}$$

CASE II : AD NEAR CHANNEL

$$\frac{2730^{K}}{29} = 94^{K}/\text{BENT}$$

WORST CASE : AD LOCATED NEAR THE CHANNEL AND UNIFORM LOADING IS NOT ASSUMED.

∴ LOAD OF 1947^{K} RESISTED BY 16 PILE BENTS

$$\frac{1947^{K}}{16} = 122^{K}$$ USE 125^{K}/BENT.

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PROJECT:		SHEET: B-1
ITEM:	CONCRETE PIER	OF
DESIGN:		REVISION:
DATE:	R.Z.	

APPENDIX B
PRESTRESSED CONCRETE FLOATING PIER CALCULATIONS

DESIGN CRITERIA

SERVICEABILITY STATE: UNDER NORMAL LIVE LOADING CONDITIONS, THE STRUCTURE WILL OPERATE IN A STRESS RANGE FROM ZERO TENSION TO 0.45 f'_c COMPRESSION.

SERVICEABILITY STATE; UNDER EXTREME DESIGN LOADING CONDITIONS, THE STRUCTURE WILL EXPERIENCE STRESSES IN THE RANGE $6\sqrt{f'_c}$ TENSION TO 0.45 f'_c COMPRESSION.

LOADING CRITERIA

LIVE LOADS ON MAIN DECK: 600 PSF LOCAL
90 TON TRUCK CRANE
HS-44-20 TRUCK
20 TON FORK LIFT TRUCK

LIVE LOADS ON LOWER DECK: SAME AS ABOVE EXCEPT NO CRANE.

HYDROSTATIC PRESSURE LOAD: EXTREME 24 FT. HEAD
OPERATING 13 FT. HEAD

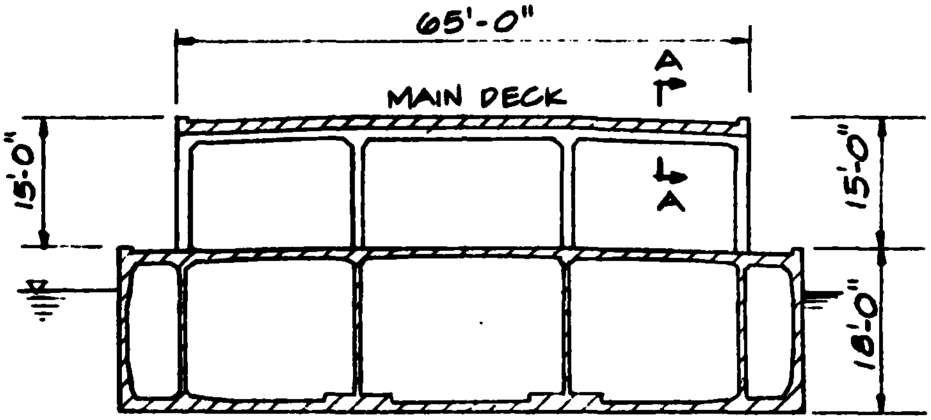
LATERAL LOADS: WIND & CURRENT LOADS ACTING TOGETHER; BERTHING & MOORING FORCES

LONGITUDINAL LOADS: WIND LOADS.
BERTHING & MOORING FORCES.

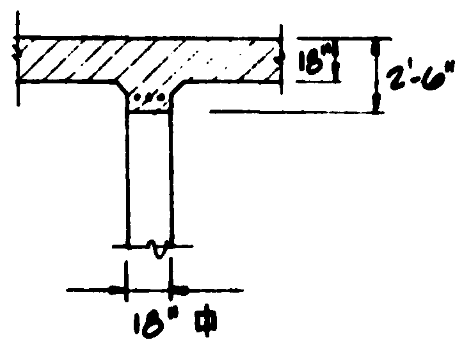
MATERIALS

LIGHTWEIGHT CONCRETE: DENSITY 125 PCF, $f'_c = 5000$ PSI
REBAR: GRADE 60
PRESTRESS (P/S): STRAND $f'_s = 270$ KSI
ROD $f'_s = 150$ KSI

DETERMINE WEIGHT & DRAFT OF DESIGN SECTION



COLUMNS CENTERED OVER PONTOON WALLS @ 20' C-C, BULKHEAD WALLS @ 40' C-C.



SECTION A-A

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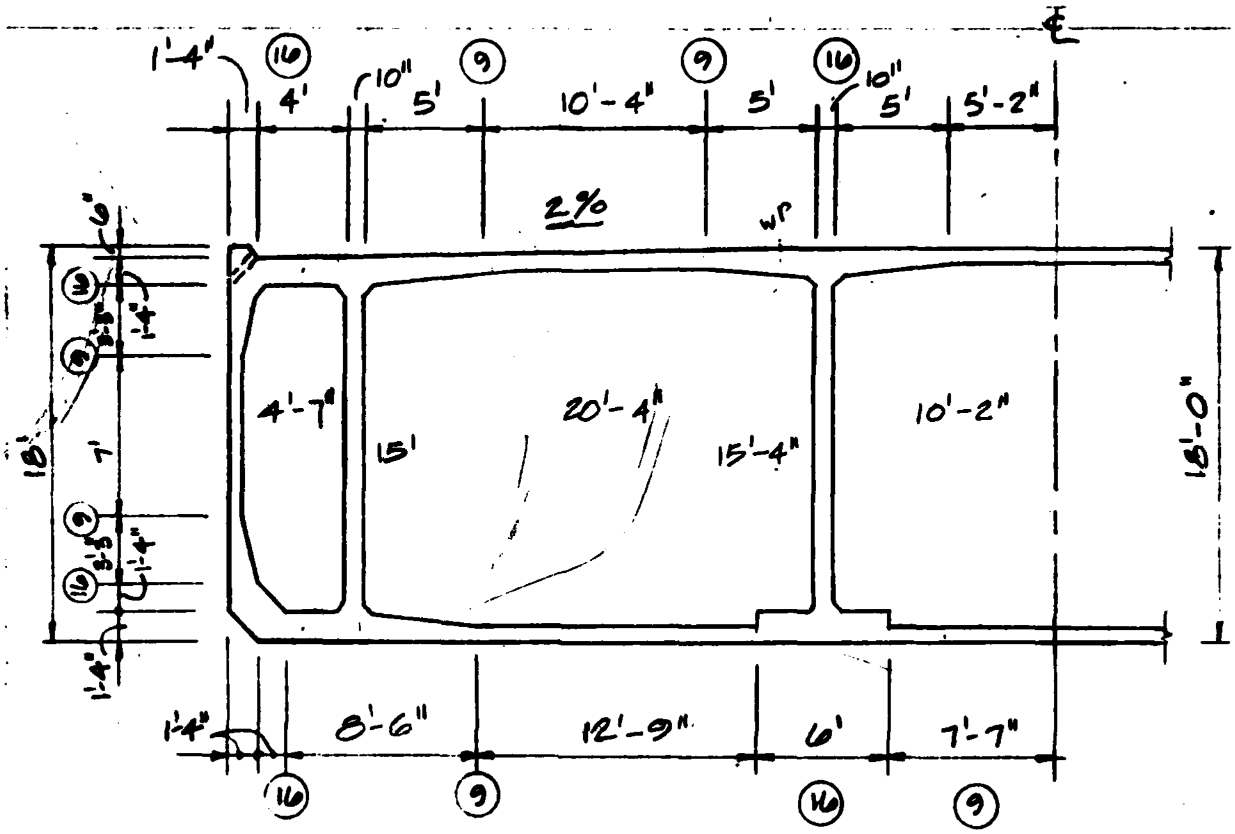

PONTOON

$$A = [16''(19.67) + 12.5'(26.5) + 10''(30.33) + 9''(42.83)]\left(\tfrac{2}{12}\right)$$

$$= 222.5\ FT^2 + 0.5\ FT^2\ (CURB) + 0.5\ FT^2\ (FILLETS) = 223.5\ FT^2$$

ADD MAIN DECK: @ 65 FT WIDE $65(1.5) = 97.5\ FT^2$

ADD CURBS: 2 @ 2'-0" ϕ $= 4.0\ FT^2$

$223.5 + 97.5 + 4 = 325\ FT^2$

ADD 2% (TOLERANCE) $= 325\ (1.02) =$ 33[illegible] FT^2

WT/FT. = (0.125)(332) = 415 K/FT

Σ W = 41.5 (1200) = 49,800 K LONG. WALLS & DECK

ADD BULKHEADS, COLUMNS AND BEAMS:

BULKHEADS @ 40' C-C.

VERTICAL PRESTRESS ONLY, T = 10" @ 40 FT. C-C.

1200 ÷ 40 = 30 CELLS.

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20 INTERIOR BULKHEADS, 2 END WALLS: SAY 12" END WALLS

$\Sigma T = \frac{10}{12}(20) + \frac{12}{12}(2) = 26.17$ FT.

$A_{NET} = 18(75) - 310 = 1040$ FT^2

WEIGHT = 26.17 (1040) (.125) = 3400K BULKHEAD WALLS

WET WELLS: @ 20.33 × 15.75 = 320 FT^2
@ 10" THICK $30(320)(\frac{10}{12})(.125)$ = 1000K WET WELLS

BEAM AND COLUMN SYSTEM: @ 20 FT. C-C

$A = \frac{[4(12.5)+65](18) - 22(6)}{12} = 161.5$ FT^2

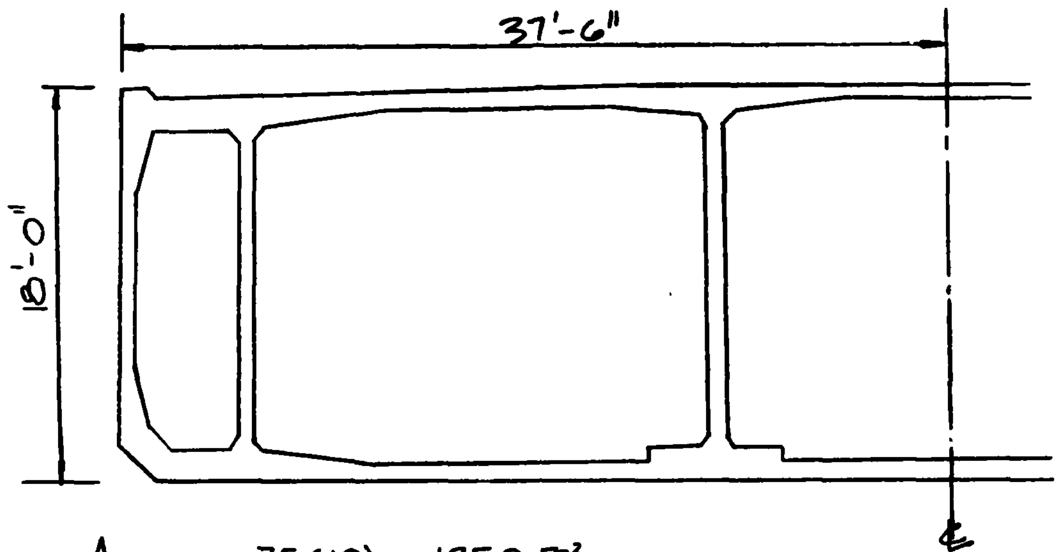
$\Sigma W \cong 61(1.5)(161.5)(.125) = 1850^K$

TOTAL WEIGHT: 49,800 + 3400 + 1000 + 1850 = 56,050 K
ADD 2000K FOR MANHOLES & UTILITY PIPES, ETC.
GRAND TOTAL ΣW = 58,000 K

NET BUOYANT AREA:
PILE WELLS ARE 8'-0" × 20'-4"
A = 1200 (75) - 30(8)(20.33) = 85,120 FT^2 @ .064 KSF/FT.
85,120 (0.064) = 5450 K/FT
DRAFT = $\frac{58,000}{5450}$ = 10.64 FT. Δ @ 5'-0" FREEBOARD:
7.35 - 5.0 = 2.35 FT.
2.35 (5450) = 12,800 K

SPREAD OVER GROSS AREA OF MAIN DECK
$\frac{12,800 K}{65'(1200')}$ = 164 PSF L.L.

LONGITUDINAL SECTION PROPERTIES
(GROSS SECTION FOR BENDING)



$A_{GROSS} = 75(18) = 1350$ FT^2

$- A_{CELLS} = 1122.2$ FT^2

$+ A_{FILLETS} = 1.0$ FT^2

NET AREA = 229 FT^2

NET I: $I_{GROSS} = \frac{75(18)^3}{12} = 36,450$ FT^4

Σ-I = 2 (12,212) = 24,424 FT4

$\boxed{\text{NET I} = 12,026 \text{ FT}^4}$

CG @ 9FT FROM BOTTOM (MIDHEIGHT)

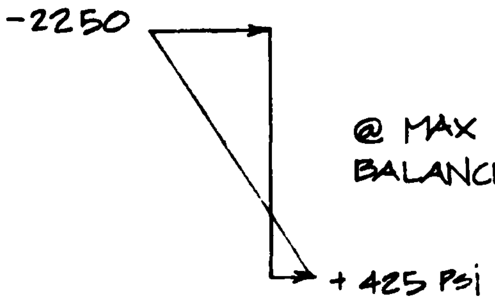
$S_T = S_B = \frac{12,026}{9} = 1,336$ FT^3

DETERMINE MAX ALLOWABLE BENDING MOMENT:

FOR 5000 psi CONCRETE,

TENSION $6\sqrt{f'_c} = 425$ psi

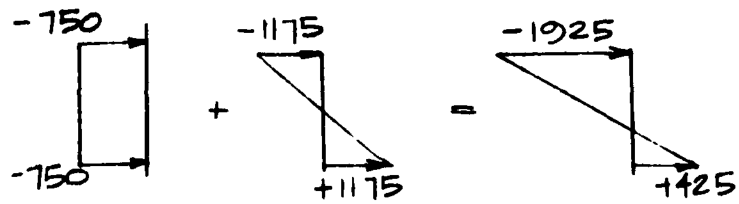
COMPRESSION $0.45 f'_c = 2250$ psi



@ MAX M

BALANCED CONDITION

ASSUME P/S = 750 psi



MAX BENDING STRESS = ± 1175 psi

$M_{MAX} = f_b \cdot S = 1.175$ Ksi $(1336\ FT^3)(144\ \frac{IN^2}{FT^2}) = 226{,}050$K FT.

DETERMINE NO. OF TENDONS REQ.D: SAY $0.6''\phi \times 7$ STRAND @ 246K EA. FOR $0.6 f'_s$

$A_{GROSS} = 229\ FT.^2 = 32{,}976\ IN^2$

P REQ'D = 0.75Ksi $(32{,}976\ IN^2) = 24{,}732$ K

NO OF TENDONS = $\frac{24{,}732\ K}{246\ K\ EA.}$ = 100 TENDONS OR 50 PER PONTOON SIDE (CENTER AREA INTERRUPTED BY PILE WELLS)

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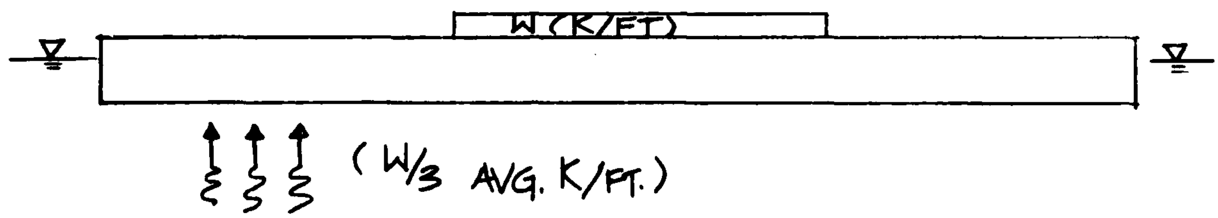
NOTE: DOCK ACTS AS AN ELASTIC BEAM:

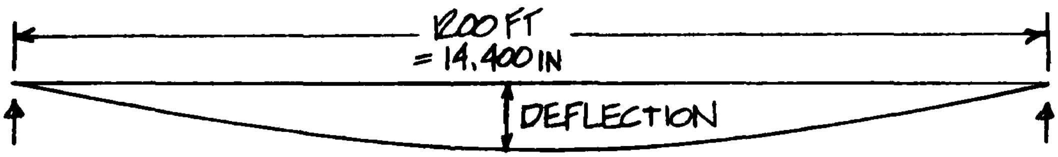
$I_{GROSS} = 12{,}026\ FT^4 = 249{,}371{,}000\ IN^4$

E FOR 5000 PSI, $W_c = 125$ PCF

$E = W_c^{1.5}\, 33\sqrt{f'c} = (125)^{1.5}\, 33\sqrt{5000} = 3{,}260$ KSI



SAY - DEFLECTED SHAPE IS PARABOLIC: CHECK TOTAL DEFLECTION CAUSED BY TRIANGULAR LOADING RESULTING FROM NONUNIFORM DRAFT DUE TO DEFLECTION



$M_{MAX} = \frac{WL}{6} = 226{,}000$ K FT.

$W = \frac{6M}{L} = \frac{6(226{,}000)}{1200} = 1{,}130$ K.

$\Delta = \frac{WL^3}{60EI} = \frac{1{,}130(14{,}400)^3}{60(3260)(249.37)(10)^6} = 69'' = \underline{5.7}$ FT

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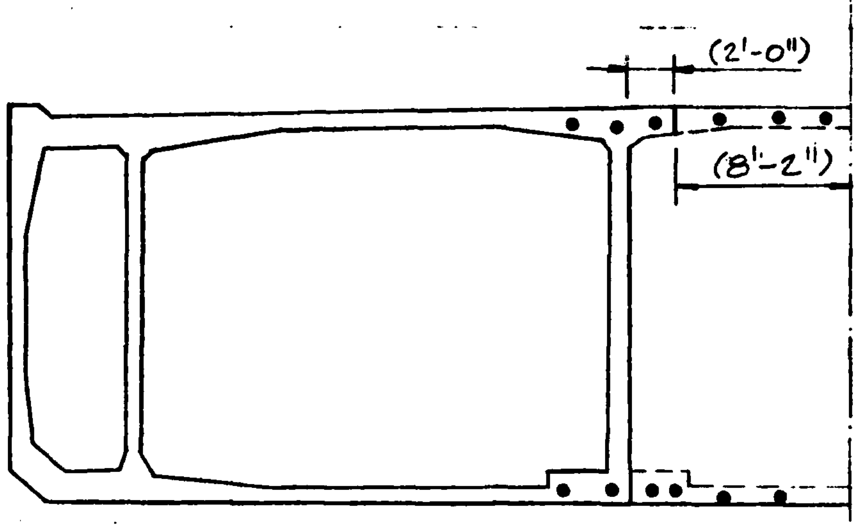
FOR BEAM ACTION: PONTOON SECTION IS INTERRUPTED BY OPENINGS FOR PILE WELLS. STRENGTH IS BASED ON NET SECTION AT WELL OPENINGS.

$\left(2\times(5.17\times0.75+3.0\times0.925)\right) = -13.31\ \text{ft}^2$



$\left(7.58\times0.75+2.58\times1.33\right) = -18.23\ \text{ft}^2$

$A_{NET} = 229 - 13.3 - 18.2 = 197.5\ \text{ft}^2$

NO. TENDONS = 100 − 14 = 86

$P/S = \dfrac{86(246{,}000)}{144(197.5)} = 744\ \text{psi} \cong 750\ (99\%)$

$I \equiv I_{GROSS} - \Sigma I_{o-o} - \Sigma AD^2$

3'-0" 5'-2"
10" 9"
16" 9"
2'-7" 7'-7"

$\Sigma I_{o-o} = \dfrac{36(10)^3 + (62+91)(9)^3 + 31(16)^3}{12} = 22{,}876\ \text{IN}^4$

$\Sigma AD^2 = (62+91)(9)(103.5)^2 + 360(103)^2 + 496(100)^2 = 23{,}530{,}008\ \text{IN}^4$

$\Sigma(-) = \dfrac{-23{,}552{,}884}{(12)^4}\times 2 = -\underline{2{,}272}\ \text{ft}^4$

NET $I = 12{,}026 - 2{,}272 = 9{,}754\ \text{ft}^4$

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SHEET: B-9
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$$S = \frac{9754}{9} = 1084 \text{ FT}^3 = \underline{1,872,827} \text{ IN}^3$$

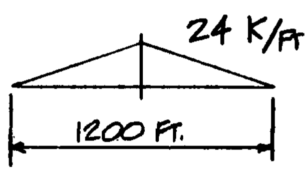
$$M_{MAX} = \frac{1175(1,872,827)}{12000} = 182,300 \text{ KFT}$$

DEFLECTION (BASED ON FULL-SECTION "I"): = 249,371,136 IN^4

$$W = \frac{6M}{L} = \frac{6(182,300)}{1200'} = 911.5 \text{ K}$$

$$\Delta = \frac{WL^3}{60EI} = \frac{911.5(14.400)^3}{(60)(3260)(249.37)(10)^6} = 56'' = 5 \text{ FT.}$$

NOTE: AT MAXIMUM MOMENT ELASTIC DEFLECTION OF 5FT ± OCCURS AT ℄ OF STRUCTURE.
AT 0.064 (75) = 4.8 K/FT. OF DISPLACEMENT, THIS IS EQUIVALENT TO A LOAD OF 5 × 4.8 = 24 K/FT.



4.8 (1200) = 5760 K/FT

$$\Sigma W = \frac{24(1200)}{2} = 14,400 \text{ K. OR } \frac{14,400}{400} = 36.0 \text{ K/FT. OVER}$$
CENTER 400 FT.

OR MAIN DECK LOADED AT $\frac{36 \text{ K}}{65 \text{ FT.}}$ = 550 PSF. FOR CENTER 400 FT.

CHECK: 400 (36) = 14,400 K

$$\text{AVG. } \Delta \text{ DRAFT} = \frac{14,400}{5760} = 2.5 \text{ FT. FOR ENTIRE LENGTH.}$$
(IF PIER WAS RIGID)

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PROJECT:
ITEM: STRUCTURAL - PONTOON
DESIGN: BOTTOM SLAB
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SHEET: B-10
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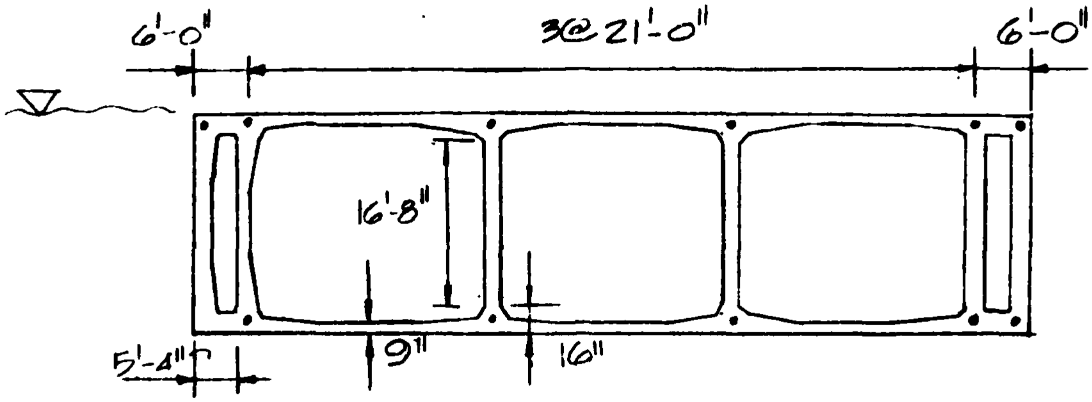


BOTTOM SLAB : DESIGN FOR 19 FT. HEAD WITH $6\sqrt{f'_c}$ TENSION ALLOWED IN CONCRETE :

MAX TENSION = $6\sqrt{5000}$ = 424 psi

TRY : 9" SLAB W/16" LAUNCHES :

NOTE : FEM = DISTRIBUTED MOMENT

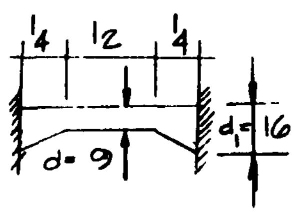
$+M_{MAX}$: W_H = 19 (.064) = 1.216 KSF.
W CONCRETE @ 9" - $\frac{9}{12}$ (.125) = 0.094 KSF.
NET = 1.122 KSF.

HAUNCHED SECTION MOMENT DISTRIBUTION :

$a = 0.25, \frac{d}{d_1} = 0.56$

$\therefore f = 0.099$

$\therefore -M = 0.099\, WL^2$



(FROM CALTRANS BDPM, 3R ED, APPENDIX 2-H-3)

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ITEM:		OF
DESIGN:	BOTTOM SLAB (CONT)	REVISION:
DATE:	R.Z.	

$\therefore\ -M = 0.099\,(1.122)(21)^2 = -\underline{49.0}$ KFT./FT.
@ t = 16": P/S @ 5½" UP: $\left(\frac{16}{6} = 2.67\right)$

SAY $e = 2\frac{1}{2}"$

$A = 12(16) = 192$ IN²

$S = \frac{12(16)^2}{6} = 512$ IN³

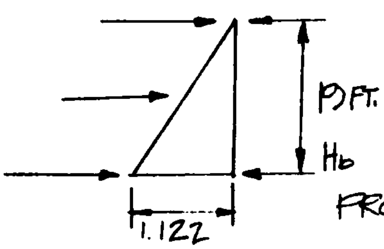
$$\overset{T}{\frac{M}{S}} - \left(\overset{C}{\frac{P}{A}} + \overset{C}{\frac{Pe}{S}}\right) = -.424 \text{ Ksi}$$

$$P\left(\frac{1}{A} + \frac{e}{S}\right) = +.424 - \frac{49(12)}{512} = -0.7244$$

$$\left(\frac{1}{A} + \frac{e}{S}\right) = \frac{1}{192} + \frac{2.5}{512} = .0101$$

$$\therefore P = \frac{0.7244}{.0101} = 71.8 \text{ K/FT. (REQ'D)}$$

NOTE: PART OF "P" IS PROVIDED BY HYDROSTATIC PRESSURE;



$W = \frac{1.122(19)}{2} = 10.66$ K/FT.

$H_b = \frac{2}{3}(10.66) = 7.1$ K

PROVIDE P/S @ 4×0.6 @ 21" = 80.37 K @ 1'-9" C-C.

$\therefore$ REQ'D NET P/S = 71.8 − 7.1 = 64.6 K/FT.

CHECK:

$$\frac{80.37 + 7.1}{192} \pm \frac{49(12)}{512} \mp \frac{80.37(2.5)}{512} = 1212 \text{ PSIC} / 300 \text{ PSIT}$$

(OK)

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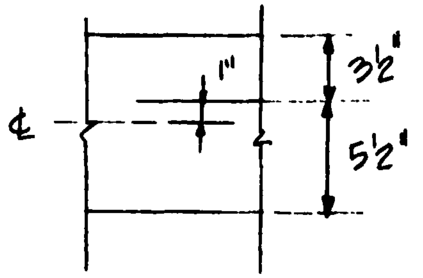
OF:
REVISION:

CHECK +M: t = 9"
@ e = 1"



$SBM = \dfrac{1.122(21)^2}{8} = 61.85$ KFT.

∴ $+M_{℄} = 61.85 - 49.0 = 12.85$ KFT

$A = 108\ IN^2, \quad S = 162\ IN^3$

$$\underset{T/b}{\dfrac{80.37+7.1}{108}} \; {+ \atop -} \; \underset{b}{\dfrac{12.85(12)}{162}} \; {- \atop +} \; \underset{T}{\dfrac{80.37(1.0)}{162}} = 1266 \text{ PSIC (BOTTOM)} / 354 \text{ PSIC (TOP)} \quad (OK)$$

CHECK @ ZERO HEAD.

$$\dfrac{80.37}{108} \; {+ \atop -} \; \dfrac{80.37}{162} = 1240 \text{ PSIC} / 248 \text{ PSIC} \quad (OK)$$

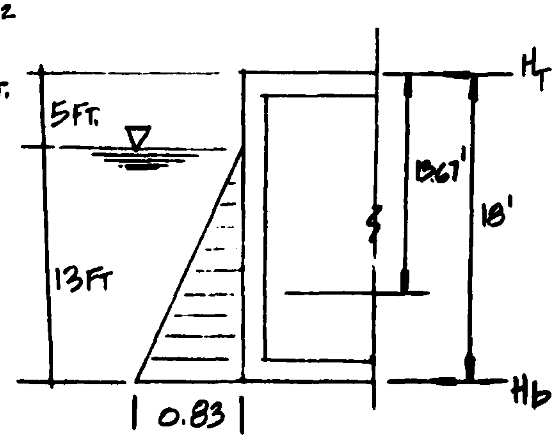
CHECK STRESS AT NORMAL DRAFT: @ 5 FT. FREEBOARD.

$W = 13(.064) - .094 = 0.738$ K/FT.²

$-M = 0.738\ (.099)(21)^2 = 32.2$ K/FT.

$\dfrac{13}{2}(0.83) = 5.4K$

$H_b = 5.4K\left(\dfrac{13.67}{18}\right) = 4.1K$



$$∴ \dfrac{4.1+80.37}{192} \; {+ \atop -} \; \dfrac{32.2(12)}{512} \; {- \atop +} \; \dfrac{80.37(2.5)}{512} = 802 \text{ PSIC} / 77 \text{ PSIC} \quad (OK)$$

∴ REQ'D TENDONS: 0.6ϕ ×4 @ 1'-9" C-C.

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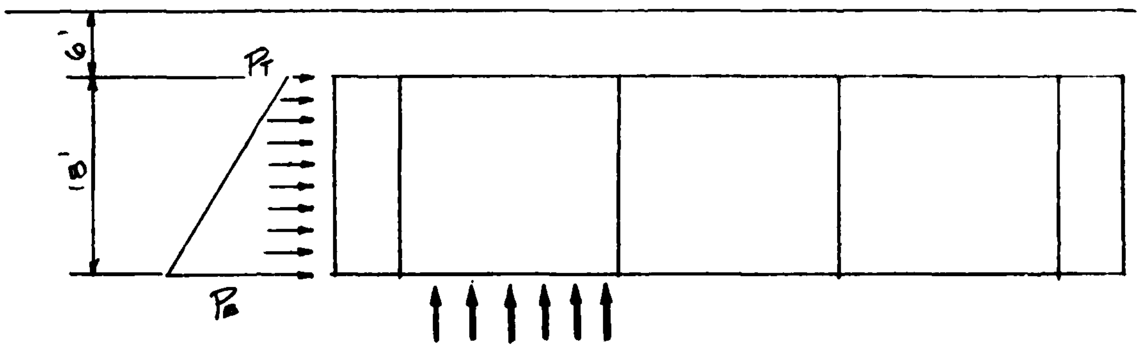
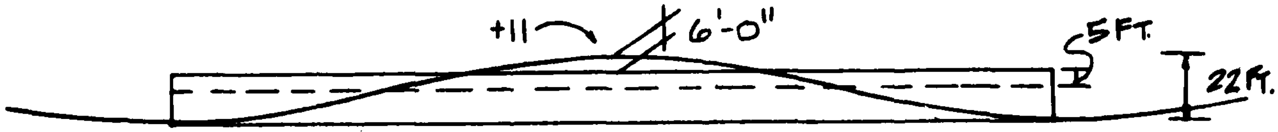
OF
REVISION:

CHECK BOTTOM SLAB OF 300 FT. LONG PIER UNIT UNDER TOW, 22 FT WAVE:



SAY - 300 FT. SECTION, 22 FT. WAVE:
- STILL-WATER FREEBOARD @ 5FT.:



CREST OF WAVE @ 6FT. OVER LOWER DECK:
DESIGN MOMENTS WERE BASED ON 19 FT. HEAD:
WAVE CRESTS 6FT. OVER DECK, PRODUCING 22 FT. HEAD.

$24\ (.064) = 1.536$ KSF

$\Delta - M = .099(1.536)(21)^2 = -67$ KFT

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OF:
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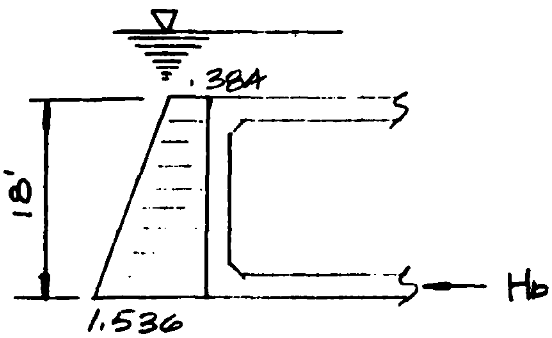
SUBTRACT −M FROM CONCRETE SLAB:

AVG $t = 12.5''$ $\therefore$ $W = 0.13$ KSF

$+M = 0.099(.13)(21)^2 = -5.7$ K-FT.

$\therefore$ NET $-M = -67 + 5.7 = -61.3$ K-FT.

$\frac{P}{A}$ DUE TO SIDE PRESSURE:



$H_b = \frac{1}{2}(.384)(18) + \frac{2}{3}\cdot\frac{1}{2}(1.536 - .384)(18) = 10.40$ K/FT.

P/S = 4 − 0.6 ϕ STRANDS @ 21'' C-C @ 0.6 F's

$= 140.6 \times \frac{12}{21} = 80.38$ K/FT.

$A = 16(12) = 192$ IN²

$S = \frac{12(16)^2}{6} = 512$ IN³

$e = \frac{16}{2} - 5.5 = 2.5$ IN

$$\Sigma \frac{P}{A} \pm \frac{Pe}{S} \mp \frac{M}{S} = \frac{10.40+80.38}{192} \pm \frac{80.38(2.5)}{512} \mp \frac{61.3(12)}{512}$$

$= 570$ PSI T, 1520 PSI C

TENSION EXCEEDS ALLOWABLE OF 425 Psi BY 33% ±

INCREASE P/S FOR BOTTOM SLAB FOR THE 300 FT LONG PIER UNIT: USE 4 − 0.6 ϕ STRANDS @ 18'' C-C

$P/S = (140.6)\frac{12}{18} = 93.7$ K/FT.

$$= \frac{10.4 + 93.7}{192} \pm \frac{93.7\,(2.5)}{512} \mp \frac{61.3(12)}{512}$$

$= 440$ PSIT, 1520 PSIC

TENSION EXCEEDS ALLOWABLE OF 425 Psi BY 3.5%

O.K. USE
4 - 0.6"ϕ STRANDS @ 18" C-C.

CHECK +M SECTION:

$$+M = \frac{WL^2}{8} - 61.3 = \frac{1.406(21)^2}{8} - 61.3 = +16.2 \text{ KFT}$$

$(W = 1.536 - 0.130 = 1.406)$

$A = 9(12) = 108 \text{ IN}^2 \qquad S = \frac{12(9)^2}{6} = 162 \text{ IN}^3$

$e = \frac{9}{2} - (9 - 5\tfrac{1}{2}) = \underline{1}"$

$$\frac{P}{A} \pm \frac{Pe}{S} \mp \frac{M}{S} = \frac{10.40 + 80.38}{108} \pm \frac{80.38(1)}{162} \mp \frac{16.2(12)}{162}$$

$= 140$ PSIC / 1545 PSIC OK

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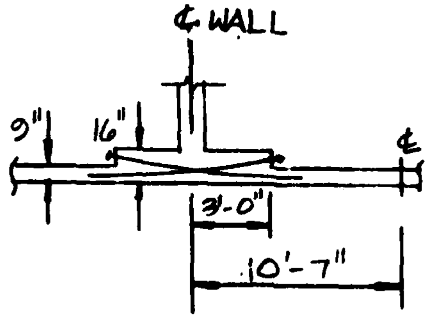
SHEET: B-16
OF
REVISION:

RECHECK TRANSVERSE P/S @ ℄ INTERIOR WALL:
d = 18'-0": WITH PRISMATIC HAUNCH, FOR 24' PRESS. HEAD

BOTTOM SLAB: @9" = .094 K/FT.

NET = $-w$ = 1.536 − 0.094 = 1.442 K/FT.

$hc = 9''$
$rhc = 16 - 9 = 7$
$\therefore r = \frac{7}{9} = 0.78$
$a = 0.28$



FEM = −M = 0.105 WL^2
(FROM PCA STRUCTURAL BUREAU, ST 81)

$\therefore -M = .105(1.442)(21)^2 = -\underline{66.8}$ Kft./ft.

PRESTRESS @ ℄ WALLS:
NO. OF STRANDS IS DOUBLED: (DUE TO LAP).

$\therefore P = 2(80.4) = 160.8K$

$e \approx 2''$

$$\frac{P}{A} \pm \frac{Pe}{S} \mp \frac{M}{S} = \frac{160.8+10.4}{192} \pm \frac{160.8(2)}{512} \mp \frac{66.8(12)}{512}$$

= 320 PSIC, 1880 PSIC OK.

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PROJECT:	
ITEM:	STRUCTURAL - PONTOON
DESIGN:	LOWER DECK
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SHEET:	B-17
OF	
REVISION:	

CHECK LOWER DECK, @ SAME T + P/S AS BOTTOM SLAB:

UNIFORM 600 PSF LIVELOAD.

DEAD LOAD - "T" AVG $\cong \frac{9+16}{2} = 12.5''$ = 125 PSF

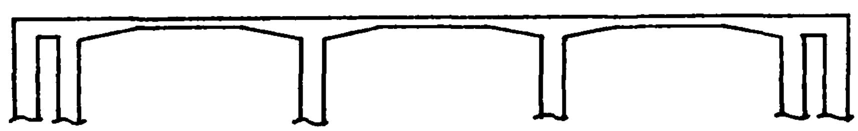
$\therefore$ D+L = 725 PSF.

FEM: $-M = .099(.725)(21.)^2 = -31.65$ K FT/FT.

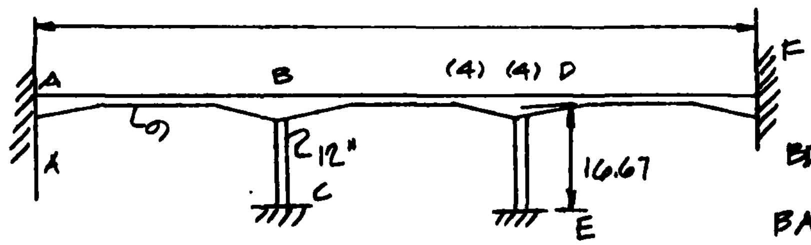
SBM: $-\frac{725(21.)^2}{8} = +39.96$ KFT./FT

$\therefore +M = 39.96 - 31.65 = +8.32$ KFT.

LOAD ALTERNATE PANELS:



$\cong$

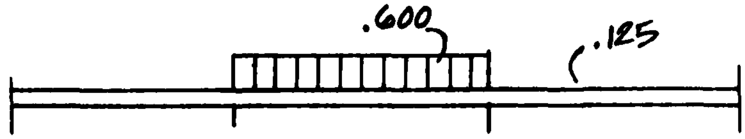


BD: $\frac{8.5(9)^3}{21} = 295$

BA: $\frac{8.5(9)^3}{21} = 295$

BC: $\frac{4(12)^3}{16.67} = 414$

MAX +M: LOAD BD.



C = 0.67

K = 8.5

$\frac{EI}{L} =$

	SAY
0.29	.3
0.29	.3
0.42	.4

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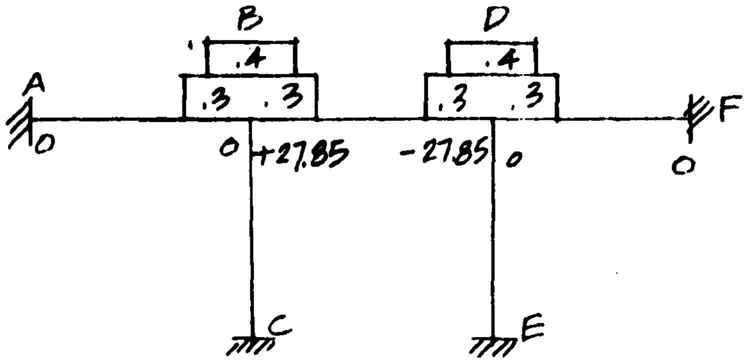


(C = 0.67)

A	B			D			F
AB	BA .3	BC .4	BD .3	DB .3	DE .4	DF .3	FD
0	0	0	+26.2	-26.2	0	0	0
0	-7.9	-10.5	-7.9	+7.9			
-5.3	0	0	+5.3				
0	-1.6	-2.1	-1.6				
-1.1	0	0	+1.1				
0	-0.3	-0.4	-0.3				
-0.2	0	0	+0.2				
6.6	-9.8	-13.0	+23.0				

$\therefore -M_L = 23.0$ KFT. $\qquad SBM_L = \frac{0.6(21.1)^2}{8} = 33.1$

$+M_L = 33.1 - 23.0 = +10.1$ KFT.

$+M_P = \frac{.125}{.725}(8.32) = +1.4$

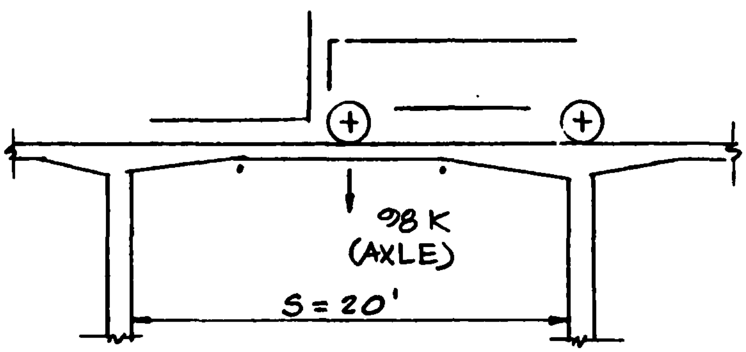
$\therefore +M_{D+L} = 10.1 + 1.4 = +11.5$ KFT. $\qquad$ ($P/S = 80.4$ K/FT.) $\quad$ ($e = 1''$)

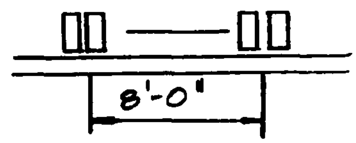
$$\frac{P}{A} \pm \frac{M}{S} \mp \frac{Pe}{S} =$$

$$\frac{80.4}{108} \pm \frac{11.5(12)}{162} \mp \frac{80.4}{162} = 1100 \text{ PSIC} / 380 \text{ PSIC} \quad \text{(OK)}$$

CHECK LOWER DECK FOR 20 TON FORK LIFT:

AXLE LOAD = 98 K (FROM DM25)
ON 8'-0" SPACING





$S = 20 FT.$ $\therefore E = 4.0 + .06\,(20) = 5.2\ FT.$

$\therefore \Sigma E$ FOR AXLE $= 8 + 5.2 = 13.2$ FT.

$\therefore P = \frac{98}{13.2} = 7.42$ K/FT.

SAY - $\underline{8}$ K/FT. (WITH 8% IMPACT)

SBM: $+M = \frac{8(21)}{4} = \underline{42}$ K FT/FT.

FEM: $V = 0.5$

MIN/MAX $a/d = 0.56$ $a = 0.25$ FEM $= .15(21)8) = -\underline{25.2}$ KFT.

$f = .15$

PLACE LOAD IN CENTER BAY AND DISTRIBUTE MOMENTS:

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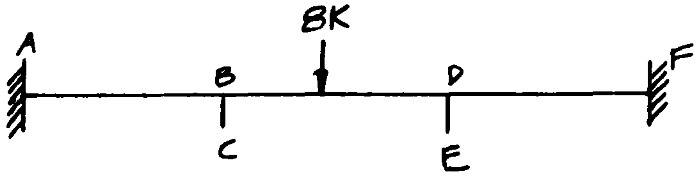
OF
REVISION:



A	B			D
	BA 0.3	BC 0.4	BD 0.3	
0	0	0	+25.2	
	-7.6	+10.0	-7.6	
-5.1	0	0	+5.1	
0	-1.5	-2.0	-1.5	
-1.0	0	0	+1.0	
0	-0.3	-0.4	-0.3	
-0.2	0	0	+0.2	
		-0.1		
-6.3	-9.4	-12.5	+22.1 KFT.	

$\therefore \ +M_L = 42 - 22 = +\underline{20}$ KFT.

$+M_D = \dfrac{.125}{.725}(8.3)2 = +\underline{1.4}$ KFT.

$M_{D+L} = \qquad +21.4$ KFT

STRESSES: $\dfrac{P}{A} \pm \dfrac{M}{S} \mp \dfrac{Pe}{S} = \dfrac{80.4}{108} \pm \dfrac{21.4(12)}{162} \mp \dfrac{80.4(1)}{162}$

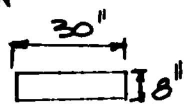
$= 1833$ PSIC / 344 PSIT (OK)

CHECK PUNCHING SHEAR ON 9' SLAB FOR 49 KIP WHEEL LOAD

CONTACT PATCH: SAY - TIRE PRESSURE = 200 PSI

$A_c = \frac{49000}{0.2} = 245 \text{ IN}^2$

SAY - 30" × 8"



"d" = 5.5"

$\therefore$ PERIMETER = $2[(30+5.5)+(8+5.5)] = 98''$

CHECK @ $V_c = (0.6\sqrt{f'c} + 700 \frac{V_u d}{M_u})(bd)$

$V_u = 1.7(49) = 83\text{ K}$ $\quad M_u = 1.4(1.4) + 1.7(20) = 36\text{ K}'$

$\therefore 0.6\sqrt{5000} + 700\left(\frac{83 \times 5.5}{36(12)}\right) bd = 782\ bd$

MAX ALLOWABLE = $5\sqrt{5000}\ bd\ (0.85) = 300\ bd$ CONTROLLING EQ.

LT. WT. SAND/CONCRETE FACTOR

$V_n = V_c = 0.300\,(98)(5.5) = 161.7\text{ K}$ $\qquad \phi = 0.85$

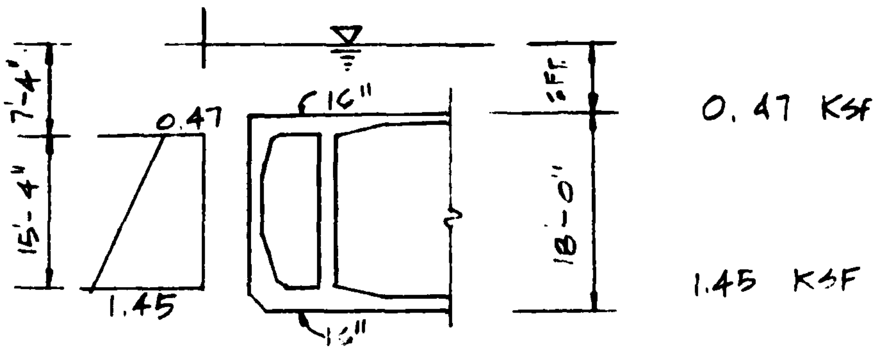
$\therefore V_u = \phi V_n = 0.85(161.7) = 137.4\text{ K}$

MAX LOAD = $\frac{137.4}{1.7} = 81\text{ K} \gg 49\text{ K}$ OK @ 9' SLAB

EXTERIOR WALLS

EXTERIOR WALLS:

DESIGN FOR HYDROSTATIC HEAD PRODUCED BY 22ft. WAVE ON 300 FT SECTION UNDER TOW:



HAUNCHED SECTION :

NOTE - ASSUME FULL FIXITY AT CORNERS FOR NEG. MOMENT:

FEM @ $a = 0.25$, $d/d_1 = 9/16$:
$f = 0.10$ (SAME AS SLAB).

FEM FOR PRISMATIC SECTION:

$$-M \text{ BOTTOM} = -\left[\frac{0.47(15.33)^2}{12} + \frac{0.98(15.33)^2}{20}\right] = -20.7 \text{ K/FT.}$$

APPROXIMATE @ $f = .0833$

-M FOR HAUNCHED SECTION:

$$\frac{0.10}{0.083}(20.7) = -\underline{25.0} \text{ KFT.}$$

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−M @ TOP:

(PRISMATIC):

$$-\left[\frac{0.47(15.33)^2}{12} + \frac{0.98(15.33)^2}{30}\right] = -16.9 \text{ KFT}$$

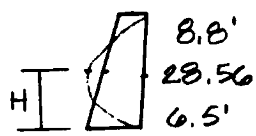
(HAUNCHED): $\frac{.10}{.083}(16.9) = -\underline{20.3}$ KFT.

SBM: $= \frac{W \square L^2}{8} + \frac{2W\Delta L}{9\sqrt{3}}$ $\qquad W\Delta = \frac{0.98(15.33)}{2} = 7.5$

$$\frac{.47(15.33)^2}{8} + \frac{2(7.5)(15.33)}{9\sqrt{3}} = +28.56 \text{ K}'$$

@ $H = 15.33\left(1 - \frac{1}{\sqrt{3}}\right) = 6.5$ FT.

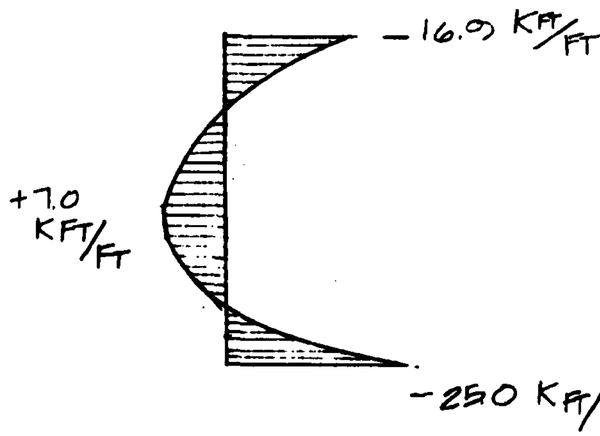
∴ +M



+M = 28.56 KFT.

$$+M = 28.56 - 16.9 - \frac{8.8}{15.3}(25.0 - 16.9) = +\underline{7.0} \text{ KFT.}$$

Mdgm:



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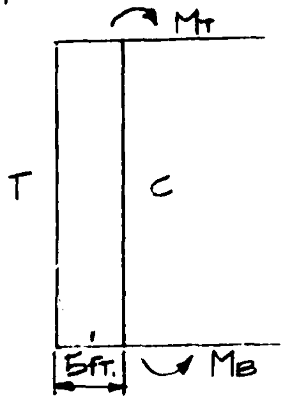
SHEET: B-24
OF
REVISION:

ADDITIONAL TENSION IN WALL FROM SLAB STRESS:

$M_B = -61.3$ K FT.

$T = \dfrac{61.3\ K'}{5\ FT.} = 12.3\ K/FT.$



PRESTRESS FORCE:

80.4 K/FT. @ "e" = $\underline{2\tfrac{1}{2}}$"

A = 192 IN^2; S = 512 IN^3

$\therefore\ f = \dfrac{P}{A} \pm \dfrac{Pe}{S} \mp \dfrac{M}{S} = \dfrac{80.4-12.3}{192} \pm \dfrac{80.4(2.5)}{512} \mp \dfrac{25(12)}{512} =$

161 PSIC, 548 PSIC AT LOWER CORNER. (OK)

$\therefore$ USE $\underline{9}$" SLAB WITH 16" HAUNCHES FOR WALL.

NOTE: STRESSES ARE VERY LOW, BUT RESERVE STRENGTH IS REQUIRED FOR RESISTANCE TO COLLISION DAMAGE.

PRESTRESS @ 1'-9" C-C: Gr. 150 DWDG

REQ. $61\ K \times \dfrac{21}{12} = 107$ K/UNIT:

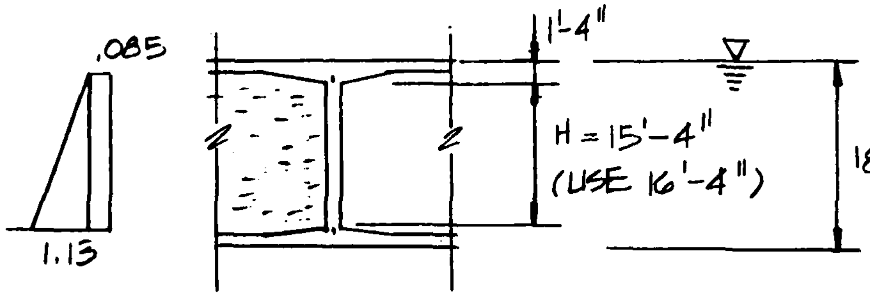
SAY - ONE - $1\tfrac{3}{8}$ ϕ BAR @ 0.6(237) = 142 K

COULD USE $\underline{1\tfrac{1}{4}}$ ϕ BAR @ 11$\underline{2.5}$ K

INTERIOR WALLS

CHECK INTERIOR LONG'L WALL

AT CONSTANT 9" THICKNESS: FULL HEAD.
(SWAMPED CONDITION).



17.67(.064) = 1.13

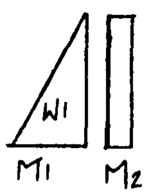
1.33(.064) = .085
16.33(.064) = 1.045

FEM @ BASE & TOP OF WELL:

$$\text{TOP: } \frac{1.045\,(16.33)^2}{30} + \frac{.085\,(16.33)^2}{12} = 11.18 \text{ KFT.}$$

$$\text{BOTTOM: } \frac{1.045\,(16.33)^2}{20} + \frac{.085\,(16.33)^2}{12} = 15.83 \text{ KFT.}$$

SBM



$$+M_1 = \frac{2WL}{9\sqrt{3}} = \frac{2(8.53)(16.33)}{9\sqrt{3}} = 17.88$$

$$@\ \frac{16.33}{\sqrt{3}} = 9.43 \text{ FT. DOWN.} = 1.26 \text{ FT. BELOW } ℄$$

$$+M2 = \frac{0.85(16.33)^2}{8} = 2.83$$

$$W = \frac{1.045(16.33)}{2} = 8.53$$

$$\therefore \Sigma +M = 20.7 \text{ KFT.}$$

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PROJECT:
ITEM:
DESIGN: INTERIOR WALL (CONT.)
DATE: R.Z.

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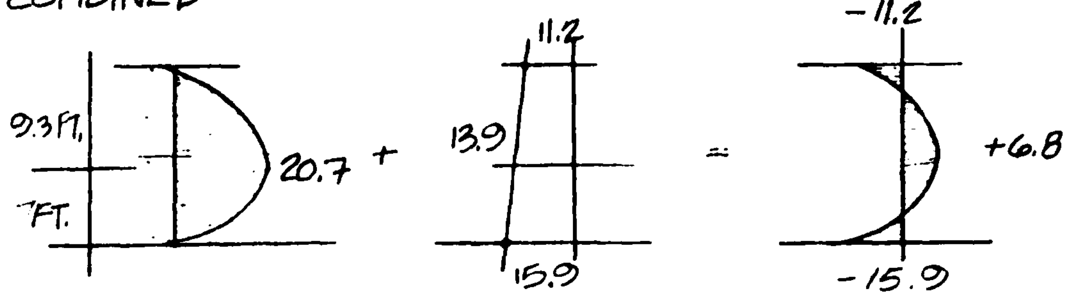
OF
REVISION:

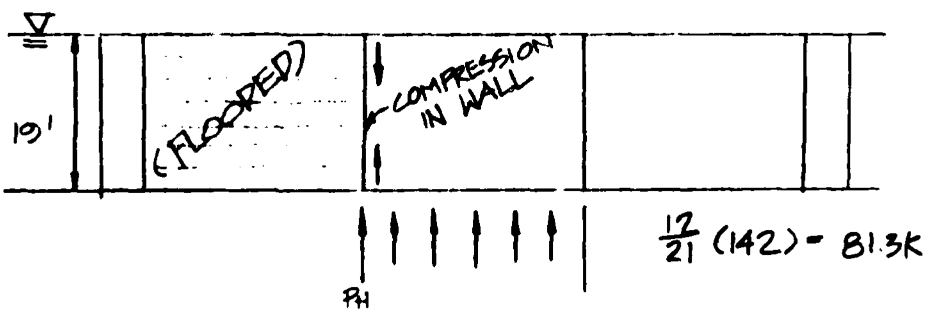
COMBINED:



∴ MAX M @ BASE = −15.9 KFT/FT

NOTE: HYDROSTATIC PRESSURE LOADS WALL: SAY – 10 FT. EFFECTIVE WIDTH

$P_H = 10(19)(.064)$
$= 12.2$ K/FT.



$\underline{10}"$ WALL $A = 120$, IN^2, $S = 200$ IN^3 $e = \underline{0}$

SAY – $1\frac{3}{8}$ φ RODS @ 21' SPACING!

$P = P/S = \underline{81.3}$ K/FT. $\Sigma P = 12.2 + 81.3 = 93.5$

$$\frac{P_H + P}{A} \pm \frac{M}{S} = \frac{93.5}{120} \pm \frac{15.9(12)}{200} = 175 \text{ PSIT} / 1733 \text{ PSIC} \quad \text{OK}$$

TRY $\underline{9}"$ WALL:

$A = 108 IN^2$; $S = 162$

$$\frac{93.5}{108} \pm \frac{15.9(12)}{162} = 2044 \text{ PSIC} / 312 \text{ PSIT} \quad \text{O.K}$$

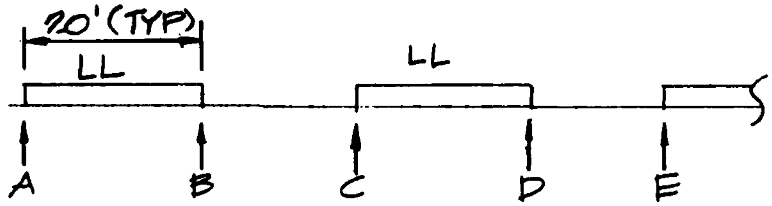
(OK) BUT USE $\underline{10}"$

(REQ'D TO CENTER RODS IN WALL.)

MAIN DECK

MAIN DECK

DESIGN AS ONE-WAY SLAB, LONGITUDINALLY PRESTRESSED - CONTINUOUS OVER 20 FT SPANS.



FOR MAXIMUM MOMENTS: SAY - USE FOUR EQUAL SPAN INFLUENCE LINES - LOAD ALTERNATING SPANS. (12" STRIP)

TRY t = 12" SLAB: LTWT. @ 125 #/FT³

W = 0.125 K/FT. $+M @ 0.4$ PT = $+.0771\,WL^2$

$-M @ B = -.1077\,WL^2$

$+M_D = .0771(.125)(20)^2 = +\underline{3.855}$ KFT.

$-M_D = .1077(.125)(20)^2 = -\underline{5.385}$ KFT.

LIVE LOAD @ 600 PSF:

$+M_L = (.0932 + .0054)(.6)(20)^2 = +\underline{23.664}$ KFT.

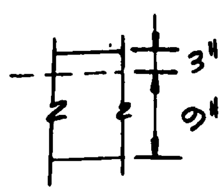
$-M_L = (.0670 + .0491 + .0045)(.6)(20)^2 = -\underline{28.944}$ KFT.

$-M_{MAX} = -5.385 - 28.944 = -\underline{34.33}$ KFT.

PRESTRESS:

CG @ 3" BELOW SURFACE.

$\therefore$ "e" $= \frac{12}{2} - 3 = 2"$



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ITEM:	OF:
DESIGN: MAIN DECK (CONT...)	REVISION:
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$A_g = 12(12) = 144 \text{ IN}^2 \qquad S = \frac{(12)^3}{6} = 288 \text{ IN}^3$

$\frac{M}{S} = \frac{34.33(12)}{288} = 1.4304 \text{ Ksi}$

$@ \; \frac{P}{A} \pm \frac{M}{S} \pm \frac{Pe}{S} = 0 :$

$\frac{P}{A} + \frac{Pe}{S} = \frac{M}{S} = 1.4304$

$P\left(\frac{1}{A} \times \frac{e}{S}\right) = 1.4304 \qquad P\left(\frac{1}{144} + \frac{3}{288}\right) = 1.4304$

$P = \frac{1.4304}{.0174} = 82.4 \text{ K/FT}$

$\frac{82.4}{144} \pm 1.4304 \mp \frac{82.4(3)}{288} = 1.4304^{KSIC}/0.00$ CAN USE LESS P/S

@ $f'_c = 5000$ PSI : ALLOW $0.45(5000) = \underline{2250}$ PSI

@ 600 PSF LOAD: ALLOW TENSION:
SAY - $6\sqrt{f'_c}$

$6\sqrt{5000} = 424$ PSI

$\frac{P}{144} + \frac{3P}{288} = +1.4304 - .424 \qquad P = \frac{(1.4304 - .424)(288)}{5}$

$= 58.0 \text{ K}$

SAY - 58.0 K/FT.

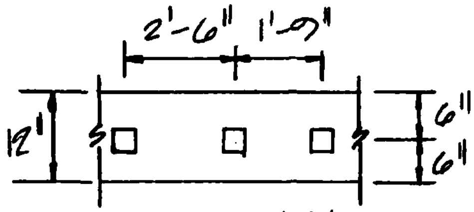
$\frac{58}{144} \pm 1.4304 \mp \frac{58(3)}{288} = 1.230^{KSIC}/0.424 \text{ KSIT}$

$(0.6 f_s' = 141K/\text{TENDON}$

@ $P/_S = 58$ K/FT. : SAY - VSL E6-4

SYSTEM: @ 0.6×4: (4-0.6 ϕ STRANDS PER TENDON, IN ROUND DUCT

TENDONS @ $\frac{141}{58}(12) = 29''$ - SAY 2'-6" C-C



END ANCHORAGE

CHECK DECK FOR VEHICLE LOADINGS:

HS20-44.
AASHTO 1.3.2 (C) CASE B.
$E = 4 + .06S$
$= 4 + .06(20) = 5.2$ FT

WHEEL LOAD = 16K

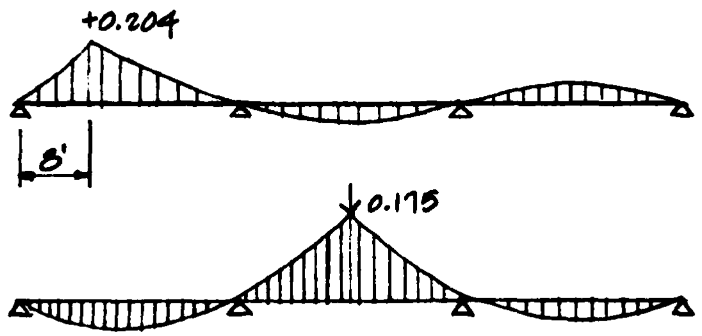
(SAY - 18K WITH IMPACT) 1ST INTERIOR SUPPORT

$-M_{L+S} = (.1029 + .0789)(18)(20) = -65.45$ KFT.

$-M_{L+S}/\text{FT.} = \frac{65.45}{5.2} = -\underline{12.6}$ KFT/FT. $\ll -28.04$

∴ VEHICLE LOADING DOES NOT GOVERN.

CHECK OUTRIGGER LOAD FROM 90-TON TRUCK CRANE
OUTRIGGER LL = 187 KIPS. ON 20' CONTINUOUS SPANS.
SAY - END SPAN : USE 3- EQUAL SPAN INFLUENCE LINE.



END SPAN

INTERIOR SPAN

+ M @ END SPAN = 0.204 (187)(20) = 763 K FT.
+ M @ INTERIOR SPAN = 0.175 (187)(20) = +655 K FT

DISTRIBUTION WIDTH = REF. AASHTO 1.3.2
BASED ON AASHTO DISTRIBUTION FORMULA, CASE "B" $E = 4 + 0.065$
$S = 20\text{ FT} - 1.5\text{ FT} = \underline{18.5\text{ FT}}$
$E = 4 + .06\,(18.5) = 5.11\text{ FT}.$

INTERIOR SPAN :

$$+M = \frac{655}{5.11} = \underline{+128\text{ KFT.}}$$

+M BASED ON AASHTO CASE "A"

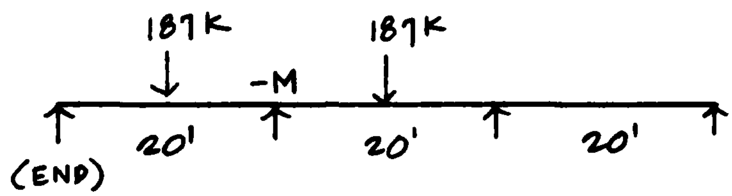
$$M = \left(\frac{S+2}{32}\right) P \times 0.8 \qquad P = 187\text{ K}$$

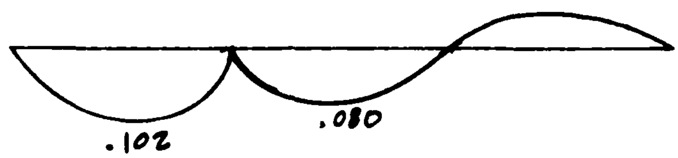
(CONTINUITY FACTOR) $S = 18.5$ FT.

$$M = \frac{18.5+2}{32} \times 187 \times 0.8 = \underline{+96\text{ KFT}}$$

NOTE : AASHTO DISTRIBUTION FORMULAS ARE BASED ON "WESTERGAARD" THEORY :
CASE "A" IS GOOD FOR BOTH POSITIVE AND NEGATIVE MOMENT.

CHECK −M OVER BEAM, USING 3 · EQUAL SPAN INFLUENCE LINE.





$-M = (.102 + .08)(187)(20) = \underline{-681 \text{ KFT}}$

FOR CASE "B" DISTRIBUTE OVER A WIDTH OF 5.11 FT:

$-M_L = \dfrac{681}{5.11} = -133 \text{ K FT.}$

FOR AASHTO CASE "A" (REINFORCEMENT PERPENDICULAR TO TRAFFIC),

$\pm M_L = \pm 96 \text{ KFT/FT.}$

FOR AASHTO CASE "B" (REINFORCEMENT PARALLEL TO TRAFFIC),

$\pm M_L = +128 \text{ KFT,}$

$-M_L = -133 \text{ KFT.}$

SAY - DESIGN FOR AVERAGE OF

ABOVE : $\dfrac{96 + 96 + 128 + 133}{4} = \underline{113 \text{ KFT.}}$

CHECK 12" SLAB WITH P/S AT
"d" = 9" SAY - 12" STRIP
WITH 0.6 ϕ STRAND EVERY 4" (3/FT)
4 - 0.6 ϕ STRAND TENDONS @ 16" C-C

$$W_{pu} = \frac{A_{ps} f_{pu}}{bd\ f'_c} \qquad A_{ps} = .2203(3) = 0.661$$
$$f'_c = 5000\ psi$$

$$W_{pu} = \frac{0.661(270)}{12 \times 9 \times 5} = 0.3305$$

$$f_{ps} = f_{pu}(1 - 0.5\,W_{pu}) = 270\left(1 - \frac{.3305}{2}\right) = \underline{225\ ksi}$$

$$W_p = \frac{A_{ps} f_{ps}}{bd\ f'_c} = \frac{.661(225)}{12(9)(5)} = .276 < 0.3 \quad (OK)$$

$$M_n \cong A_{ps} f_{ps}\left(d - 0.59\,\frac{A_{ps} f_{ps}}{b f'_c}\right)$$

$$M_n \cong \frac{0.661(225)}{12}\left(9 - 0.59\,\frac{.661(225)}{12(5)}\right)$$

$$M_n \cong \underline{93.4\ KFT}$$

$$+M_u = M_D = +\frac{WL^2}{24} = \frac{.125(20)^2}{24} = 2.08\ K\,FT.$$

$$M_u = 1.4(2.08) + 1.7(113) = 195\ KFT.$$

$195 >> 93.4 \quad \therefore$ 12" SLAB <u>NO GOOD</u>

CHECK USING 18" SLAB W/ MORE P/S.

$d = 14"$: P/S = SAY 4 × 0.6φ @ 12" C-C. $A_{ps} = 0.881$

$$W_{pu} = \frac{A_{ps}\, f_{pu}}{b d\, f'_c} = \frac{0.881(270)}{12(14)(5)} = 0.2832$$

$$f_{ps} = 270\left(1 - \frac{0.2832}{2}\right) = 232 \text{ Ksi}$$

$$W_p = \frac{.881\,(232)}{12\,(14)\,(15)} = 0.2431 < 0.3 \quad \therefore \text{ O.K}$$

$$M_n \cong \frac{.881\,(232)}{12}\left(14 - 0.59\,\frac{.881\,(232)}{12\,(5)}\right)$$

$$M_n \cong 204 \text{ KFT.} > 195 \text{ KFT} \quad \therefore \text{ O.K.}$$

CHECK AT SERVICE LOAD :

$$M_D = \frac{1.5\,(.125)(20)^2}{24} = 3.12 \text{ KFT.}$$

$$\Sigma\, M_{D+L} = 113 + 3 = 116 \text{ KFT.}$$

$$S = \frac{12\,(18)^2}{6} = 648 \text{ IN}^3$$

$A = 18\,(12) = 216 \text{ IN}^2$

$P = 0.6(270)(.881) = 143 \text{ K/FT.}$

$e = 14 - 9 = 5"$

$P/A = 662$ psi c

$$\frac{P}{A} \pm \frac{M}{S} \mp \frac{Pe}{S}$$

$$\frac{143}{216 \text{ IN}^2} \pm \frac{116\,(12)}{648} \mp \frac{143\,(.5)}{648}$$

= 1708 psi c top, 383 psi + BOTTOM   OK

USE 18" SLAB W/ 4 × 0.6φ STRAND TENDONS AT 12" C-C.

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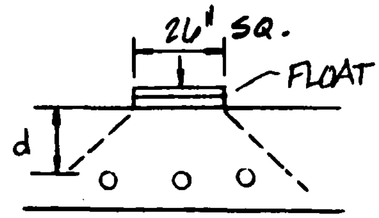
OF
REVISION:

CHECK FOR PUNCHING SHEAR ON 18" MAIN DECK

LOAD = 187^{k}

A = 4.69 FT^2



CONTACT PRESS = $\frac{187}{4.69}$ = 40 ksf

d = 14"

PERIMETER = 4(26 + 14) = 156 IN

CHECK $V_c = \left(0.6\sqrt{f'c} + 700 \frac{V_u d}{M_u}\right) bd$

$(V_u) = 1.7(187) = 318K$

$M_u = 1.4 M_D + 1.7 M_L$ $\quad M_D = \frac{WL^2}{24} = \frac{(.1875)(20^2)}{24} = 3$ K FT

$W = (1')(1.5)(1')(.125) = .1875$ K/FT.

$M_u = 1.4(3) + 1.7(113) = 196$ K FT

$$V_c = \left[.6\sqrt{5000} + 700 \frac{318^{k}(14)}{196^{kFT}(12)}\right] bd = 1367\ bd.$$

$$42 \quad + \quad 1325$$

MAX. ALLOWABLE

$5\sqrt{5000}\ bd\ (.85) = 300\ bd$ $\quad$ CONTROLS

C FACTOR

$V_c = (.300)(156)(14) = 655$ K.

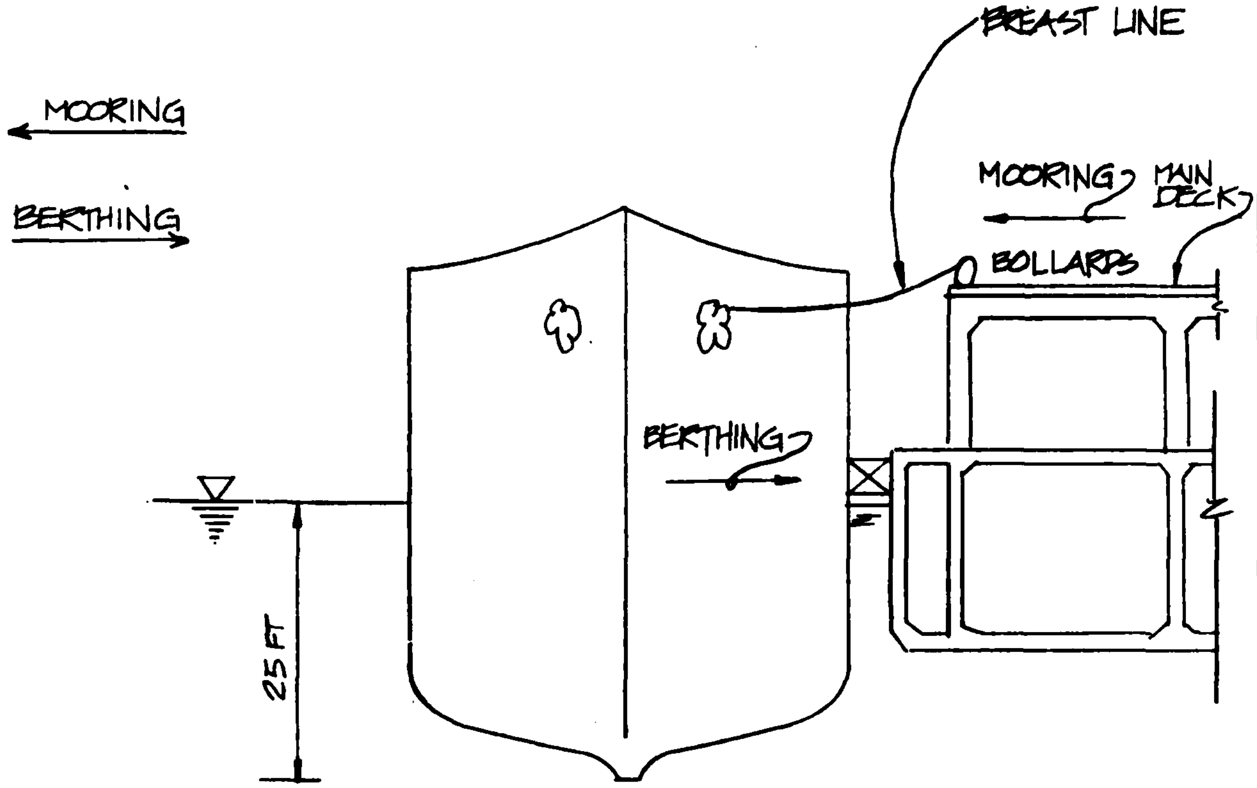
$V_u = \phi V_c = 0.85(655) = 557$ K

MAX. ALLOWABLE LOAD = $\frac{557^{K}}{1.7}$ = 328 K >> 187^{K} OK

NO PUNCHING SHEAR PROBLEM

SHEAR LOAD ON TOP DECK

MOORING LINE RESTRAINS SHIP FROM CURRENT FORCES



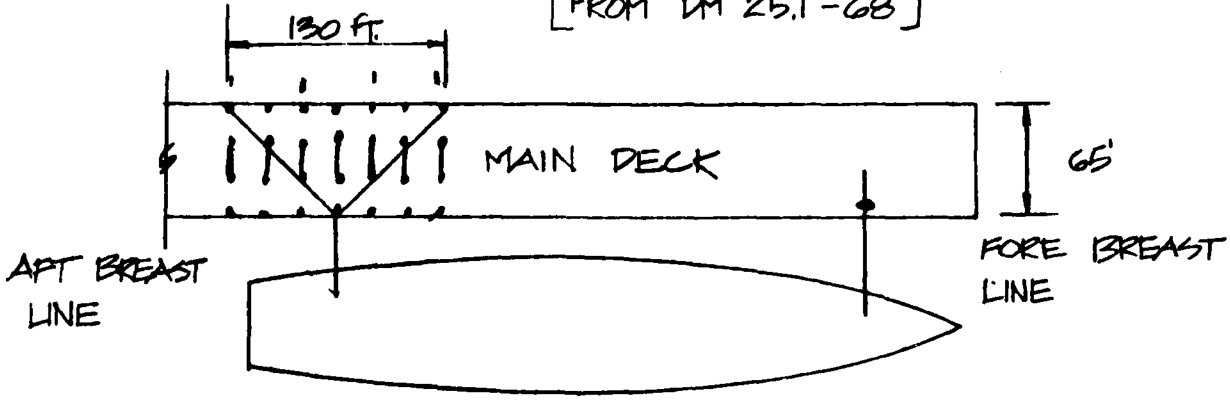
MOORING

BREAST LINE FORCE IS SPREAD OVER 65x2 = 130 ft LENGTH OF MAIN DECK:

BERTHING

FORCE IS TAKEN BY FENDERS AT WATER LINE;

[FROM DM 25.1-68]



AFT BREAST LINE ENGAGES 130 FT LENGTH OF DECK:
WITH SHEAR WALLS @ 40 FT. C-C.

$\frac{130}{40} \cong \underline{3}$ SHEARWALLS ARE ENGAGED.

REF. "CHELLIS", P190: $F_{cv} = KB_v V_c^2$ $\underline{K = 0.8}$

SPRUANCE CLASS DESTROYER:

SAY - L = 500 FT, DRAFT = 25 FT. (AVG.)

NOTE SHIP IS DOWN STREAM FROM DOCK: USE <u>NET</u> AREA.

v = 10 FT/SEC. ASSUME DRAFT = 15 FT. BELOW DOCK BOTTOM

$P = K v^2 = 0.8(10)^2 = 80$ PSF $= 0.08$ KSF.

$\Sigma P = 250(15)(0.08) = 300$ KIPS;

$\frac{300\text{K}}{3\text{WALLS}} = \underline{100}$ K/WALL USE AS DESIGN LOAD

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PROJECT:
ITEM: STRUCTURAL - MAIN DECK
DESIGN: SHEAR WALLS
DATE: R.Z

SHEET: B-37
OF
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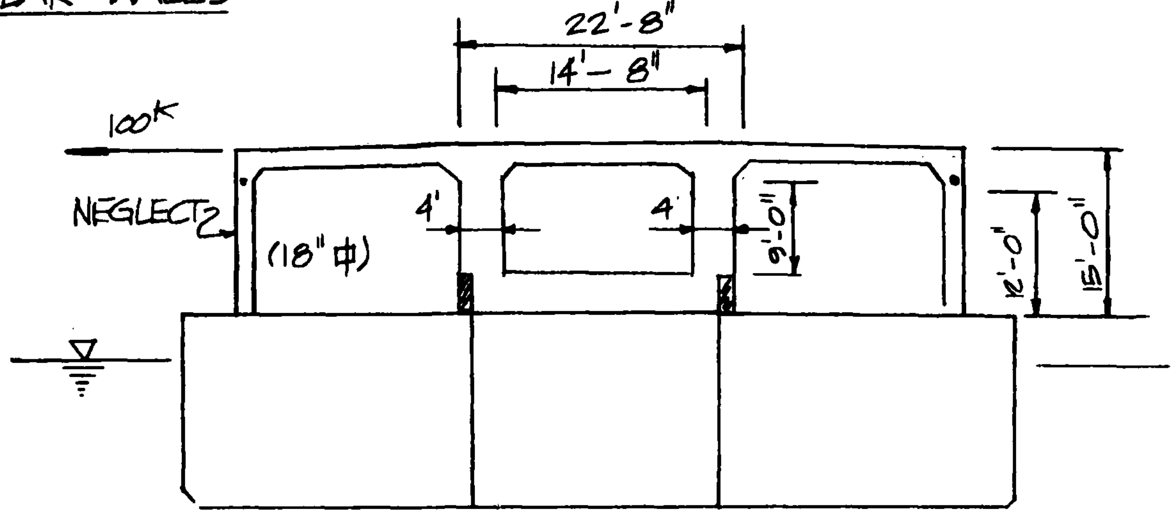
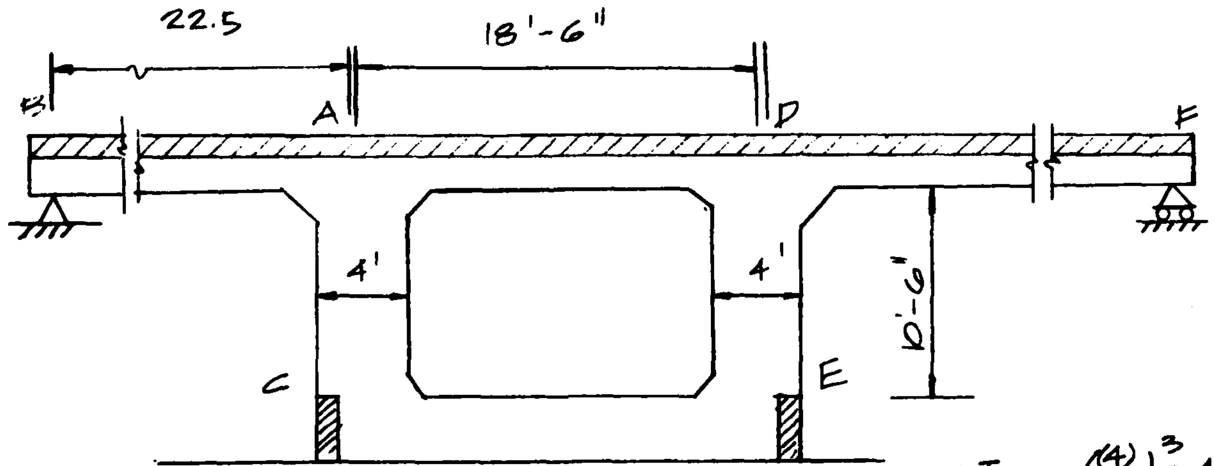
SHEAR WALLS





$$I_{AC} = \left(\frac{4}{2.5}\right)^3 = 4.1$$

$$FEM = \frac{100}{4}(10.5) = 260 \text{ K FT.}$$

AB $\frac{3EI}{L} = \frac{3(1)}{22.5} = 0.07$

AC $\frac{4EI}{L} = \frac{4(4.1)}{10.5} = 0.82$

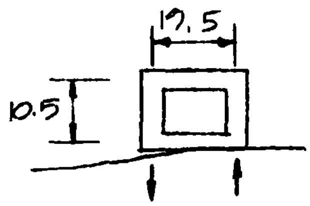
AD $\frac{4EI}{L} = \frac{4(1)}{18.5} = 0.11$

AB	AC	AD	CA
.07	.82	-11	0
0	+260	0	260
-18	-213	-28	0
0	0	0	-107
-18	+47	-39	153

ADJUST $\frac{520}{47+153} = 2.60$

M CA = 153(2.60) = 400 K FT.

$M_{AC} = 47(2.6) = 123$ KFT.



OVERTURNING:

$100\,K\left(\frac{10.5}{19.5}\right) = \pm\ \underline{54}\ K$

DEAD LOAD: $A_T = 433\ FT^2$ @ 12" SLAB:

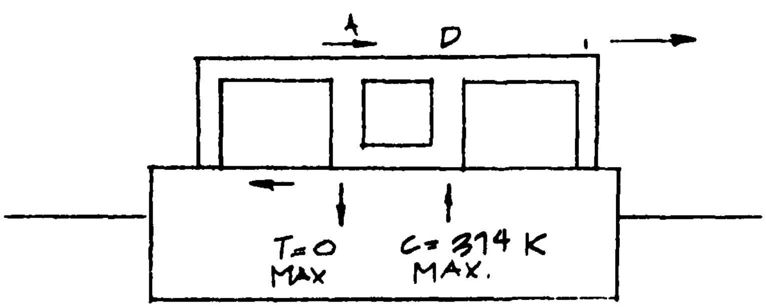
$P_D = \underline{54}$ K : SAY - $\underline{60}$ K

LIVE LOAD: SAY - 433 FT^2 @ 600 PSF OR $\underline{0}$ K

$P_L = \underline{260}$ K OR $\underline{0}$ K

$\therefore$ C MAX. = 54 + 60 + 260 = $\underline{374}$ K COMP.

T MAX. = 54 − 60 + 0 = $\underline{0}$ K TENSION



CHECK A AS BEAM: (NO AXIAL LOAD)

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ITEM: SHEAR WALLS
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OF
REVISION:

$\Sigma M = M_u = 400 \text{ Kft.} \times 1.7 = \underline{680} \text{ K}'$

$d \cong (4'\text{-}0'') - 4'' = \underline{44''}$

$f'_c = 5000 \text{ psi} \quad f_y = 60 \text{ Ksi}$

$F = \frac{18(44)^2}{12000} = 2.90$

$\therefore K_u = \frac{M_u}{F} = \frac{680}{2.90} = 234$

@ $P = .006$: $A_s = Pbd = .006\,(18)(44) = 4.75 \text{ IN}^2$

SAY - 5 - #9 BARS: EACH END.

SHEAR: $V_u = 1.7\left(\frac{100^K}{2}\right) = 85 \text{ K/col.}$

$V_c = 2\sqrt{f'_c}\, bd\,(0.85)$ (LT. WT SAND/CONCRETE REDUCTION FACTOR)

$= 2\sqrt{5000}\,(18)(44)(.85) = 95 \text{ K}$

$V_u = \phi V_n = 0.85\, V_n$: $V_n = \frac{V_u}{\phi} = \frac{85}{0.85} = 100 \text{K}$

$\therefore V_s = V_n - V_c = 100 - 95 = \underline{5} \text{ K}$

$V_s = \frac{A_v f_y d}{s}$ - ; SAY $A_v = 0.4 \text{ in.}^2$ FOR #4 TIES

$s_{req} = \frac{A_v f_y d}{V_s} = \frac{0.4\,(60)(44)}{5} = 211''$

$\therefore$ USE NOMINAL SPACING:

SAY - $\underline{\underline{12''}}$ C-C- FOR TIES.

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PROJECT:	
ITEM:	SHEAR - WALLS
DESIGN:	
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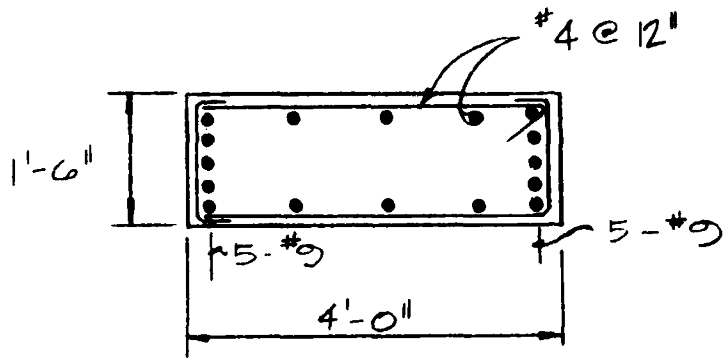
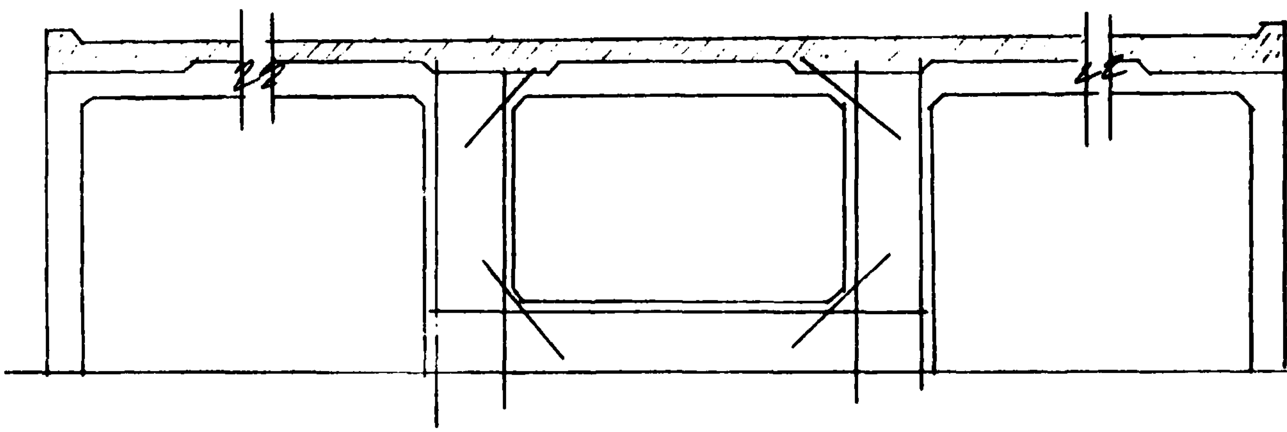
OF
REVISION:





SQUARE COLUMNS:
SAY - NEGLECT RIGID FRAME ACTION AND
DESIGN FOR AXIAL LOAD! (ALL MOMENTS INTO SHEAR WALLS)

INTERIOR COLUMNS

$A_{TRIB} = 21.17(20)$
$= 423.3 \text{ ft}^2$

LL @ 600 $\#/\text{ft}^2$ = 423.3 (0.6) = 254 K

DL SLAB : 100 ft^2 @ 18"
323 ft^2 @ 12"
= 150 + 323 = 473 ft^3

DL = 0.125 Kcf × 473 ft^3 = 59 K

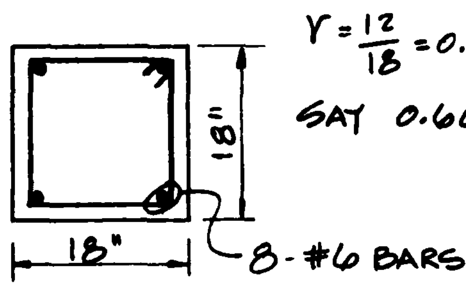
FACTORED LOAD:

$P_u = 1.4(59) + 1.7(254) = 515$ K

COLUMN : 18" ϕ

COLUMN TABLE :

R 5 - 60.60
(P 292, ACI Hdbk)



$\gamma = \frac{12}{18} = 0.67$

SAY 0.60

$$\frac{P_u}{A_g} = \frac{515}{(18)^2} = 1.59 \text{ Ksi}$$

e/h = SAY - 0.1 (NOMINAL)

$\therefore \rho_g = 0.01$ OK (MAX $e/h = 0.25$)

A_s REQ. = $.01(18)^2 = 3.24 \text{ IN}^2$ = 8 - #6 BARS
3.52 IN^2

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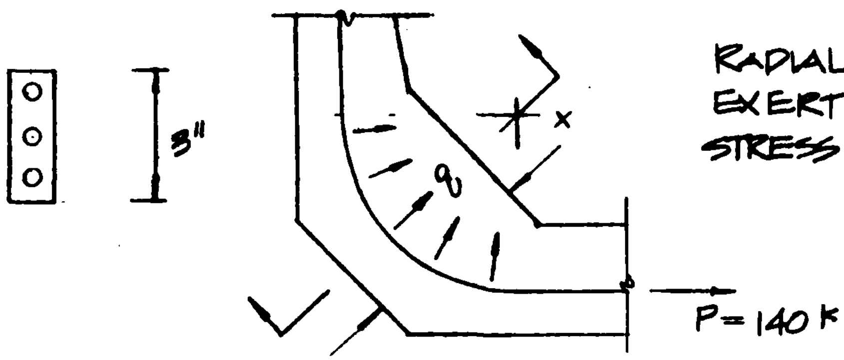
PROJECT:
ITEM: STRUCTURAL - PONTOON
DESIGN: CORNER DETAILS
DATE:
R.T.

SHEET: B-42
OF
REVISION:

CORNER DETAIL

$$P = 140\ K / 3\,IN. = 46.7\ K/IN$$



RADIAL FORCE EXERTS BEARING STRESS ON CONCRETE.

T = 46.7K HOOP TENSION FORMULA:

$$P = q\, r_c \;:\; r_c = 24"$$

$$f_{bearing} = q = \frac{P}{r_c} = \frac{46{,}667}{24} = 1944\ psi$$

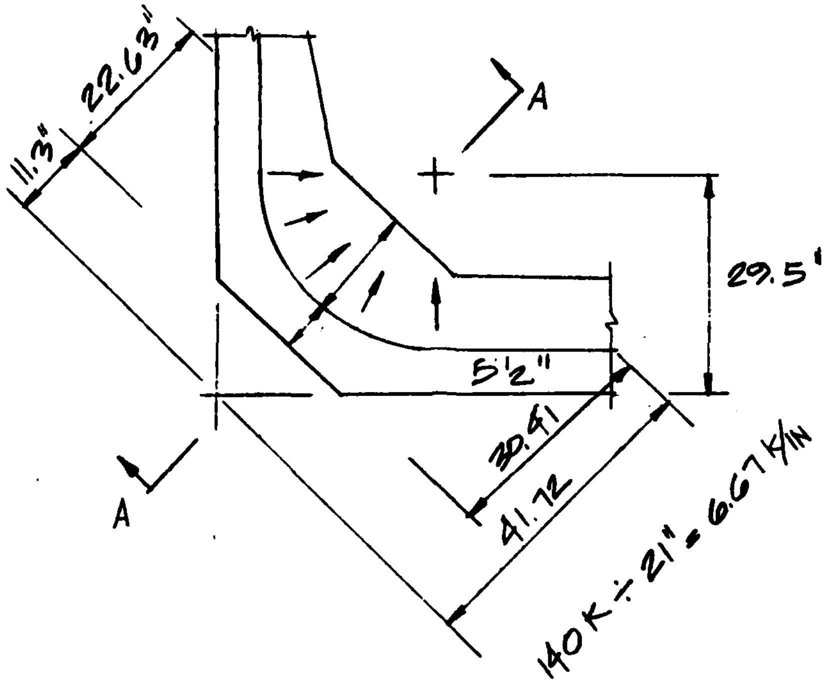
ALLOW

$$f_{bearing} = 0.6 f'c \sqrt{A_2/A_1} \geq f'c$$

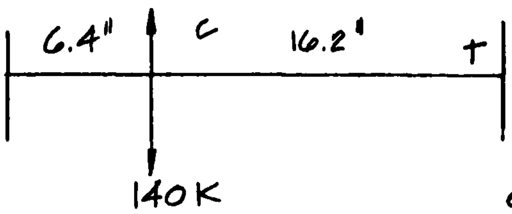
$$0.6 f'c = 0.6(5000) = 3000\ psi :$$

$\therefore$ OK @ 1944 psi

LOOK AT COMPRESSION IN CONCRETE.



A–A:



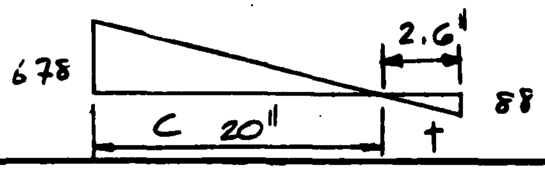
$$e = 16.2 - \frac{22.6}{2} = 4.9'$$

$$\frac{P}{A} = \frac{6.67}{22.6} = 0.295 \quad \text{Ksi}$$

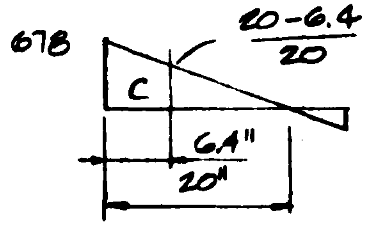
$$Pe = 6.67\,(4.9) = 32.7 \text{ K/in.}$$

$$S = \frac{bh^2}{6} = \frac{(1)(22.63)^2}{6} = 85.35 \text{ IN}^3$$

$$\therefore \frac{P}{A} \pm \frac{M}{S} = 0.295 \pm \frac{32.7}{85.35} = 678 \text{ PSIC},\ 88 \text{ PSIT.}$$



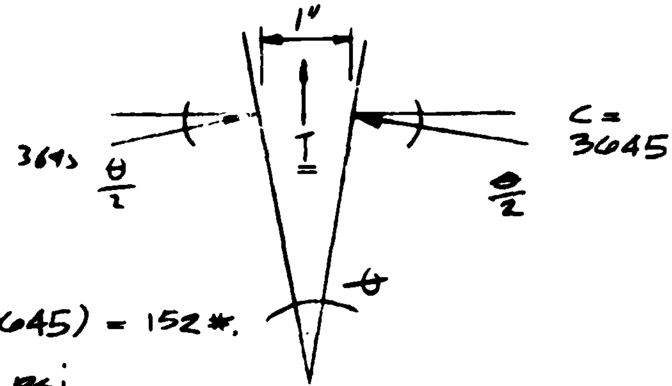
NOTE: TANGENTIAL COMPRESSION IN CONCRETE OUTSIDE OF P/S PRODUCES A RADIAL TENSION.



$\frac{20-6.4}{20}(678) = 461$ PSI

$C = \frac{678+461}{2}(6.4'') = 3645$ K/IN.

RADIAL COMPONENT:



SAY – 1" LENGTH:

$\theta = \frac{1}{24} = .0417$ RAD.

$T = \theta \times C = .0417(3645) = 152$ #.

$f_t = 152$ # IN. $1' = 152$ PSI

CONSIDER THIS RADIAL TENSION TO BE EQUIVALENT TO DIAGONAL TENSION (SHEAR), AND ALLOW CONCRETE TO CARRY $1.1\sqrt{f'_c}$ (Wkg STRESS)

(ACI APPENDIX B.3).

$\therefore v_c = 1.1\sqrt{5000} = 78$ PSI

∴ TAKE REMAINDER WITH
STIRRUPS: 152 - 78 = 74 PSI

SAY: #4 BARS: @ 24 KSI.

24,000 × 0.20 = 4,800 #/BAR

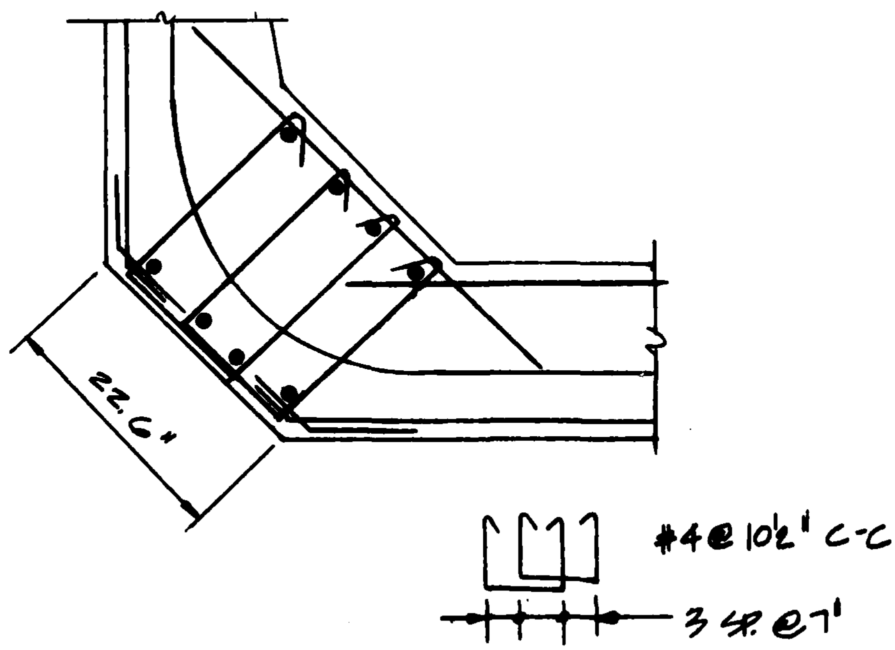
$$A_c = \frac{4800}{74} = 65 \text{ IN}^2 \text{ PER BAR.}$$

SAY - 2 STIRRUP SETS PER TENDON:

TENDONS @ 21" C-C. ∴ STIRRUPS @ 10½"

$$\therefore \text{STIRRUP SPACING} = \frac{65 \text{ IN}^2}{10\frac{1}{2}"} = 6\frac{1}{2}" \text{ MAX.}$$

SAY -



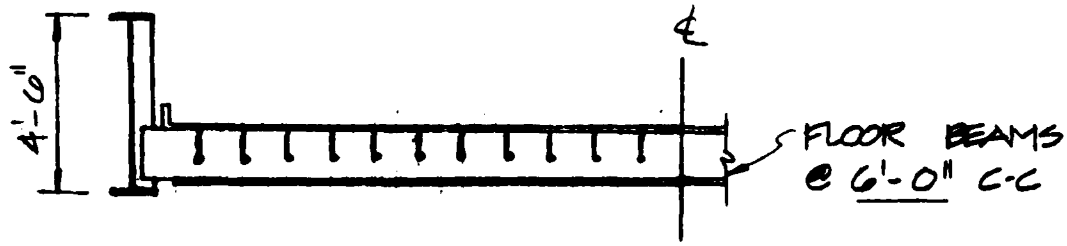
RAMPS

RAMP: SAY- FROM ASTMA - 588 STEEL.
(F_y = 50 Ksi)

DESIGN AS THROUGH - GIRDER WITH ORTHOTROPIC DECK (OPEN - RIB).

L = 60 ft, W = 25 ft (C-C GIRDS) (24' CLEAR OPG)

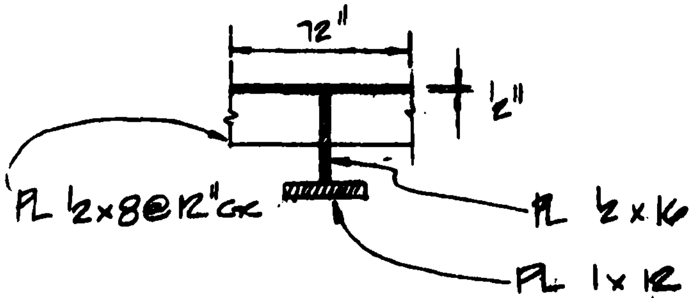
SAY - 4'-0" DEPTH OF GIRDERS

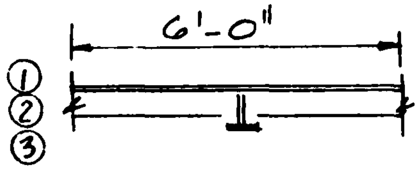


FLOOR: PL · 1/2 W/ PL 1/2 x 8 OPEN STIFFENERS @ 12" C-C.

NOTE: FOR DESIGN - REF. ORTHOTROPIC BRIDGES: THEORY AND DESIGN; TROITSKY, 1967

FLOOR BEAM;
a_u = 15'





① = PL ½ x 72 (TOP PL)

② = PL ½ x 18 (WEB)

③ = PL 1 x 12 (BOTTOM FLANGE)

PROPERTIES: $Y_b = 13.84"$ $Y_T = 5.66"$

$S_b = 258 \text{ IN}^3$ $S_T = 630 \text{ IN}^3$

$I = 3567 \text{ IN}^4$

$W_T = 194$ #/FT.

ADD WT. STIFFENERS: 81.6 #/FT.

Σ STEEL W = 265 #/FT.

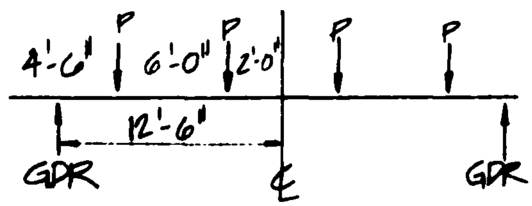
SAY - 2" AC TOPPING: $W = \frac{144}{6} \times 6 = 144$ #/FT. (144 PCF)

Σ W = 420 #/FT (70 PSF)

$\therefore +M_D = \frac{.42(25)^2}{8} = 32.8$ KFT.

LL:

$I = 0.3$



$+ M_{L+I} = 2P(10.5) - P(6) = 15P$

$P_{L+I} = 1.3(16K) = 20.8K$

$\therefore +M = 15(20.8) = \underline{312}$ KFT.

FACTORED LOADS:

$1.3(D + 1.67L) = 1.3[33 + 1.67(312)] = 720$ KFT.

STRESSES: $\frac{M}{S_b} = \frac{720(12)}{258} = 33.5$ KSIT
(ALLOW 36)

TOP: $\frac{M}{S_T} = \frac{719(12)}{549} = \frac{15.7 \text{ KSIC}}{\text{(OK)}}$

CHECK LL DEFLECTION: $L = 25' = 300''$

$M_{LL+I} = 312$ K FT. $M = \frac{WL^2}{8}$

SAY - EQUIV. TO UNIF. LOAD OF

$\frac{8M}{L^2} = \frac{8(312)}{(25)^2} = 4.0$ K/FT. $= 0.333$ K/IN

$\Delta = \frac{5WL^4}{384EI} = \frac{5(0.333)(300)^4}{384(29000)(3567)} = 0.34''$

$\frac{0.34'}{300'} = 1/882$ ∴ OK (ALLOW 1/800)

MAIN GIRDER:

DEAD LOAD: SAY 200 #/FT FOR GIRDER
200 #/FT FOR UTILITIES
875 #/FT FOR ROADWAY

$\Sigma = 1275$ K/FT.

$M_D = \frac{1.275(60)^2}{8} = 574$ K'

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PROJECT:
ITEM: MAIN DECK - RAMPS
DESIGN:
DATE: R.Z.

SHEET: B-51
OF
REVISION:

DECK PLATE @ t = ½": SAY - A36
(f_u = 58 Ksi)

$P_u = 6.2 \times \frac{f_u A}{a} \sqrt{\epsilon_u}$ (TROITSKY FORMULA 125)

$= 6.2 \times \frac{1}{2} \times \frac{58{,}000}{12} \times \sqrt{0.21} = 6866$ PSI

SAY: $A_{brg} = 24'' \times 12''$: (W/2" OVERLAY)

$\therefore P = 6866 \times \frac{24 \times 12}{1000} = 1977$ K

$n = \frac{P_u}{P_{ACT}} = \frac{1977}{15.6} = 127$ (SAFTY FACTOR) $(P_{WHEEL} = 1.3(12) = 15.6\text{K})$

t_{req} = (FORMULA 88) = $.0065\, a \sqrt[3]{P}$

$P = \frac{15{,}600}{24 \times 12} = 54.2$ PSI

$a = 12''$

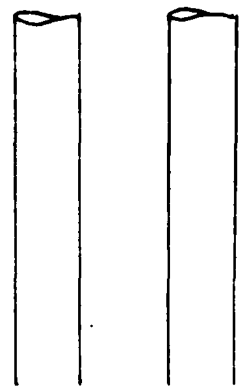
$.0065(12)\sqrt[3]{54.2} = 0.295''$ $\therefore$ $\underline{\tfrac{1}{2}''}$ OK

NOTE: USE DIAMOND - PATTERN (CHECKER PL)
TO HELP HOLD OVERLAY ON.

APPENDIX. C : ANCHORING SYSTEM CALCULATIONS.

VERTICAL PILES

LATERAL LOAD FROM ENVIRONMENTAL FORCES = 125 K PER PILE BENT



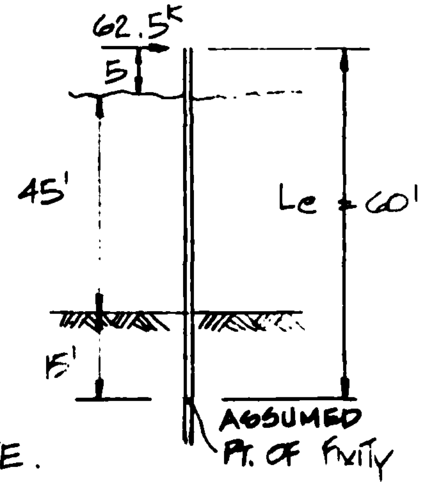
TRY: TWO PILES @ BENT
DIA = 48"
t = 1"
Le = 60'

$I = \frac{\pi(D_o^4 - D_e^4)}{64} = \frac{\pi(48^4 - 46^4)}{64}$
$= 40,790 \text{ IN}^4$

$S = \frac{40,790}{24} = 1700 \text{ IN}^3$

$M = PLe = 62.5^K(60) = 3750^K\text{FT.}$

$f = \frac{M}{S} = \frac{3750 \text{ K FT. } (12)}{1700 \text{ IN}^3} = 26.5 \text{ KSI}$ OK



ALLOW FOR 1/4 IN. CORROSION OVER 40YR LIFE.

t EFFECTIVE = 3/4 IN

$I = \frac{\pi(47.5^4 - 46^4)}{64} = 30,100 \text{ IN}^4$

$S = 1267 \text{ IN}^3$

$f = M/S = \frac{3750 (12)}{1267} = 35.5 \text{ KSI}$ USE $f_y = 50$ KSI
$f_b = 0.70 f_y$ O.K.

FILL PILES WITH SAND TO INCREASE LOCAL BUCKLING STRENGTH AND TO INHIBIT CORROSION.

USE CATHODIC PROTECTION FOR EXTERIOR OF PILES.

DEFLECTION OF PILES : $\Delta = \frac{PL^3}{3EI}$

$= \frac{62.5K(60)^3(12)^3(1000)}{3(29 \times 10^6)(40,790 \text{ IN}^4)} = 6.6 \text{ IN.}$

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PROJECT:	
ITEM:	ANCHORING
DESIGN:	BATTER PILES
DATE:	R.Z.
SHEET:	C-2
OF	
REVISION:	

BATTER PILES

PILES: SAY - 36"ϕ, 1" WALL:

A_s = 110 IN^2 W = 374 #/FT.

I = 14,125 IN^4 S = 785 IN^3

FILLED WEIGHT: @ 150 PCF CONCRETE:

W_{CONC} = 946 #/FT. DISPL. WATER = 0.45 K/FT.
50'(0.45) = 22.5 K

SAY - L = 100 FT.:
W_{STEEL} = 37 K

FILL 60 FT. W/CONCRETE: 60 × 0.95 = 57 K

NET BUOYANT WT. ≅ 37 + 57 − 22 = 72 K

BATTER @ 1 TO 6 IP @ +75 FT.

LOAD OCCURS @ 60 FT.

SAY - LOAD IS 125 K

APPLIED 15 FT. BELOW P.I.

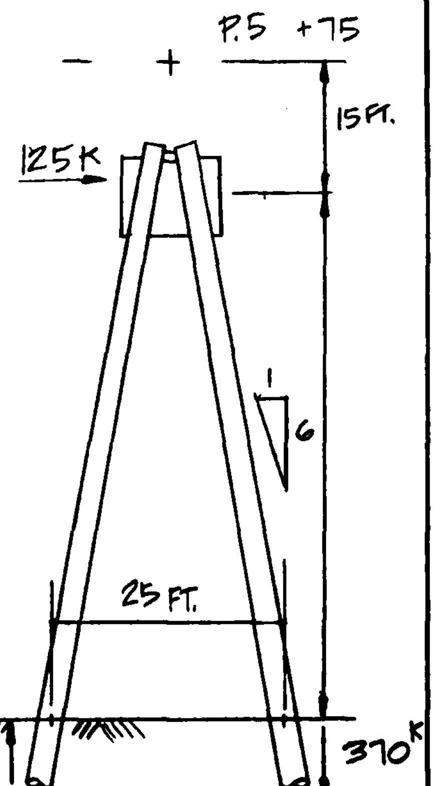
@ H = 100K, V = $\frac{60}{25}$ (125) = 300 K ↑↓

NET V↑ = 300K − 72 K = 228 K

NET. T = 1.0138(233) = 230 K ↑

NET. C = 230 + 2(72) ≅ 374 K ↓

UPLIFT = 0.62 × DOWNFORCE.



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PROJECT:
ITEM: BATTER PILES
DESIGN:
DATE: R.Z.

SHEET: C-3
OF
REVISION:

PILES ARE DRIVEN INTO "MARL":
(STIFF CLAY: TERZAGHI + PECK, P8)
SAY – ULTIMATE SKIN FRICTION IS 1000 #/FT.²
(TERZAGHI + PECK P533, TABLE 56.1; B.K. HOUGH, P335, FIG. 11-11)
36" φ PILE: A = 9.425 FT.²/FT.

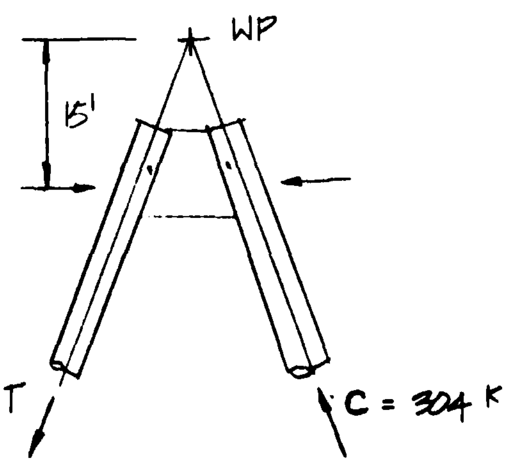
∴ ULTIMATE CAPACITY = 9.425 K/FOOT OF EMBEDMENT:
AT SF = 1.5, GOOD FOR 6.3 K/FT.
EMBEDMENT REQ'D FOR PULLOUT
RESISTANCE = $\frac{230\text{ K}}{6.3\text{ K/FT}}$ = 36.5 FT.
SAY – 40 ft OK. @ 1.5 S.F.

CHECK STRESSES IN PILES:
NOTE: MAX BENDING STRESS OCCURS @ PILE TOP, DUE TO FRAME ACTION:
T = C = 1.0138 (300) = 304 K
"e" = $\frac{15}{3}$ = 5 FT. = 60"

∴ M = 304 (60) = 18,240 K-IN

∴ $f_b = \frac{18240}{2(785)}$ = 11.62 Ksi

$f_a = \frac{304\text{ K}}{11.0\text{ IN}^2}$ = 2.16 Ksi



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PROJECT:
ITEM: BATTER PILES
DESIGN:
DATE: R.Z.

SHEET: C-4
OF
REVISION:

CHECK @ $\frac{3}{4}$" WALL ($\frac{1}{4}$" LOST BY CORROSION), OD = 35.5 IN

$A_s = 81.88 \text{ IN}^2$

$I = 12{,}365 \text{ IN}^4, \quad S = 697 \text{ IN}^3$

$f_a = \dfrac{304\text{ K}}{81.88 \text{ IN}^2} = 3.71 \text{ KSI}$

$f_b = \dfrac{18240}{2(697)} = 13.08 \text{ KSI}$

SAY - 36 KSI STEEL: $F_b = 0.66(36) = 24$ KSI ALLOWABLE

$L = 60 \text{ FT.} = 720"$ $\quad K = 0.8$

$r = \sqrt{\dfrac{I}{A}} = 12.3"$

$\therefore \dfrac{KL}{r} = \dfrac{0.8(720)}{12.3} = 46.8$

$\therefore F_a = 18.6 \text{ KSI}$ $\qquad \dfrac{3.71}{18.6} = 0.2$

$F'_e = 67.6$

$C_m = 0.85$

$$\frac{f_a}{F_a} + \frac{C_m f_b}{\left(1 - \frac{f_a}{F'_e}\right)F_b} = \frac{3.71}{18.6} + \frac{0.85(13.08)}{\left(1 - \frac{3.71}{67.6}\right)(24)}$$

$$= 0.20 + 0.49 = 0.69 < 1.0$$

$\therefore$ OK (COULD TAKE 180 K)

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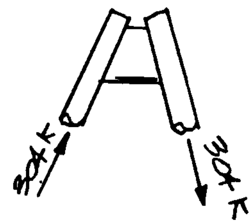
PROJECT:
ITEM: BATTER PILES
DESIGN:
DATE: R.Z.

SHEET: C-5
OF
REVISION:

SHEAR PLATES



$V = \underline{304}$ K

$F_v = 14.5$ ksi

A_v REQ'D $= \frac{304}{14.5} = 21$ IN^2

SAY - 3/4" PLATE.

L REQ'D $= \frac{21}{0.75} = 28"$

MAKE PLATE 5'-0" LONG,
TO ALLOW FOR SECTION LOSS.

$A_v = 60(0.75) = 45$ IN^2

$f_v = \frac{304}{45} = 6.75$ KSI $< 0.5 \times F_v$

THUS $\underline{50\%}$ SECTION LOSS CAN OCCUR.

WELDS: DOUBLE FILLET: SAY $\underline{5/16}$

$A_w = 2(60)(0.221) = 26$ IN^2

APPENDIX D : FENDER DESIGN

FROM DM 25.1

$$E = \frac{1}{2} M V^2 C_E C_H = \frac{1}{2}\frac{W}{g} V^2 C_E C_H$$

C_E = BERTING Coef. = 0.5

C_H = HYDRODYNAMIC Coef.

Fig 47 $C_H = 3.1$ FOR AD

$C_H = 3.6$ FOR CG.

V = MODERATE SITE 0.6 fps AD
0.5 fps CG.

W = DISPL. TONNAGE 22,800 T. AD
10,000 T. G

AD $E = \frac{1}{2}\frac{22{,}800 \text{ TON}}{32.2}(.5)^2(.5)(3.1) = 137$ ft. TONS

= 306 ft Kips CONTROLS DESIGN

CG. $E = \frac{1}{2}\frac{10{,}000 \text{ TON}}{32.2}(.6)^2(.5)(3.6) = 100$ ft. TON

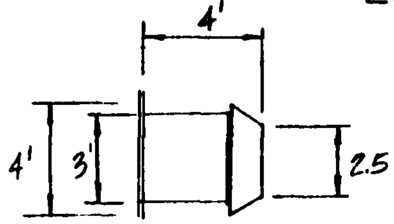
= 224 ft Kips

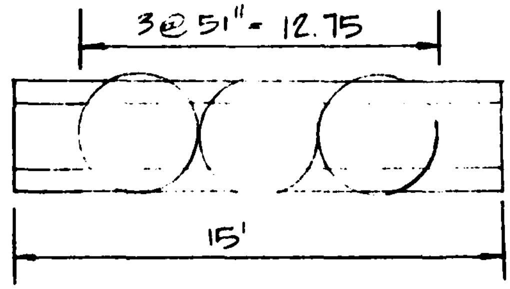
LORD'S CELL TYPE FENDERS

SUC 1000H - R1 RATED E = 113 ft KIPS @ 52.5% DEFLECTION
MAX E = 120 ft KIPS @ 55%

RATED REACTION = 78.5 KIP
MAX REACTION @ 55% DEFLECTION = 83.6 K

No OF CELLS = $\frac{E_{DESIGN}}{E_{CELL}} = \frac{306 \text{ ft.K}}{113 \text{ Ft. K}}$ = 2.7 CELLS
USE 3 CELLS





CONTACT AREA ON SHIP
(2.5')(15')(144) = 5400 IN²

MAX REACTION ON SHIP 3(83.6^K) = 250.8K

PRESS. ON SHIP HULL $\frac{250.8K}{5400 \text{ IN}^2}$ = 46 psi OK

ALLOWABLE 45 psi

TIME TO STOP SHIP

MAX. DEFLECTION OF CELL = (.55) (3') ≃ 1.65'

v_0 = 0.6 fps

v_1 = 0 fps

$t = \frac{2d}{v_0 + v} = \frac{2(1.65)}{.6}$ = 5.5 SEC.

REACTION ON PIER

REACTION FROM FENDER CELLS = 3(83.6) = 250.8 K

THE PIER UNDER LATERAL LOADING
IS A DEEP BEAM
AND THEREFORE VERY STIFF

ASSUME LOAD DISTRIBUTED OVER LENGTH OF 5 (WIDTH)

(5)(75) = 375 FT

SAY. 400 FT. WHICH IS 10 PILE BENTS

EACH PILE BENT LOADED @ $\frac{250.8}{10} = 25\text{ K}$

DESIGN LOAD FOR PILE BENTS 125K WHICH PRODUCES
A DISPLACEMENT OF ABOUT 6 IN.

DISPLACEMENT FOR 25 K LOAD = $\frac{25}{125}(6) = 1.2''$
SAY 1"

FOR 1" DISPLACEMENT THE PILES ALONE WILL RESIST THE SHIP BERTHING FORCES BUT A WATER MASS HAS TO BE MOVED BY THE PIER WHICH ALSO REQUIRES FORCE

DRAFT OF PIER = 13'
AREA FOR 10 PILE BENTS = 13(400) = 5200 FT^2

Wt OF WATER THAT MUST BE MOVED FOR PIER TO MOVE 1IN.

$Wt = (0.064)(5200)(\frac{1}{12}) = 27.7^K > 25^K$

BERTHING FORCE CAN BE ABSORBED BY PIER ALONE (NEGLECT PILES) MOVING 1 IN.

WITH PILES & WATER MASS WORKING TOGETHER DISPLACEMENT WILL BE ½ IN.

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PROJECT:	SHEET: E-1
ITEM: NAVAL ARCHITECTURE	OF
DESIGN: WAVE HEIGHT	REVISION:
DATE:	

APPENPIX . E : NAVAL ARCHITECTURAL CALCULATIONS.

DETERMINE LONGITUDINAL BENDING MOMENT
FROM TROCHOIDAL WAVE

CALCULATIONS ARE BASED ON THE FOLLOWING ASSUMPTIONS

CONDITION (1) A, WEIGHT DISTRIBUTION EVEN

B, WAVE LENGTH EQUAL TO LENGTH OF PONTOON = 600 FT.

C, WAVE HEIGHT = 14.3 FT. (ASSUMED HEIGHT)

CONDITION (2) A, & B, SAME AS CONDITION 1

C, WAVE HEIGHT = 10 FT. (ASSUMED HEIGHT)

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PROJECT:
ITEM:
DESIGN: WAVE HEIGHT
DATE:

SHEET: E-2
OF
REVISION:

CONDITION 1)

STATION	LOAD (WAVE HEIGHT = 14.3')		
0	7.15 x 75 x 1/35	= 15.32 TONS	= 34.3 KIPS
1	5.15 x	= 11.04	= 24.7
2	1.85 x	= 3.96	= 8.9
3	−1.85 x	= −3.96	= −8.9
4	−5.15 x	= −11.04	= −24.7
5	−7.15 x	= −15.32	= −34.3
6	−5.15 x	= −11.04	= −24.7
7	−1.85 x	= −3.96	= −8.9
8	1.85 x	= 3.96	= 8.9
9	5.15 x	= 11.04	= 24.7
10	7.15 x	= 15.32	= 34.3

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PROJECT:
ITEM:
DESIGN: WAVE HEIGHT
DATE:

SHEET: E-3
OF
REVISION:

STATION	SHEAR (WAVE HEIGHT 14.3')	
0		= 0 KIPS.
1	$(1.25+2\times1.52+1.72)\times1\times\frac{1}{2}\times20\times30$	= 1803
2	$(0.45+2\times0.85+1.25)\times300+1803$	= 2923
$\frac{1}{2}$	$(0.45\times1)\times300+2923$	= 3058
3		= 2923
4		= 1803
5		= 0
6		= -1803
7		= -2923
$\frac{1}{2}$		= -3058
8		= -2923
9		= -1803
10		= 0

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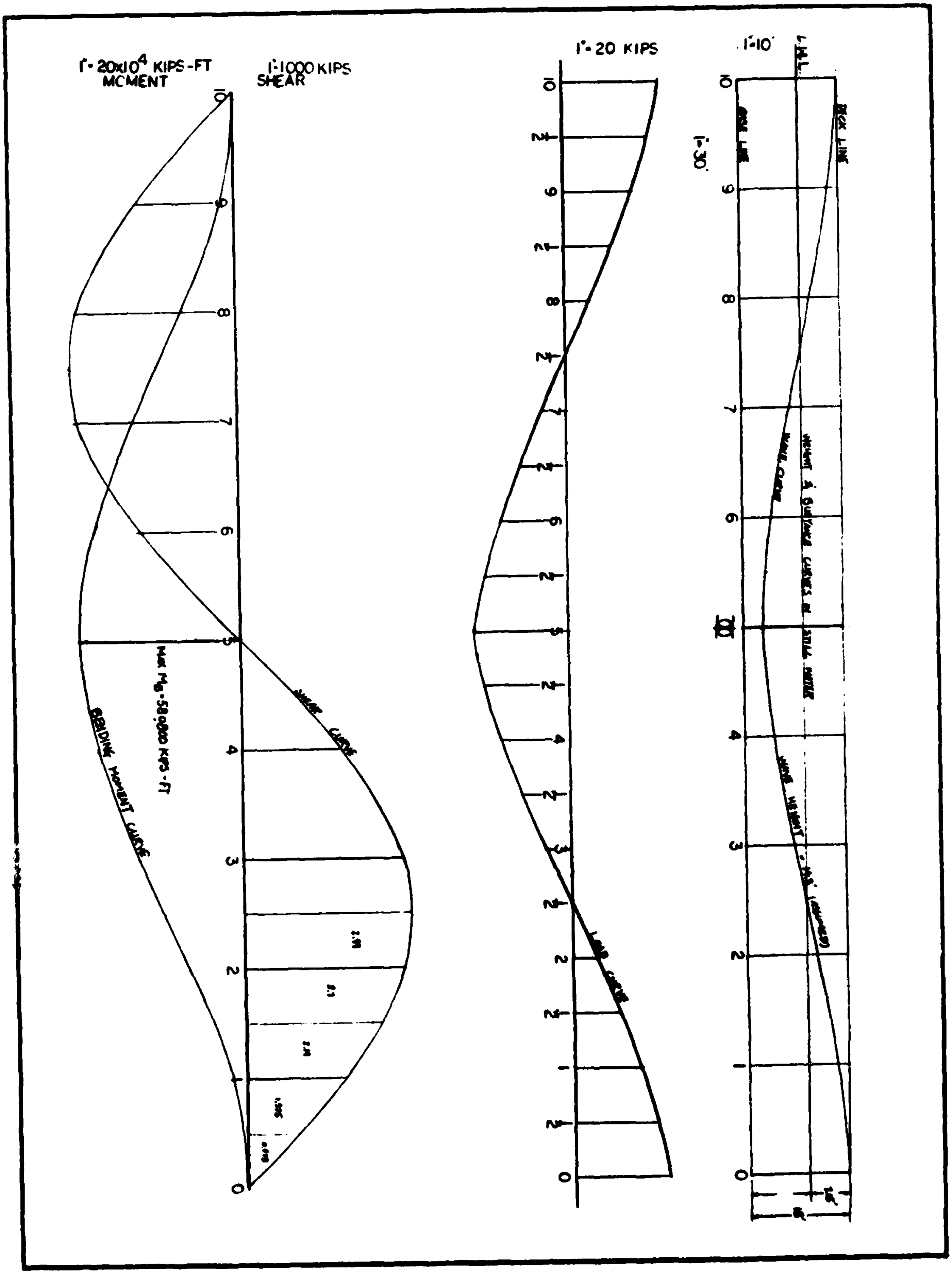
PROJECT:
ITEM:
DESIGN: WAVE HEIGHT
DATE:

SHEET: E-4
OF
REVISION

STATION	BENDING MOMENT (WAVE HEIGHT = 14.3')	
0		= 0 KIPS - FT.
1	$(0.475 + 1.375) \times 1000 \times 30$	= 55500
2	$\overset{145200}{(2.14 + 2.70) \times 3 \times 10^4} + 55500$	= 200700
3	$\overset{179400}{2.99 \times 2 \times 3 \times 10^4} + 200700$	= 380100
4	145200 + 380100	= 525300
5	55500 + 525300	= 580800 MAX.
6		= 525300
7		= 380100
8		= 200100
9		= 55500
10		= 0


1" = 20x10^4 KIPS-FT
MOMENT
1" = 1000 KIPS
SHEAR
1" = 20 KIPS
1" = 10'
1" = 30'
L.W.L.
DECK LINE
BASE LINE
SHEAR CURVE
MAX M_B = 58,000 KIPS-FT
BENDING MOMENT CURVE
LOAD CURVE
WAVE CURVE

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PROJECT:
ITEM:
DESIGN: WAVE HEIGHT
DATE:

SHEET: E-5
OF
REVISIONS

CONDITION II

STATION	LOAD (WAVE HEIGHT = 10')		
0	5.0 x 75 x 1/35	= 10.71 TONS	= 24.0 KIPS
1	4.15 x	= 8.89	= 19.9
2	1.5 x	= 3.21	= 7.2
3	-1.5 x	= -3.21	= -7.2
4	-4.15 x	= -8.89	= -19.9
5	-5.0 x	= -10.71	= -24.0
6	-4.15 x	= -8.89	= -19.9
7	-1.5 x	= -3.21	= -7.2
8	1.5 x	= 3.21	= 7.2
9	4.15 x	= 8.89	= 19.9
10	5.0	= 10.71	

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STATION	SHEAR (WAVE HEIGHT = 10')	
0		= 0 KIPS
1	$(0.99 + 2 \times 1.16 + 1.2) \times 1 \times \frac{1}{2} \times 20 \times 30$	= 1353
2	$(0.35 + 2 \times 0.69 + 0.99) \times 300 + 1353$	= 2169
$2\frac{1}{2}$	$(0.35 \times 1) \times 300 + 2169$	= 2274
3		= 2169
4		= 1353
5		= 0
6		= -1353
7		= -2169
$7\frac{1}{2}$		= -2274
8		= -2169
9		= -1353
10		= 0

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PROJECT:
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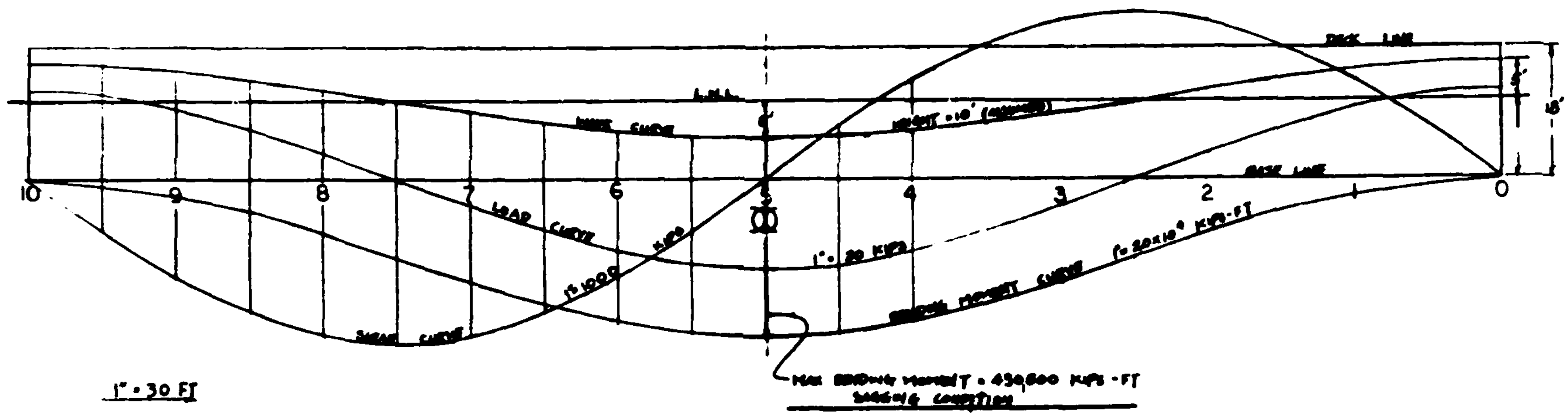
STATION	BENDING MOMENT (WAVE HEIGHT = 10')
0	= 0 KIPS - FT
1	$(1.35 + 2 \times 0.72 + 0) \times 1 \times \frac{1}{2} \times 1000 \times 30$ = 41850
2	$(2.17 + \overset{106800}{2 \times 1.8} + 1.35) + 15000 + 41850$ = 148650
3	$(2.17 + \overset{133200}{2 \times 2.27} + 2.17) \times 15000 + 148650$ = 281850
4	= 388650
5	MAX = 430500
6	= 388650
7	= 281850
8	= 148650
9	= 41850
10	= 0



DECK LINE
L.W.L.
BASE LINE
WAVE CURVE
LOAD CURVE
SHEAR CURVE
BENDING MOMENT CURVE
SAGGING CONDITION
1" = 30 FT
10
9
8
7
6
5
4
3
2
1
0
18'

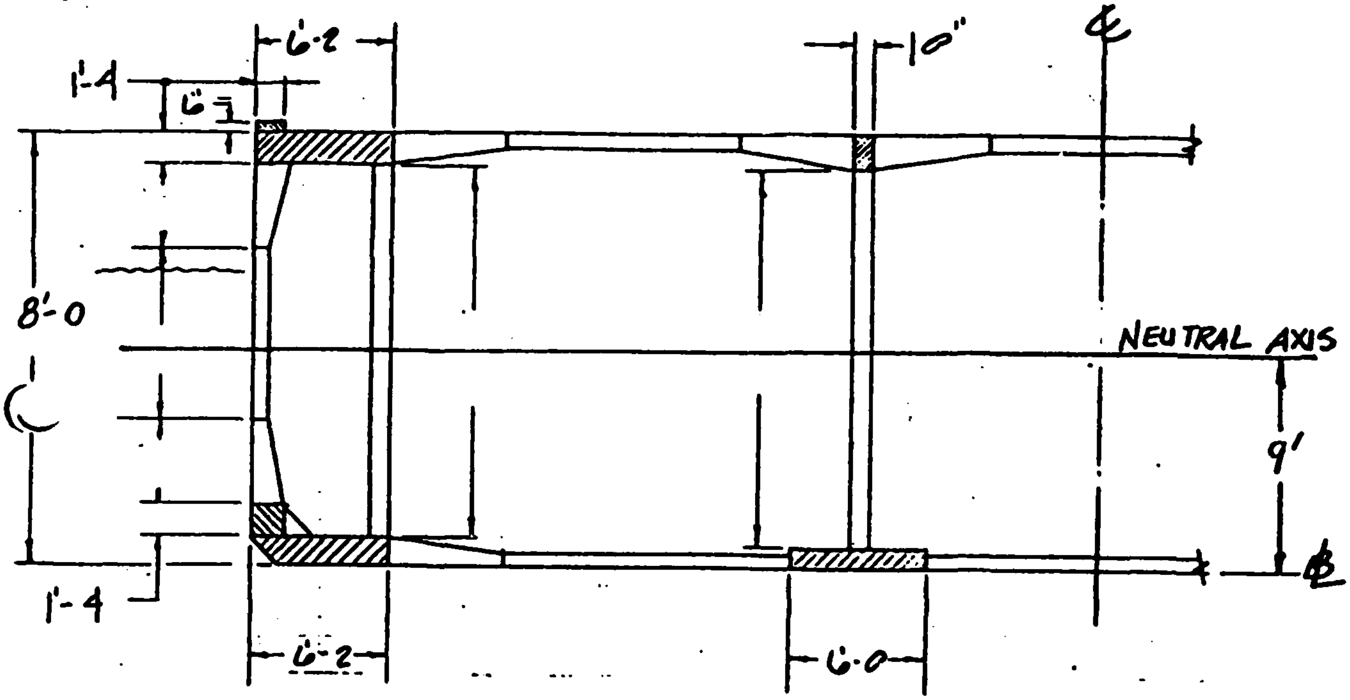


FIG. 1

$I = 12{,}026\ FT.^4$

DETERMINE ALLOWABLE BENDING MOMENT FOR PONTOON

M_B - ALLOWABLE BENDING MOMENT

I = MOMENT OF INERTIA OF CROSS SECTION
= 12,026 FT^4

σ = MAX ALLOWABLE BENDING STRESS = 1175 PSI

$$\sigma = \frac{M_B}{SM} = M_B / \frac{I}{c}$$

ASSUMED NEUTRAL AXIS = 9'-0 ABOVE BASE LINE

σ = 1175 × 144/1000 = 169.2 $KIPS/FT^2$

$$M_B = \sigma \times \frac{I}{c}$$
$$= 169.2 \times \frac{12{,}026}{9} = 226{,}090 \text{ KIPS-FT.}$$

USE ABS RULES

SECTION 6.3.2 TOTAL BENDING MOMENT:

$M_t = M_{sw} + M_w$

M_{sw} = STILL WATER BENDING MOMENT
= 0

M_w = MAXIMUM WAVE - INDUCED BENDING MOMENT
$= C_2 L^2 B H_e K_b$

K_b = 1.0 FOR $C_b \geq 0.80$

C_b = 0.97

$C_2 = [6.53\ C_b + 0.57]\ 10^{-4} = [6.33 + 0.57]\ 10^{-4} = 6.9 \times 10^{-4}$

L = 600 FT.

B = 75 FT.

H_e = EFFECTIVE WAVE HEIGHT OF STANDARD WAVE IN FT.
= 0.018 L + 11.535 = 22.335 FT. 490 < L < 720 FT.

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$$M_w = 6.9 \times 10^{-4} \times 600^2 \times 75 \times 22.395 \times 1 \times 2.24\ ^{k}/_{TON}$$

$$= 934,570 \text{ KIPS - FT}$$

DETERMINE ALLOWABLE WAVE HEIGHT BY PROPORTION:

$$\frac{\text{ALLOW. H}}{\text{MAX } M_B} = \frac{H_e}{M_w}$$

1) ABS METHOD $H = \frac{M_B}{M_w} \times H_e$

$$= \frac{226,090}{934,570} \times 22.395 = \underline{5.4 \text{ FT.}} \quad \text{FOR } L = 600 \text{ FT.}$$

2) TROCHOIDAL WAVE METHOD:

A. WAVE HEIGHT = 14.3' (ASSUMED)

M_B MAX = 580,800 KIPS - FT (SEE CONDITION 1)

$$H = \frac{M_B}{M_{B\,MAX}} \times 14.3$$

$$= \frac{226,090}{580800} \times 14.3 = 5.6 \text{ FT}$$

CHECKS WITH ABS METHOD

B. WAVE HEIGHT = 10' (ASSUMED)

M_B MAX = 430500 KIPS - FT (SEE CONDITION 2)

$$H = \frac{226,090}{430500} \times 10 = \underline{5.3 \text{ FT}}$$

CHECKS WITH ABS METHOD

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USE ABS RULES TO DETERMINE ALLOWABLE WAVE HEIGHT FOR SHORTER LENGTH PONTOON UNITS

FOR $L = 400$ FT.

$H_e = 0.0172 L + 11.98 = 18.86$ FT.

$M_w = 6.9 \times 10^{-4} (400)^2 (75)(18.86)(1)(2.24)$

$= 349{,}800$ K FT.

$H = \frac{226{,}090}{349{,}800} (18.86) = \underline{12.2 \text{ FT}}$

FOR $L = 300$ FT

$H_e = 0.0172 L + 11.98 = 17.14$ FT.

$M_w = 6.9 \times 10^{-4} (300)^2 (75)(1)(2.24)$

$= 178{,}818$ K FT.

$H = \frac{226{,}090}{178{,}818} (17.14) = \underline{21.7 \text{ FT.}}$

ROLLING PERIOD

CALCULATION IS BASED ON THE FOLLOWING ASSUMPTIONS:

A) WEIGHT DISTRIBUTION EVEN

B) TOTAL LENGTH OF PIER (TWO PONTOONS) = 1200 FT.

C) TOTAL WEIGHT = 64900 KIPS, CORRESPONDING DRAFT = 11.65 FT.

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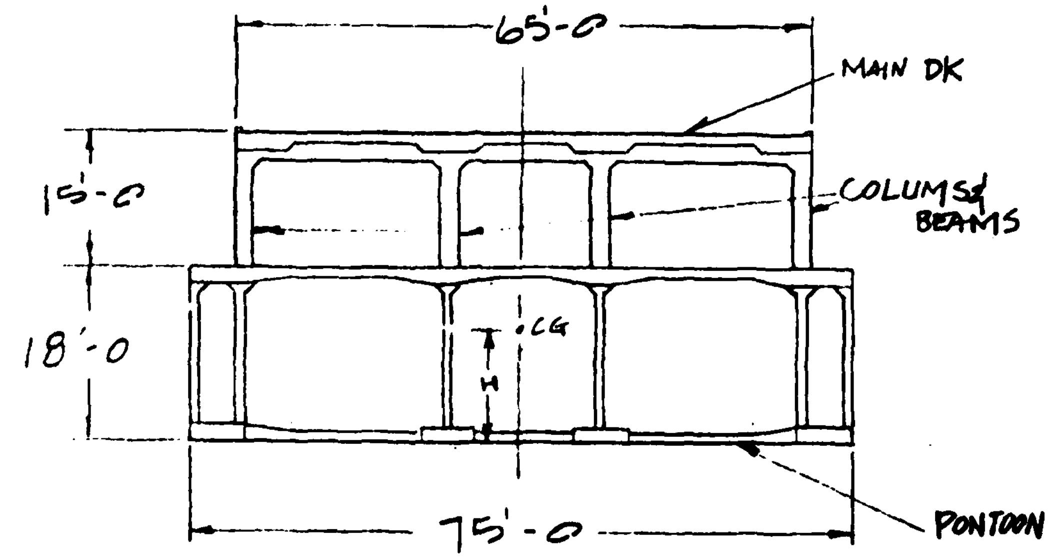

FIG. 2

WEIGHT:

PONTOON (1200')		46400	KIPS
MAIN DECK & COLUMN, ETC...		14000	KIPS
WEIGHT	=	60400	KIPS
CORRESPONDING DRAFT	=	10.85	FT.
UTILITIES 50#/FT.²	=	4500	KIPS
TOTAL WEIGHT	=	64900	KIPS
CORRESPONDING DRAFT	=	11.65	FT.

CENTER OF GRAVITY H:

$46400 \times 9 + 14000 \times (18+10) + 4500 \times (18+15+0.5) = 64900 \times H$

$H = 960350/64900 = 14.8$ FT ABOVE BASE LINE

CENTER OF BUOYANCE AT DRAFT 11.65 FT.

$11.65 \times \frac{1}{2} = 5.825$ FT ABOVE BASE LINE

BLOCK COEFFICIENT AT DRAFT 11.65 FT

$$C_b = 35\Delta / L \times B \times d = 64900 / 1200 \times 75 \times 11.65 \; \frac{2.24}{35}$$

$$= 0.97$$

Δ = DISPLACEMENT IN TONS SALT WATER

TRANSVERSE METACENTRIC HEIGHT:

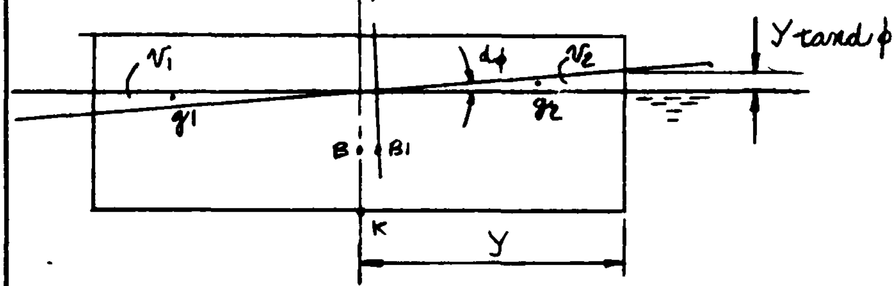

FIG. 3

$$\overline{BM} = \frac{\overline{BB_1}}{\tan d\phi} = \frac{v \times g_1 g_2}{\tan d\phi}$$

∇ = VOLUME OF DISPLACEMENT (35Δ)

v = VOLUME OF EACH WEDGE

WHEN $d\phi = 5°$

$v = (y \times y \tan d\phi)\frac{1}{2} \times L = \frac{1}{2} y^2 \times \tan d\phi \times L$

$= (\frac{75}{2})^2/2 \times \tan 5° \times 1200 = 73406$ CU FT

$\nabla = 1200 \times 75 \times 11.65 \times 0.97 = 1017045$ CU FT.

$\overline{g_1 g_2} = 2 \times \frac{2}{3} y = 50$ FT.

$\overline{BM} = 73406 \times 50 / 1017045 \times 0.087$

$= 41.5$ FT.

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

$= 5.82 + 41.5 = 47.32$ FT.

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

$= 47.32 - 14.8 = 32.52$ FT.

ROLLING PERIOD $T\phi = \frac{1.108k}{\sqrt{GM}}$

$= \frac{C \times B}{\sqrt{GM}}$

k - RADIUS OF GYRATION

B = BEAM

C = CONSTANT . 0.44 FOR SURFACE TYPE SHIP
0.37 FOR SUBMARINE
0.52 ASSUMED FOR PONTOON

$T\phi = \frac{0.52 \times 75}{\sqrt{32.52}} = 6.8$ SECONDS

WHEN $d\phi = 1°$

$v = (\frac{75}{2})^2/2 \times \tan 1° \times 1200 = 14727.655$ CU.FT.

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$\overline{BM} = 14727.655 \times 50 / 1017045 \times 0.17455$

$= 41.48$ FT.

APPROXIMATE MOMENT TO HEEL ONE DEGREE $= W.\overline{GM} \sin 1°$

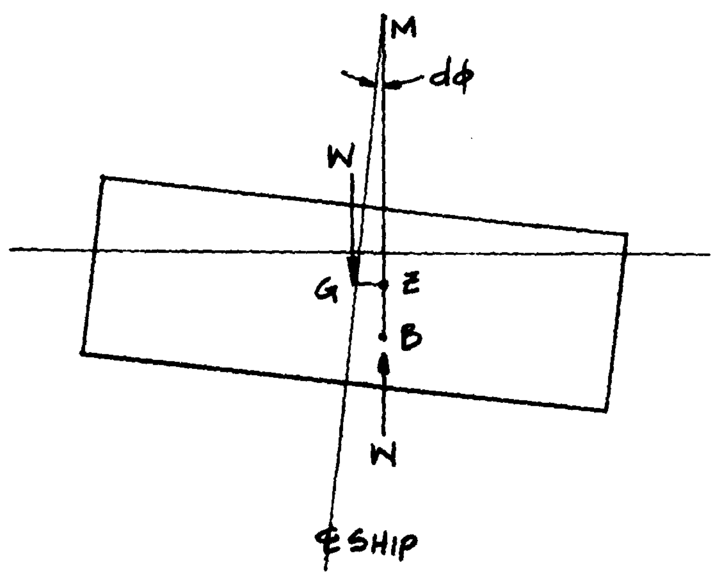
W = WEIGHT OF SHIP

$\overline{GZ}$ = RIGHTING ARM

$= \overline{GM} \sin d\phi$

$= 64900 \times 32.5 \times 0.017$

$= 35,880$ KIPS-FT



HYDRAULIC STATIC CURVES

CALCULATIONS ARE BASED ON ONE PONTOON.

$L \times B \times D = 600 \text{ FT} \times 75 \text{ FT} \times 18 \text{ FT}$

BLOCK COEFFICIEN = 0.97 (ASSUMED)

4 FT WL   AREA OF WATERPLANE, BM_L, BM ETC.

STATION	HALF ORDINATE	$\frac{1}{3}$SM	FUNCTION OF ORDINATE	ARM	FUNCTION OF LONGL MT	ARM	FUNCTION OF LONGL MT OF INERTIA	CUBES HALF-ORDINATE	$\frac{1}{3}$SM	FUNCTION OF HALF-CUBES
FP 0	36.375	$\frac{1}{4}$	9.1	5	45.5	5	227.5	48129	$\frac{1}{4}$	12032
$\frac{1}{2}$	TAKEN AVERAGE DEDUCTION OF PILE TRUNK (37.5×0.97) = 36.375	1	36.4	$4\frac{1}{2}$	163.8	$4\frac{1}{2}$	737.1		1	48129
1		$\frac{3}{4}$	27.3	4	109.2	4	436.8		$\frac{3}{4}$	36097
2		2	72.8	3	218.4	3	655.2		2	96258
3		1	36.4	2	72.8	2	145.6		1	48129
4		2	72.8	1	72.8	1	72.8		2	96258
					+ 682.5					
⊡ 5		1	36.4						1	48129
6		2	72.8	1	72.8	1	72.8		2	96258
7		1	36.4	2	72.8	2	145.6		1	48129
8		2	72.8	3	218.4	3	655.2		2	96258
9		$\frac{3}{4}$	27.3	4	109.2	4	436.8		$\frac{3}{4}$	36097
$9\frac{1}{2}$		1	36.4	$4\frac{1}{2}$	163.8	$4\frac{1}{2}$	737.1		1	48129
AP 10		$\frac{1}{4}$	9.1	5	45.5	5	227.5		$\frac{1}{4}$	12032
					− 682.5 + 682.5		4550		$f(I)$ =	721935
				$f(M)$ =	0					
						$\frac{[f(M)]^2}{f(A)}$ =	0			
						$f(I_L)$ =	4550			

TOTAL AREA = $f(A) \times m_1$ = 43680 FT^2

CENTER OF GRAVITY OF WATERPLANE FROM ORDINATE 5 = $f(M)/f(A) \times S$ = 0 FT

MOMENT OF WATERPLANE ABOUT ORDINATE 5 = $f(M) \times m_2$ = 0 FT^3

LONGITUDINAL I OF WP ABOUT AXIS THROUGH ITS CENTER OF GRAVITY = $f(I_L) \times m_3$ = 1.3×10^7 FT^4

TRANSVERSE I OF WP ABOUT CENTERLINE = $f(I) \times m_4$ = 1.9×10^7 FT^4

$m_1 = \frac{4}{3} \times S$, $m_2 = \frac{4}{3} \times S^2$, $m_3 = \frac{4}{3} \times S^3$, $m_4 = \frac{4}{9} \times S$, $S = 60$

MOMENT OF INERTIA ÷ (VOLUME DISPLACEMENT 174600 CU FT) = $\overline{BM}_L$ = 7446 $\overline{BM}$ = 108.8

CENTER OF BOUYANCY ABOVE BASE LINE = $\overline{KB}$ = 2 $\overline{KB}$ = 2

METACENTERS ABOVE BASE LINE = $\overline{KM}_L$ = 7448 $\overline{KM}$ = 110.8

TONS PER INCH IMMERSION = AREA ÷ 420 = 104

WATERLINE COEFFICIENT = AREA ÷ (L×B) = 0.97

DISPLACEMENT = (L×B×d) × C_B = Δ = 4989 TONS C_B = 0.97, BLOCK COEFFICIENT

8 FT WL

Δ = 9977 TONS ∇ = 349200 CU. FT.
C_B = 0.97
VCB = 4 FT.
LCB = 0 LCF = 0
$\overline{BM_L}$ = $1.3\times10^9/349200$ = 3722 $\overline{BM}$ = $1.9\times10^7/349200$ = 54.4
$\overline{KM_L}$ = 3726 $\overline{KM}$ = 58.4

12 FT WL

Δ = 14966 TONS ∇ = 523800 CU. FT.
C_B = 0.97
VCB = 6 FT
LCB = 0 LCF = 0
$\overline{BM_L}$ = $1.3\times10^9/523800$ = 2481.9 $\overline{BM}$ = $1.9\times10^7/349200$ = 36.3
$\overline{KM_L}$ = 2488 $\overline{KM}$ = 42.3

16 FT WL

Δ = 19354 TONS ∇ = 698400 CU. FT.
C_B = 0.97
VCB = 8 FT.
LCB = 0 LCF = 0
$\overline{BM_L}$ = $1.3\times10^9/698400$ = 1861.4 $\overline{BM}$ = $1.9\times10^7/698400$ = 27.2
$\overline{KM_L}$ = 1869 $\overline{KM}$ = 35.2

MOMENT TO ALTER TRIM ONE INCH;

WL, FT	M_L ABOVE BASE = KML, FT.	KG	GM_L, FT.	$\frac{\Delta \text{ TONS, SW} \times GM_L}{12\times \text{LENGTH}}$	MTI, FT-TONS
4	7448	14.8	7433	4989 x 7433/12x600	5150
8	3726	↓	3711	9977 x 3711/12x600	5142
12	2488		2473	14966 x 2473/12x600	5140
16	1869		1854	19354 x 1854/12x600	5138

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PROJECT:
ITEM:
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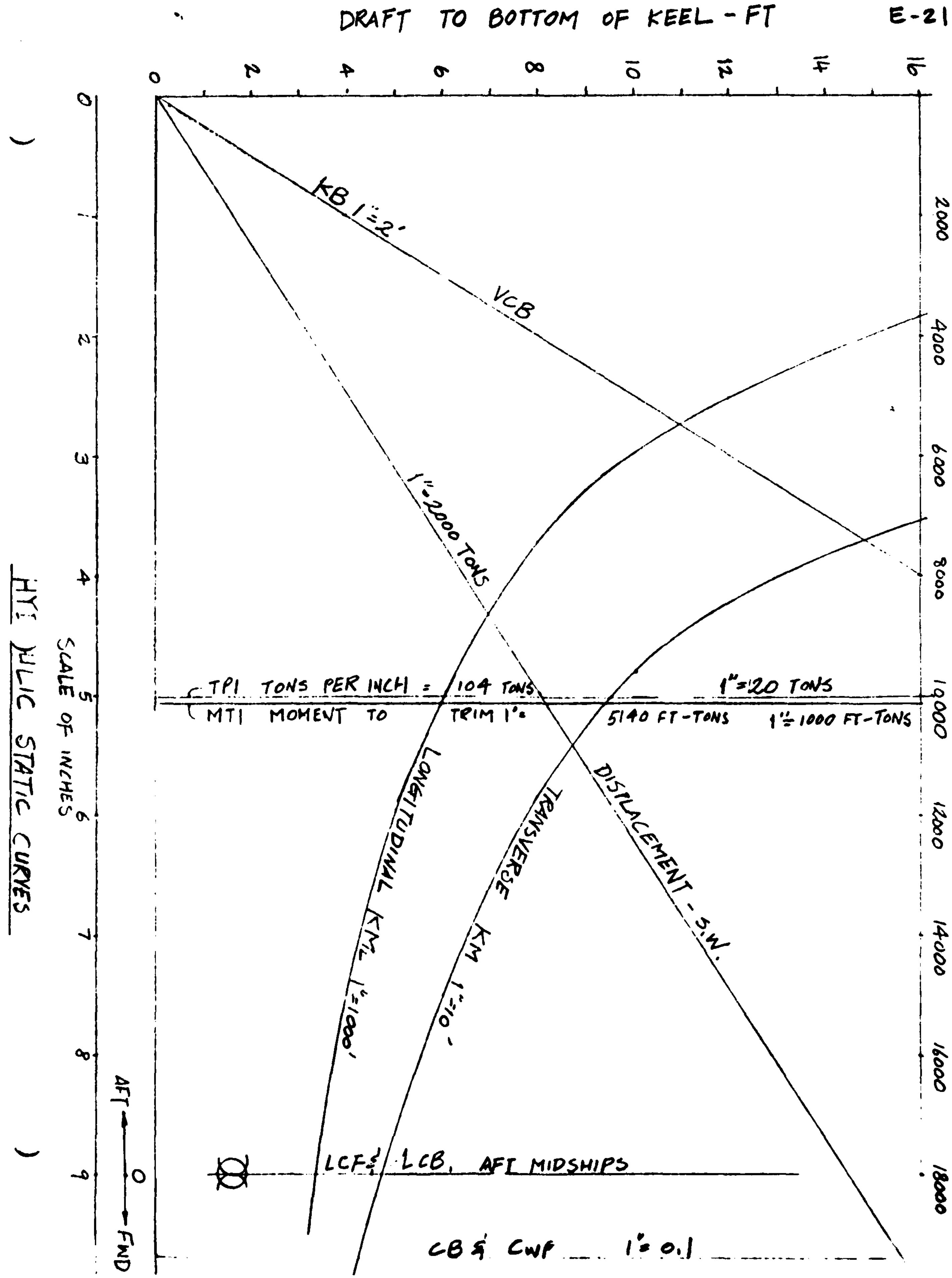
SUMMARY OF RESULTS OF CALCULATIONS

ITEMS, UP TO WL	4 FT. WL	8FT. WL	12FT. WL	16FT WL
DISPLACEMENT, TONS, SW(Δ)	4989	9977	14966	19954
BLOCK COEFFICIENT (C_B)	0.97	0.97	0.97	0.97
CENTER OF BUOYANCY ABOVE LINE (VCB)	2	4	6	8
CENTER OF BUOYANCY FROM ℄ (LCB)	0	0	0	0
LONGITUDINAL METACENTER ABOVE BASE LINE (FT)(KML)	7442	3726	2488	1869
TRANSVERSE METACENTER ABOVE BASE LINE (KM)	110.8	58.4	42.3	35.2
MOMENT TO ALTER TRIM 1" (MT 1)	5150	5142	5140	5138
TON PER INCH IMMERSION (TPI)	104	104	104	104
WATER PLANE COEFFICIEN (CWP)	0.97	0.97	0.97	0.97
C.G. OF WATERPLANE FROM ℄ (LCF)	0	0	0	0


DRAFT TO BOTTOM OF KEEL - FT
0
2
4
6
8
10
12
14
16
KB 1"=2'
VCB
1"=2000 TONS
TPI TONS PER INCH = 104 TONS
1"=20 TONS
MTI MOMENT TO TRIM 1" 5140 FT-TONS
1"=1000 FT-TONS
LONGITUDINAL KML 1"=1000'
TRANSVERSE KM 1"=10'
DISPLACEMENT - S.W.
LCF & LCB, AFT MIDSHIPS
CB & CWP 1"= 0.1
2000
4000
6000
8000
10000
12000
14000
16000
18000
SCALE OF INCHES
0
1
2
3
4
5
6
7
8
9
AFT
0
FWD
HYDROSTATIC CURVES

AD-A121 865 CONCEPTUAL DESIGN OF NAVY FLOATING PIER(U) LIN (T Y) INTERNATIONAL SAN FRANCISCO CA SEP 82 NCEL-CR-82-031 N62474-81-C-9404 3/3

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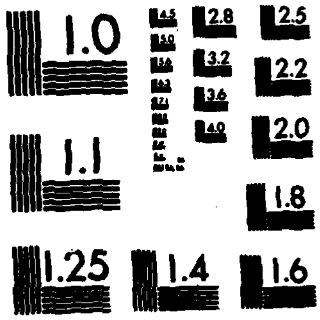


MICROCOPY RESOLUTION TEST CHART
NATIONAL BUREAU OF STANDARDS-1963-A

DAMAGE STABILTY

CALCULATIONS WERE CARRIED OUT FOR INDVIDUAL PONTOON, OF WHICH END CAMPARTMENTS No 1 PORT OR STABOARD & CENTER (CASE I) AND CONPARTMENTS No 1 & No 2 PORT OR No 1 & No 2 STBD (CASE II), AS SHOWN IN FIG, AND FIG. , ARE CONSIDERED TO BE THE MOST CRITICAL COMPARTMENTS IN CASE OF DAMAGE.

CASE I,

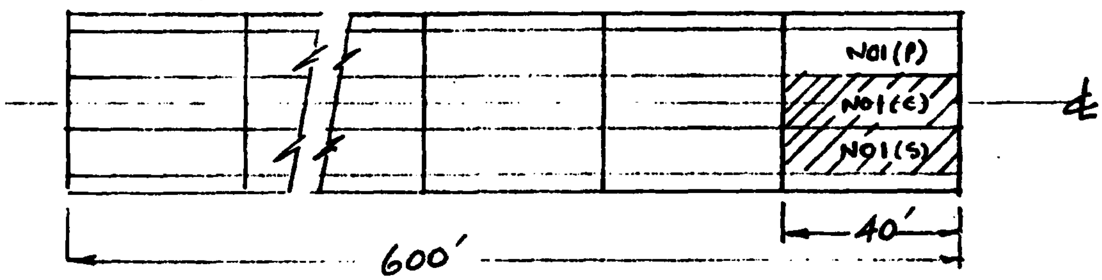
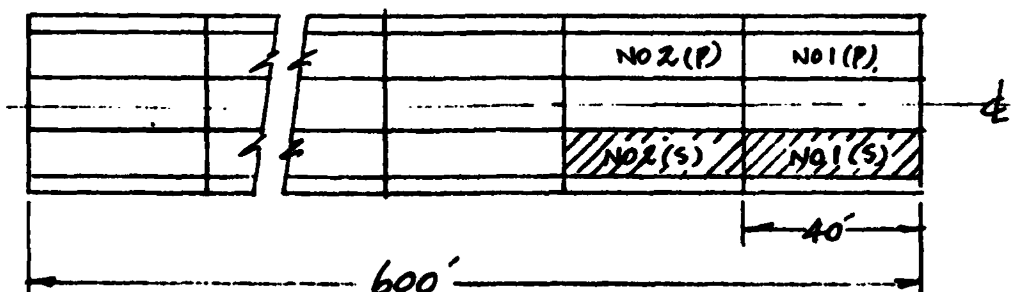
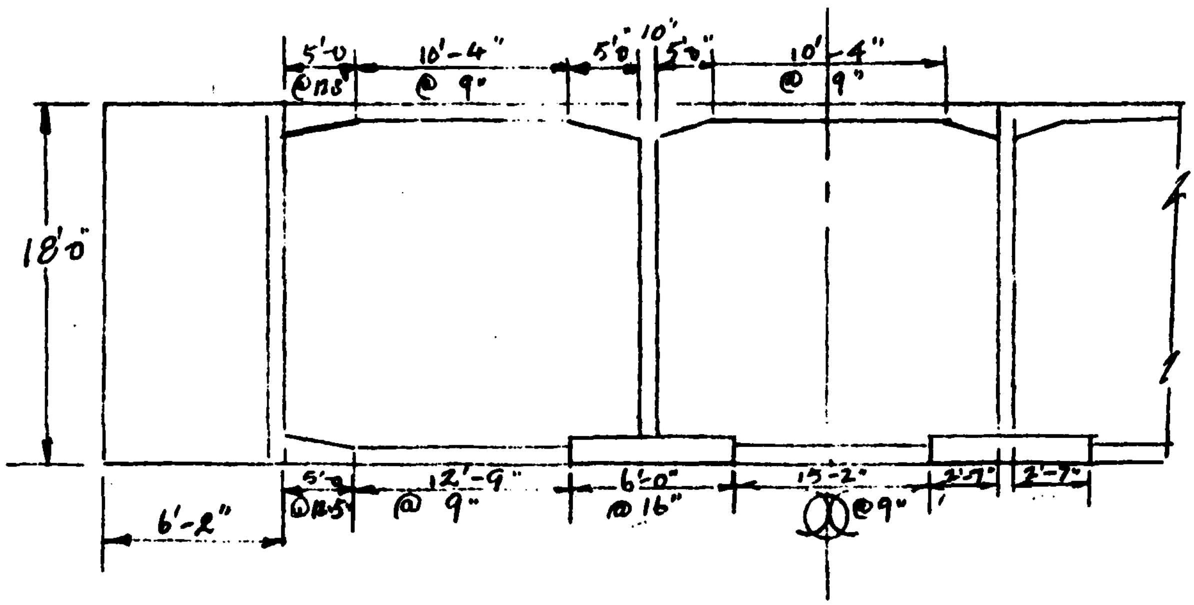


FIG.

CASE II,





FIG

AND BHD THICKNESS 12 INCHES.

NET LOST BUOYANCY AT DRAFT 11.65:

CASE I, $v' = (5+12.75+6+15.17+2.58) \times (40-1) \times 11.65 - [5 \times 1.04 + 12.75 \times 0.75 + 6 \times 1.3 + 15.17 \times 0.75 + 2.58 \times 1.3 + (11.65 - 1.3) \times 0.83] \times (40-1)$

$= 18855.5 - 1789.3$

$= 17066$ CU FT $= 487.6$ TONS

CASE II, $v' = [(5+12.75+2.58) \times (40-1) \times 11.65 - (5 \times 1.04 + 12.75 \times 0.75 + 2.58 \times 1.3) \times (40-1)] \times 2$

$= (9236.9 - 706.5) \times 2$

$= 17061$ CU FT $= 487.5$ TONS

CENTER OF v' FROM ℄:

CASE I, $3 + \frac{15.17}{2} = 10.59$ FT.

CASE II, $(5+12.75+2.58) \times \frac{1}{2} + 3.42 + \frac{15.17}{2} = 21.17$ FT.

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LOST TONS PER INCH

CASE I, $V/L \times 12 = 487.6 / 11.65 \times 12 = 3.5$ TONS

CASE II, $= 487.5 / 11.65 \times 12 = 3.5$ TONS

WATER PLANE AREA AFTER FLOODING

CASE I $\sqrt{2.52^2 + 600^2} \times 75 \times 0.97 = 34650$ SQFT

CASE II $\sqrt{2.31^2 + 600^2} \times 75 \times 0.97 = 34650$ SQFT

TRANSVERSE MOM. INERTIA/35 OF WATERPLANE AFTER FLOODING ABOUT ℄.

CASE I & II

$1.9 \times 10^7 / 35 = 542857$ FT-TONS

$\mu_s \times$ LOST WATERPLANE AFTER FLOODING

μ_s = SURFACE PERMEABILITY OF COMPARTMENT
$= 1.0$

CASE I, $1 \times (5 + 10.33 + 5) \times 2 \times (40 - 1) \times 0.97 = 1538.2$ FT^2

CASE II $1 \times (5 + 10.33 + 5) \times (80 - 2) \times 0.97 = 1538.2$ FT^2

$\mu_s \times i_T / 35$ OF LOST WATERPLANE AFTER FLOODING!

CASE I & II i_T = MOMENNT OF INERTIA OF LOST WATERPLANE

$= \frac{lb^3}{12}$ IN RECTANGULAR TANK

$= \frac{39 \times 20.33^3 \times 2}{12} = 54617$ FT^4

$\mu_s \times i_T / 35 = 1560.5$ FT - TONS

CASE I)

DAMAGE STABILITY-LOST BUOYANCY METHOD- SINKAGE AND TRIM-BASIC DATA --REF.5

ITEM No.	SOURCE	DESCRIPTION	UNIT					
1	-------------	T, DRAFT BEFORE DAMAGE	FT	11.65				
2	CURVES OF FORM	DISPLACEMENT	TONS	14529				
3	CURVES OF FORM	TONS PER INCH	TONS	104				
4	CURVES OF FORM	LCF F- A+	FT	0				
5	CURVES OF FORM	[illegible]	FT	43				
6	CURVES OF FORM	$\overline{KB}$	FT	5.83				
7	COMPARTMENT CALCULATIONS	NET LOST BUOYANCY *	TONS	487.6				
8	COMPARTMENT CALCULATIONS	VERTICAL CENTER OF (7)	FT	6.2				
9	COMPARTMENT CALCULATIONS	CENTER OF (7) FROM MIDSHIPS F- A+	FT	-280				
10	COMPARTMENT CALCULATIONS	CENTER OF (7) FROM CL P- S+	FT	+10.59				
11	COMPARTMENT CALCULATIONS	LOST TONS PER INCH	TONS	3.5				
12	COMPARTMENT CALCULATIONS	CENTER OF (11) FROM MIDSHIPS F- A+	FT	-280				
13	COMPARTMENT CALCULATIONS	CENTER OF (11) FROM CL P- S+	FT	+10.59				
14	$(3) - (11)$	REMAINING TONS PER INCH	TONS	100.5				
15	$\frac{(3)\times(4) - (11)\times(12)}{(14)}$	CENTER OF (14) FROM MIDSHIPS F- A+	FT	+9.75				
16	$\frac{(7)}{12\times(14)}$	APPROXIMATE SINKAGE	FT	0.4				
17	$(1) + (16)$	APPROXIMATE DRAFT AFTER FLOODING	FT	12.05				
18	CURVES OF FORM	TONS PER INCH AT (17)	TONS	104				
19	COMPARTMENT CALCULATIONS	LOST TONS PER INCH AT (17)	TONS	3.5				
20	$(18) - (19)$	REMAINING TONS PER INCH	TONS	100.5				
21	$\frac{(7)}{6\times[(14) + (20)]}$	SINKAGE	FT	0.4				
22	$(1) + (21)$	DRAFT AFTER FLOODING	FT	12.05				
23	CURVES OF FORM	TONS PER INCH AT (22)	TONS	104				
24	CURVES OF FORM	LCF AT (22) F- A+	FT	0				
25	CURVES OF FORM	MOM TO TRIM ONE INCH AT (22)	FOOT TONS	5140				
26	COMPARTMENT CALCULATIONS	LOST TONS PER INCH AT (22)	TONS	3.5				
27	COMPARTMENT CALCULATIONS	CENTER OF (26) FROM MIDSHIPS F- A+	FT	-280				
28	COMPARTMENT CALCULATIONS	CENTER OF (26) FROM CL P- S+	FT	+10.59				
29	$\frac{(23)\times(24) - (26)\times(27)}{(23) - (26)}$	CENTER OF REMAINING TPI F- A+	FT	+9.75				
30	$(9) - \frac{[(15) + (29)]}{2}$	TRIMMING LEVER F- A+	FT	289.75				
31	$(7)\times(30)$	TRIMMING MOMENT F- A+	FOOT TONS	141282				
32	$\frac{(23)\times(26)\times[(24) - (27)]^2}{L\times[(23) - (26)]}$	------------	FOOT TONS	473.2				
33	------------	AVERAGE LENGTH OF LOST WATERPLANE	FT	39				
34	$\frac{(26)\times(33)^2}{12L}$	------------	FOOT TONS	0.74				
35	$12[(25) - (32) - (34)]$	NET MOMENT TO TRIM ONE FOOT	FOOT TONS	55993				
36	$(31)/(35)$	TRIM IN FEET F- A+	FT	-2.52				

* NOTE ITEM (7), NET LOST BUOYANCY = $\Sigma \frac{v}{35}$ (or T) FOR ALL SPACES OPENED TO SEA LESS TOTAL TONS OF LIQUIDS WHICH WERE IN ANY BREACHED TANKS BEFORE DAMAGE

Fig. 24 Form for calculation of sinkage and trim, lost buoyancy method

CASE I,

DAMAGE STABILITY - LOST BUOYANCY METHOD - HEEL AND $\overline{GM}$ --- REF. 5

ITEM No.	SOURCE	DESCRIPTION	UNIT					
1	------------	T, DRAFT BEFORE DAMAGE	FT	11.65				
37	(22) - (36) (0.5 + (29)/L)	DRAFT FORWARD AFTER FLOODING (WL_2)	FT	13.35				
38	(22) + (36) (0.5 - (29)/L)	DRAFT AFT AFTER FLOODING (WL_2)	FT	10.83				
39	DIRECT CALCULATIONS OR CURVES OF FORM	WATERPLANE AREA AT WL_2	FT^2	43150				
40	DIRECT CALCULATIONS OR CURVES OF FORM	TRANS. MOM. INERTIA / 35 OF (39) ABOUT C.L.	FOOT TONS	54285				
41	COMPARTMENT CALCULATIONS	μ_S = LOST WATERPLANE AT WL_2	FT^2	1538.2				
42	COMPARTMENT CALCULATIONS	CENTER OF (41) FROM C.L. P- S+	FT	10.59				
43	COMPARTMENT CALCULATIONS	$\mu_S i_T$/35 OF (41) ABOUT C.L.	FOOT TONS	1560.5				
44	(41) × (42)	------------	FT^3	16289.5				
45	$(44)^2$/35 ((39) - (41))	------------	FOOT TONS	180				
46	(40) - (43) - (45)	------------	FOOT TONS	54117				
47	(5) - (6) - (46)/(2)	$\overline{BM}$ LOSS	FT	0.07 ~0				
48	((1) + (22))/2	VERTICAL CENTER OF PARALLEL SINKAGE LAYER	FT					
49	(48) - (8)	$b\,b_1$	FT					
50	(7) × (49) / (2)	VCB RISE DUE TO SINKAGE	FT					
51	$(36)^2$	$TRIM^2$	FT^2					
52	(35) × (51) / (2 × L × (2))	VCB RISE DUE TO TRIM	FT					
53	(47) + 0.17 - (50) - (52)	INTACT GM FOR 2" GM DAMAGED	FT					
54	TRIM AND HEEL DIAGRAM	MINIMUM TANGENT TO MARGIN LINE	--					
55	------------	CORRESPONDING ANGLE OF HEEL	DEG					
56	------------	SELECTED ANGLE ϕ	DEG					
57	------------	TAN (56)	--					
58	(21) + ((27) - (29)) (36)/L	HEIGHT WL TO WL_2 IN WAY OF (27)	FT					
59	(-12 × (26) + (28) × (58)) / (7)	CENTER OF NET ADDED BUOYANCY FROM CL P- S+	FT					
60	(59) - (10)	TRANSVERSE SHIFT OF BUOYANCY P- S+	FT					
61	-(7) × (60)	TRANSVERSE MOMENT P- S+	FOOT TONS					
62	COMPARTMENT CALCULATIONS	MOM. CORRECTION FOR CHANGE IN μ ETC. HEELING THROUGH ϕ P- S+	FOOT TONS					
63	[(61) + (62)] / ((57) × (2))	------------	FT					
64	FIGS. 43,44 OR DIRECT CALCULATIONS	"F"	--					
65	(53) + (63) - [(64) × (49)] / (2) - 0.17	INTACT $\overline{GM}$ TO LIMIT HEEL TO (56)	FT					
66	(53) OR (65) WHICHEVER IS LARGER	REQUIRED INTACT $\overline{GM}$	FT					
67	(66) + 0.17 - (53)	CORRESPONDING $\overline{GM}$ IN DAMAGE CONDITION	FT					

IF (53) IS MORE THAN (65), EQUILIBRIUM ANGLE OF HEEL AND RELATED "F" ARE THOSE WHICH SATISFY EQUATION: $0.17 + "F" = \frac{(46)}{(2)} + \frac{(61) + (62)}{(2) \times TAN\ \phi}$

MOMENT TO HEEL ONE DEGREE
= W × $\overline{GM}$ Sin 1° — PAGE 21
= 14529 × 28.2 × 0.017
= 6965 TONS - FT

∵ KG = 14.8 FT — PAGE 19
$\overline{GM} = \overline{KM} - \overline{KG}$
= 43 - 14.8
= 28.2 FT

ANGLE OF HEEL =
(9) × (10) / 6965 = 0.7 DEG

X = 35.5 (12 "/ft) (Tan 0.7°) = 5.2"
FREEBOARD CHANGE

Fig. 25 Form for calculation of heel and GM, lost buoyancy method

CASE II,

DAMAGE STABILITY - LOST BUOYANCY METHOD - SINKAGE AND TRIM - BASIC DATA. — REF. 5

ITEM No.	SOURCE	DESCRIPTION	UNIT					
1	------------	T, DRAFT BEFORE DAMAGE	FT	11.65				
2	CURVES OF FORM	DISPLACEMENT	TONS	14529				
3	CURVES OF FORM	TONS PER INCH	TONS	104				
4	CURVES OF FORM	LCF F- A+	FT	0				
5	CURVES OF FORM	$\overline{KM}$	FT	43				
6	CURVES OF FORM	$\overline{KB}$	FT	5.83				
7	COMPARTMENT CALCULATIONS	NET LOST BUOYANCY *	TONS	487.5				
8	COMPARTMENT CALCULATIONS	VERTICAL CENTER OF (7)	FT	6.2				
9	COMPARTMENT CALCULATIONS	CENTER OF (7) FROM MIDSHIPS F- A+	FT	-260				
10	COMPARTMENT CALCULATIONS	CENTER OF (7) FROM C.L. P- S+	FT	+21.17				
11	COMPARTMENT CALCULATIONS	LOST TONS PER INCH	TONS	3.5				
12	COMPARTMENT CALCULATIONS	CENTER OF (11) FROM MIDSHIPS F- A+	FT	-260				
13	COMPARTMENT CALCULATIONS	CENTER OF (11) FROM C.L. P- S+	FT	+21.17				
14	(3) - (11)	REMAINING TONS PER INCH	TONS	100.5				
15	$\frac{(3)\times(4) - (11)\times(12)}{(14)}$	CENTER OF (14) FROM MIDSHIPS F- A+	FT	+9.05				
16	$\frac{(7)}{12\times(14)}$	APPROXIMATE SINKAGE	FT	0.4				
17	(1) + (16)	APPROXIMATE DRAFT AFTER FLOODING	FT	12.05				
18	CURVES OF FORM	TONS PER INCH AT (17)	TONS	104				
19	COMPARTMENT CALCULATIONS	LOST TONS PER INCH AT (17)	TONS	3.5				
20	(18) - (19)	REMAINING TONS PER INCH	TONS	100.5				
21	$\frac{(7)}{6\times[(14)+(20)]}$	SINKAGE	FT	0.4				
22	(1) + (21)	DRAFT AFTER FLOODING	FT	12.05				
23	CURVES OF FORM	TONS PER INCH AT (22)	TONS	104				
24	CURVES OF FORM	LCF AT (22) F- A+	FT	0				
25	CURVES OF FORM	MOM. TO TRIM ONE INCH AT (22)	FOOT TONS	5140				
26	COMPARTMENT CALCULATIONS	LOST TONS PER INCH AT (22)	TONS	3.5				
27	COMPARTMENT CALCULATIONS	CENTER OF (26) FROM MIDSHIPS F- A+	FT	-260				
28	COMPARTMENT CALCULATIONS	CENTER OF (26) FROM C.L. P- S+	FT	+21.17				
29	$\frac{(23)\times(24) - (26)\times(27)}{(23) - (26)}$	CENTER OF REMAINING TPI F- A+	FT	+9.05				
30	$(9) - \frac{[(15)+(29)]}{2}$	TRIMMING LEVER F- A+	FT	269.05				
31	(7) × (30)	TRIMMING MOMENT F- A+	FOOT TONS	131162				
32	$\frac{(23)\times(26)\times[(24)-(27)]^2}{L\times[(23)-(26)]}$	------------	FOOT TONS	408				
33	------------	AVERAGE LENGTH OF LOST WATERPLANE	FT	78				
34	$\frac{(26)\times(33)^2}{12L}$	------------	FOOT TONS	2.96				
35	$12[(25) - (32) - (34)]$	NET MOMENT TO TRIM ONE FOOT	FOOT TONS	56748				
36	(31)/(35)	TRIM IN FEET F- A+	FT	-2.31				

* NOTE ITEM (7), NET LOST BUOYANCY = $\Sigma \frac{\mu v}{35}$ (or T) FOR ALL SPACES OPENED TO SEA LESS TOTAL TONS OF LIQUIDS WHICH WERE IN ANY BREACHED TANKS BEFORE DAMAGE

Fig. 26 Form for calculation of sinkage and trim, lost buoyancy method

CASE II,

DAMAGE STABILITY - LOST BUOYANCY METHOD - HEEL AND $\overline{GM}$ - REF. 5

ITEM No.	SOURCE	DESCRIPTION	UNIT					
1	------------	T, DRAFT BEFORE DAMAGE	FT	11.65				
37	(22) - (36) (0.5 + (23)/L)	DRAFT FORWARD AFTER FLOODING (WL_2)	FT	13.24				
38	(22) + (36) (0.5 - (23)/L)	DRAFT AFT AFTER FLOODING (WL_2)	FT	1090				
39	DIRECT CALCULATIONS OR CURVES OF FORM	WATERPLANE AREA AT WL_2	FT^2	43650				
40	DIRECT CALCULATIONS OR CURVES OF FORM	TRANS MOM. INERTIA / 35 OF (39) ABOUT C.L.	FOOT TONS	542857				
41	COMPARTMENT CALCULATIONS	μ_3 × LOST WATERPLANE AT WL_2	FT^2	1532				
42	COMPARTMENT CALCULATIONS'	CENTER OF (41) FROM C.L. P- S+	FT	+21.17				
43	COMPARTMENT CALCULATIONS	$\mu_3 i_T/35$ OF (41) ABOUT C.L.	FOOT TONS	1560.5				
44	(41) × (42)	--------------	FT^3	32584				
45	(44)²/35 ((39) - (41))	--------------	FOOT TONS	719.5				
46	(40) - (43) - (45)	--------------	FOOT TONS	540577				
47	(5) - (6) - (46)/(2)	$\overline{BM}$ LOSS	FT	-.003 ~0				
48	((1) + (22))/2	VERTICAL CENTER OF PARALLEL SINKAGE LAYER	FT					
49	(48) - (8)	b b_1	FT					
50	((7) × (49))/(2)	VCB RISE DUE TO SINKAGE	FT					
51	(36)²	TRIM²	FT^2					
52	((35) × (51))/(2 × L × (2))	VCB RISE DUE TO TRIM	FT					
53	(47) + 0.17 - (50) - (52)	INTACT GM FOR 2" GM DAMAGED	FT					
54	TRIM AND HEEL DIAGRAM	MINIMUM TANGENT TO MARGIN LINE	--					
55	------------	CORRESPONDING ANGLE OF HEEL	DEG					
56	------------	SELECTED ANGLE Φ	DEG					
57	------------	TAN (56)	--					
58	(21) + ((27) - (29)) (36)/L	HEIGHT WL TO WL_2 IN WAY OF (27)	FT					
59	(-12 × (26) × (28) × (58))/(7)	CENTER OF NET ADDED BUOYANCY FROM C L P- S+	FT					
60	(59) - (10)	TRANSVERSE SHIFT OF BUOYANCY P- S+	FT					
61	- (7) × (60)	TRANSVERSE MOMENT P- S+	FOOT TONS					
62	COMPARTMENT CALCULATIONS	MOM. CORRECTION FOR CHANGE IN μ ETC. HEELING THROUGH Φ P- S+	FOOT TONS					
63	((61) + (62))/((57) × (2))	--------------	FT					
64	FIGS. 43,44 OR DIRECT CALCULATIONS	"r"	--					
65	(53) + (63) - ((64) × (46))/(2) - 0.17	INTACT $\overline{GM}$ TO LIMIT HEEL TO (56)	FT					
66	(53) OR (65) WHICHEVER IS LARGER	REQUIRED INTACT $\overline{GM}$	FT					
67	(66) + 0.17 - (53)	CORRESPONDING $\overline{GM}$ IN DAMAGE CONDITION	FT					

IF (53) IS MORE THAN (65), EQUILIBRIUM ANGLE OF HEEL AND RELATED "r" ARE THOSE WHICH SATISFY EQUATION: $0.17 + "r" \times \frac{(46)}{(2)} = \frac{(61) + (62)}{(2) \times \tan\Phi}$

ANGLE OF HEEL =
(7) X (10) / 6965 = 1.5 DEG

X = 35.5' (12"/') tan 1.5° = 11.2" FREEBOARD CHANGE

Fig. 27 Form for calculation of heel and GM, lost buoyancy method

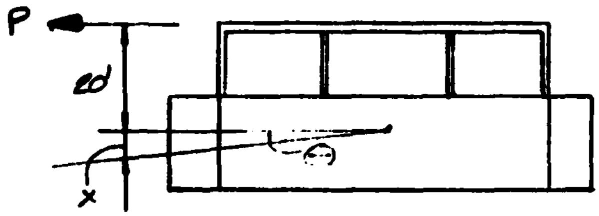
TYLIN INTERNATIONAL
STRUCTURAL ENGINEERING
315 Bay St., San Francisco, Ca. 94133

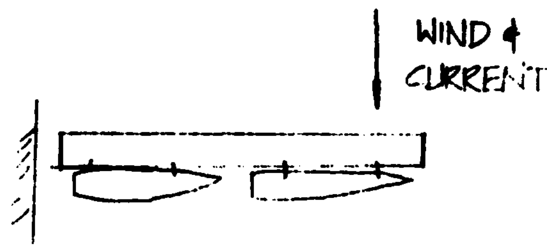
PROJECT:
ITEM: HEEL
DESIGN:
DATE:

SHEET: E-29
OF
REVISION:

HEEL FROM BERTHED LOADS

P FROM WIND & CURRENT ON SHIPS ON LEE SIDE OF PIER





ASSUME LOAD, P, IS 1/2 OF MAX WIND & CURRENT ACTING ON 4 SHIPS BERTHED AT PIER. THIS IS CONSERVATIVE ASSUMPTION BECAUSE PIER WILL ABSORB SOME OF THE WIND AND MOST OF THE CURRENT FORCE.

$$P = \frac{2730\,K}{2} = 1365\,K$$

$$M = (1365)(20') = 27{,}300\ K\text{-FT.}$$

$$\text{HEEL } \theta = \frac{27{,}300}{35{,}880_K} = 0.763°$$

MOMENT TO CAUSE HEEL OF 1°, FROM NAVAL ARCHITECT CALCS.

$$x = (37.5')(12)(\tan 0.763°) = 6.0"$$ HEEL IN WORST STORM.

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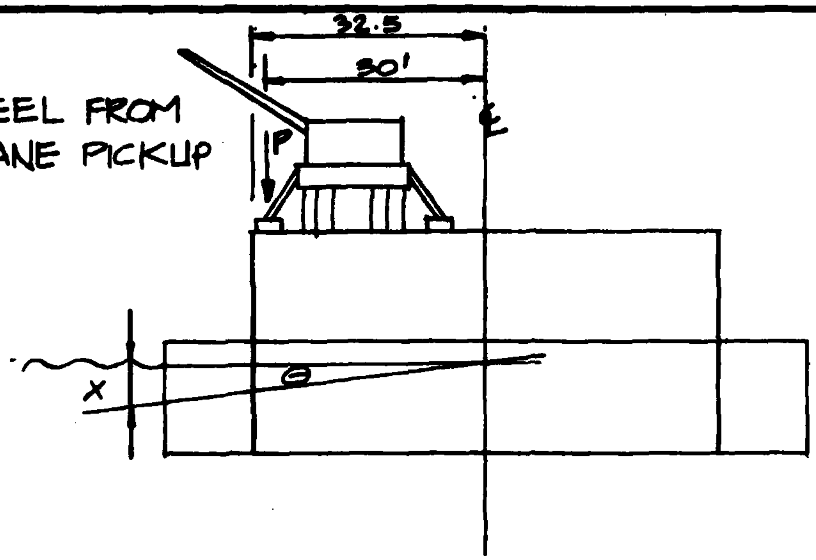
PROJECT:
ITEM: HEEL
DESIGN:
DATE:

SHEET: E-30
OF
REVISION:

HEEL FROM CRANE PICKUP



WT CRANE = 140 K
MAX LOAD = 180 K
320 K

MAX $P = 320^K$ FOR CRANE ABOUT TO TIP OVER

$M = 320(30') = 9600$ KFT.

MOMENT TO CAUSE 1° HEEL IS 35,880 KFT.

HEEL $\theta = \frac{9600}{35,880} = 0.268°$

$\tan \theta = \frac{x}{37.5'}$

$x = 37.5'(12)(\tan 0.268) = \underline{\underline{2.1}}''$

APPENDIX F: MATERIAL QUANTITY TAKE OFFS.

@ 125 PCF = 3.375 K/CY

CONCRETE: $\frac{56{,}050 \text{ K}}{3.375} = \underline{16{,}600}$ CY

PRESTRESSING STRAND:

LONG'L PONTOON: SAY - 102 TENDONS,

0.6 ϕ × 7 - STRAND: = 714 STRANDS

LONG'L MAIN DECK:

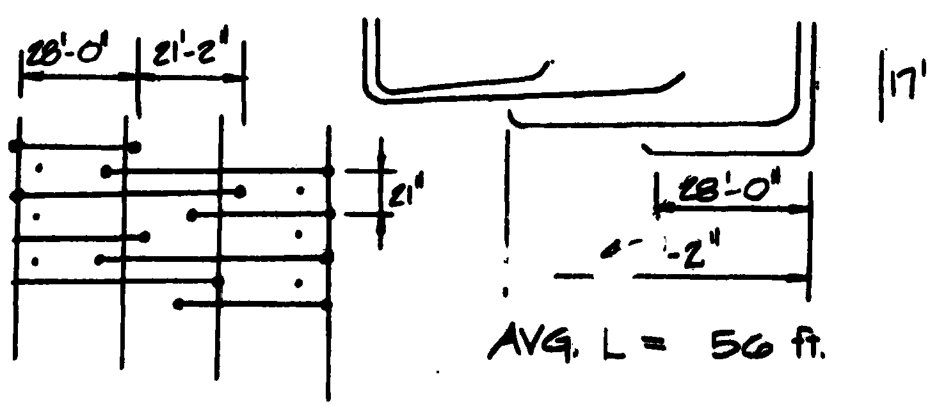
18" SECTIONS: 4×0.6 @ 1'-0" C-C: = 66 TENDONS

4(66) = 264 STRANDS

$\Sigma\Sigma$ = 714 + 264 = 978 STRANDS × 1200 ft.

= 1,173,600 ft.

TRANSVERSE PONTOON P/S:



AVG. L = 56 ft.

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PROJECT:	SHEET: F-2
ITEM: QUANTITIES	OF
DESIGN:	REVISIONS
DATE: R.E.	

111 FT. EVERY 1.75 FT = 63.5 FT./FT. @ 4×0.6 ϕ

Σ = 63.5(1200)(4) = 304,800 FT.

∴ ΣΣΣ STRAND = 1,173,600 + 304,800 = 1,478,400 LF

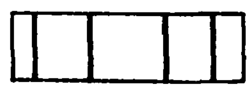
ADD 5% FOR WASTE:

SAY – 1,552,320 LF @ 0.7375 #/FT.

Wt = 1,145,300 Lb (P/S STRANDS)

PRESTRESSING ROD: (1 3/8 ϕ DYWIDAG)

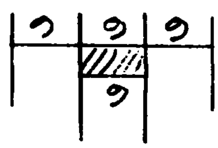
(USED IN INTERIOR WALLS AND BULKHEADS): (GR 150)



LONG'L WALLS: L = 17'-6" × 4 = 70 FT. SPACING 21" OR 1.75'

70 ÷ 1.75 = 40 FT/FT. ΣL = 40(1200) = 48,000 FT.

BULKHEADS: 36 RODS/BULKHEAD



17.5 (36)(30) = 18,900 FT.

Σ = 66,900 FT. + 5% = 70,250 FT.

WT. @ 5.56 #/FT. = 390,600 # (P/S ROD)

ΣWT 1,145,300 + 390,600 = 1,535,900 lb TOTAL P/S

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PROJECT:
ITEM: QUANTITIES
DESIGN:
DATE: R.Z.

SHEET: F-3
OF
REVISION:

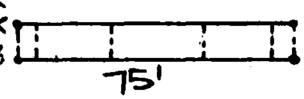
MILD STEEL (REBAR).

SAY - #4 @ 12" C-C ALL FACES:

MAIN DECK

LOWER DECK

BOTTOM SLAB



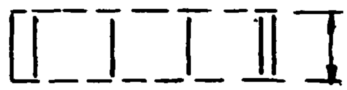
HORIZONTAL:

$\Sigma A = (65 + 150)(1200)$

$= 258,000 \text{ ft}^2$

VERTICAL:

LONG'L



$\Sigma A_{VERT} = 6(16)(1200) = 115,200 \text{ ft}^2$

BULKHEADS : $30(16)(90) = 43,200 \text{ ft}^2$

$\Sigma\Sigma A = 258,000 + 115,200 + 43,200 = 416,400 \text{ ft}^2$

$A/\text{ft}^2 = 4 \text{ ft} @ 0.668 \#/\text{ft} = 2.672 \#/\text{ft}^2$

$\Sigma Wt = 2.672(416,400) = 1,112,620 \#$

COLUMNS @ SAY - 2½% $.025(18)^2 = 8 \text{ IN}^2$

(8 - #9)

TOTAL COLUMNS: (27 #/ft)

$\Sigma L = 60 \text{ ft} \times 30 = 1800 \text{ ft}$

$WT. = 1800(27) \equiv 48,600 \#$

$\therefore \Sigma\Sigma$ WT. REBAR = 1,161,220 #

ADD 5% FOR LAPS & DETAILS

WT = 1.05 (1,161,200 #) : 1,220,000# REBAR

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PROJECT:	
ITEM:	QUANTITIES
DESIGN:	
DATE:	R.Z.

SHEET: F-4
OF
REVISION:

BATTER PILES : (L.F.)

As @ 1" WALL, 36" OD, = 110 IN^2

W = 375 #/FT SAY - A36 OR A53

(58 PILES) (100' LONG) (375 #/FT) = 2,175,000 LBS

VERTICAL PILES

As @ 1" WALL, 48" OD, = 147.7 IN^2

W = 500 #/FT SAY - A 53

(58 PILES) (100' LONG) (502 #/FT) = 2,911,600 LBS.

END

DATE
FILMED

1-88

DTIC