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# PORT MUHAMMAD-BIN-QASIM PROJECT

# DETAILED DESIGN REPORT

## IRON ORE & COAL BERTH

DECEMBER 1975

JAPAN INTERNATIONAL COOPERATION AGENCY

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## CHAPTER I PLANNING OF IRON ORE AND COAL BERTH

### I-1 PRINCIPLE OF PLANNING

In 1970, it was decided to establish a major up-to-date Steel Mill in Pakistan. Port Qasim, with the advantage of deep sea waters for the transport of raw materials, was selected as the location of the Steel Mill.

From the beginning of the feasibility study, alternate sites were considered for the location of the Steel Mill. From the point of marine transport, the area around the entrance of Phitti Creek will be a favorable site with access of deep water. However, a wide space on land with a favorable foundation may be secured in the rear area of Gharo Creek. Comparing the two sites, it was decided to locate the Steel Mill in the Gharo Creek area.

The Gharo Creek is 24 km inland from the entrance of Phitti Creek. As iron ore and coal must be transported to the inland location by large carriers, a wide channel must be dredged over a long distance. In general, ports are developed for medium and small size vessels in the early stages, and gradually develop into major ports for large vessels. Contrary to the general port development policy, Port Qasim will be developed to meet the demand of large ore carriers in the first stage, and general cargo berths for medium vessels will be constructed in the second stage.

The construction of the Iron Ore and Coal Berth will be a siginificant project in the development of Port Qasim.

Though situated at a considerable distance form the sea,the Gharo

Creek area prossesses advantages for the construction of a port complex. Besides the Iron Ore and Coal berth, various terminals may be constructed in the area. Wtih a Port Control Office in the area as well, with various facilities centered in the Gharo Creek area, it will be a great advantage from the point of port operation.

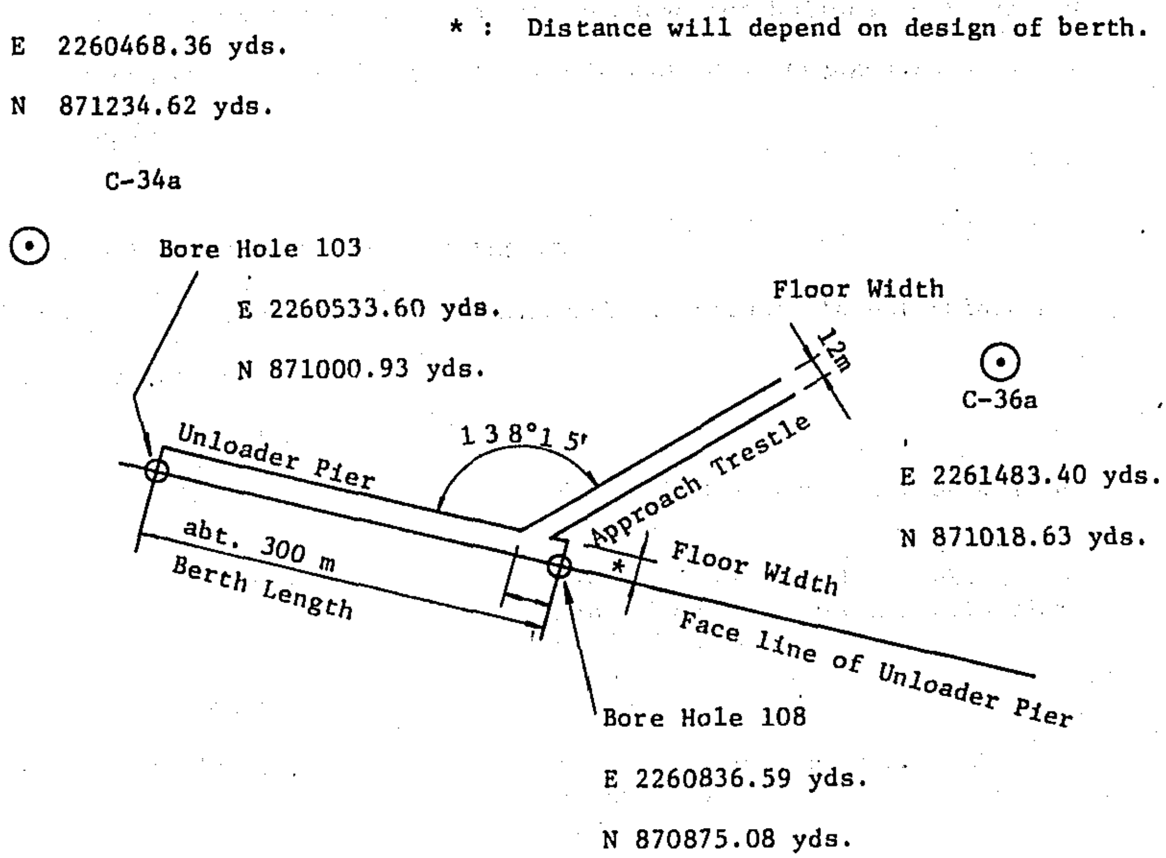
However, the Iron Ore and Coal Berth will be a facility for the exclusive use of the Steel Mill, while other facilities will be for public use. An exclusive berth will be operated independently from public berths. From the point of ore dust and noise during loading and unloading, it will be advisable to construct the Iron Ore and Coal Berth at a certain distance from the public wharves.

It will be quite feasible to develop a port complex along the banks of Gharo Creek, and construct the Iron Ore and Coal Berth in the adjacent area.

## I-2 LOCATION AND ALIGNMENT

The proposed site for the Iron Ore and Coal Berth at the entrance of Gharo Creek cannot be said to be a favorable location for the construction of berth facilities from the geographic and hydraulic point of view, with a crossing of creek systems, subject to the effect of tidal currents and littoral drift. However, at the shortest distance from the Steel Mill, the site has been selected with regard to the flow of Gharo Creek. With considerations to avoid hydraulic changes which may affect sedimentation and erosion within the creek as far as possible, and to fall in line with the flow in the creek, the direction of the pier face line has been determined as follows.

Fig. I-1 Alignment of Berth




## I-3 OUTLINE OF FACILITIES

The berth is to be designed to meet requirements of vessels of 25,000 DWT - 50,000 DWT in the first stage of port development, and vessels of 75,000 DWT in the second stage. However, structures will be designed for vessels of 75,000 DWT to provide for the second stage.

### 1. Berth

The berth will include berthing facilities, unloader pier, belt conveyor system, ducts for water supply, electric power and fueling, and mooring facilities.

### 2. Approach Trestle

Iron ore and coal will be delivered from the berth of the ore carrier to the Steel Mill by a belt conveyor system. The belt conveyor will be installed on an approach trestle. The approach trestle will also serve as the passage for bulldozers to unload iron ore and coal, as well as workers to be engaged in unloding operations.

## I-4 ALTERNATE TYPES OF CONSTRUCTION

The following three alternate plans will be studied to determine the structure of the berth.

1) <u>Gravity type wharf</u>

| | |
|---|---|
| Advantage: | The method of construction will be simple. |
| Disadvantage: | The foundation will be subject to scour by tidal currents with risks of failure. |

2) <u>Open type pier</u>

| | |
|---|---|
| Advantage: | May be easily connected with rear areas. The foundation will withstand scouring. |
| Disadvantage: | With a wide area, the cost of construction will be high. |

3) <u>Detached pier type wharf</u>

| | |
|---|---|
| Advantage: | The foundation will withstand scouring, and cost of construction will be low. |
| Disadvantage: | An approach will be required for connection with rear areas. |

From comparison of the above three types of construction, it is recommended that the berth be designed as a detached pier type construction.

# CHAPTER II SOIL CONDITIONS

Subsoil investigations at the proposed site for the Iron Ore and Coal Berth have been carried out by M/s Volkerman Ltd. (Pakistan) from 1945 to 1975. The ground foundation and soil conditions will be evaluated on the basis of the report of the above investigations, "Iron Ore and Coal Berths, Memorandum, Stratigraphy and Subsoil Conditions, September 1975," prepared by the National Engineering Services, Ltd.

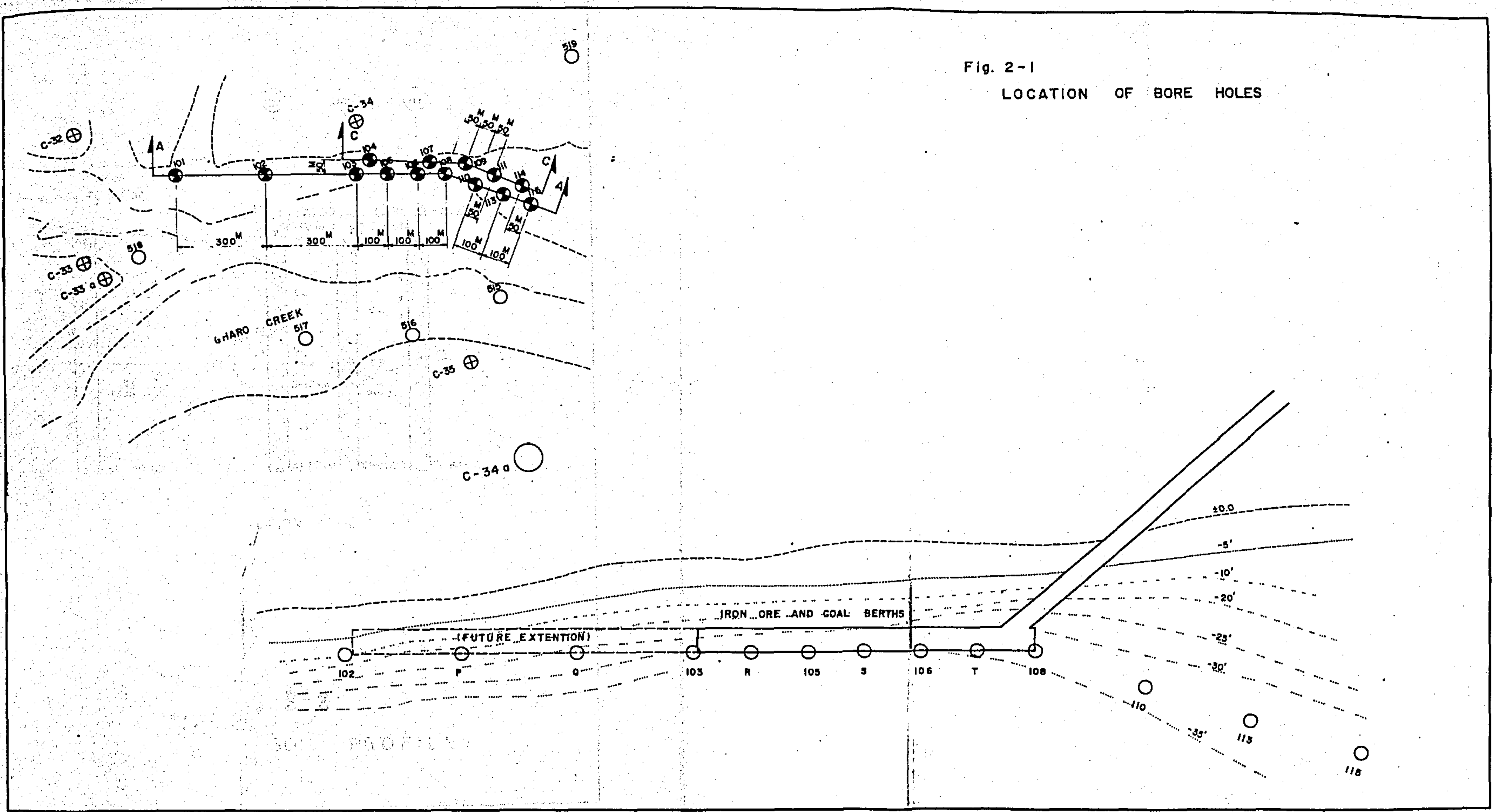
The location of bore holes and the soil profile are as shown in Fig. 2-1 and Fig. 2-2.

Furthermore, from October 1975, particularly in regard to the soil conditions at the proposed site of the berth, supplementary investigation has been planned at three points, R,S, and T along the proposed berth alignment to obtain accurate data for designing, and to check the results of previous investigations. The investigation is being carried out by M/s Volkerman Ltd, with technical advice from specialists in soil investigation from Japan.

Results obtained from the supplementary investigation so far are boring logs of the above three bore holes given in Fig. 2-3.

Fig. 2-1

LOCATION OF BORE HOLES



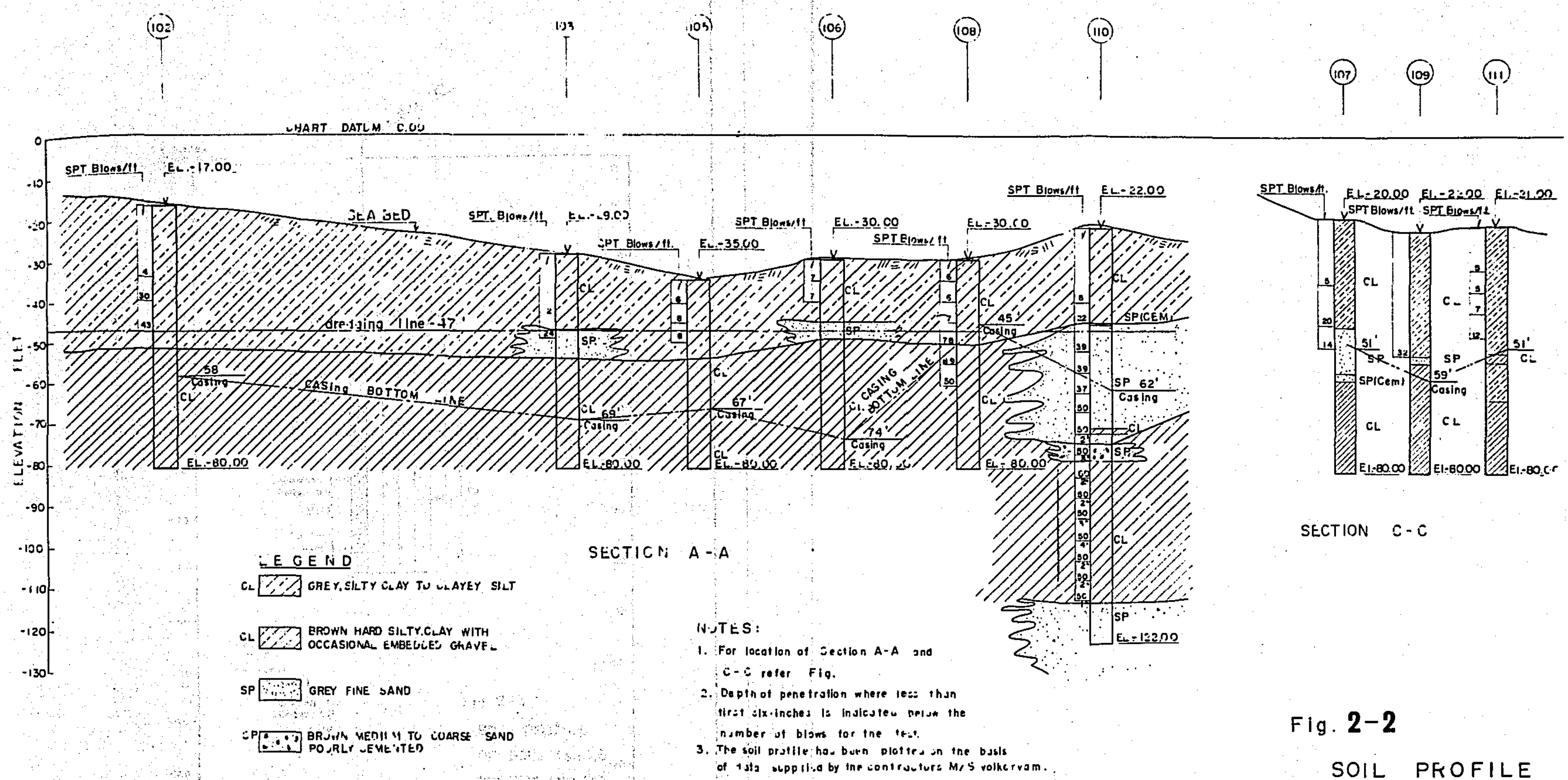



Fig. 2-2

SOIL PROFILE

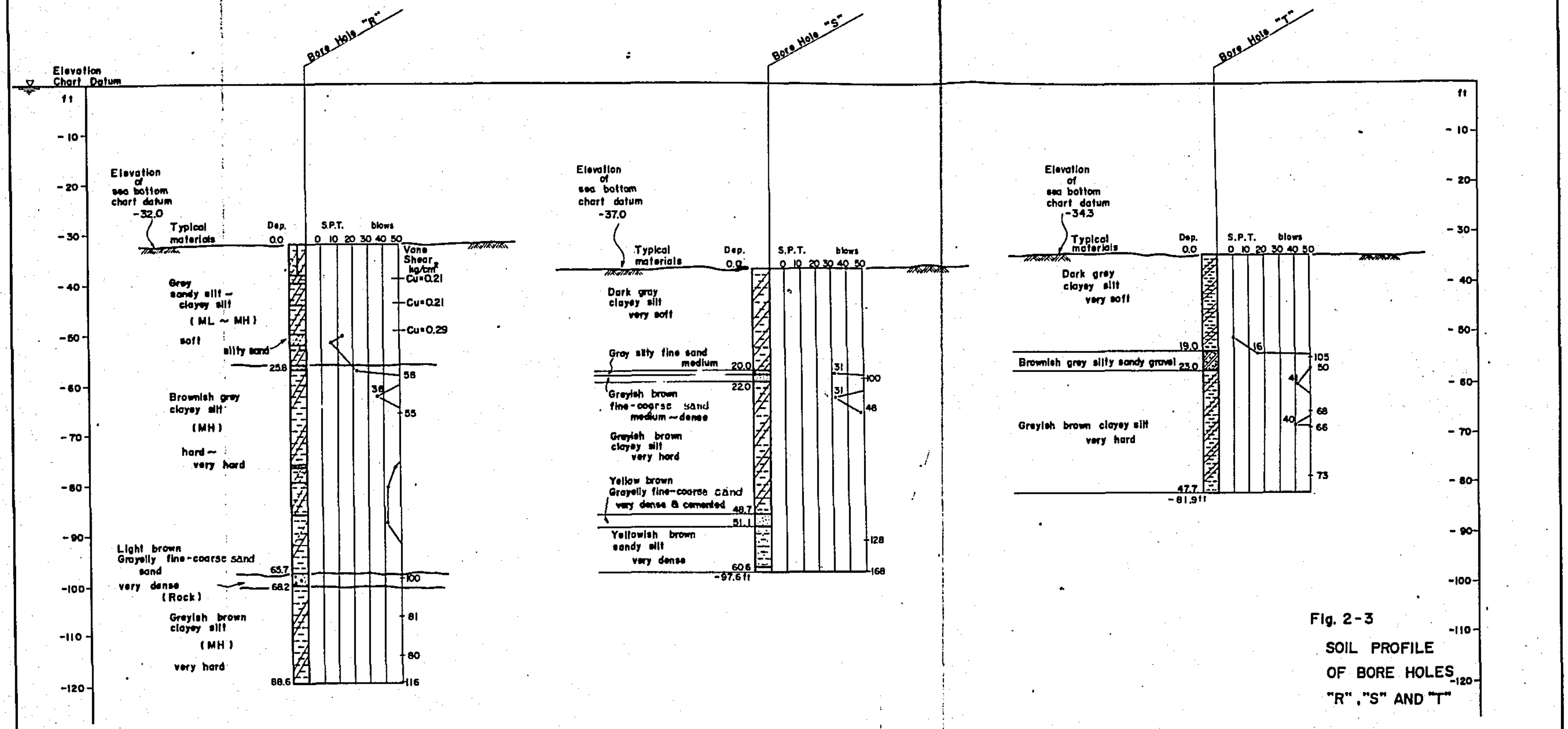



Fig. 2-3

SOIL PROFILE OF BORE HOLES "R", "S" AND "T"

According to Fig. 2-2 and 2-3, with elevation level -50' ∿ 55" as roughly the border, the subsoil of the present sea bottom may be largely divided into the upper stratum of soft clayey silt and the lower stratum of hard mudstone.

However, the assumed border line between the hard and soft layers drawn along the proposed berth site (bore hole 103 ∿ 108) must be closely evaluated in designing the berth structure in regard to the judgement of hard and soft soil and the accuracy of depths.

1. Upper Stratum

From site observation the upper stratum consists of gray clay to silt.

According to soil investigations carried out up to September 1975, results of most standard penetration tests in the upper stratum record N values = 6 ∿ 8.

According to laboratory tests of samples collected, the moisture content is 10 ∿ 16%, the liquid limit 26 ∿ 47%, and the plasticity index 3 ∿ 19%, revealing extremely low values, which do not seem to represent the soil at the site.

From supplementary investigation, it was found that the upper stratum is so soft that boring rods penetrated into the bore holes at points R, S, and T by their own weight.

According to results of vane tests, the distribution of shearing strength shows the maximum value of Cu ≑ 0.3 kg/cm$^2$ around elevation -50', and it may be assumed that the values show a gradual decrease to N = 1 or less around the sea bottom.

In general, the upper stratum cannot be said to be favorable soil as supporting foundation for principal structures.

2. Lower Stratum

From site observation, the lower stratum may be judged to be hard brown silty clay. As shown in Fig. 2-2, the layers extend from elevation -50' ∿ 55' to depths beyond -100', and in investigations carried out up to September 1975, the soil has been classified uninformly as hard silty clay. Laboratory tests of cores collected from bore holes, in the proposed site for the Iron Ore and Coal Berth and the vicinity reveal that the unconfined compressive strength of the soil is,

$$q_u = 2.7 - 9.2 \text{ ton/ft}^2 \doteqdot 3\text{-}10\text{kg/cm}^2$$

The value is a slightly lower test value than the assumed hardness. However, in addition to data from the above comprehensive report, particularly in the surface layers of the lower stratum from elevation -50' ∿ 55' to elevation -65' ∿ 70' penetrated by the casing pipe in the course of boring, close examination of results of supplementary investigation at bore holes R and S reveals an obvious variation in hardness of the soil. In both bore holes, the surface layers are of N value = 50, while immediately below in depths of -62°, the hardness decreases to N value = 31 ∿ 35, and further down, immediately, layers of N = 50 are encountered again.

As the variation in hardness is also observed at bore hole T, it may be assumed that the trend continues throughout the section of bore holes 103 - R - 105 - S - 106.

On the other hand, at bore hole 108, the casing pipe could not be penetrated beyond depths of -50', with N values = 78, 89 and 50, revealing a sudden solidification of the soil.

The surface layers of the lower stratum show a variation of hardness in the course of changing to mudstone. It may be assumed that the layers are not of uniform characteristics along the same elevation or in different depths.

However, on the whole, the lower stratum may be said to possess sufficient load bearing power as supporting foundation for heavy structures.

Investigations carried out up to October 1975, consisted mainly of geological survey as mentioned above. On the other hand, the detailed supplementary survey carried out from October 1975, particularly in regard to the hardness of the surface layers of the lower stratum, has provided useful information for the designing of the berth structure. However, the survey cannot be said to be quite sufficient in the number of holes surveyed.

The berth structure will be designed on the basis of available data. In the course of construction, it will be necessary to carry out further investigation to check partial variation of the soil in advance.

# CHAPTER III BERTH CONSTRUCTION SYSTEM

## III-1 ALTERNATE BERTH STRUCTURES

Prior to the detailed designing, various alternate plans have been studied for the berth construction of the Iron Ore and Coal Berth, and it has been concluded that a detached pier type structure completely independent in the creek system would be the optimum plan. In the present study, the following three alternate types of construction will be compared for the foundation of the pier.

1) P.C. pile structure
2) S.P. pile structure
3) Caisson structure

Comparison of the above three alternate types of structure will be conducted on the basis of various problems involving the effect on tidal currents in the creek system, subsoil conditions, and berthing of vessels.

a) Effect on tiadal currents

The above structures 1) and 2) may be recommended from the point that a foundation of pile construction will not affect tidal currents to any large extent.

In case of a caisson structure 3), if the caissons are lined along the entire extension of the berth, an obstacle of island form will be created in the creek, and the structure will not be a favorable type of construction from the point of tidal currents. Therefore, the caisson type structure will be designed to place the caissons at uniform intervals to allow the flow of tidal currents between the caissons.

b) Subsoil conditions

At the proposed site of the Iron Ore and Coal Berth, surface layers of the subsoil consist of relatively soft silty clay. Below the design water depth (-47') in lower layers beyond depths of approximately -50' ∿ -55', continuous layers of hard silty clay with N values over 40 are found.

Difficulties to be encountered in driving long piles into the hard material must be considered in designing pile structures 1) and 2).

c) Berthing and unloading equipment

Generally, in iron ore and coal berths, the structure of the pier is designed to resist external forces of vessels and unloading equipment.

In case the impact force and pulling force of vessels are particularly strong, to avoid unfavorable effects on the unloader, breasting dolphins and mooring dolphins will be constructed as separate structures from the pier.

However, if it will be possible to construct an unloader pier with sufficient strength, the pier may be expected to perform functions of breasting and to a certain extent, of mooring for vessels, an advantage from the economic point as well. The case may be applied particularly to a pier of caisson structure 3).

With consideration for the above three conditions, alternate plans for berth construction will be designed with structures and framework as shown in Fig. 3-1 ∿ 3-3.

1. <u>P.C. Pile Structure</u> (Refer to Fig. 3-1)

The length of the berth of approximately 300m will be composed of blocks of approximately 30m in length. In each block, in mesh arrangement of 3 ∿ 4m, a large number of long prestressed concrete piles of large diameter will be driven in as vertical piles. The superstructure of each block will be a combined floor system structure of beams and slab of reinforced concrete, and rails for the unloader track will be laid on the beams.

A pier of P.C. pile structure will be able to function as both breasting and mooring facilities for berthing vessels.

2. <u>S.P. Pile Structure</u> (Refer Fig. 3-2)

A S.P. pile structure will be studied as the most favorable design among alternate plans under comparison.

In a berth of steel pipe pile structure, for the unloader pier, a group of piles will be driven in at intervals of approximately 20m in the direction of the pier axis. The pile groups will be composed of raking piles with comparatively shallow penetration into the foundation ground.

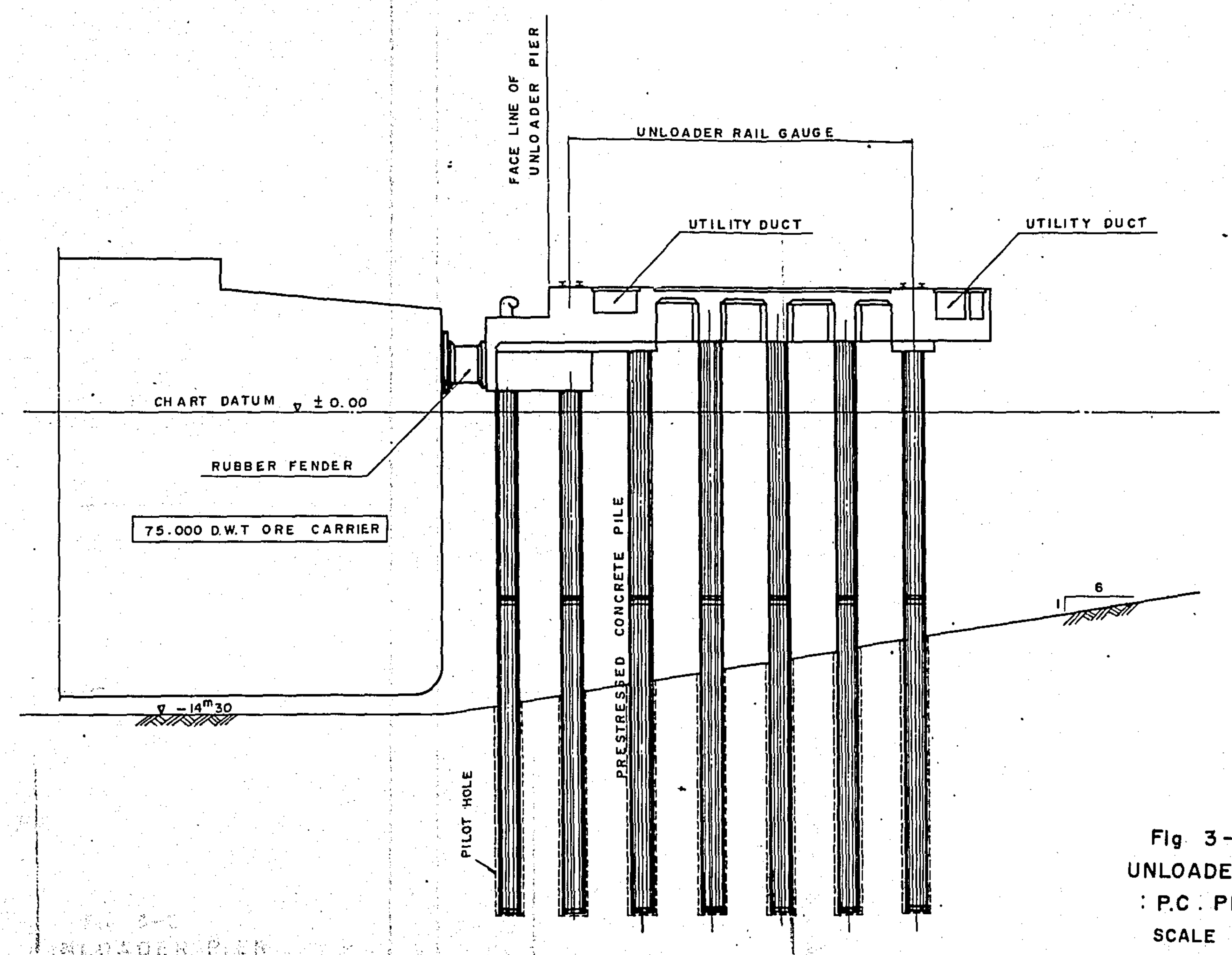



Fig. 3-1
UNLOADER PIER
: P.C. PILE TYPE
SCALE 1 : 200

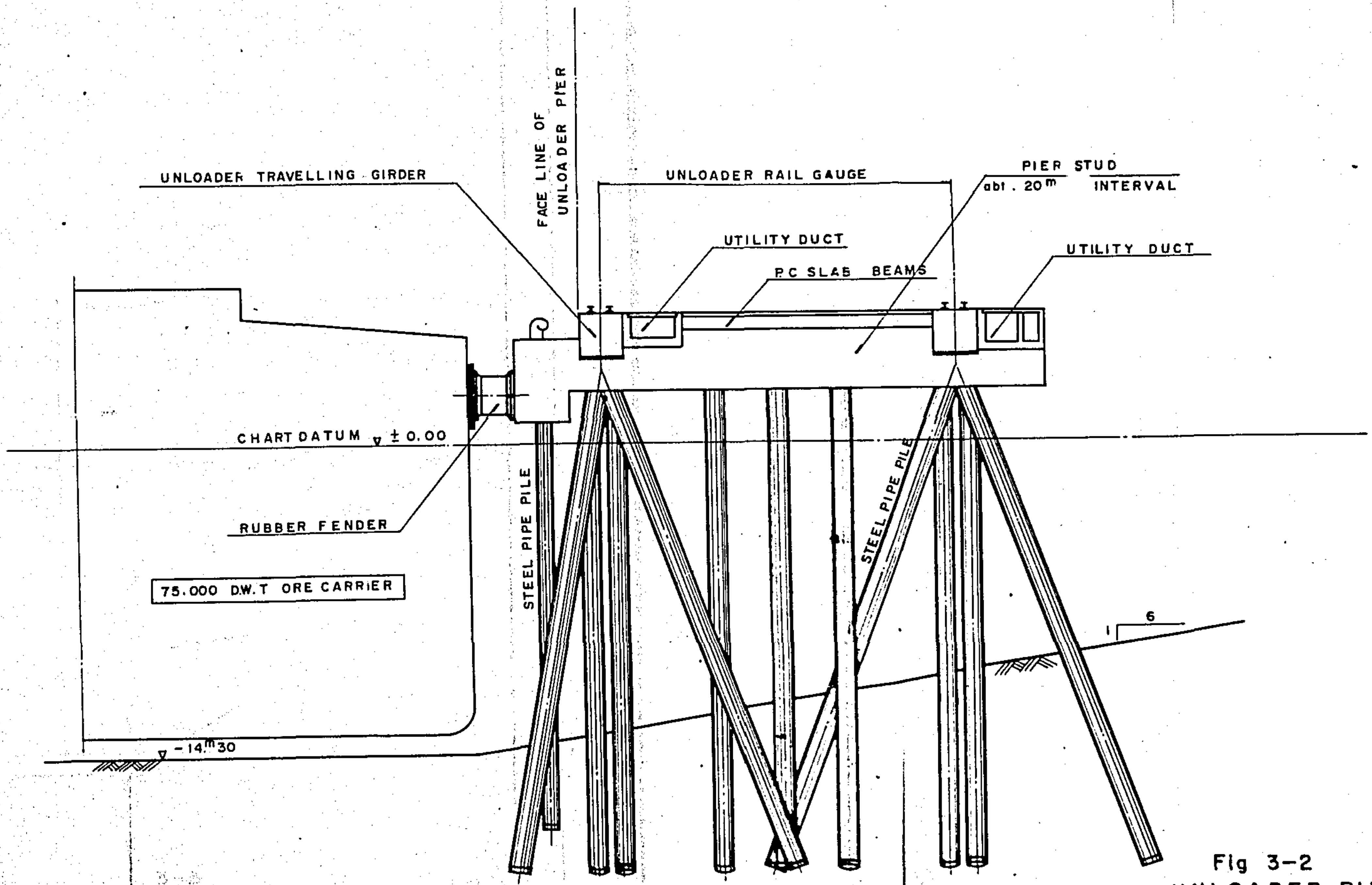



Fig 3-2
UNLOADER PIER
:S.P. PILE TYPE
SCALE 1:200

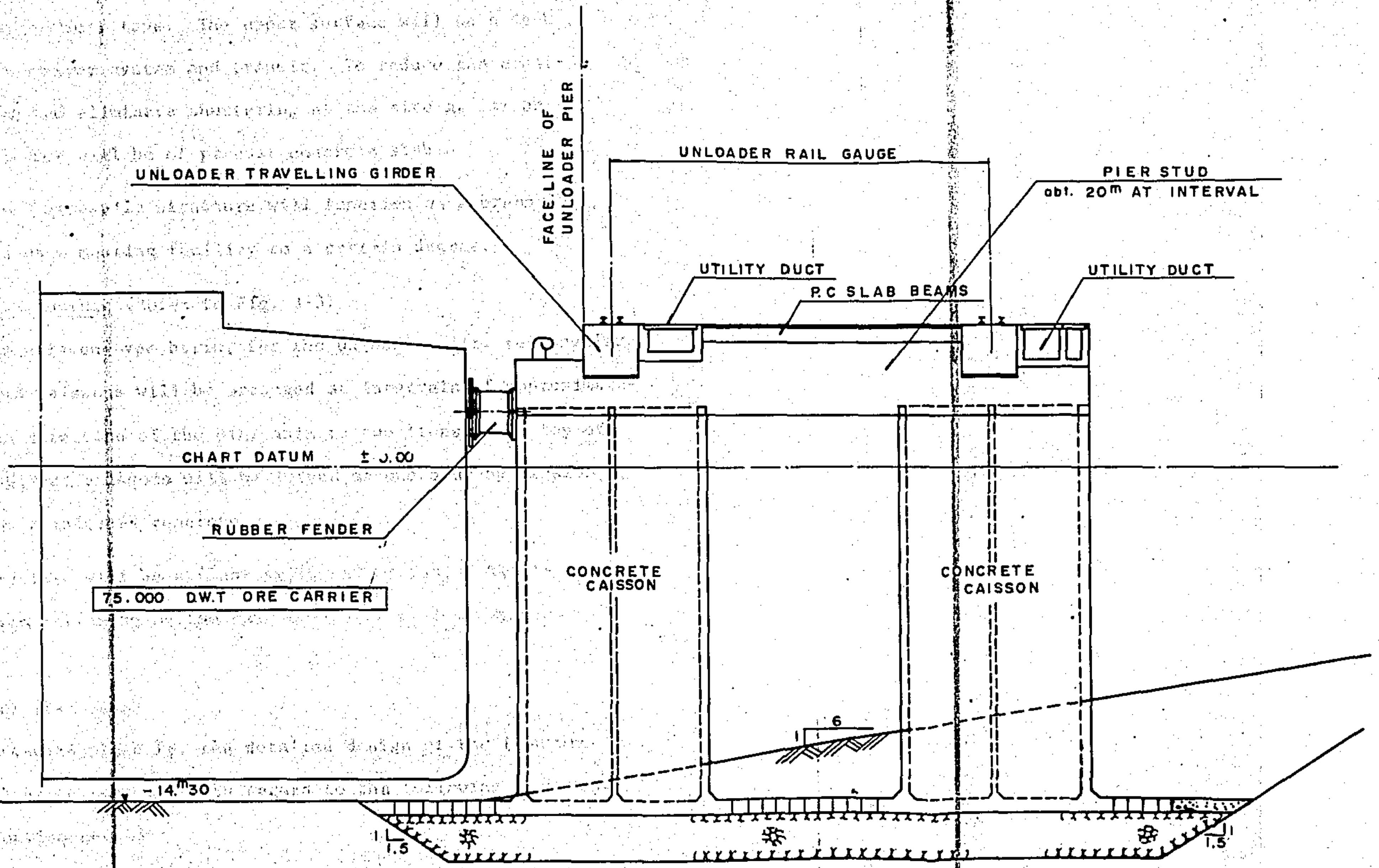



Fig 3-3
UNLOADER PIER
: CAISSON TYPE
SCALE 1:200

P.C. girders will be installed between the pier studs, and rails for the unloader track will be laid on the girders. In a pile type foundation, in order to secure the rail gauge in position, piles on the sea side and land side will be joined in pairs by cast-in-place concrete tops. The upper surface will be a deck for the belt conveyor system and transit. To reduce the construction period and eliminate shuttering at the site as far as possible, the deck will be of precast concrete slab.

A pier of steel pipe pile structure will function as a breasting facility, and as a mooring facility to a certain degree.

3. Caisson Structure (Refer to Fig. 3-3)

In a concrete caisson type berth, for the unloader pier, rectangular parallelepiped caissons will be arranged at intervals of approximately 20m in the direction of the pier axis in two lines. The top of the front and rear caissons will be joined as one body by massive cast-in-place reinforced concrete.

The superstructure will be a floor system of precast concrete along the same reasoning as the case of a S.P. pile structure.

## III-2 COMPARISON OF STRUCTURES

A comparison of alternate plans for the detailed design of the Iron Ore and Coal Berth will be carried out with regard to the following problems.

1. Construction method
2. Construction cost and period

The type of structure to be adopted must be of easy construction at the site, with advantages in the construction cost and period.

1. P.C. Pile Structure

If the unloader pier is to be of raking pile structure, deep penetration of piles will not be required. However, it will be extremely difficult to drive in P.C. piles (raking piles) of heavy weight directly into the hard ground.

On the other hand, in case of a vertical pile structure, an embedded length of piles including the calculated safety factor must be secured. Judging from available soil data, though it may be difficult to drive in piles directly to the necessary embedded length, it will be possible to drive in piles by drilling pilot holes by auger machines in advance.

However, in case of P.C. piles, from the limit to the size of diameters of piles to be prefabricated (from the function of the section), a great many number of piles will be required for the dolphins, involving a large amount of ground drilling. The pile driving will require engineering technique of high level.

Notwithstanding the advantage that basic materials for P.C. piles are produced locally, the above problems, together with the problem of providing for the P.C. pile prefabrication plant, will result in an increase in the cost and period of berth construction. The rate of foreign currency in the construction cost will also be a considerable figure.

Therefore, involving various difficult problems, the P.C. pile type structure will be ruled out from the comparison of alternate plans.

2. S.P. Pile Structure

In case of steel pipe piles, even in soil layers with N values over 50, if the soil is not of base rock, generally, it is possible to drive in piles to depths of 1 ∿ 2m. From the soil conditions at the proposed site, it may be judged that in case S.P. piles are used, a certain degree of embedded length may be secured, but the value is unknown. However, the problem may be solved by conducting test driving prior to the actual pile driving.

Therefore, in case a sufficient embedded length cannot be secured for raking piles of the unloader pier, it will be advisable to design the pier head as a gravity type structure so that pulling of piles will not occur.

In case vertical long steel pipe piles are to be used for the mooring dolphins, piles will be driven in by drilling pilot holes. As steel pipe piles possess higher sectional function, the number of piles to be driven in will be far less than the case of P.C. pile dolphins, and pile driving will be easier.

The construction period will be shorter compared to P.C. pile structures. As S.P. piles must be imported, the use of the piles will be contrary to the principle to use domestic material as far as possible. However, from the point of safety and reliability of the entire berth construction, the use of S.P. piles must be tolerated.

3. Caisson Structure

The caisson structure will be considered from the point of construction cost, as a method to take advantage of the hard foundation of the site.

In relatively small depths (-50' ~ -55') from the design water depth (-47') at the proposed berth site, hard silty clay deposits are encountered. By removing the upper soft layers by dredging, and replacing the material with foundation rubble stones, a large ground bearing capacity may be obtained.

However, difficulties will be encountered in the course of making caissons. The construction of a caisson yard requires a long period of time and an enormous amount of funds. In case a floating dock is to be used, the cost of transport from abroad will amount to a large sum. If the number of caissons to be formed is small, the construction cost of the berth will be extremely costly. As a third method, the caissons may be cast-in-place on the shoals in the creek. When the caissons have been formed, the material in front of the caissons may be dredged, and the caissons may be floated and towed to site. However, engineering technique of high level will be required for this method of construction.

## III-3 RECOMMENDED CONSTRUCTION SYSTEM

From the above discussion of alternate plans for the berth structure, it has been concluded that the P.C. pile structure will be ruled out from the detailed designing as it may be judged that it will be extremely difficult to drive in P.C. piles at the proposed site for the foundation of the pier.

A steel pipe pile type structure and a caisson type structure will be studied as alternate plans for the detailed design of the berth construction.

From various studies and the detailed designing to be carried out, the structure to be finally adopted will be determined from the two alternate plans.

From discussions conducted so far, it may be concluded that an unloader pier of S.P. piles will be the most favorable structure for the Iron Ore and Coal Berth.

In case of a caisson structure, it will be advisable to study methods to prepare the caissons at low cost.

(Refer to Appendix:"Structural Analysis and Study of a Caisson Type Pier)

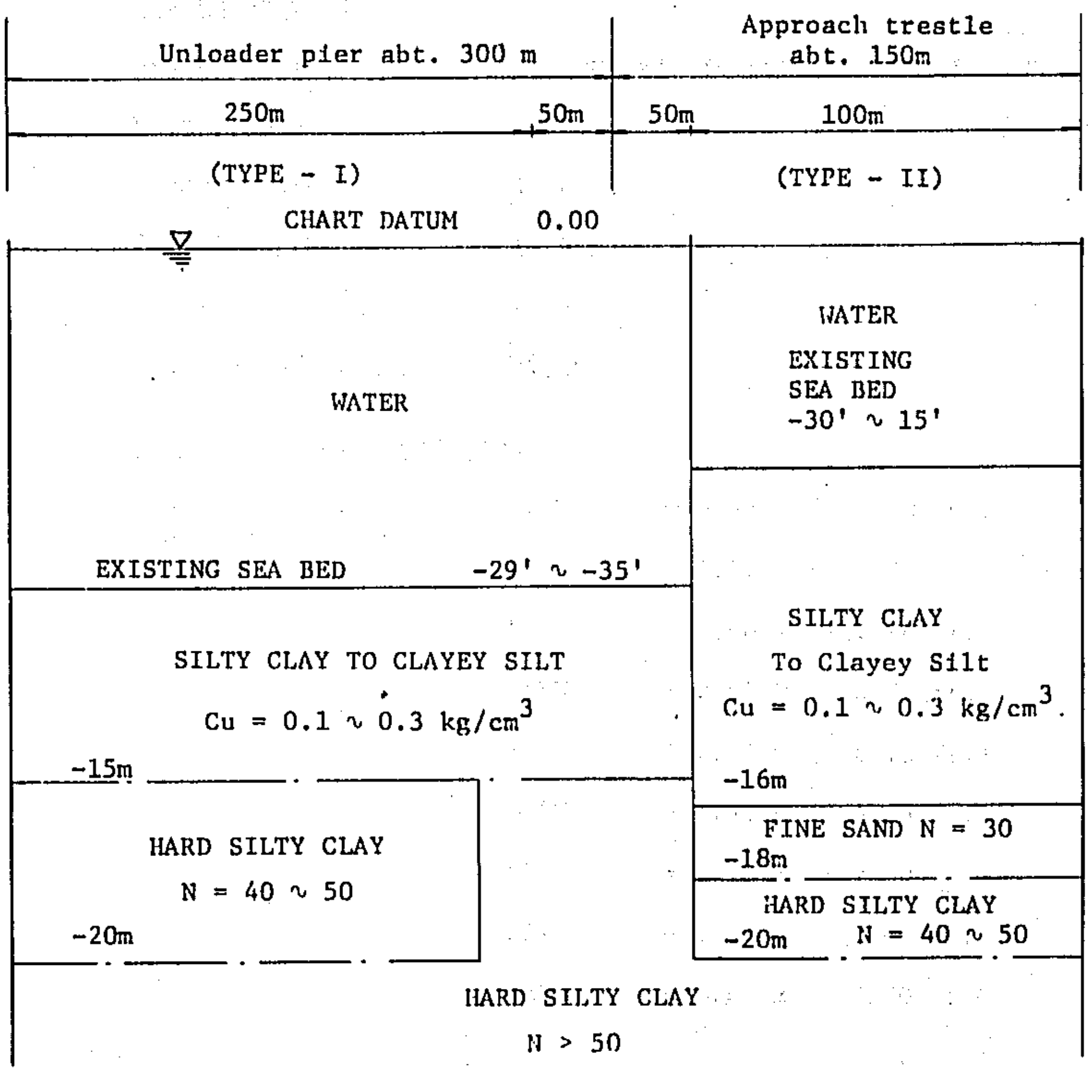
## CHAPTER IV DESIGN CRITERIA

The berth structure and unloader pier foundation will be designed according to the following criteria.

1. <u>Soil Conditions</u>

From the evaluation of subsoil conditions discussed in Chapter II, the design soil conditions will be largely divided into Type I and Type II as shown in Fig. 4-1.

Fig. 4-1 Design Soil Conditions



## 2. Berth Structure

| | |
|---|---|
| Length | abt. 300m (1000 ft.) |
| Length of approach trestle | abt. 150m (450 ft.) |
| Crown height of structure | |
| dolphin | + 4.50m (14.8 ft.) |
| unloader | + 6.00m (19.7 ft.) |
| approach trestle | + 6.00m (19.7 ft.) |
| Berth depth (75,000 DWT) | - 14.30m (-47 ft.) |
| Design berth depth | |
| (allowing for over-dredging) | - 14.90 m (-48.9 ft.) |

## 3. Design ship size

| | |
|---|---|
| Ore carriers | 75,000 DWT, 50,000 DWT, 25,000 DWT |
| Vessel length | 247.0 m |
| Vessel width | 35.4 m |
| Vessel depth | 18.9 m |
| Full-load draft | 13.4 m |
| Displacement tonnage | 97,000 |

## 4. Weather and Sea Conditions

| | | |
|---|---|---|
| Design tidal levels (chart datum ±0.00) | ERAW | + 3.96 m (- 13.00 ft.) |
| | HAT | + 3.96 m ( " ) |
| | MHHW | + 3.40 m (+11.16 ft.) |
| | MLHW | + 2.67 M (+ 8.76 ft.) |
| | MSL | + 2.05 m (+ 6.73 ft.) |
| | MHLW | + 1.43 m (+ 4.69 ft.) |
| | MLLW | + 0.70 m (+ 2.30 ft.) |
| | LAT | - 0.61 m (- 2.0 ft.) |
| | ERLW | - 0.83 m (- 2.70 ft.) |

| | |
|---|---|
| Tidal currents | |
| Currents parallel to berth | 2.0 m/sec. |
| Currents perpendicular to berth | 0.5 m/sec. |
| Wind velocity | |
| On unloaders working | 10.3 m/sec (20 knots) |
| On unloaders in storms | 31.0 m/sec (60 knots) |
| On vessels in berth | 20.0 m/sec (39 knots) |
| Temperature | |
| In air | 40°C |
| In water, underground | to be ignored |
| Average for structural analysis | 20°C |
| Seismic coefficient | |
| Lateral | 0.1 |
| Vertical | 0.0 |

5. Load Conditions of Unloading Equipment

Unloading equipment for iron ore and coal will include unloaders, bulldozers, trucks and conveyors. Mobil cranes are not in common use to carry ore and coal or buckets for repairs. Unloaders are used in place of mobile cranes. If a pier with a mobile crane is to be constructed, the foundation of the pier and deck slab must be reinforced in strength, requiring a large amount of construction cost. Therefore, a mobile crane will be ruled out in the course of designing.

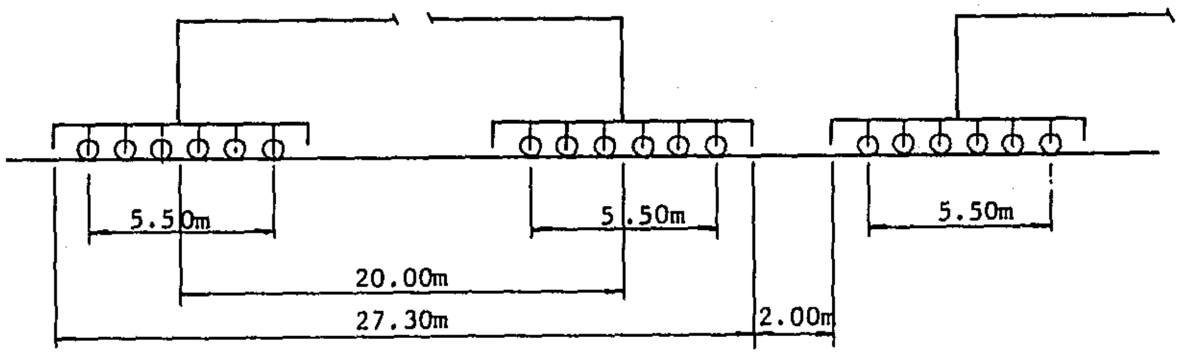
a) Unloader

i Capacity: 1,000 ton/hr

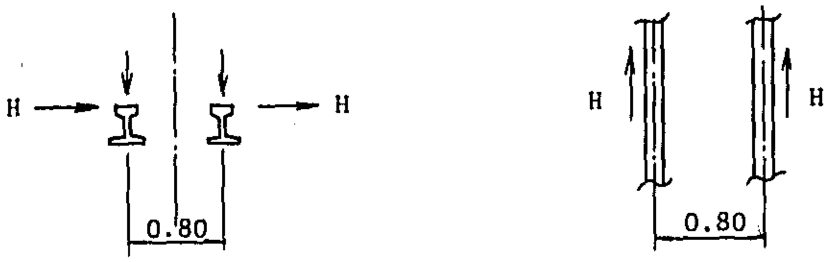
ii Dead weight: 1,000 ton

iii Number of wheels: 6 wheels x 2 = 12 wheels/corner

Fig. 4-2 Wheel Layout of Unloader




SECTION of rail track (one side) PLAN of rail track (one side)




IV Wheel load

| Load Condition | Wind Velocity | Vertical | Horizontal (perpendicular) | Duration of Load |
|---|---|---|---|---|
| Working in Position | 20 knots | 30 t/wheel | 1 t/wheel | Long period load |
| Travelling | do | 37.5 t/wheel* | 4 t/wheel** | Short period load |
| In storms | 60 knots | 35 t/wheel | 4 t/wheel | do |

Note:

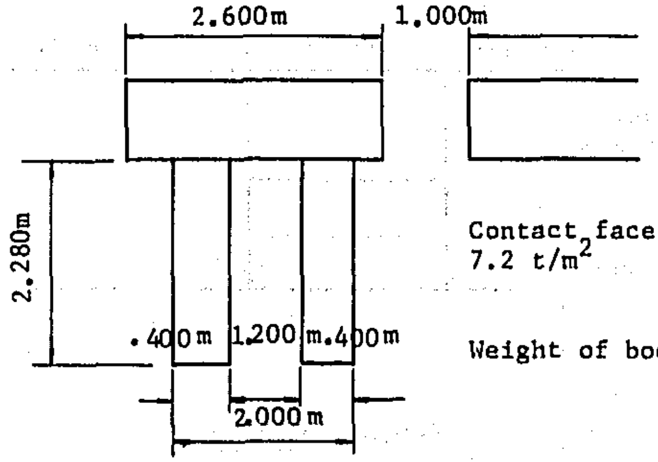
Approach of two unloaders will be considered during operation.

* 30t x 1.25 = 37.5 t/wheel

**30t x 0.1 + 1t = 4t/wheel

b) Small Bulldozers

Fig. 4-3 Body Space of Bulldozer

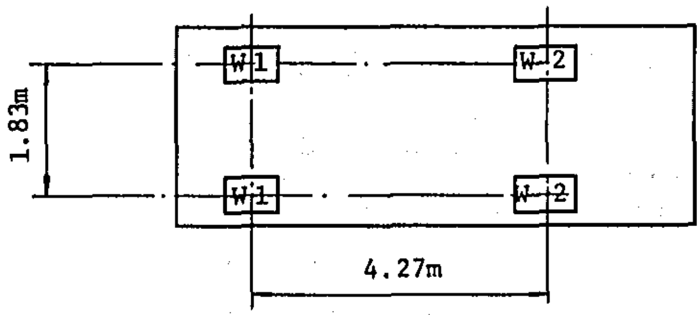



Contact face pressure
7.2 $t/m^2$

Weight of body 13.13 ton

c) AASHO H-20 truck

Fig. 4-4 Wheel Layout of Truck

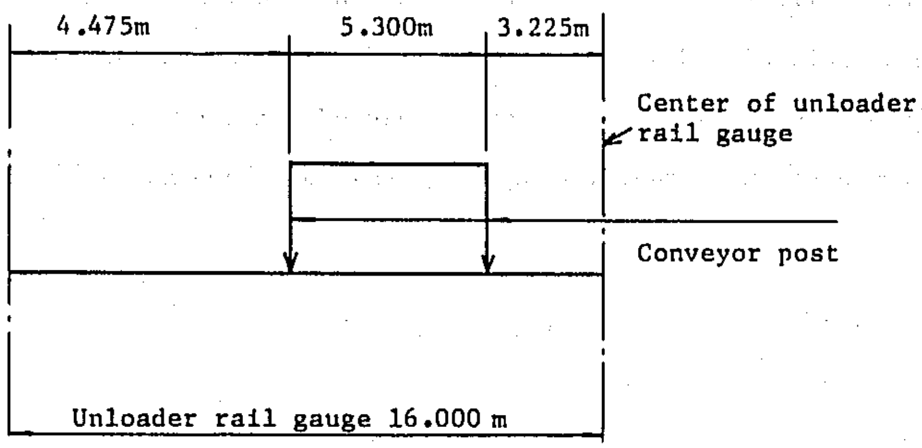



W1 = load of front wheel = 1.81 t/wheel

W2 = load of rear wheel = 7.56 t/wheel

d) Conveyor System

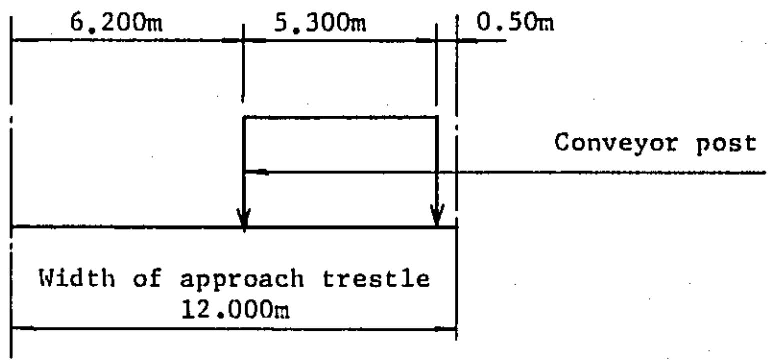
Fig. 4-5 Conveyor Load Positions on Unloader Pier




Pitch of support 7.50 m

Reaction force of support 5 t/support

Fig. 4-6 Conveyor Load Positions on Approach Trestle




Pitch of support 7.50m

Reaction force of support 5 t/support

e) Erection of unloader

Large cranes must not be placed directly on the pier for erection of the unloader and test working. The assembly of large parts of the unloader shall be carried out by floating cranes. A derrick crane may be used on the pier for the erection of the unloader on condition that a temporary beam frame is used.

6. Surcharge

| | | | |
|---|---|---|---|
| Unloader | U.D.L. | Ordinary | 2.0 $t/m^2$ |
| | | Extraordinary | 1.0 $t/m^2$ |
| Approach trestle | U.D.L. | Ordinary | 0.5 $t/m^2$ |
| | | Extraordinary | 0.25 $t/m^2$ |

7. Wind Velocities on Vessels in Berth

It will be difficult to provide effective storm bollards, as the conveyor system on the pier will be in the way, and the size of vessels to berth is uncertain. Therefore, on condition that vessels will evacuate when the wind velocity is expected to exceed 15 - 20 m/sec, storm bollards will not be installed.

The design wind velocity when vessels are in berth will be 20 m/sec. In cases of extraordinary storms, with wind velocities over 20 m/sec, vessels in berth will evacuate to the water area in front of the berth with sufficient space and depth for anchoring of vessels.

8. Berthing of Vessels

At times of berthing of vessels of 50,000 DWT - 75,000 DWT at the Iron Ore and Coal Berth, the engine will be stopped at a distance of 1 - 2 km from the berth. The vessel will then berth with the aid of a tug approaching parallel to the berth as far as possible.

Berthing speed 10 cm/sec.

(at the center of gravity of the vessel)

Berthing angle 6° or less

In deberthing, the vessel will stay back while the tug pulls the vessel out to the turning basin. Then the vessel will turn and leave port. Refer to Fig. 4-7 and Fig. 4-8.

9. <u>Strength of Construction Materials</u>

a. Steel

i. Steel pipe pile $\delta y \geqq 24.61$ kg/mm$^2$ A.S.T.M. A252 GRADE 2

ii. Reinforcement $\delta y \geqq 24.00$ kg/mm$^2$ -

( y: yield point strength, allowable stress will be based on A.S.T.M. 615.)

iii. P.C. steel wire $\delta t \geqq 175$ kg/mm$^2$ A.S.T.M. A416 GRADE 250

iv. P.C. steel bar $\delta t \geqq 102$ kg/mm$^2$ A.S.T.M. A416-68 GRADE 250

( $\delta t$: tensile strength, allowable stress will be based on A.C.I. 318-71.)

b. Concrete

i. Prestressed concrete $\delta 28 \geqq 350$ kg/cm$^2$

ii. Precast reinforced concrete $\delta 28 \geqq 300$ kg/cm$^2$

iii. Cast-in-place reinforced concrete $\delta 28 \geqq 240$ kg/cm$^2$

( $\delta 28$: 28 day strength, allowable stress will be based on A.C.I. 318-63 1003.)

10. Corrosion of Steel Pipe Piles

| | | |
|---|---|---|
| Rate of corrosion | one side | 0.1 mm/yr |
| Durable years | Corrosion 30 yrs + cathodic protection 20 years | |

The effect of cathodic protection is reduced when the steel is exposed to air. The part of steel pipe piles exposed above water level of -1.00 m, will be coated.

Fig. 4 - 7

MANEUVERING OF BERTHING VESSELS

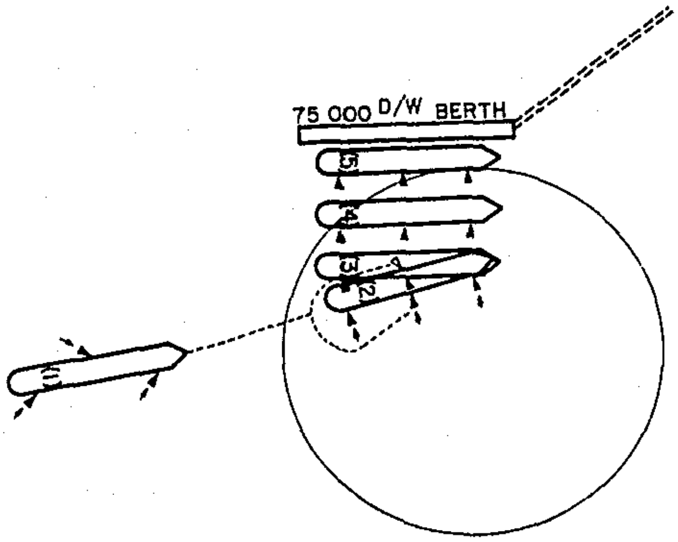
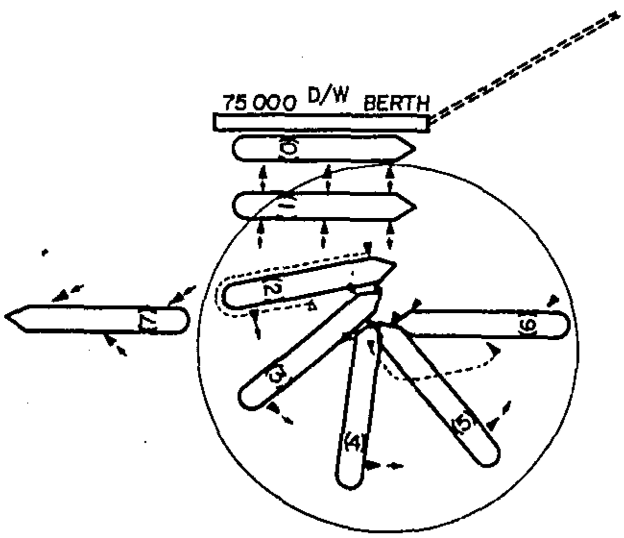



Fig. 4 - 8

MANEUVERING OF DEBERTHING VESSELS



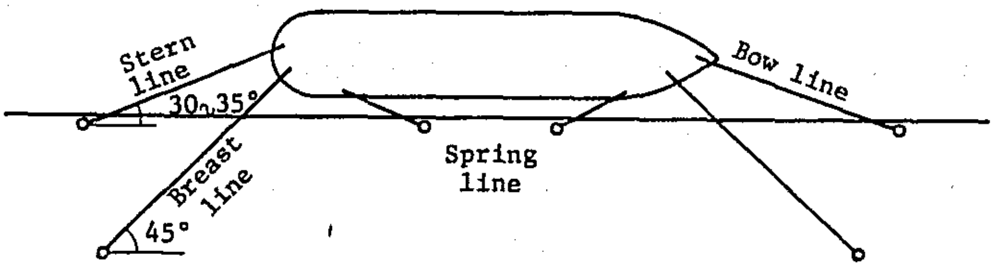
## Chapter V Mooring and Berthing of Vessels

### V-1 MOORING OF VESSELS

#### 1. Arrangement of Pier Studs and Mooring Posts

(1) General line handling

Fig. 5-1 Arrangement of Mooring Lines




a. Lines in frequent use are the bow and stern lines. A favorable angle between the bow and stern lines and the shoreline of the berth will be of 30° ∿ 35°, and the maximum allowable angle will be in the range of 25° ∿ 45°. The mooring posts will be bitts.

b. In case of storms, breast lines will be pulled at a distance longer than the vessel width from the shoreline at an angle of 45° as storm lines. The mooring posts will be bollards.

c. The standard arrangement of bitts in regard to number and intervals on a berth for large ocean-going vessels is as given in the following table:

Table 5-1 Arrangement of Bitts

| Displacement Tonnage of Vessel | Maximum Interval of Bitts | Minimum Requirement per Berth |
|---|---|---|
| 20,000 ∿ 50,000 tons | 35 m | 8 bitts |
| 50,000 ∿ 100,000 tons | 45 | 8 |

(2) Particular line handling at proposed berth

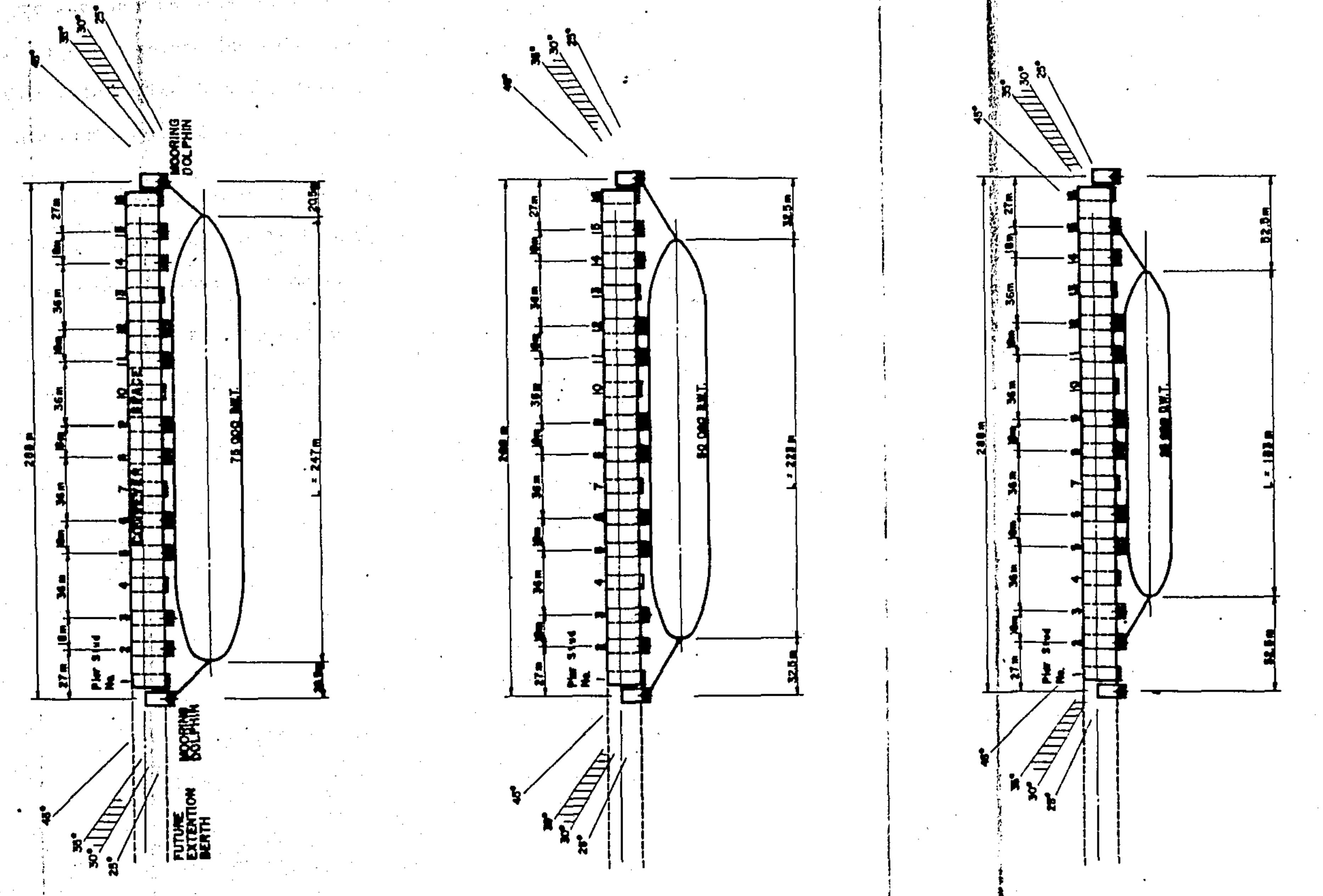
As discussed in Section II of Chapter III Berth Construction System, for a steel pipe pile structure, the pier studs will be arranged at intervals of 18 m along the entire length of the berth, from the desirable span of P.C. girders for the travelling unloader from the economic point of view.

As bitts will be installed on the pier studs, the arrangement will provide for the installing of bitts at intervals of 18 m and 36 m.

Even under wind velocity of 20 m/sec. on vessels in berth, a considerably large pulling force will act on the bitts on both ends of the berth. The end bitts will be installed on the mooring dolphins of independent structure from the unloader pier.

Nevertheless, at the proposed berth, it will be difficult to instal efficient storm bollards for breast lines due to the conveyor system and uncertain vessel sizes. Therefore, it will be necessary to plan the evacuation of vessels in times of storms.

The arrangement of bitts and line handling for various vessels sizes at the proposed berth are shown in Fig. 5-2.




◆ : BITT POSITION

Fig. 5 - 2

ARRANGEMENT OF PIER STUDS, DOLPHINS AND BITTS

SCALE 1:2000

2. <u>Strength of Mooring Posts</u>

(1) Wind pressure and tidal current pressure to act on berthing vessels

Wind pressure due to winds of wind velocity of 20 m/sec., and tidal current pressure due to tidal currents of 20 m/sec., from the ship axial direction, and tidal currents of 0.5 m/sec. from the ship width direction may be calculated as follows.

i. Calculation of wind pressure

Wind pressure on vessels in berth may be calculated according to the following formula.

$$R = \frac{1}{2}\rho C\upsilon^2(A\cos^2\Theta + B\sin^2\Theta)$$

Where R: resultant force of wind pressure (kg)

$\rho$: air density 0.123 ($kg \cdot sec^2/m^4$)

$\upsilon$: wind velocity (m/sec)

A: area of front reflection of vessel on water ($m^2$)

B: area of side reflection of vessel on water ($m^2$)

Ore carriers

Full-load $\log A = 0.42 + 0.480 \log D.W.$

$\log B = 0.648 + 0.550 \log D.W.$

Light-load $\log A = 0.377 + 0.533 \log D.W.$

$\log B = 0.733 + 0.601 \log D.W.$

: angle of wind direction and center line of hull

C: wind pressure coefficient

*where*, $\Theta = 0°$ $C = 0.69$

where, $\Theta = 90°$ $C = 1.122$

ii. Calculation of tidal current force

a. Resisting force against currents from the bow direction.

Resisting force between the vessel and flow of tides from the bow direction may be calculated according to the following formula.

$$R_f = 0.14\ SV^2$$

where

$R_f$: resisting force (kg)

S : waterline area ($m^2$)

Ore carrier

Full-load $\log S = 0.871 + 0.660 \log D.W.$

Light-load $\log S = 0.632 + 0.661 \log D.W.$

V : velocity of tidal currents

b. Resisting force against currents from the perpendicular direction to pier axis.

Resisting force may be calculated according to the following formula.

$$R = \frac{1}{2} \rho\ CV^2\ B'$$

where

R : current pressure (kg)

$\rho$ : sea water density 101.5 ($kg \cdot sec^2/m^4$)

C : current pressure coefficient, 4.60

B': alongside area below draft line

Ore carrier

Full-load $\log B' = 0.484 + 0.612 \log D.W.$

Light-load $\log B' = 0.499 + 0.463 \log D.W.$

iii. Summary of calculations

Wind Pressure Force and Tidal Current Force

| Vessel | | Perpendicular to Berth | | | Parallel to Berth | | |
|---|---|---|---|---|---|---|---|
| | | Tidal Current Pressure | Wind Pressure | Total | Tidal Current Pressure | Wind Pressure | Total |
| DWT 75,000 | Full-load | 174 t | 58.9 t | 233 t | 6.9 t | 9.9 t | 16.8 t |
| | Light-load | 8.9 | 127.0 | 136 | 4.0 | 16.0 | 20.0 |
| DWT 50,000 | Full-load | 137.4 | 50.8 | 188 | 5.3 | 8.2 | 13.5 |
| | Light-load | 7.4 | 107.4 | 115 | 3.1 | 12.9 | 16.0 |
| DWT 25,000 | Full-load | 90.0 | 32.2 | 122 | 3.3 | 5.9 | 9.2 |
| | Light-load | 5.4 | 65.6 | 71 | 1.9 | 8.9 | 10.8 |

(2) Pulling force of vessel

i. Pulling force R of the bow and stern lines against wind pressure and tidal current pressure from the pier side may be obtained by vector analysis as shown in the following table.

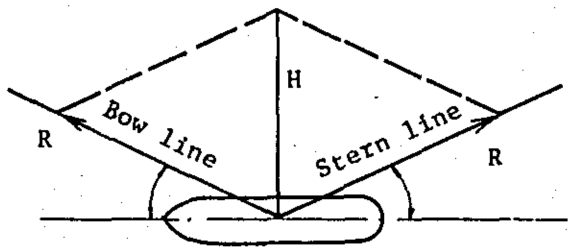


Table 5-3 Pulling Force of Vessel

| | H | | R |
|---|---|---|---|
| 75,000 DWT | Full-load 233 ton | 45° | 165 t |
| 50,000 DWT | " 188 ton | 30° | 188 t |
| 25,000 DWT | " 122 ton | 30° | 122 t |

ii. Pulling force R of the bow and stern lines against wind pressure and tidal current pressure from the ship axial direction may also be obtained by vector analysis. However, as the wind pressure and tidal current pressure are small, the pulling force may be ignored.

(3) Mooring posts

According to I.S.O. standards, the breaking strength of mooring lines for vessels of 75,000 DWT ∿ 25,000 DWT is 75 ton/line. (Equipment number 7,400 ∿ 13,400) Bitts will be installed on the pier studs and dolphins according to the above standard as shown in the following table.

i. Mooring bitts

Bitts for the bow and stern lines at either end of the berth

Table 5-4 Bitt Capacity

| | Position of Bitts | Actual Pulling Force | Bitt Capacity | Breaking Strength of Line |
|---|---|---|---|---|
| 75,000 DWT | on dolphin | 165 ton | 70 ton x 3 = 210 ton | 75 ton x 3 = 225 ton |
| 50,000 DWT | on dolphin | 188 ton | 70 ton x 3 = 210 ton | 75 ton x 3 = 225 ton |
| 25,000 DWT | Pier stud No. 2 and 15 | 122 ton | 70 ton x 2 = 140 ton | 75 ton x 2 = 150 ton |

ii. Maneuvering bitts

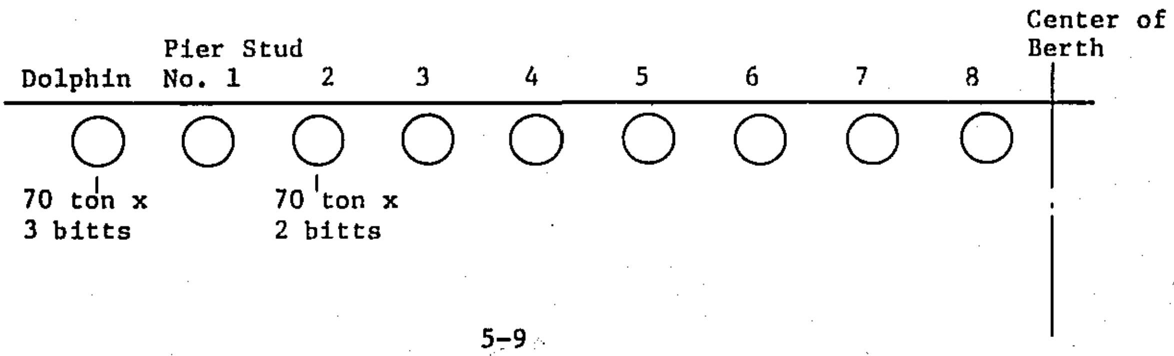
Spring line bitts in the middle part of the berth and the end bitts will be used for maneuvering of vessels in berthing and deberthing. In maneuvering, the bitts may be used for only one line at a time. Therefore, as the breaking strength of one line is 75 tons, allowing for poor maneuvering, a bitt of 100 ton breaking strength will be installed for maneuvering of vessels.

iii. Conclusion

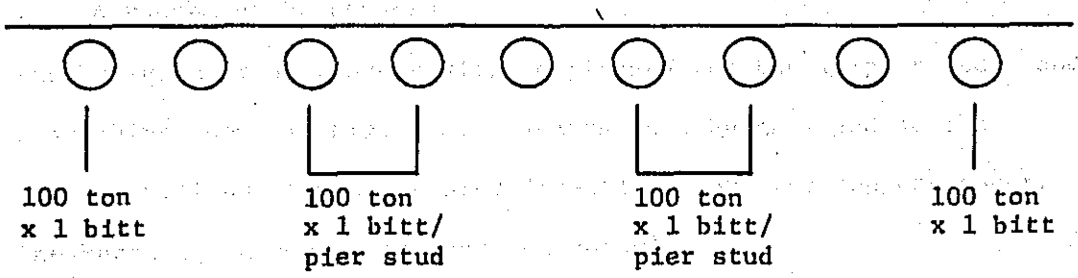
From the above discussion, the arrangement of bitts for the entire berth will be planned as follows.

Fig. 5-3 Arrangement of Bitts

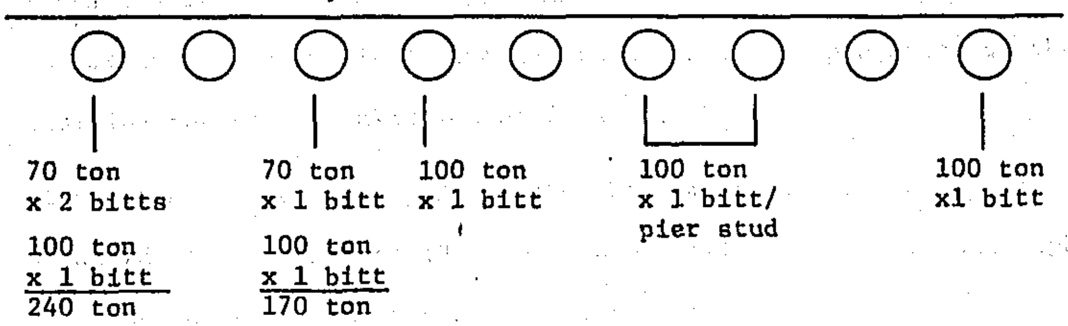
Mooring Bitts



Manuevering Bitts




Therefore,



## V-2 BERTHING OF VESSELS

### 1. Arrangement of Fenders

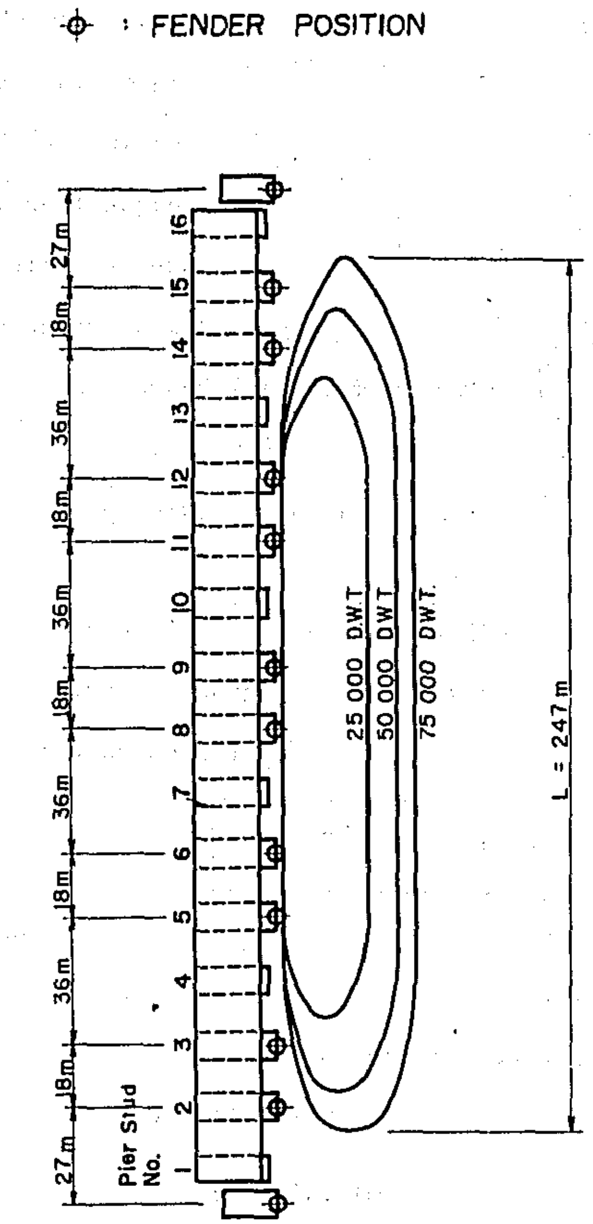
The arrangement of fenders will be planned for the 16 pier studs and 2 dolphins shown in Fig. 5-2. Fenders must be arranged so that vessels will not touch the pier directly before the fenders absorb the berthing energy of approaching vessels.

Generally, in berths for vessels of large dimensions, fenders are installed at intervals of approximately 20 m. Fenders may be installed at larger intervals if the concrete surface is projected at the position for the installation of fenders.

At the proposed berth, applying the above method, fenders will be arranged at intervals of 18 $\sim$ 36 m in symmetry to the left and right from the center of the berth, as shown in Fig. 5-4.

Fenders must also be installed on the mooring dolphins at both ends of the berth to provide for emergency in berthing and deberthing.

## Fig. 5 - 4   ARRANGEMENT OF FENDERS



## 2. Impact Force and Pressing Force of Vessels

### (1) Berthing impact of vessels

The impact reaction of fenders will be calculated for the following conditions.

Design conditions

| | |
|---|---|
| Vessel type | 75,000 DWT ore carrier |
| Vessel length | 247.0 m |
| Vessel width | 35.4 m |
| Vessel depth | 18.9 m |
| Full-load draft | 13.4 m |
| Displacement tonnage | 97,000 D.T. |
| Berthing speed (at berthing point) | 12 cm/sec. |
| Maximum berthing angle | 6 |
| Berthing point of vessel | 1/4 length point or<br>1/3 length point of vessel |

i. Vessel's berthing energy (E)

The kinetic energy (ES) of a vessel is calculated by the following equation.

$$\boxed{ES = \frac{Wv^2}{2g} \qquad (t\text{-}m)}$$

where

ES = kinetic energy of a vessel (t-m)

W = estimated weight of a vessel (ton)

$$W = (1+\frac{2D}{B})\ Wo$$

D : Draft (m)

B : Beam (m)

Wo: Displacement tonnage of vessel (D.T.)

V = berthing speed of a vessel (m/sec)

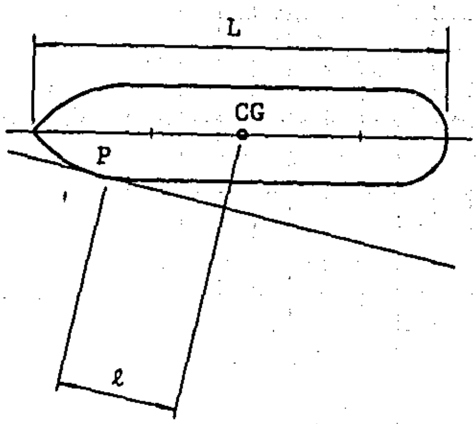
g = acceleration of gravity (9.8 m/sec$^2$)

$$W = (1 + \frac{2D}{B})\ Wo$$

$$= (1 + \frac{2\times13.4}{35.4}) \times 97{,}000$$

$$= 170{,}435\ t$$

ii. Energy lost by turning of the vessel (ER)




In most cases, a ship berths with either the bow or stern at an angle of a certain degree to the quay-wall or dolphin. At the time of berthing, the ship turns and rolls simultaneously. For this reason, the total kinetic energy held by the vessel is consumed partially as its turning energy, and the remaining energy is conveyed to the mooring quay-wall. The energy lost (ER) by the turning of the vessel can be obtained from the following equation.

$$\boxed{ER = \frac{Wv^2}{2g} \times \frac{(\frac{\ell}{r})^2}{1+(\frac{\ell}{r})^2} \qquad (t\text{-}m)}$$

where

$\ell$ = distance alongside the waterline of the quay-wall from the center of gravity of the vessel to the berthing point.

r = turning radius (m) around the vessel's center of gravity on a level surface.

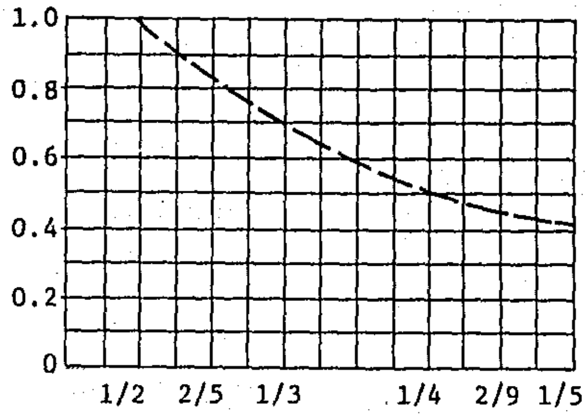
iii. Vessel's effective berthing energy (E)

The effective berthing energy (E) may be expressed by the following formula.

$$E = ES = ER$$

$$= \frac{Wv^2}{2g} \times \frac{1}{1+(\frac{\ell}{r})^2}$$

$$= \frac{Wv^2}{2g} \times K$$




Berthing point of vessel

The value of coefficient K may be read from the above diagram.

Berthing energy in case of berthing at $\frac{1}{4}$ length point,

$$K = 0.5$$

$$E = \frac{Wv^2}{2g} K$$

$$= \frac{170,435 \times 0.12^2}{2 \times 9.8} \times 0.5$$

$$= 62.6 \text{ ton.M}$$

Berthing energy in case of berthing at $\frac{1}{3}$ length point,

$K' = 0.7$

$$E = \frac{Wv^2}{2g} K$$

$$= \frac{170{,}435 \times 0.12^2}{2 \times 0.8} \times 0.7$$

$$= 87.7 \text{ ton·M}$$

iv. Impact reaction

The impact reaction in berthing of vessels depends on the characteristics of fenders.

Description of fenders

| | |
|---|---|
| Type | cell type |
| Size | C-1700 H x 1 |
| Rubber | RH |
| Frame size | width 2,890 mm |
| | height 2,970 mm |

During actual berthing conditions, the fender is generally loaded at an angle which is equivalent to the approach angle of the vessel. These angular loadings change the characteristics of load-deflection and the resulting capacity of the fender to absorb energy. These changes can be predicted based on the results of model testing.

The performance of fenders described above to efficiently absorb the berthing energy calculated in paragraph iii will be as follows.

Table 5-5 Performance of Fenders

| DEFLECTION (%) | PERFORMANCE OF C1700H $\theta = 0^0$ E | R | RUBBER GRADE: RH $\theta = 6^0$ E | R |
|---|---|---|---|---|
| 5 | 2.7 | 63.9 | 2.6 | 60.6 |
| 10 | 10.3 | 114.7 | 9.8 | 110.0 |
| 15 | 21.6 | 150.3 | 20.8 | 146.5 |
| 20 | 34.9 | 163.6 | 33.6 | 158.7 |
| 25 | 49.0 | 167.0 | 47.2 | 159.7 |
| 30 | 63.1 | 165.3 | 60.7 | 159.2 |
| 35 | 77.1 | 161.8 | 74.2 | 156.9 |
| 40 | 90.7 | 157.8 | 87.8 | 161.0 |
| 45 | 104.3 | 161.6 | 101.3 | 172.9 |
| 47.5 | 109.0 | 167.0 | - | - |

Remarks:

$\theta$ : Berthing angle (degree)

E : Energy absorption (t-m)

R : Reaction force (tons)

The following table may be obtained from the above discussions.

Table 5-6 Reaction Force of Fender against Berthing Energy of Vessels

| Item | Calculated Energy | Berthing Angle $\theta = 0°$ | $\theta = 6°$ |
|---|---|---|---|
| Energy absorption (ton-m) | 87.7 | 109.0 | 101.3 |
| Reaction force (ton) | - | 167.0 | 172.9 |
| Deflection (%) | - | 47.5 | 45.0 |
| Face pressure (ton/$m^2$) | - | 22.5 | 23.1 |

From the above table, the maximum reaction force will be $\boxed{172.9\ t}$ and the maximum face pressure will be $\boxed{23.1\ t/m^2}$.

(2) Pressing force due to winds and tidal currents

Reaction force of pier studs against wind pressure and tidal current pressure from the channel side may be calculated as follows.

Table 5-7 Reaction Force of Pier Studs against Winds and Currents

| | H | No. of Pier Stud Pressed | Reaction Force |
|---|---|---|---|
| 75,000 DWT | Full-load 233 ton | 6 | 39 ton/pier stud |
| 50,000 DWT | " 188 ton | 6 | 31 " |
| 25,000 DWT | " 122 ton | 4 | 30 " |

H: Refer to Table 5-3.

The number of pier studs pressed by the vessel may be obtained from Fig. 5-4.

The pressing force against fenders due to winds and tidal currents is far smaller than the berthing impact force of 172.9 tons.

# CHAPTER VI DESIGN AND STRUCTURE

## VI-1 MOORING DOLPHINS

### 1. Design

Mooring dolphins to sustain the ship pulling force will be designed from the point of restricting the stress to be exerted in foundation piles, disregarding the lateral displacement of piles.

The dolphin foundation, subject to strong lateral external force, to be constructed on hard ground, will be designed as a vertical pile structure with piles driven in by drilling pilot holes into the hard ground to assure reliable stability and accurate construction.

The structural analysis of the mooring dolphins will be carried out by the method of Radosavljecvic based on the lateral resistance theory of piles of Y.L. Chang. According to this method of analysis, the lower part of piles will be fixed to the foundation ground, and the upper part will be fixed to concrete of strong rigidity. The stress and displacement of piles are calculated considering the effect of inclination of the top concrete due to the lateral elasticity of the ground and the elastic deformation of piles. The numerical computation will be carried out by electronic computers.

To satisfy the lateral resistance theory of piles, the penetration depth of piles into the ground will be $\pi/\beta$. $\beta$ is related to the elasticity of the ground and the stiffness of piles, and may be expressed by the following formula.

$$\beta = \sqrt[4]{\frac{Dkh}{4EI}}$$

where, D = pile diameter

kn = coefficient of horizontal subgrade reaction

E = elastic modulus of pile

I = geometrical moment of inertia of piles

The value of kh will be assumed to be 7.5 $kg/cm^3$ in case of hard clayey ground with N values in the range of 40 - 50. A slight error in the assumption of the value of kh will not have an appreciable effect on the structural analysis.

The top reinforced concrete will be designed with sections to assure hard stiffness for sufficient allowance for the bending moment of pile heads.

2. Structure (refer to Fig. 6-1)

Foundation piles will be steel pipe vertical piles. The end will be penetrated into the hard soil layer to a depth of 10 m, and the top will be embedded into the top concrete to a length of 1.30 m. The steel pipe piles will be filled with sand for a length of 2.00 m below the bottom of the top concrete and the upper part will be filled with concrete.

The surface of the steel pipe piles above ±0.00 m will be coated

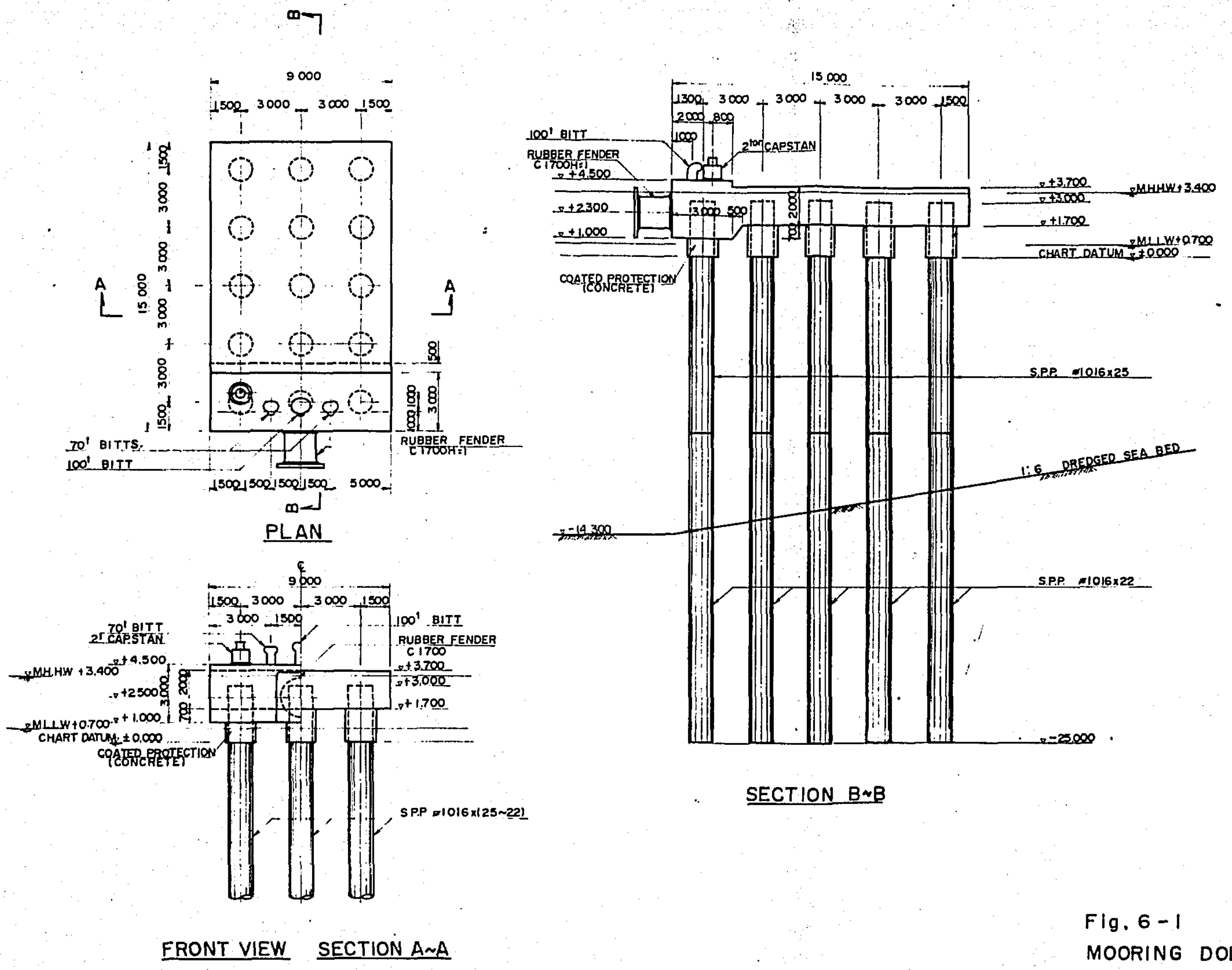



Fig. 6 - 1
MOORING DOLPHIN
SCALE 1 : 200

with concrete of 12.5 cm in thickness.

Dimensions of dolphins

| | |
|---|---|
| Width | 9.00 m x length 15.00 m |
| Crown height | +4.50 m ∿ +3.70 m |
| Depth of foundation piles | -25.00 m |
| Steel pipe piles | ø 1016.0 mm, t = 25 ∿ 22 mm<br>15 piles |
| Concrete | $\delta 28$ = 245 kg/cm$^2$ |

Auxiliary facilities

| | | |
|---|---|---|
| 1) | fenders | cell type 1,700 H |
| 2) | mooring posts | Type A 100$^T$ bitt x 1 |
| | | Type B 100$^T$ bitt x 1, 70$^T$ bitt x 2 |
| 3) | capstan | capacity 2.0$^T$ |
| | | motor 11 KW |

3. Summary of Calculations

| | Vessels Pulling (long period load) | Vessels Pulling (Short period load) | Vessels Berthing (Long period load) |
|---|---|---|---|
| Weight of dolphin | 761.05$^t$ | 761.05$^t$ | 761.05$^t$ |
| Horizontal force | 188.0$^t$ (net pulling force) | 240.0$^t$ (bitt strength) | 172.9$^t$ |
| Direction | Angle of 45° to pier face line | Angle of 45° to pier face line | Perpendicular to pier face line |

| | | | |
|---|---|---|---|
| (Pile with maximum indentation force) | | | |
| Safety factor for pile bearing capacity | $Fs = \frac{Q}{Vmax}$ $= \frac{1,024.8^t}{169.38^t} = 6.1>2.5$ | $Fs = \frac{Q}{Vmax}$ $= \frac{1,024.8^t}{190.54^t} = 5.3>1.5$ | $Fs = \frac{Q}{Vmax}$ $= \frac{1,024.8^t}{76.76^t} = 13.4>2.5$ |
| Requirement of combined stress | $\frac{fa}{Fa} + \frac{Cmfb}{(1 - \frac{fa}{Fe'})Fb}$ $= 0.82<1.0$ | $\frac{fa}{Fa} + \frac{Cmfb}{(1 - \frac{fa}{Fe'})Fb}$ $= 0.95<1.5$ | $\frac{fa}{Fa} + \frac{Cmfb}{(1 - \frac{fa}{Fe'})Fb}$ $= 1.52<1.0$ |
| (Pile with minimum indentation force) | | | |
| Minimum indentation force | — | — | $Vmin = 25.12^t$ |
| (Pile with maximum extraction force) | | | |
| Safety factor for pile bearing capacity | $Fs = \frac{-Q}{-Vmax}$ $= \frac{183.9^t}{39.99^t} = 4.6>3.0$ | $Fs = \frac{-Q}{-Vmax}$ $= \frac{183.9^t}{65.97^t} = 2.8>2.5$ | — |
| Tensile stress | $\sigma_s = 717$ kg/cm$^2$ $< 1,392$ kg/cm$^2$ | $\sigma_s = 936$ kg/cm$^2$ $< 2,088$ kg/cm$^2$ | — |
| (Lateral displacement of dolphin) | | | |
| Perpendicular to pier face line | $\Delta = 3.08$ cm | $\Delta = 3.92$ cm | $\Delta = 3.89$ cm |
| Along pier face line | $\Delta = 3.31$ cm | $\Delta = 4.23$ cm | — |

The horizontal seismic force to act on the dolphin (short period load) is smaller than the 2,400 ton pulling force of vessels.

where,

$Q$ = ultimate bearing capacity of pile

$V_{max}$ = maximum axial indentation force against pile

$F_a$ = axial stress that would be permitted if axial force alone existed

$F_b$ = compressive bending stress that would be permitted if bending moment alone existed

$f_a$ = computed axial stress

$f_b$ = computed compressive bending stress at the point under consideration

$C_m$ = a coefficient whose value shall be taken as 0.85 for members whose ends are restrained.

## VI-2 PIER STUDS

### 1. Design

In regard to the pier studs to be loaded with the girder and floor system, particular consideration must be given to the lateral displacement to occur at times of travelling of the unloader, berthing of vessels, and pulling of vessels. In order to reduce the lateral displacement, the foundation will be constructed with raking and vertical piles. It will be difficult to penetrate raking piles into the hard ground in long lengths. Therefore, the penetration depth of the pier stud piles, including vertical piles will be shallow, and the lower end of the piles will be considered as a hinge support in the structural analysis.

In the hinge support, the horizontal force to act on the lower end of the piles will be resisted by the friction between the bottom surface of piles and the ground, and the shearing strength of the ground around the piles.

Therefore, to function as a hinge support, the piles will be driven in to a penetration depth of over 3m into the hard stratum with N value = 40 ∿ 50 beyond depths of approximately -15m (end of pile will be at -18m). Thus the piles will acquire the necessary bearing capacity and the pulling force of piles will be resisted by the weight of the pier.

As discussed in Chapter II Soil Conditions of Site, and Chapter IV Design Criteria, of the entire length of 300m of the proposed berth, in the section of approximately 50m towards bore hole No. 108, from depths of -15m, the hardness of soil increases sharply. It may be difficult to drive in vertical and raking piles directly to depths of -18m.

A practical method of pile driving must be studied thoroughly in the "Construction Program".

It will not be advisable to excavate the hard stratum in the entire section to a depth of -18m to replace the soil with sand as a method for pile driving as one solution to the problem. An effective hinge support will not be assured at the pile ends.

In the remaining section of approximately 250m of the entire berth length, a penetration depth of over -18m may be secured by piling driving equipment.

The structural analysis of the pier studs will be carried out by the deflection method of analyzing the stress of various members, assuming that the deflection caused when a load acts on the structure is of unknown value. The numerical computation will be carried out by a computor according to the structural analysis program by the deflection method, "STRUDL". "STRUDL" is a structural analysis program in the ICES (Integrated Civil Engineering System) developed at MIT (Massachusetts Institute of Technology).

The reinforced concrete to bind the upper end of the pier stud piles will be designed to possess sufficient strength against stress due to fixed load and surcharge, and the bending moment and shearing strength of pile heads.

External forces to act on the pier studs may be divided into the fixed load from the weight of various structures supported by the pier studs, and variable load of the unloader and vessel.

2. Structure (Refer to Fig. 6-2)

The pier studs will be designed as a raking pile type foundation of steel pipe piles, with the pile tops fixed in concrete. Both the vertical and raking piles of the foundation will be penetrated into the hard soil layer to depths of 3.00 m, and the pile heads will be embedded in the top concrete for a length of 1.10 m. The foundation piles will be filled with concrete, for a length of 1.00 m from the bottom of the top concrete and the surface of piles above ±0.00 will be coated with concrete of 12.5 cm in thickness.

Dimensions and material of the pier stud structure will be as follows.

| | | |
|---|---|---|
| Size | Width 8.00 m x length 23.10 m<br>$\sim$ 24.05 m | |
| Crown height | Floor | +6.00 m |
| | Girder support | +3.90 m |
| Depth of foundation piles | | -18.00 m |
| Steel pipe piles | $\phi$ 914.4 x 16 mm | 16 piles |
| Concrete | $\sigma_{28}$ = 245 kg/cm$^2$ | |
| Auxiliary facilities | | |

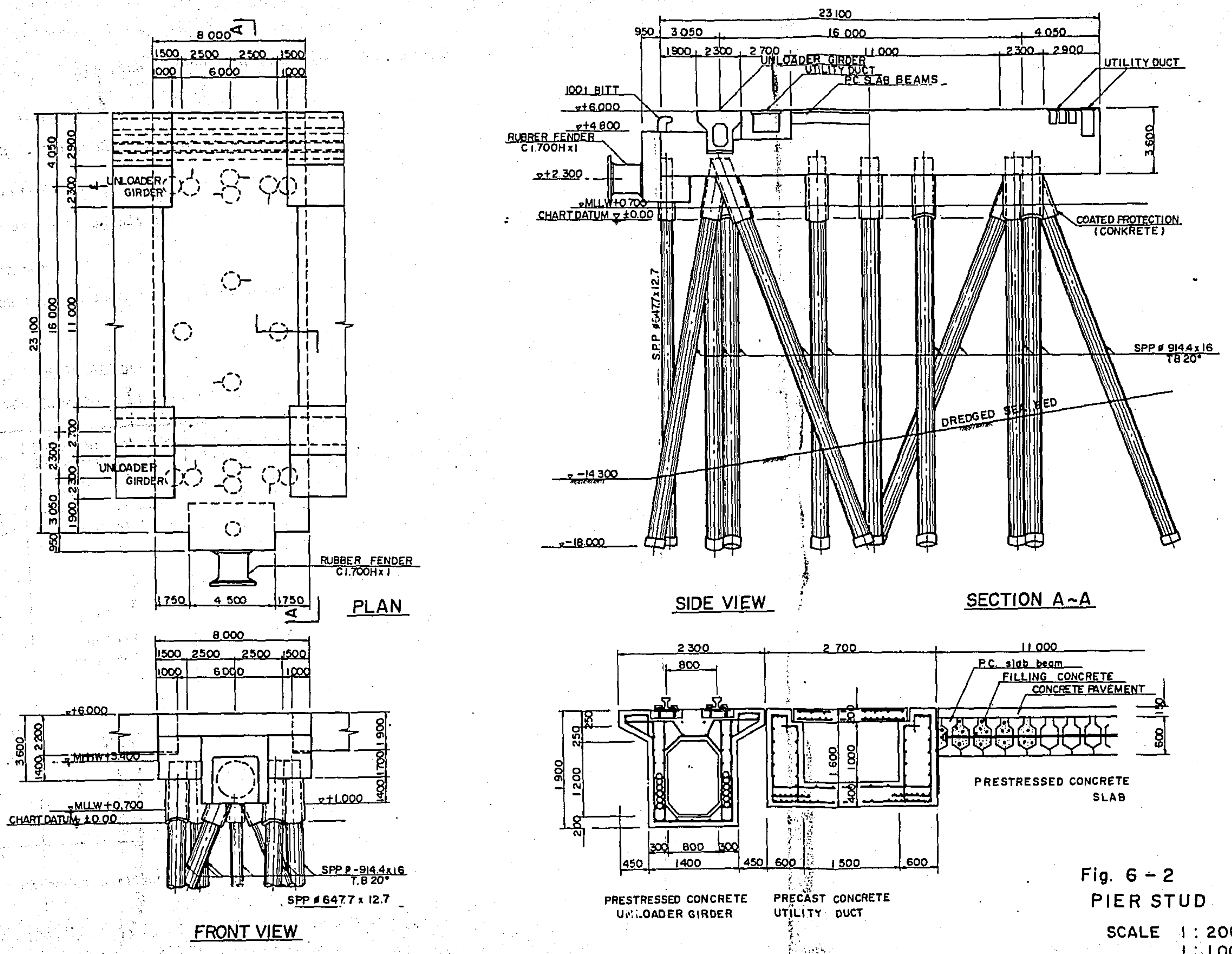



Fig. 6 - 2
PIER STUD

SCALE 1 : 200
1 : 100

1) Fenders cell type 1,700H (for dolphins and pier studs)

2) Water supply bitt

Width 1.50 m x Length 2.00 m x Height 1.10 m

3) Bunkering bitt

Same as water supply bitt

4) Light box

Width 13 cm x Length 1.41 m x Height 3.10 cm

3. <u>Summary of Calculations</u>

1) Common Case for All Pier Studs

| | Unloader working in position (Long period load) | Unloader travelling in work (Short period load) | During Earthquakes (Short period load) |
|---|---|---|---|
| Weight of pier stud | $1,415.3^t$ | $1,415.3^t$ | $1,415.3^t$ |
| Floor, ducts and conveyor load | | | |
| Unloader surcharge | $1,272^t$ (2 unloaders on pier) | $1,590^t$ (2 unloaders on pier) | $500.0^t$ (1 unloader on pier) |
| Horizontal force | | | |
| perpendicular to unloader pier axis line | $42.4^t$ | $169.6^t$ | $249.04^t$ |
| direction of unloader pier axis line | — | — | $249.04^t$ |
| Perpendicular to unloader pier axis | | | |

| | | | |
|---|---|---|---|
| (Pile with maximum indentation force) | | | |
| Safety factor for pile bearing capacity | $F_s = \frac{Q}{Vmax}$ <br> $= \frac{788.7^t}{303.8^t} = 2.6 > 2.5$ | $F_s' = \frac{Q}{Vmax}$ <br> $= \frac{788.7^t}{378.8^t} = 2.1 > 1.5$ | $F_s = \frac{Q}{Vmax}$ <br> $= \frac{788.7^t}{328.5^t} = 2.4 > 1.5$ |
| Requirement of combined stress | $\frac{fa}{Fa} + \frac{Cmfb}{(1 - \frac{fa}{Fe'})Fb}$ <br> $= 0.73 < 1.0$ | $\frac{fa}{Fa} + \frac{Cmfb}{(1 - \frac{fa}{Fe'})Fb}$ <br> $= 0.90 < 1.0$ | $\frac{fa}{Fa} + \frac{Cmfb}{(1 - \frac{fa}{Fe'})Fb}$ <br> $= 0.78 < 1.0$ |
| (Pile with minimum indentation force) | | | |
| Minimum indentation force | $Vmin = 160.5^t$ | $Vmin = 131.7^t$ | $Vmin = 41.3^t$ |
| (Lateral displacement of pier) | | | |
| | $\Delta = 0.4$ cm | $\Delta = 0.9$ cm | $\Delta = 1.1$ cm |
| Direction of unloader pier axis line | | | |
| (Pile with maximum indentation force) | | | |
| Safety factor for pile bearing capacity | — | — | $F_s = \frac{Q}{Vmax}$ <br> $= \frac{788.7^t}{361.6^t} = 2.2 > 1.5$ |
| Requirement of combined stress | — | — | $\frac{fa}{Fa} + \frac{Cmfb}{(1 - \frac{fa}{Fe'})Fb}$ <br> $= 0.86 < 1.0$ |
| (Pile with minimum indentation force) | | | |
| Minimum indentation force | — | — | $Vmin = 26.3^t$ |
| (Lateral displacement of pier) | | | |
| | — | — | $\Delta = 1.3$ cm |

The wheel load (short period load) of the unloader to be sustained by the pier studs is smaller in times of storms than the load when the unloader is travelling during work.

2) Cases for Breasting and Mooring Pier Studs

| | Pier Stud No. 2 & 15<br>Vessels pulling (Long period load) | Pier Stud No. 3, 5, 6, 8, 9, 11, 12 & 14<br>Vessels pulling (Short period load) | Pier Stud No. 2, 3, 5, 6, 8, 9, 11, 12, 14 & 15<br>Vessels Berthing (Long period load) |
|---|---|---|---|
| Weight of pier stud | $1,415.3^t$ | $1,415.3^t$ | $1,415.3^t$ |
| Floor, ducts and conveyors load | $575.08^t$ | $575.08^t$ | $575.08^t$ |
| Unloader surcharge | (no unloader on pier) | (1 unloader on pier) | (1 unloader on pier) |
| Horizontal force | $122.0^t$ (net pulling force) | $100.0^t$ (bitt strength) | $172.9^t$ |
| Direction | Angle of 30° to pier face line | Angle of 10° to pier face line | Perpendicular to pier face line |
| (Pile with maximum indentation force) | | | |
| Safety factor for pile bearing capacity | $Fs = \frac{Q}{Vmax} = \frac{788.7^t}{247.0^t} = 3.2>2.5$ | $Fs = \frac{Q}{Vmax} = \frac{788.7^t}{275.0^t} = 2.9>1.5$ | $Fs = \frac{Q}{Vmax} = \frac{788.7^t}{312.6^t} = 2.5\geqq 2.5$ |
| Requirement of combined stress | $\frac{fa}{Fa} + \frac{Cmfb}{(1 - \frac{fa}{Fe'})Fb} = 0.60<1.0$ | $\frac{fa}{Fa} + \frac{Cmfb}{(1 - \frac{fa}{Fe'})Fb} = 0.66<1.0$ | $\frac{fa}{Fa} + \frac{Cmfb}{(1 - \frac{fa}{Fe'})Fb} = 0.74<1.0$ |

| | | | |
|---|---|---|---|
| (Pile with minimum indentation force) | | | |
| Minimum indentation force | $V_{min} = 7.9^t$ | $V_{min} = 59.4^t$ | $V_{min} = 110.5^t$ |
| (Lateral displacement of pier) | | | |
| Perpendicular to pier face line | Δ = 0.3 cm | Δ = 0.2 cm | Δ = 0.5 cm |
| Along pier face line | Δ = 0.5 cm | Δ = 0.6 cm | —— |

VI-3 UNLOADER GIRDER

1. <u>Design</u>

The unloader girder will be designed as a simple beam structure supported by pier studs at either end with a span of 11.0 m. The girder will be composed of post-tensioned prestressed concrete beams.

The design external force will be composed of vertical and horizontal wheel loads of unloaders. It will be necessary to provide for cases of wheel load conditions other than those given in Section 5 of Chapter IV Design Criteria.

The wheel load conditions may be summarized as follows.

| Load conditions | Wind velocity | Vertical load | Horizontal L. (perpendicular) | Duration of load |
|---|---|---|---|---|
| Off-work ordinary times | 20 knots | 20.8 t/wheel | 1.0 t/wheel | Long period load |
| Working in position | do. | 30.0 t/wheel | 1.0 t/wheel | do. |
| Traveling in work | do. | 37.5 t/wheel | 4.0 t/wheel | Short period load |
| Off-work during earthquakes | - | 28.6 t/wheel | 2.9 t/wheel | do. |
| Off-work in storms | 60 knots | 35.0 t/wheel | 4.0 t/wheel | do. |

From the above comparison of loads, the girder will be designed for load conditions of the unloader at work in position, and travelling in work.

2. Structure (Refer to Fig. 6-2)

The unloader girder will be composed of P.C. beams with box type sections. The upper flange will be designed with consideration for the case of alteration of the unloader rail tracks to a single track on one side, but it will be necessary to check the calculation for the final wheel layout.

| | |
|---|---|
| Outer dimensions | width : upper flange 2.30 m |
| | height : 2.00 m |
| | length : 11.90 m |
| P.C. steel wire | seven wire strand |
| | $\sigma_t \geqq 175$ kg/mm$^2$ |
| Concrete | $\sigma_{28} = 350$ kg/cm$^2$ |

3. Summary of Calculations

| | Working in Position (Long period load) | Travelling in Work (Short period load) |
|---|---|---|
| Weight of girder | | |
| Unloader wheel load | (2 unloaders on girder) | (2 unloaders on girder) |
| Vertical | 30 t/wheel | 37.5 t/wheel |
| Horizontal | 1.0 t/wheel | 4.0 t/wheel |

| | | |
|---|---|---|
| Stress of concrete | | |
| upper compressive stress | $\sigma_c$ = 83 $kg/cm^2$ < 158 $kg/cm^2$ | $\sigma_c$ = 105 $kg/cm^2$ < 158 $kg/cm^2$ |
| lower compressive stress | $\sigma_c$ = 21.2 $kg/cm^2$ > -29.8 $kg/cm^2$ | $\sigma_c$ = -5.2 $kg/cm^2$ > -29.8 $kg/cm^2$ |
| Stress of steel wire | $\sigma_p$ = 84.6 $kg/cm^2$ < 105 $kg/cm^2$ | $\sigma_p$ = 105 $kg/cm^2$ $\leq$ 105 $kg/cm^2$ |

## VI-4 FLOOR SYSTEM OF UNLOADER PIER

### 1. Design

The floor system will be a slab structure with P.C. beams arranged in line as a single body to sustain horizontal post tension. The average bending moment due to concentrated loads and partial surcharge to act on the entire width of the slab will be calculated. The section of the beam will be designed so that the bending moment will be supported by the beams composing the slab according to the respective sharing ratio. The ratio will be calculated according to the methods of Y. Yuyon and C. Massonet, which allow for the bending stiffness and torsional stiffness of the slab.

The average bending moment of the slab will be calculated for a simple beam of one way slab supported at both ends by the pier studs. The span length will be 10.5 m and the width 11.0 m.

The maximum stress of the slab will be exerted in the beam supporting the stay of the belt conveyor system when a uniform load acts on the entire slab.

2. Structure (Refer to Fig. 6-2)

Pretensioned prestressed concrete beams will be arranged in line, and cast-in-place concrete will be filled in between the beams to give horizontal post tension so that the beams will compose a single body. The upper surface of the slab will be paved with concrete.

Dimensions of a slab beam will be a width (flange) of 32 cm and a height of 60 cm. The length of the vertical section of the I beam will be 11.00 m.

Materials

| | |
|---|---|
| Prestressed concrete | $\sigma_{28}$ = 350 kg/cm$^2$ |
| P.C. steel wires | Seven wire strand<br>$\sigma_t \geqq$ 175 kg/mm$^2$ |
| cast-in-place concrete | $\sigma_{28}$ = 245 kg/cm$^2$ |

3. Summary of Calculations

| | Uniform load in case of full load (Long period load) | H-20 on floor (Long period load) |
|---|---|---|
| Surcharge load | —— | AASHO H-20 x 2 |
| | Uniform load<br>2.0 t/m$^2$ x 10.5 m x 11 m = 231$^t$ | Uniform load<br>2.0 t/m$^2$ x 10.5 m x 5.55 m = 116.6$^t$ |
| | Conveyor post<br>5 t/post x 2 = 10$^t$ | Conveyor post<br>5 t/post x 2 = 10$^t$ |
| Weight of slab | 1.84 t/m$^2$ x 10.5 m x 11 m = 212.5$^t$ | 1.84 t/m$^2$ x 10.5 m x 11 m = 212.5$^t$ |

| | | |
|---|---|---|
| Axial direction of unloader pier | | |
| Stress of concrete | | |
| upper compressive stress | $\sigma_c$ = 105.6 kg/cm$^2$ < 158 kg/cm$^2$ | Internal force exerted in P.C. slab will be smaller than the uniform load of full load condition. |
| lower compressive stress | $\sigma_c$ = 38.5 kg/cm$^2$ > -13.8 kg/cm$^2$ | |
| Stress of steel wire | $\sigma_p$ = 87.6 kg/cm$^2$ < 119 kg/cm$^2$ | |
| *Perpendicular to unloader pier axis* | | |
| Stress of steel bar | $\sigma_p$ = 53 kg/mm$^2$ < 57 kg/mm$^2$ | ditto to above. |

## VI-5 MAINTENANCE PLATFORM

### 1. Design

The platform will be designed as a vertical pile structure. Piles will be driven in to penetrate the surface layers, and secure a penetration depth of 3 m into the hard ground with N values of 40 - 50 (end of pile will be at -18 m). The penetration length will provide bearing capacity for the piles and sufficient depth to fix the lower part of the piles in position.

The value of Kh which will bear effect on the lateral resistance of piles will be 1.0 kg/cm$^3$. The method of structural analysis will be similar to the case of the mooring dolphins. The design external force will be the maximum in case seven bulldozers are loaded on the platform and a seismic force should act simultaneously.

2. Structure (Refer to Fig. 6-3)

The structure of the maintenance platform will be a steel pipe vertical pile foundation with the top fixed in the concrete beam. Foundation piles will be penetrated into the ground to a depth of 8 m, and the top will be embedded in concrete for a length of 1.00 m. Steel pipe piles will be filled with concrete for a length of 1.00 m from the bottom of the concrete beams. The surface of the piles above ±0.00 m will be coated with concrete of 12.5 cm in thickness.

Concrete beams and slab will be formed by cast-in-place concrete.

Superstructure of platform

Dimensions

| | |
|---|---|
| Slab | Width 5.90 m x length 24.00 m x height 25 cm |
| Beam | Width 80 cm x height 150 cm |

Foundation of platform

| | | |
|---|---|---|
| Steel pipe piles | $\phi$ 812.8 mm x 12.7 mm | 10 piles |
| Concrete | $\sigma_{28}$ = 245 kg/cm$^2$ | |

## VI-6 APPROACH TRESTLE TO UNLOADER PIER

1. Design

The approach trestle will provide a base for the belt conveyor system.

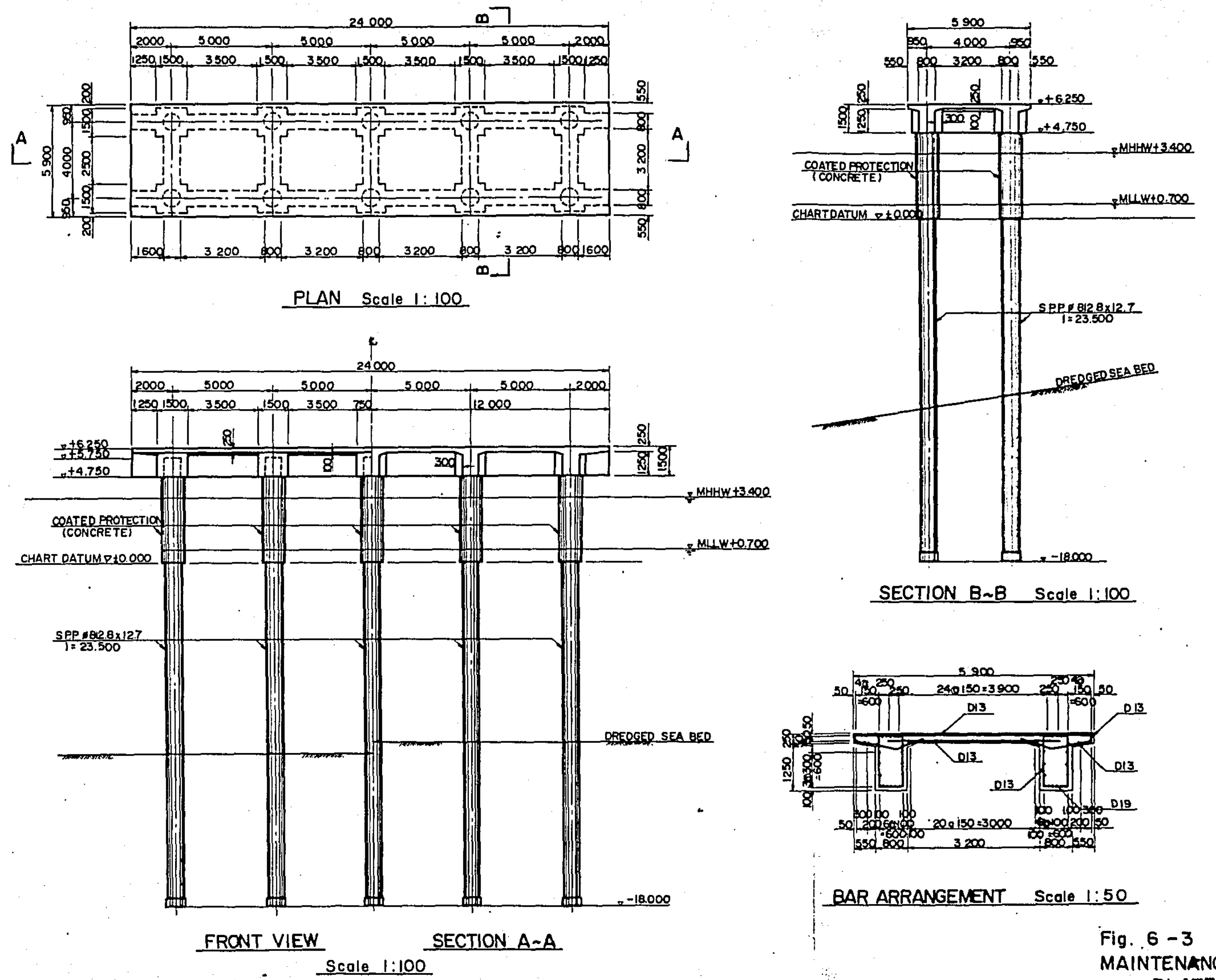



Fig. 6 -3
MAINTENANCE PLATFORM
SCALE 1:200
1:100

The foundation of the pier stud to compose the trestle will be of raking pile structure to reduce the relative lateral displacement between adjacent pier studs at times of earthquakes.

The end pier stud of different type, adjacent to the unloader pier will be designed for similar soil conditions as the case for the unloader pier. The structural analysis will carried out by the same method as the pier stud of the unloader pier, assuming that the lower end of the pile is a hinge support.

The number and angle of inclination of the raking piles will be determined on the assumption that pier stud piles will acquire bearing capacity from a penetration depth of approximately 3 m into the ground of N value ≈ 40 - 50, and the pulling force will be resisted by the weight of the pier stud.

The standard pier studs comprising the trestle will be designed according to different soil conditions from the end pier stud of different shape. Sufficient penetration depth into the ground to fix the lower part of the piles may be secured when the lower end of the piles are driven in to depths of -18 m.

Though a raking pile structure, the structural analysis will be carried out similarly as the case of mooring dolphins, assuming that the coefficient of reaction of the surface layer of the ground which will affect the lateral resistance of piles will be kh = 1.0 $kg/cm^3$.

The top concrete of the pier studs of the trestle will be designed with sections to assure hard stiffness with sufficient allowance for the bending moment of pile heads and the stress due to floor load.

Maximum stress will be exerted in the pier stud piles when a seismic force acts on the pier stud supporting full uniform load.

The structure and design of the floor system of the trestle will be planned similarly as the unloader pier.

2. <u>Structure</u> (refer to Fig. 6-4, 6-5 and 6-6)

The approach trestle will be composed of the pier studs and slab, extending for 137 m. The end pier stud of different type adjacent to the unloader pier will be of pier structure with foundation piles, beam, and slab as a single body.

a)' Pier studs (end pier stud to include beam and slab)

The pier studs will be of raking pile type steel pipe structure, with pile heads fixed in the beams and the lower end penetrated into the ground to depths of 8 - 10 m. Pile heads will be embedded in concrete to a length of 1.00 m.

Steel pipe piles will be filled with concrete to 1.00 m below the bottom of the beam, and the surface above $\pm$ 0.00 m will be coated with concrete of 12.5 cm in thickness.

Superstructure of pier studs

| | |
|---|---|
| Standard pier studs | Width 4.00 m x height 2.00 m x length 16.30 m |
| End pier stud | Width of floor beam 80 cm x height 160 cm x slab height 25 cm |

Foundation of pier studs

| | | |
|---|---|---|
| Steel pipe piles | in water | $\phi$ 812.8 mm, t = 12.7 mm |
| | on land | $\phi$ 647.7 mm, t = 12.7 mm |
| Concrete | $\delta_{28}$ = 245kg/cm$^2$ | |

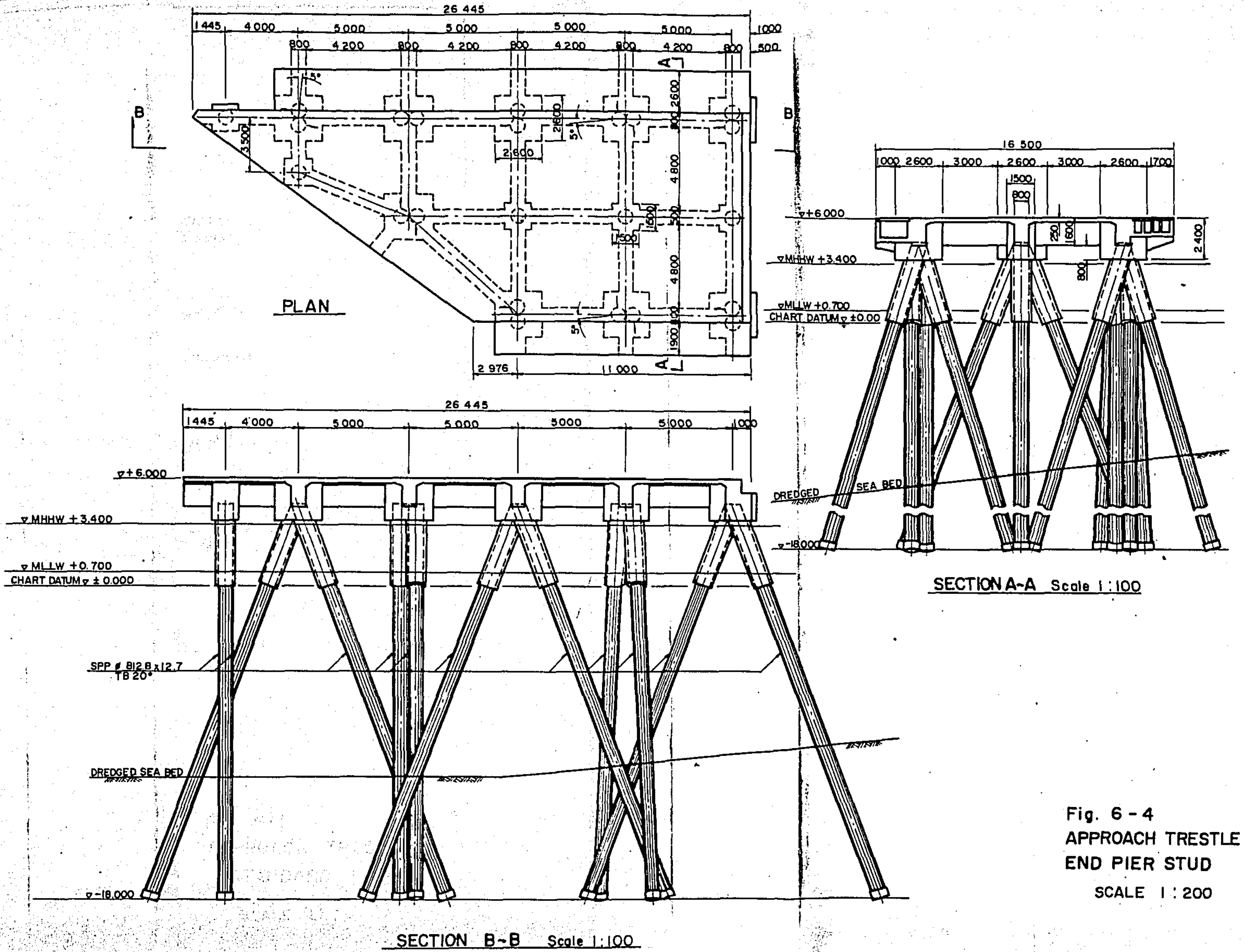
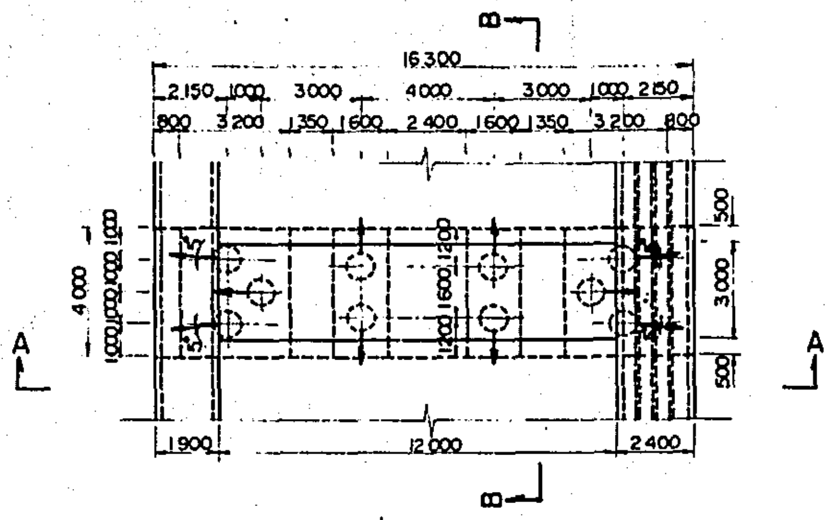
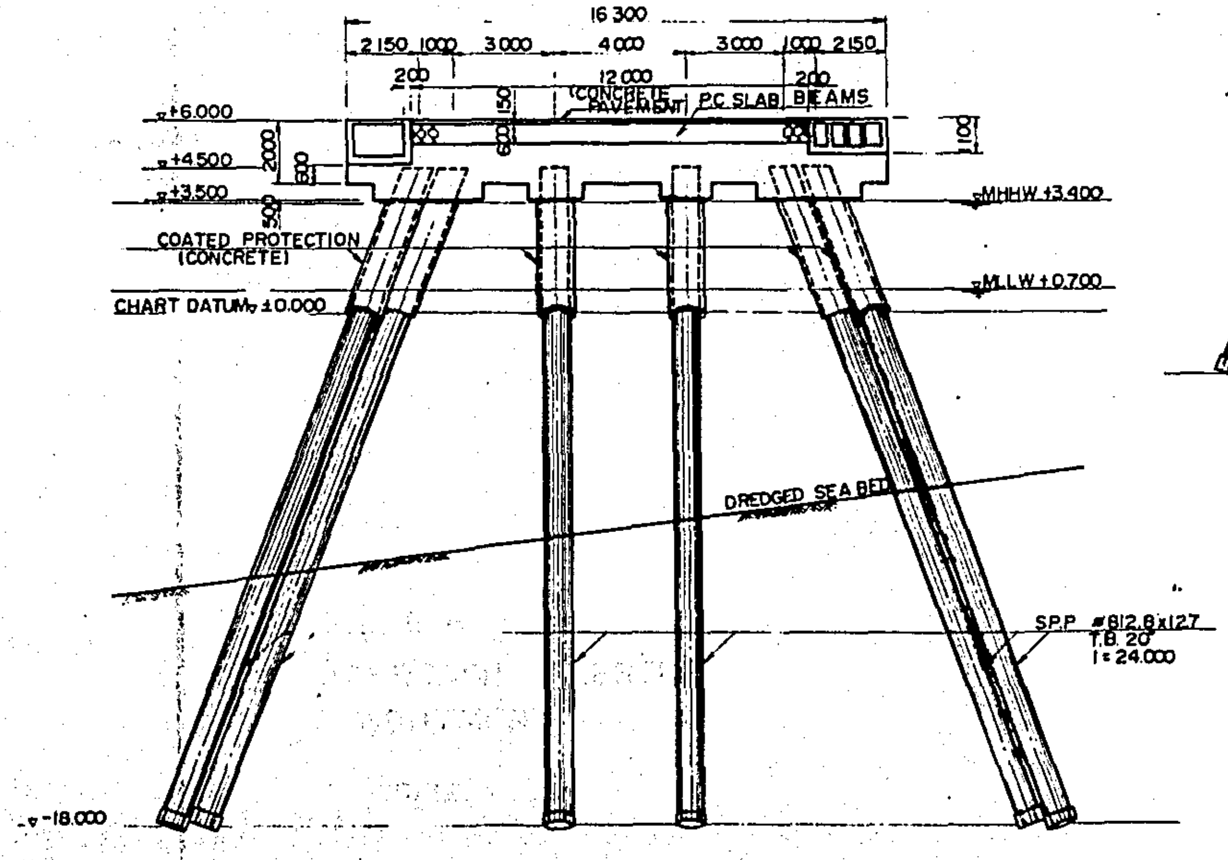


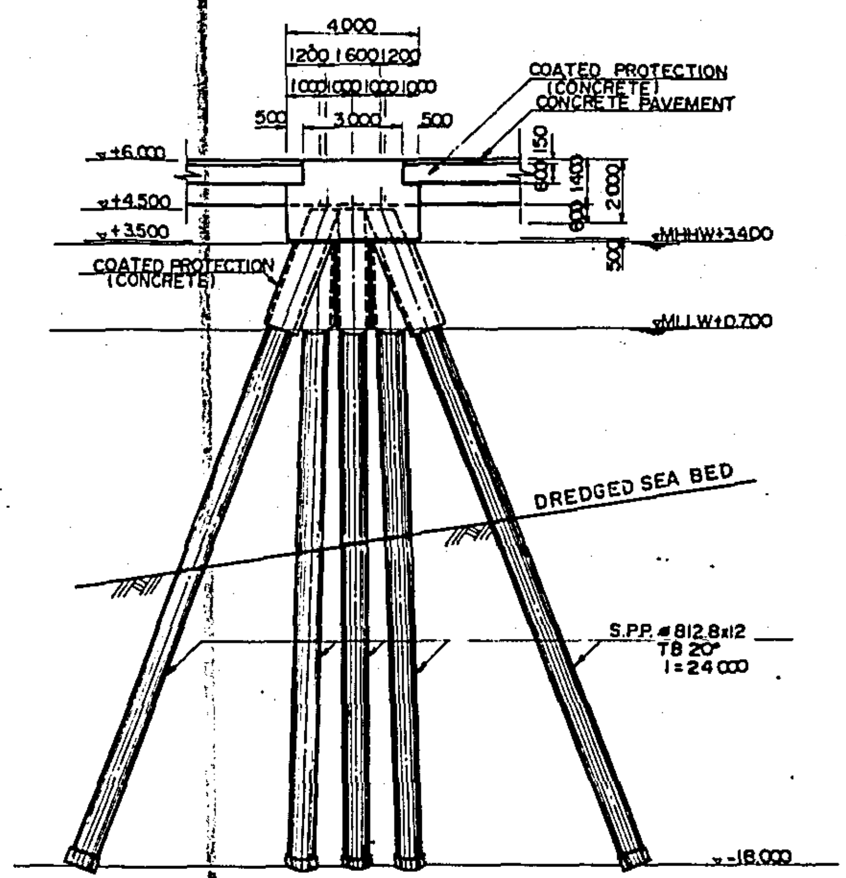

Fig. 6 - 4
APPROACH TRESTLE
END PIER STUD
SCALE 1 : 200




PLAN



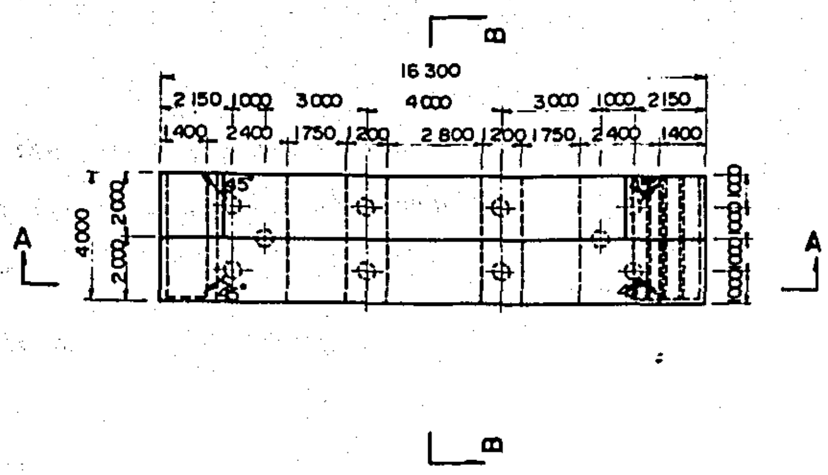

SECTION A~A




SECTION B~B

Fig. 6-5
APPROACH TRESTLE
- STANDARD PIER

SCALE 1 : 200

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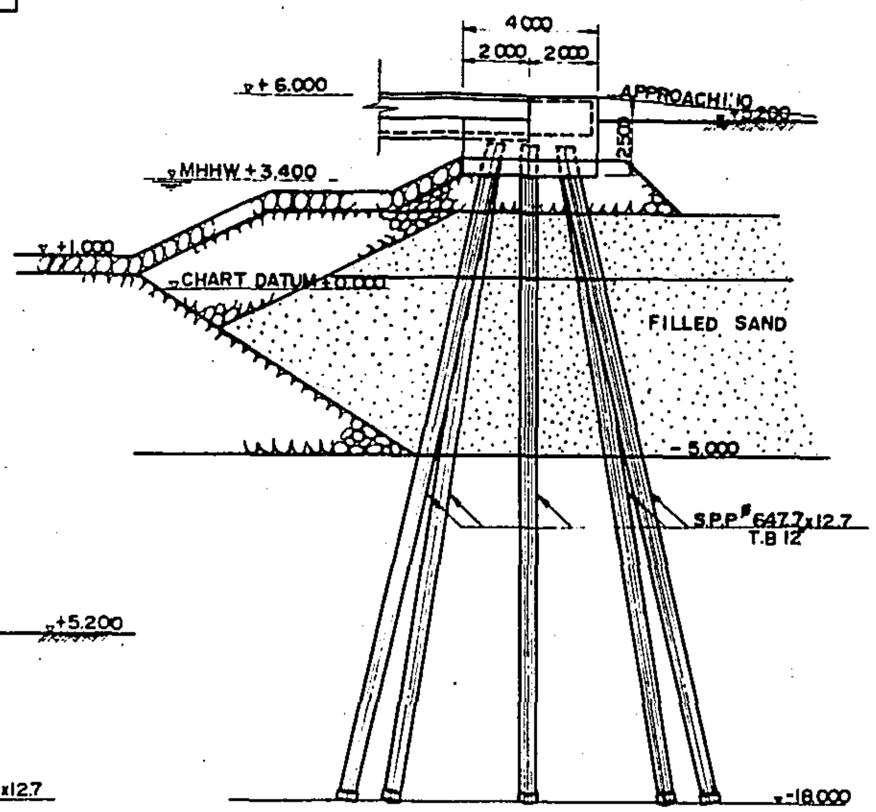
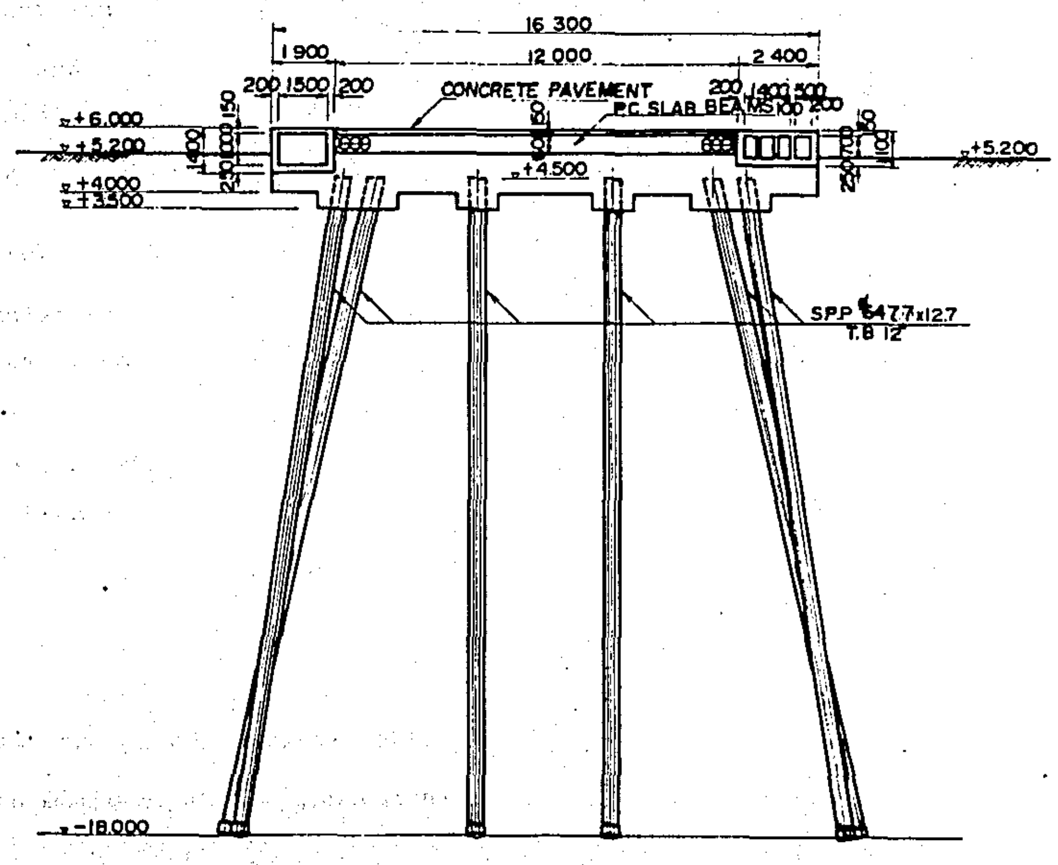



SECTION B~B




SECTION A~A

Fig. 6-6
APPROACH TRESTLE
-ABUTMENT

SCALE 1: 200

b) Slab

The slab will be of the same structure as the floor system of the unloader pier (V-4) except that the lengths of the slab will be 15.00 m.

b) Auxiliary facilities Guard rails

## 3. Summary of Calculations

a) Standard pier

| | Uniform load in case of full load (Long period load) | During Earthquakes (Short period load) |
|---|---|---|
| Weight of pier stud | $289.98^t$ | $289.98^t$ |
| Floor, duct and conveyor load | $510.98^t$ | $510.98^t$ |
| Uniform load | $108.0^t$ | $54.0^t$ |
| Horizontal force | | |
| Perpendicular to to approach tres-le axis | 0 | $85.5^t$ |
| Axial direction of approach. trestle | 0 | $85.5^t$ |
| Perpendicular to approach trestle axis (Pile with maximum indentation force) | | |

| | | |
|---|---|---|
| Safety factor for pile bearing capacity<br><br>Requirement of combined stress | Internal force exerted in piles will be smaller than the case of earthquakes. | $F_s = \frac{Q}{Vmax}$<br>$= \frac{595.9^t}{126.9^t} = 4.7>1.5$<br><br>$\frac{fa}{Fa} \quad \frac{Cmfb}{(1-\frac{fa}{Fe'})\ Fb}$<br>$= 0.87>1.5$ |
| (Pile with minimum indentation force) | | |
| Minimum indentation force | | $Vmin = 47.97^t$ |
| (Lateral displacement of pier) | | $= 1.6$ cm |
| Axial direction of approach trestle | | |
| (Pile with maximum indentation forc.) | | |
| Safety factor for pile bearing capacity<br><br>Requirement of combined stress | ditto to above. | $Fs = \frac{Q}{Vmax}$<br>$= \frac{595.9t}{138.9t} = 4.3>1.5$<br><br>$\frac{fa}{Fa} + \frac{Cmfb}{(1-\frac{fa}{Fe'})\ Fb}$<br>$= 0.62 < 1.5$ |
| (Pile with minimum indentation force) | | |
| Minimum indentation force | | $Vmin - 29.63^t$ |
| (Lateral displacement of pier) | | $\Delta = 0.7$ cm |

b) Slab

| | Uniform load in case of full load (Long period load) | H-20 on Floor (Short period load) |
|---|---|---|
| Surcharge | Uniform load $0.5^t/m^2 \times 14.5^m \times 12^m = 87^t$ | AASHO H-20 x 2<br>Uniform load $0.5^t/m^2 \times 14.5^m \times 5.55^m = 40.2^t$ |
| | Belt conveyor post $5^t$/position x 2 | Belt conveyor post $5^t$/position x 2 $= 10^t$ |
| Weight of slab | $1.84^t/m^2 \times 14.5^m \times 12^m = 320.2^t$ | Same as left = 320.2t |
| Axial direction of Approach trestle | | |
| Stress of concrete | | |
| upper compressive stress | | $\delta c = 156.1\ kg/cm^2 < 158\ kg/cm^2$ |
| lower compressive stress | Internal force exerted in the P.C slab is smaller than the case of surcharge of H-20 truck. | $\delta c = -1.1\ kg/cm^2 > -13.8 kg/cm^2$ |
| Stress of steel wire | | $\delta p = 92\ kg/mm^2 < 119 kg/mm^2$ |
| Perpendicular to approach trestle Axis | | |
| Stress of steel bar | ditto to above. | $\delta p = 53\ kg/mm^2 < 57\ kg/mm^2$ |

# CHAPTER VII CONSTRUCTION PROGRAM

## VII-1 SCOPE OF WORKS

The sequence of engineering works and methods of working for the Iron Ore and Coal Berth and Related Facilities will be planned under one construction program as the facilities are closely related to each other.

A berth for handling iron ore and coal will be constructed for the Steel Mill to be constructed in Port Qasim. The construction of the berth will be commenced with the beginning of preparations and the transport of construction equipment to site on July 1, 1976, and the construction of the main berth will be completed by the end of December 1977.

Works on the pier and related facilities will be commenced with the area for the pier dredged to a depth of -12.8m, and the land reclamation area for related facilities dredged to a depth of -5.0m by a cutter suction dredger in advance.

The dredging of the navigation channel and the construction and installation of aids to navigation will be undertaken simultaneously with the construction of the berth and related facilities.

From October 1977, the erection of the unloader and installation of the belt conveyor system will be commenced on the upper part of the pier.

The principle structures and facilities of the project will be as follows:

| | | |
|---|---|---|
| ° Mooring dolphins | 9.0m x 15.0m | 2 sets |
| ° Unloader pier | 23.1m x 297m | |
| ° Approach trestle | 16.5m x 137m | |
| ° Maintenance platform | 5.9m x 24.0m | 2 sets |

- Small craft trestle
- Water supply
- Power supply and telephone system
- Bunkering and oil facilities
- Fire protection system

Office and operation building complex

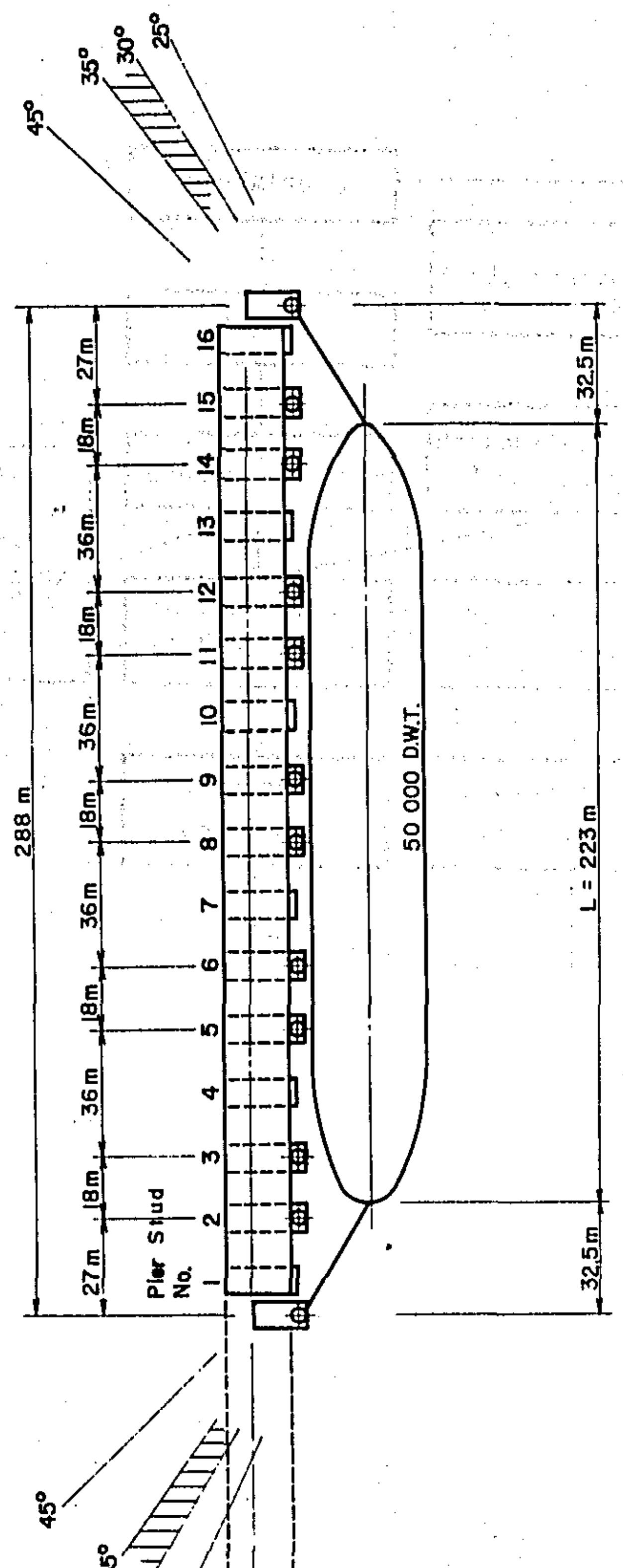
| | | |
|---|---|---|
| | Terminal office | 1317 $m^2$ |
| | Gate house | 60 $m^2$ |

- Mooring facilities

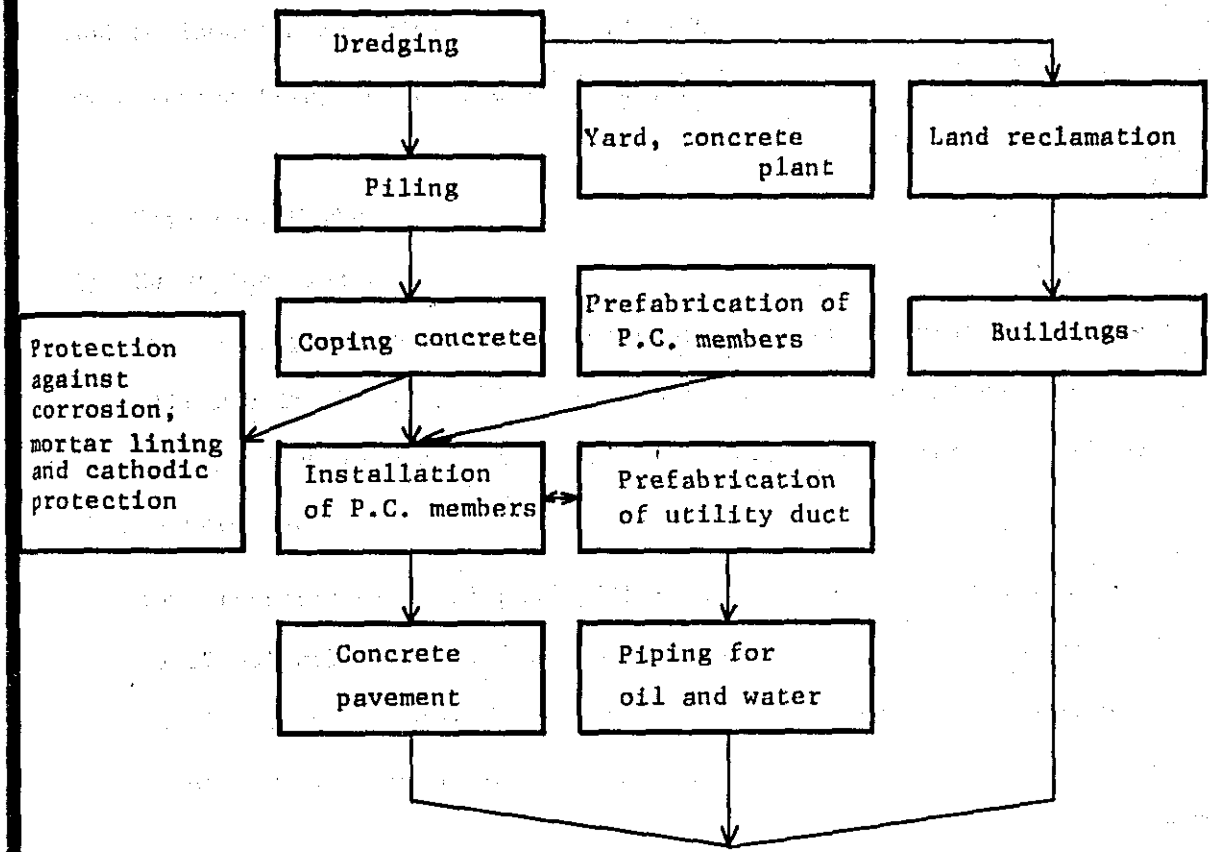
| | |
|---|---|
| 70 ton bollards | 8 sets |
| 100 ton bollards | 9 sets |
| Capstans | 6 sets |

| | |
|---|---|
| ◦ Utility ducts and covers | 48 ducts |
| ◦ Cathodic protection | |
| ◦ Rubber fenders | 9 sets |
| ◦ Road and guard fence | |

Plan of Iron Ore and Coal Berth



The flow of works for the construction of the berth and related facilities may be outlined as follows:



Principle materials for the construction works will be as follows:

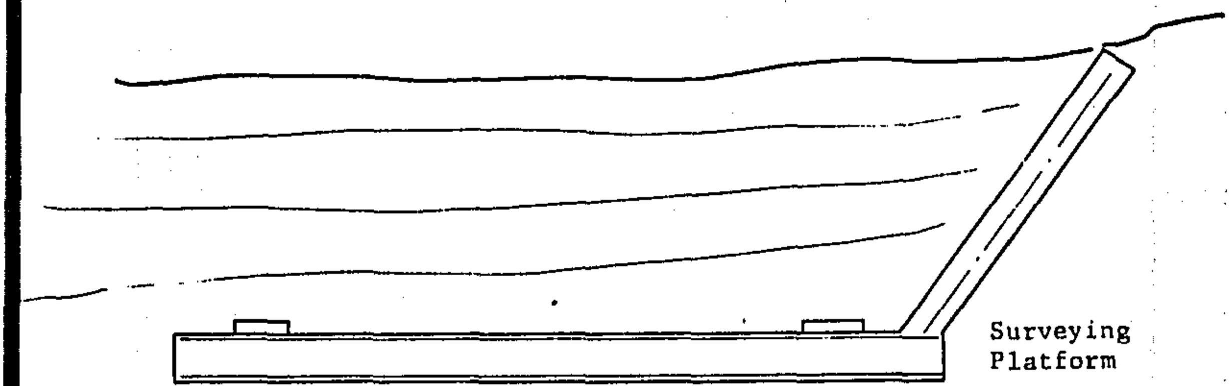
- Concrete 15,700 $m^3$
- Reinforcement 630 tons
- Steel pipe piles 3,300 tons
- Land reclamation material 132,700 $m^3$
- Armor stones (slope protection) 87,600 $m^3$

VII-2 Temporary Works

1) Surveying platforms

Surveying platforms will be necessary for pile driving in locations offshore in the sea. Platforms in the sea will be planned in locations where the navigation of vessels in the area will not be interfered.

Two platforms of steel pipe pile structure ($\phi$660.4 piles will be supplied) will be installed as given in the following diagram. The location for pile driving will be determined by the line of sight from two points on land and sea.

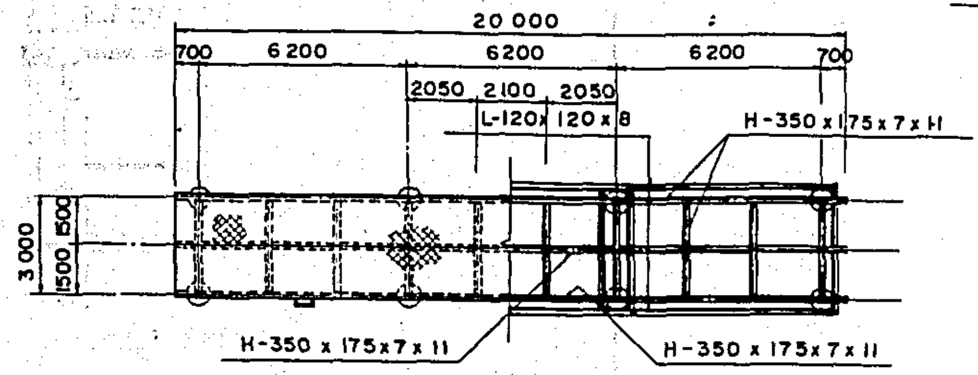



Surveying Platform

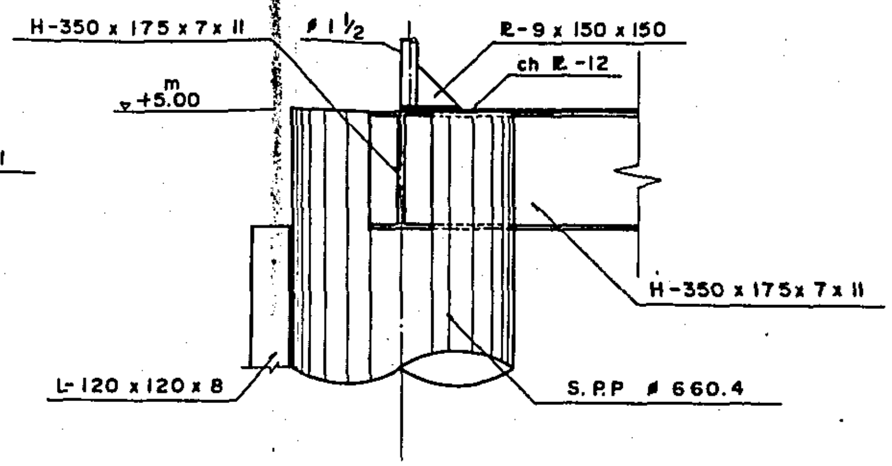
Location of Surveying Platforms

# SURVEYING PLATFORM TYPE-B

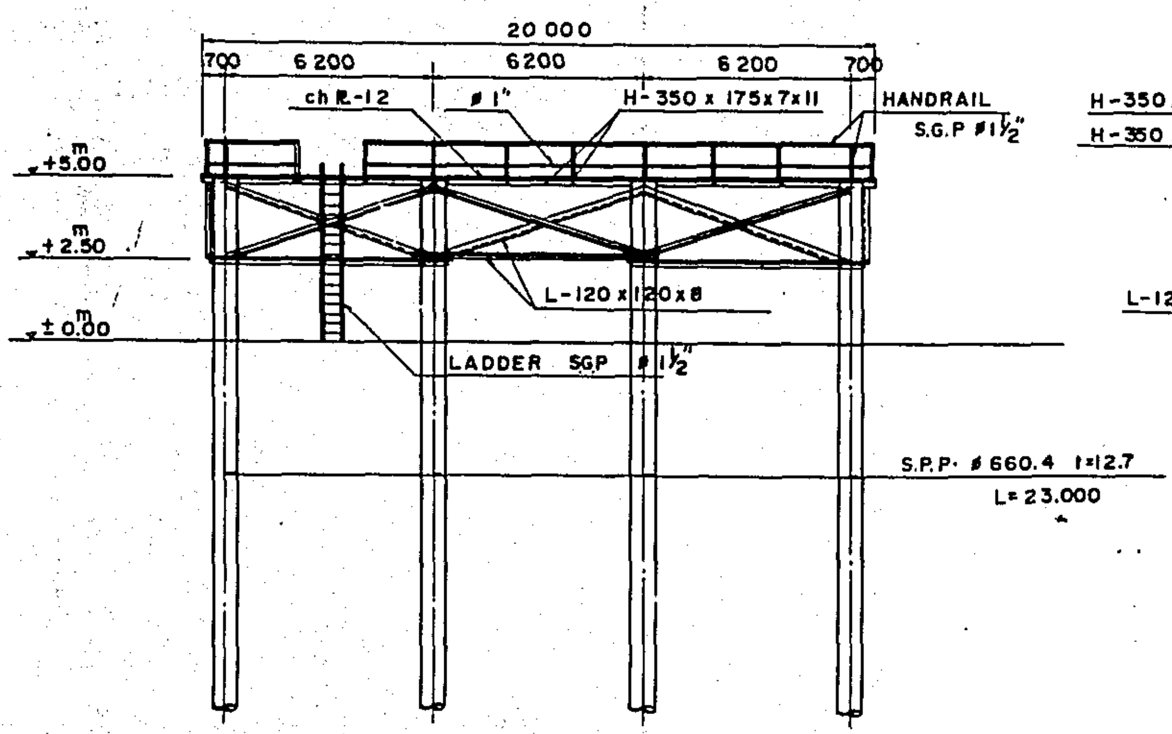
PLAN SCALE 1:200



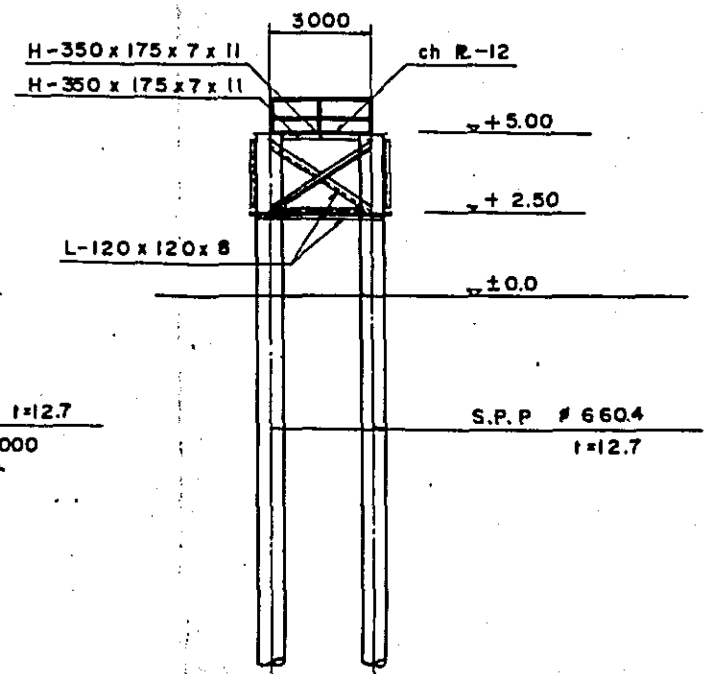

DETAIL OF PILE HEAD SCALE 1:20

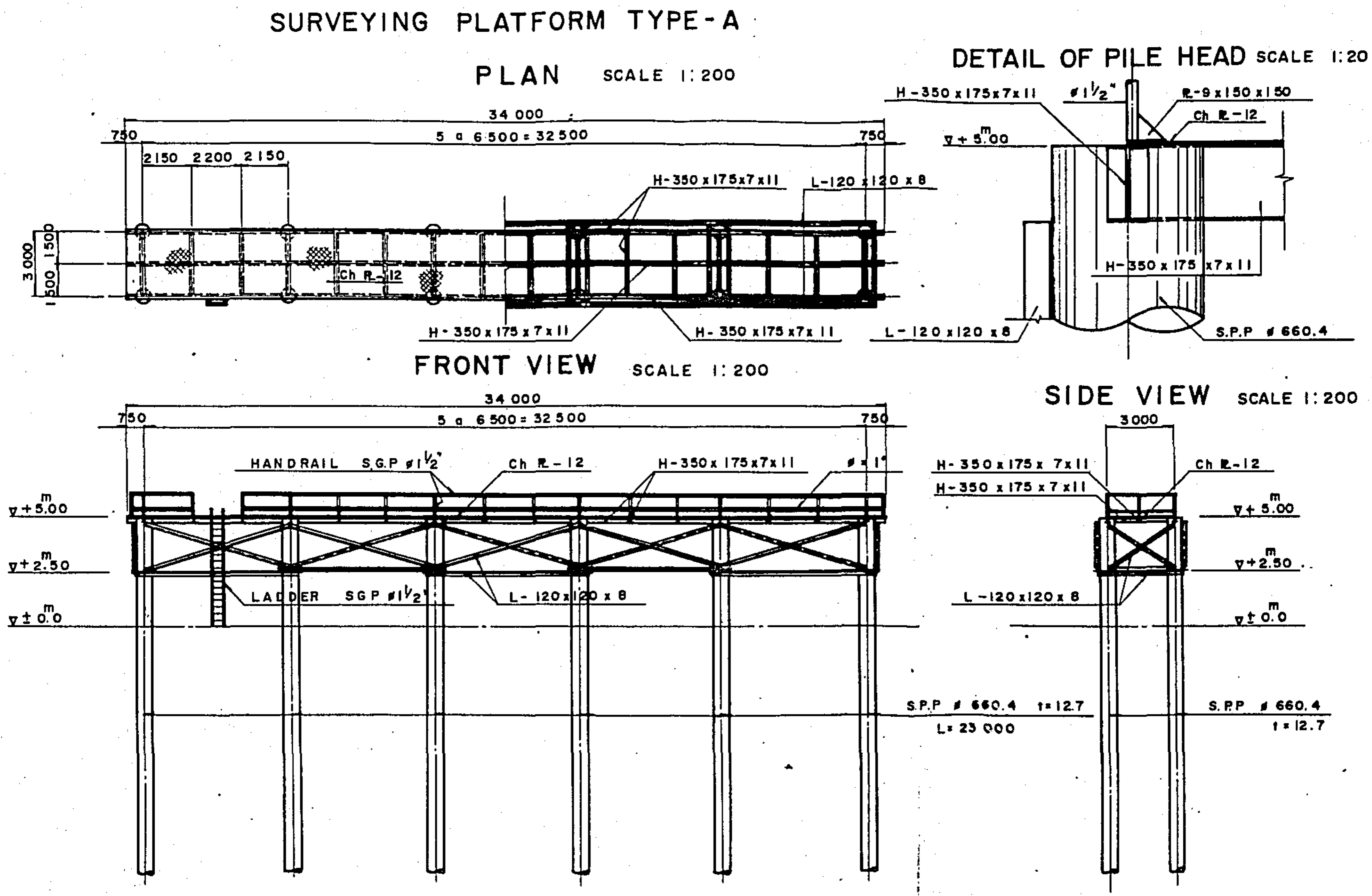



FRONT VIEW SCALE 1:200




SIDE VIEW SCALE 1:200




SURVEYING PLATFORM TYPE-A
PLAN SCALE 1:200
34 000
750
5 @ 6 500 = 32 500
750
2150 2200 2150
3 000
1500
1500
Ch R-12
H-350 x 175 x 7 x 11
L-120 x 120 x 8
H-350 x 175 x 7 x 11
H-350 x 175 x 7 x 11
DETAIL OF PILE HEAD SCALE 1:20
H-350 x 175 x 7 x 11
ø 1 1/2"
R-9 x 150 x 150
Ch R-12
▽+5.00m
H-350 x 175 x 7 x 11
L-120 x 120 x 8
S.P.P ø 660.4
FRONT VIEW SCALE 1:200
34 000
750
5 @ 6 500 = 32 500
750
HANDRAIL S.G.P ø 1 1/2"
Ch R-12
H-350 x 175 x 7 x 11
ø 1"
▽+5.00m
▽+2.50m
▽±0.0m
LADDER SGP ø 1 1/2"
L-120 x 120 x 8
S.P.P ø 660.4 t=12.7
L=23 000
SIDE VIEW SCALE 1:200
3000
H-350 x 175 x 7 x 11
H-350 x 175 x 7 x 11
Ch R-12
▽+5.00m
▽+2.50m
L-120 x 120 x 8
▽±0.0m
S.P.P ø 660.4
t=12.7

2) Construction Base

An area of approximately 600m x 150m will be secured as the construction base on the south side of the Karachi Steel Mill approximately 7 km east of the construction site. A base for the construction of the marginal wharf is already planned in the area by P.Q.A., and the base for the present project will be located in the adjacent area on the east side.

The headquarters for the construction of the project will be established in the construction base as the center of the management of construction works.

Various materials required for the construction works, including steel pipe piles, reinforcement, lumber, and cement will be brought into the base. After necessary working and processing, the materials will be conveyed to site. The following facilities will be established in the construction base.

- Welding shop for steel pipes
- Welding inspection room (radiographic inspection)
- Stock yard for steel pipe piles
- Concrete plant and inspection room
- Prefabrication yard for pretension P.C. slabs and beams
- Prefabrication yard for pretension P.C. unloader girders
- Temporary storage yard for P.C. slabs, beams and girders
- Motor pool
- Shop for reinforcement
- Lumber shop

The arrangement of the various facilities to be established in the construction base will be planned to meet the following requirements.

1. The prefabrication yard for P.C. girders will be located immediately behind the temporary jetty, as the girders will be pulled aside and conveyed to the temporary jetty by large trolleys along the rails.

2. As the P.C. beams will be prefabricated in a series of 8-6 beams in a straight line, the yard will be of a narrow long shape with minimum dimensions of 110m x 50m.

3. The length of the steel pipe piles joined by welding will be 20m - 30m. The minimum width required for the welding shop will be 30m. As two sets of welding equipment will be provided for the project, the shop will be planned with a width of 60m.

3) Electric Power System

(1) Power required for works at sea will be supplied by generators. For welding in places where generators cannot be installed, the generator will be set on a pontoon. For other welding purposes, the generator will be installed on the structures completed.

(2) In the base, power will be supplied from the power plant to be established at a site approximately 4 km northwest from the base. Electric power will be transmitted from the power plant to the receiving equipment of the facilities in the base by high voltage of 3,300V - 6,600V. The electric

power will then be distributed to various electric apparatuses.

Electric power demand in the respective shops and plants may be estimated as follows.

| | |
|---|---|
| ∘ Concrete plant | 60 KVA |
| ∘ Welding shop | 110 KVA |
| ∘ P.C. Beam yard | 90 KVA |
| ∘ Lumber shop | 20 KVA |
| ∘ Reinforcement shop | 15 KVA |
| ∘ Motor pool | 50 KVA |
| ∘ Lighting of office and living quarters | 155 KVA |
| ∘ Total | 500 KVA |

4) Water Supply

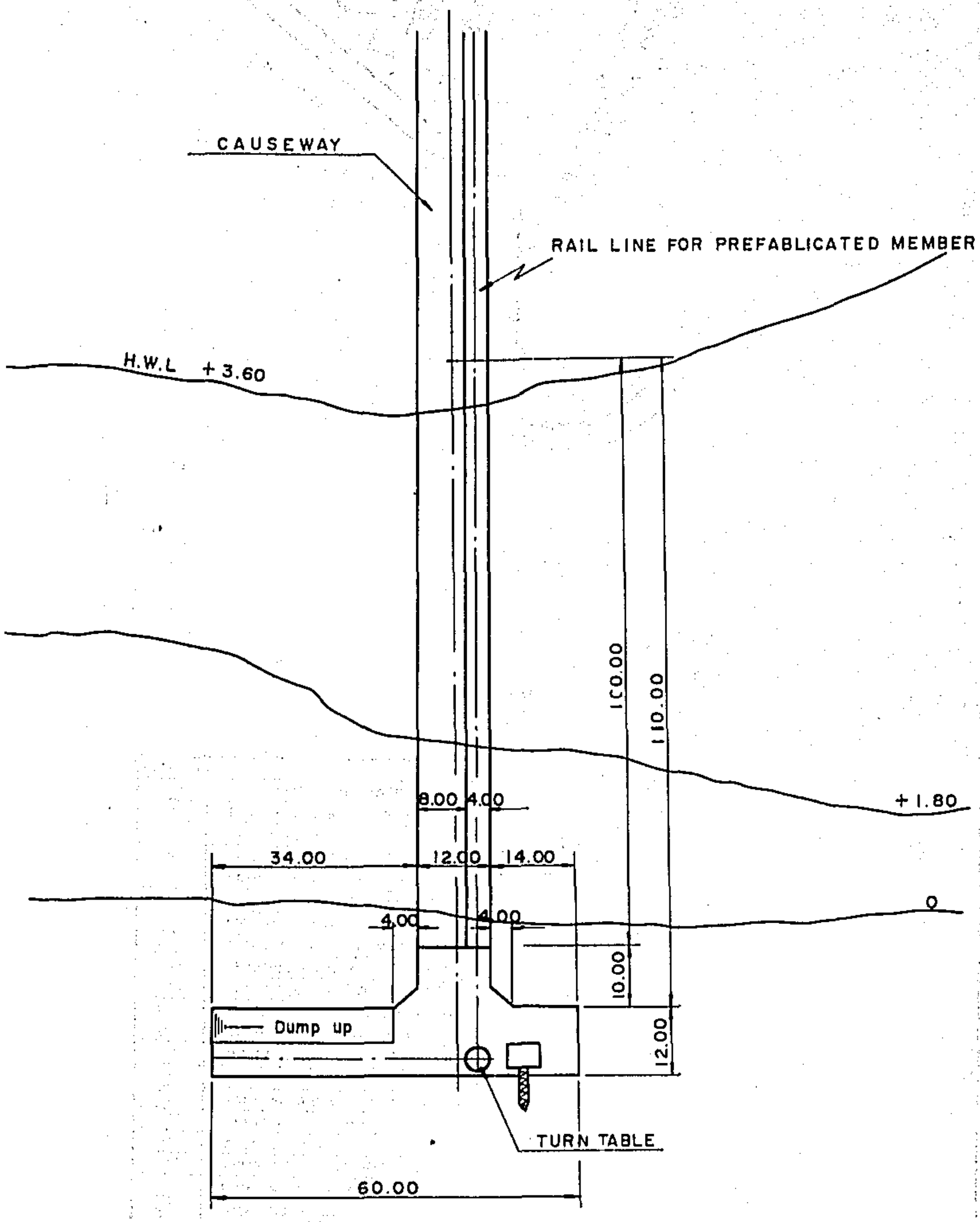
Water supply will be in demand in the concrete plant at an average of about 300 tons per day, and in the P.C. beam prefabrication yard for steam curing purposes. Water will be supplied from the vicinity of the electric power plant.

## VII-3 Piling

1) Preparation of Piles

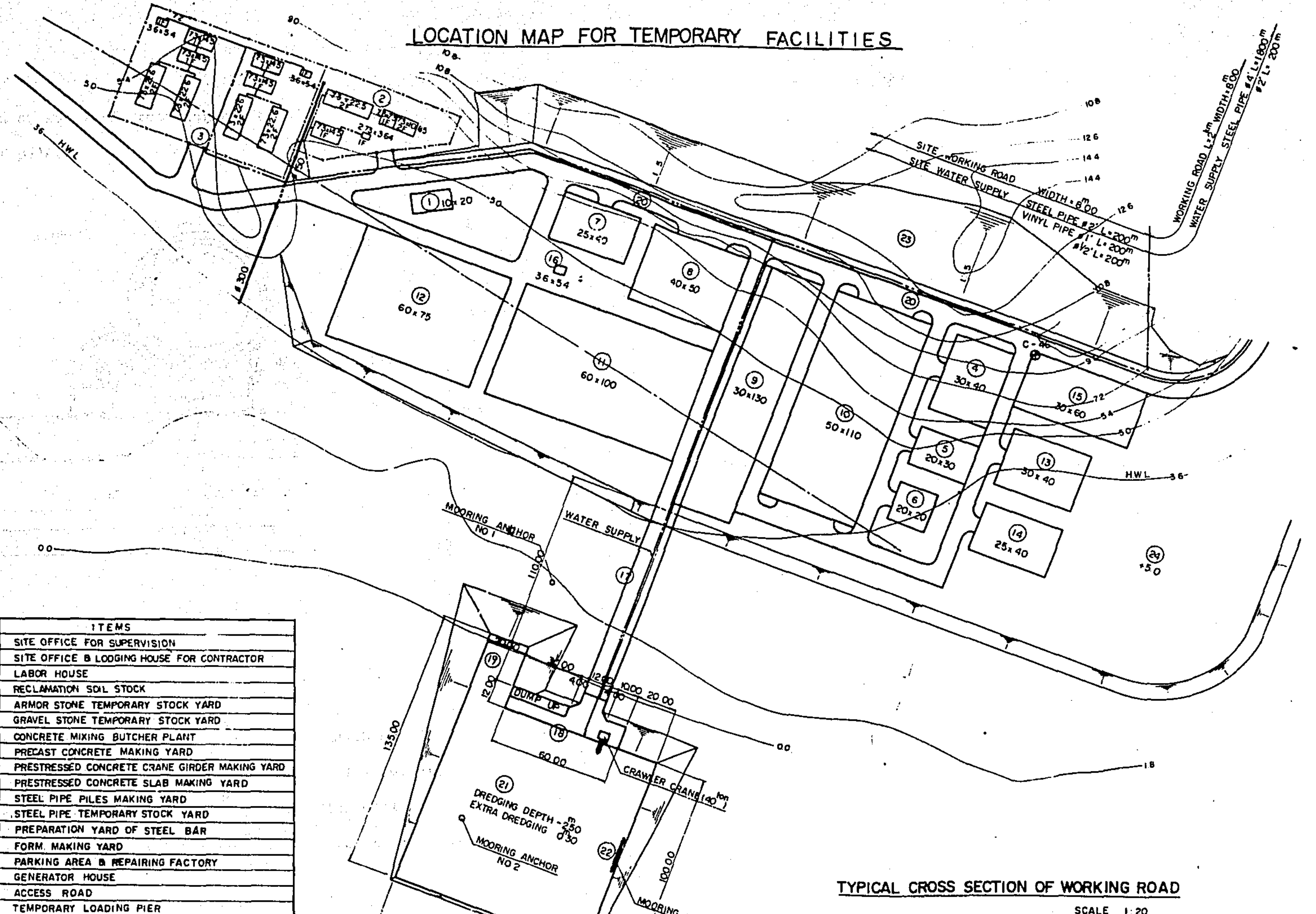
Steel pipe piles will be unloaded from cargo vessels in the Port of Karachi and brought in to the steel pipe stock yard in lengths of 10.0m - 15.0m.

After bevelling by an automatic gas cutter, slag, dirt, rust, oil, paint and other foreign matter will be completely removed from the base metal. The steel pipes will be joined by




TEMPORARY JETTY

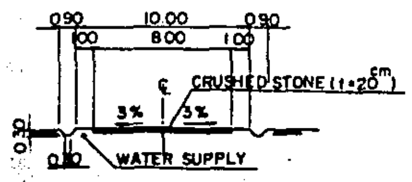
# LOCATION MAP FOR TEMPORARY FACILITIES



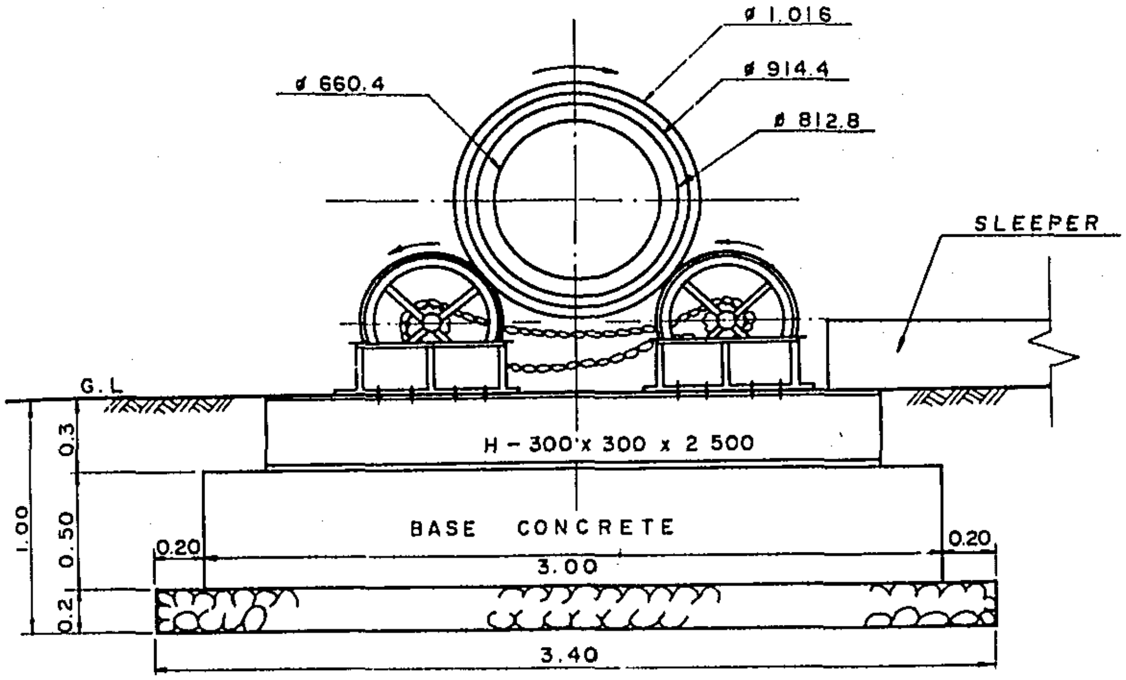

| NUMBER | ITEMS |
|---|---|
| 1 | SITE OFFICE FOR SUPERVISION |
| 2 | SITE OFFICE & LODGING HOUSE FOR CONTRACTOR |
| 3 | LABOR HOUSE |
| 4 | RECLAMATION SOIL STOCK |
| 5 | ARMOR STONE TEMPORARY STOCK YARD |
| 6 | GRAVEL STONE TEMPORARY STOCK YARD |
| 7 | CONCRETE MIXING BUTCHER PLANT |
| 8 | PRECAST CONCRETE MAKING YARD |
| 9 | PRESTRESSED CONCRETE CRANE GIRDER MAKING YARD |
| 10 | PRESTRESSED CONCRETE SLAB MAKING YARD |
| 11 | STEEL PIPE PILES MAKING YARD |
| 12 | STEEL PIPE TEMPORARY STOCK YARD |
| 13 | PREPARATION YARD OF STEEL BAR |
| 14 | FORM MAKING YARD |
| 15 | PARKING AREA & REPAIRING FACTORY |
| 16 | GENERATOR HOUSE |
| 17 | ACCESS ROAD |
| 18 | TEMPORARY LOADING PIER |
| 19 | DUMPING AREA FOR RECLAMATION SAND |
| 20 | WORKING ROAD |
| 21 | DREDGING |
| 22 | MOORING PILE |
| 23 | CUTTING AREA |
| 24 | BANKING AREA |
| | |
| | |

## TYPICAL CROSS SECTION OF WORKING ROAD

SCALE 1:20



welding on an automatic rotating equipment to be finished as steel pipe piles.




Steel pipe piles welded and joined to the length specified in the design diagrams will be marked with the pile number, and scale marks for every 20m in paint. The piles will be loaded on pontoons by a crane and conveyed to the side of the pile driving barge.

2) Piling Works

A total of 481 steel pipe piles will be used for the construction of the berth. Details of the dimensions and number will be as follows.

Table of Steel Pipe Piles

| Dimension | No. | Remarks |
|---|---|---|
| ø1,016 x 22-25mm x 28m | 30 | Mooring dolphin |
| ø914.4 x 16mm x 21.5-25m | 256 | Pier |
| ø812.8 x 12.7mm x 22.5-24m | 103 | Approach trestle |
| ø647.7 x 12.7mm x 21.5-230m | 92 | Small craft trestle and pier |

Pile driving works of the above steel pipe piles at the site will be commenced from the middle of November 1976, and completed in approximately 240 days by the middle of July 1977. Considering the weather and natural conditions and holidays, the actual number of days available for pile driving may be estimated to be approximately 200 days.

The majority of the piles are of large diameters of 900m/m - 1,000m/m, the foundation ground is hard with N values of 50, and bore holes must be drilled for pile driving in some locations.

The number of piles to be driven in per day may be calculated as follows.

Piling Schedule

) Pile Profile

| | Approach Trestle and Maintenance Platform | Dolphin and Small Craft Trestle | Unloader Pier (driving only) | Unloader Pier (driving, drilling, jet) | Dolphin (driving, drilling, jet) |
|---|---|---|---|---|---|
| Diameter | 812.2(mm) | 647.7mm | 914.4(mm) | 914.4(mm) | 1,016(mm) |
| Length | 22.5∿24.0 | 21.5∿23.0 | 21.5∿24.0 | 21.5∿24.0 | 28.0 |
| Embedded length | 10∿14 | 50∿ | 8.0∿12.0 | 8.0∿12.0 | 12.2m |
| Number | 103 | 92 | 208 | 48 | 30 |
| Ultimate Bearing Capacity | | | 737 ton | 752 ton | |

) Pile Driving & Drilling Works

| | | | | | |
|---|---|---|---|---|---|
| Type of Driver | D-40 or 3B-270 orMRB-1000 | D-40 or SB-270 MRB-1000 | D-40 or SB-270 MRB-1000 | ∿ D-70 ∿ or SB-400 ∿ MRB-1500 | D-70 or SB-400 MRB-1500 |
| Weight of Ram | 4,000 Kg | 4,000 kg | 4,000 ∿ 7,000 kg | 7,000 kg | 7,000 kg |
| Number of Blows per 1m penetration | 250 | 100 | 500 | 500 | 500 |
| Number of Blows per 1 minute | 50/Min | 50/Min | 50/Min | 50/Min | 50/Min |
| Driving time per 1 pile(minutes) | 60 Min | 10 Min | 100 Min | 100 Min | 120 Min |
| Drilling time per 1 pile(minutes) | - | - | - | 50 Min | 70 Min |
| Pitching time per 1 pile | 60 Min | 40 Min | 60 Min | 60 Min | 60 Min |
| Shifting and Positioning | 60 Min | 30 Min | 60 Min | 60 Min | 60 Min |
| Change of barge | - | - | - | 60 Min | 60 Min |
| Total Piling time per 1 pile(minutes) | 230 Min | 130 Min | 220 Min | 330 Min | 370 Min |
| Number of Piling per day | 1.3 Piles | 2.3 Piles | 1.3 Piles | 0.9 Piles | 0.8 Piles |

* Working hours 8.0 hours/day

* Effective working hours 5 hours/day

Two sets of pile driving barges will be prepared, equipped with the following pile drivers.

| | | |
|---|---|---|
| Type; number | H40 .... 1 set | H-70 .... 1 set |
| Weight of ram | 4,000 kg | 7,200 kg |
| Number of blows | 40 ∿ 60 blows/min | 38 ∿ 60 blows/min |
| Energy per blow | 11,600 kg-m | 21,500 kg-m |
| Length of stroke | 2,750 mm | 2,750 mm |
| Weight of driving hammer | 9,500 kg | 18,500 kg |
| Cooling system | Water-cooling | Water-cooling |

The specification of the pile driving barge to be used will be as follows.

| | |
|---|---|
| Displacement tonnage | 1460 ton |
| Size of pontoon | 40m x 18m x 3.6m x 2.0m |
| Main engine | D.E 700 HP |
| Height of leader | 52.3m |

The following data will be recorded in pile driving.

Pile number

Pile type

Pile inclination

Type of driving hammer

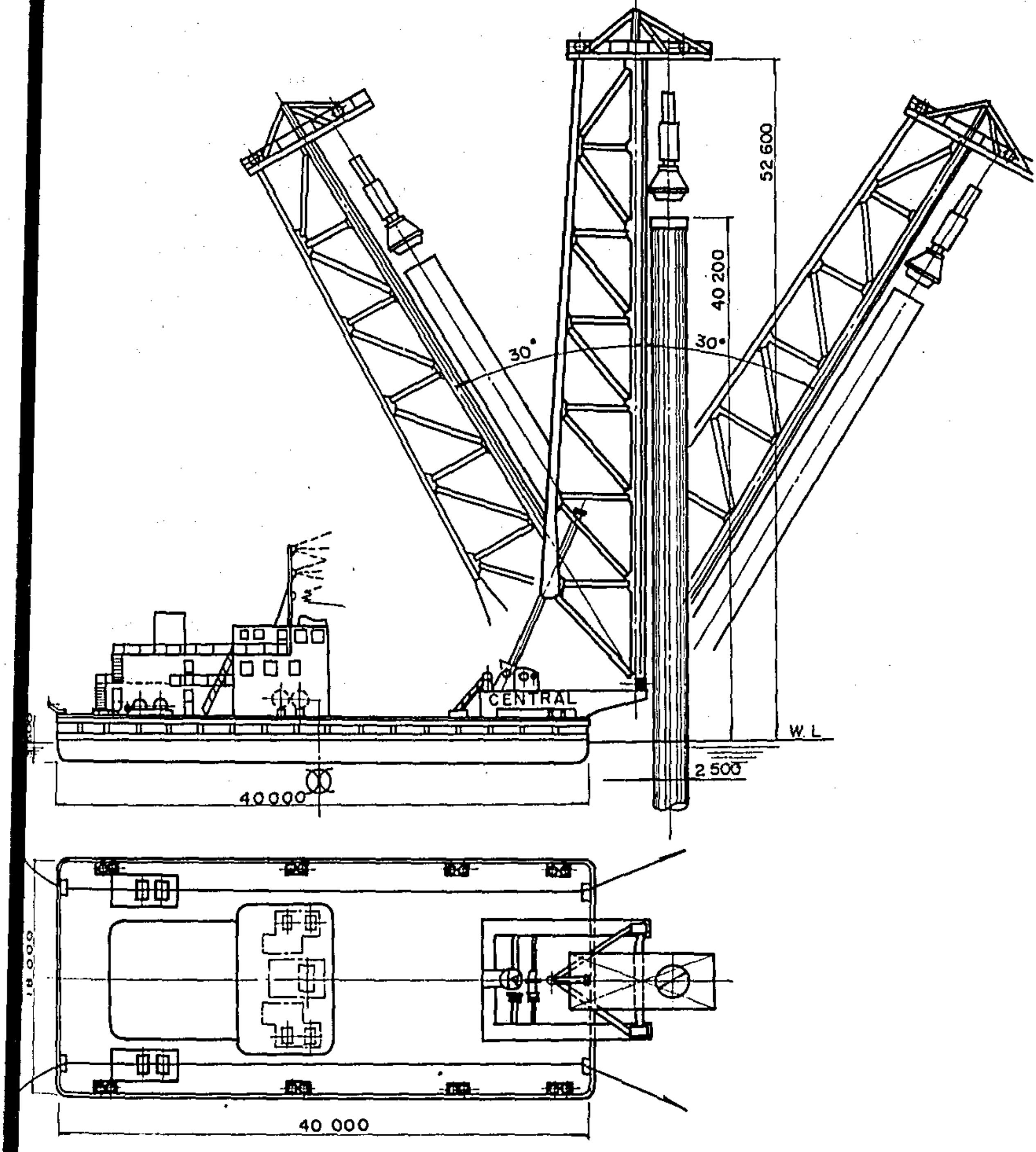
Degree of penetration in last 3 series of blows (10 blows each)

Final level and position of pilehead

Number of blows per penetration interval

Total number of blows

Degree of rebound, if encountered




PILE DRIVING BARGE.

In the construction works, piles must be driven in to depths specified in the design. Piles shall not be severed on grounds that piles cannot be driven in further.

Pile driving of steel pipe piles will be suspended and drilling will be commenced in case the following conditions are encountered.

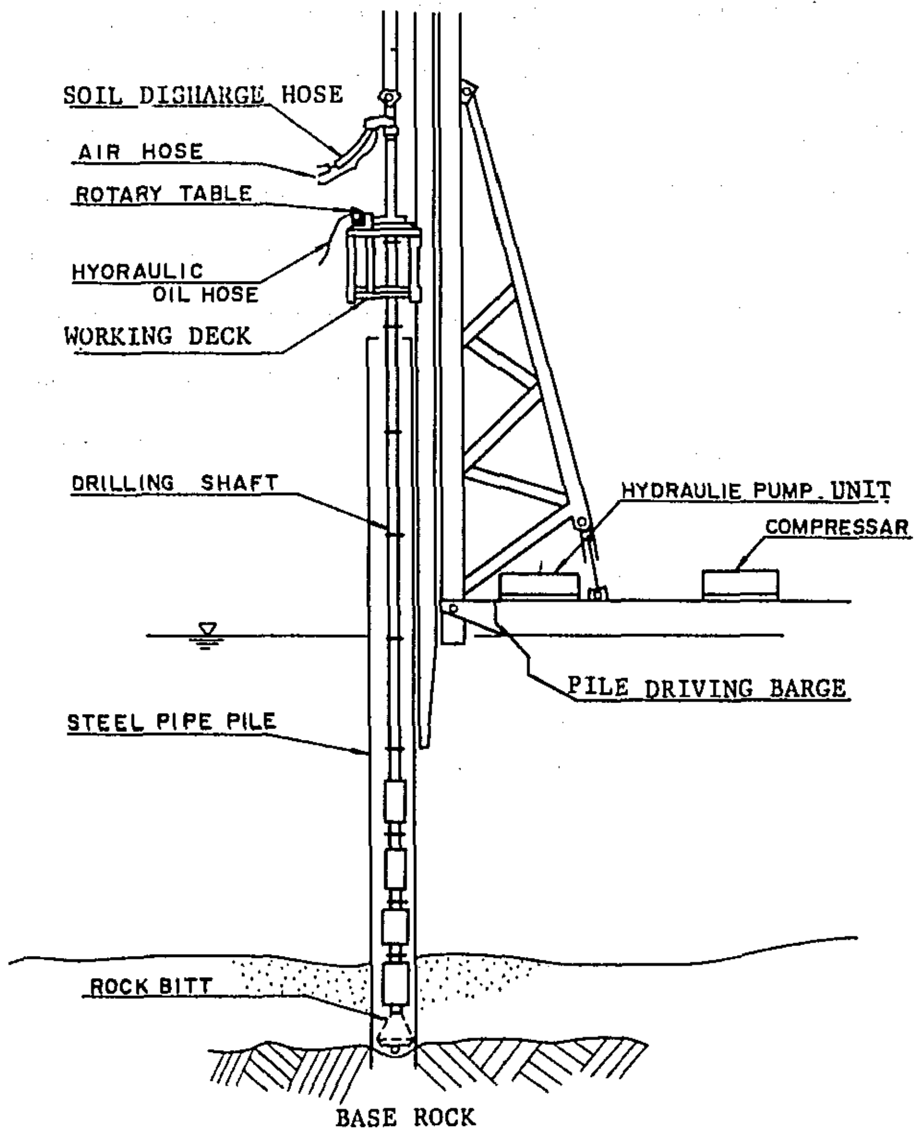
- When buckling occurs in the pile head.
- When the total number of blows exceeds 3000 blows, or when the degree of penetration per blow is 2m/m or less.

3) Drilling Works

In hard ground where steel pipe piles cannot be driven in by a pile driver (pile driving barge), piles will be driven in by a diesel hammer as far as possible. With the steel pipe pile as the casing, boring holes with a diameter of a difference within 5cm from the outer diameter of the piles will be drilled, and the piles will be driven in to the depth specified in the design.

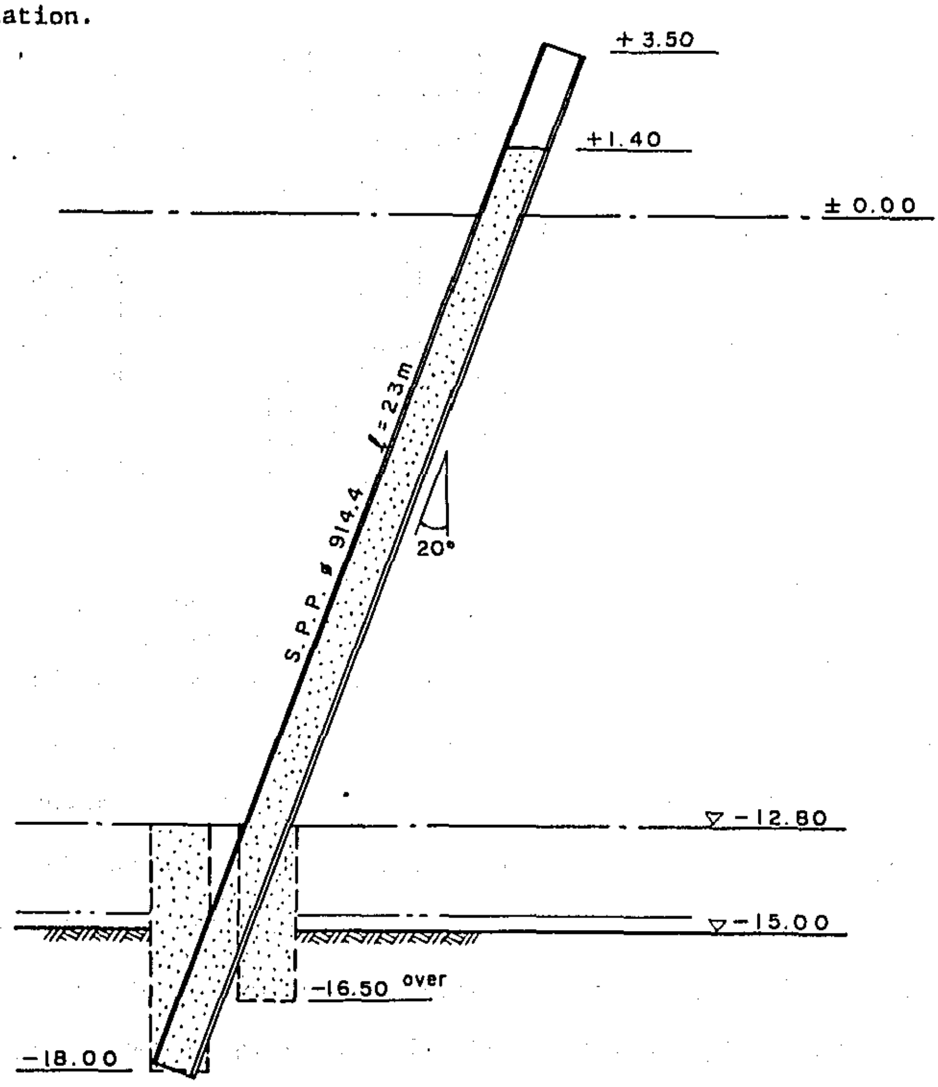
IHI-WIRTH Boring Machine on a Pile Driving Barge

Suspended Boring Machine




VERTICAL PILE

In case of piling of raking steel pipe piles with an angle of 12.5° or more, equipment to drill holes with the pile as the casing is not easily available due to the shortage of equipment. Judging the soil condition from soil survey and piling of vertical piles in the vicinity, in hard ground, it will be allowable to drill a hole with a casing (ϕ1219.6) in advance (2 holes for piling of one steel pipe pile), and the casing will be filled with returned sand. The casing will be pulled out, and the steel pipe pile will be driven in at the specified location.

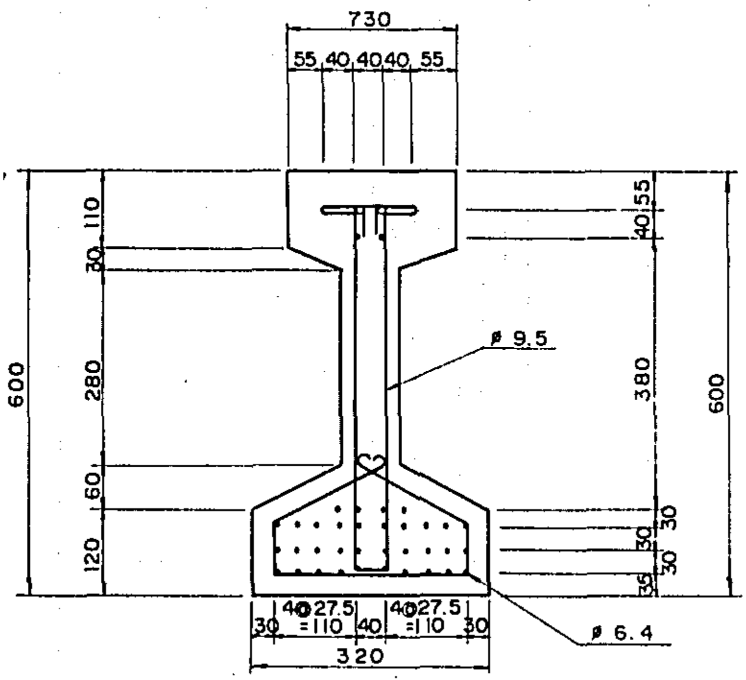



RAKING PILE

## VII-4 Prefabrication of P.S. Concrete Units

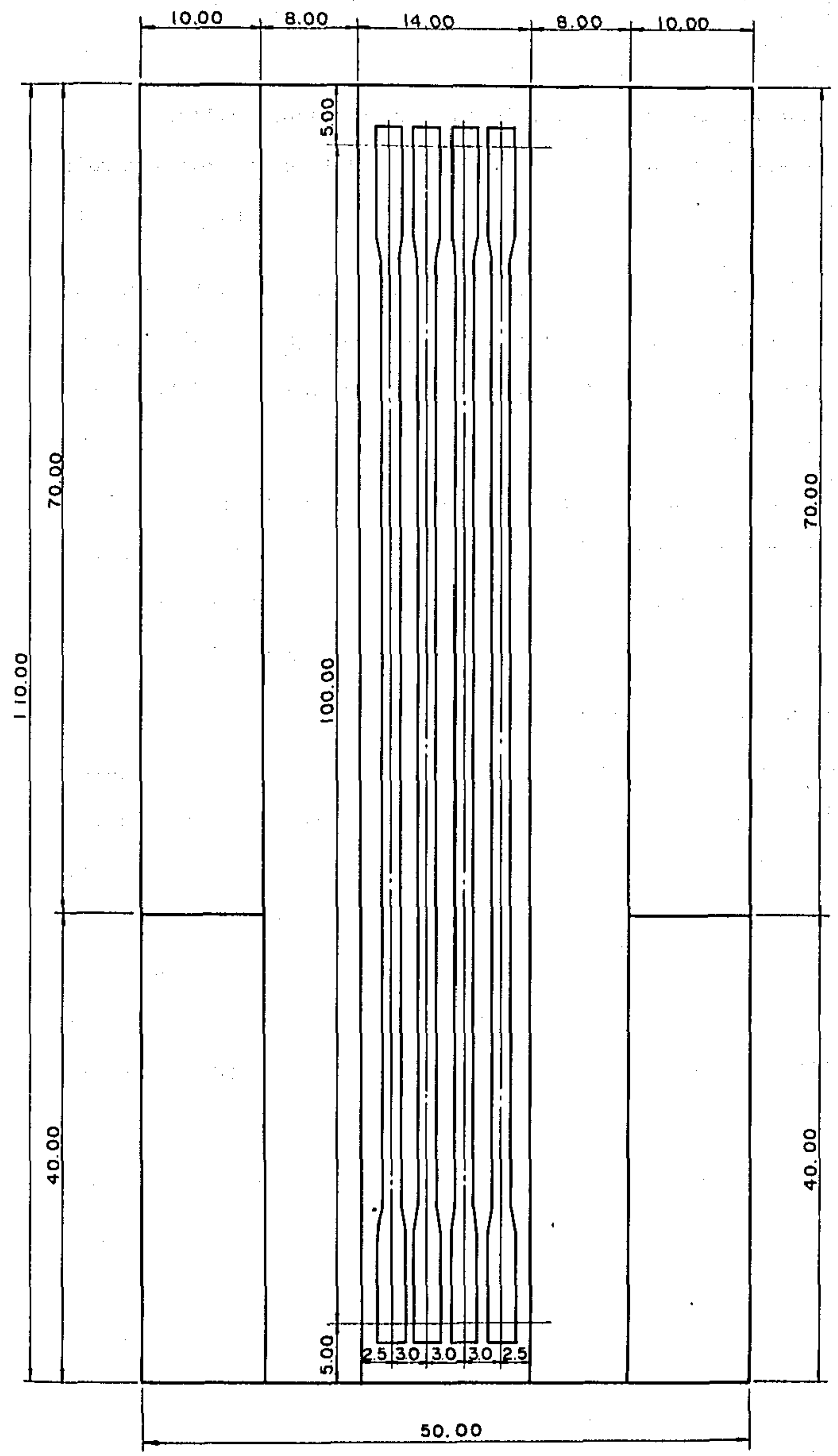
1) P.C. Beams

Pretensioned prestressed concrete beams will be used for the construction of the approach trestle and unloader pier. Precast concrete beams for the approach trestle will be of 15m in length, and beams for the pier will be of 11.0m in length. The section of the beams will be as follows.




DETAILS OF P.C BEAM
(MEMBER OF SLAB OF APPROACH TRESTLE AND UNLOADING PIER)

According to the design drawing, 252 precast concrete beams will be required for the approach trestle, and 507 beams will be required for the unloader pier. The total 759 precast concrete beams will be prefabricated in the concrete plant approximately 7 km from the site, and conveyed by pontoons over the sea to the site. The beams transported to site will be placed on the pier, and will be prestressed after concrete has been cast between the beams.




P.G BEAM. PREFABLICATING YARD

From the entire construction schedule, the construction period allowable for the prefabrication of the P.C. concrete beams will be 8.5 months.

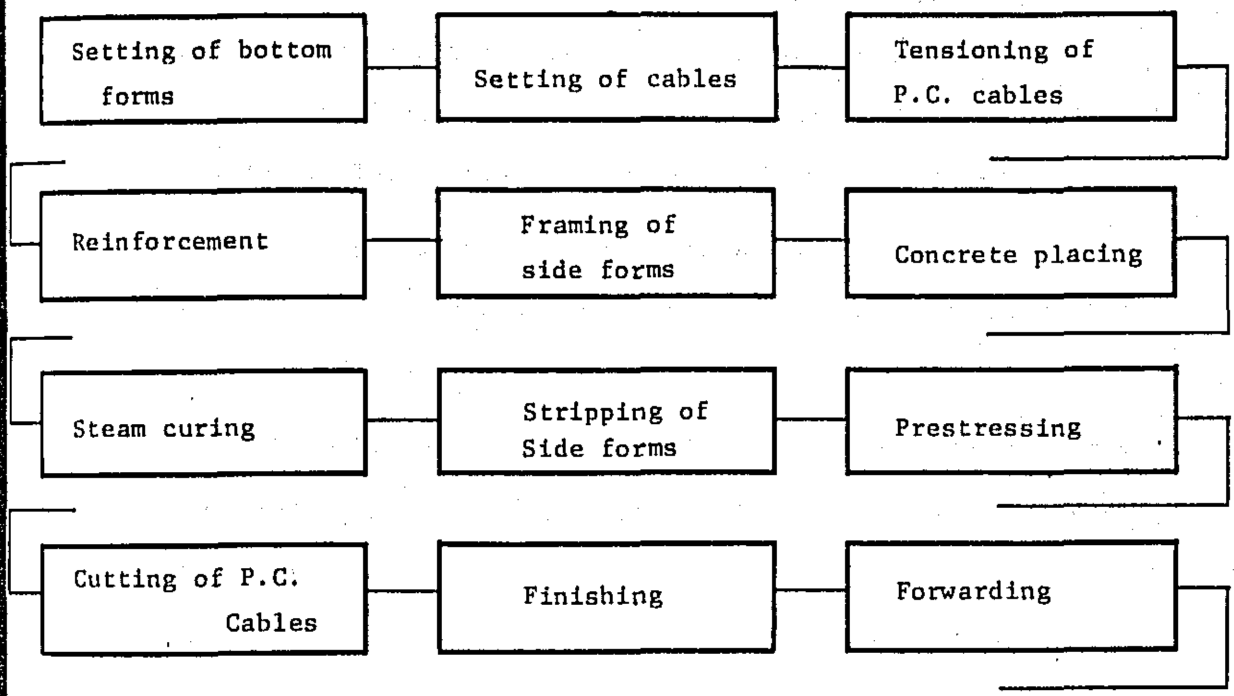
Prefabrication of P.C. beams as members of the slab will be commenced in January 1977, when the concrete mixing plant to be established in the construction base starts working, and the prestressing at the site must be completed on Sept. 15, 1977.

The demand for precast prestressed concrete beams on the design drawings is a total of 759 beams. Allowing a loss rate of 2% for damage during transport and errors in prefabrication, 775 beams will be prepared.

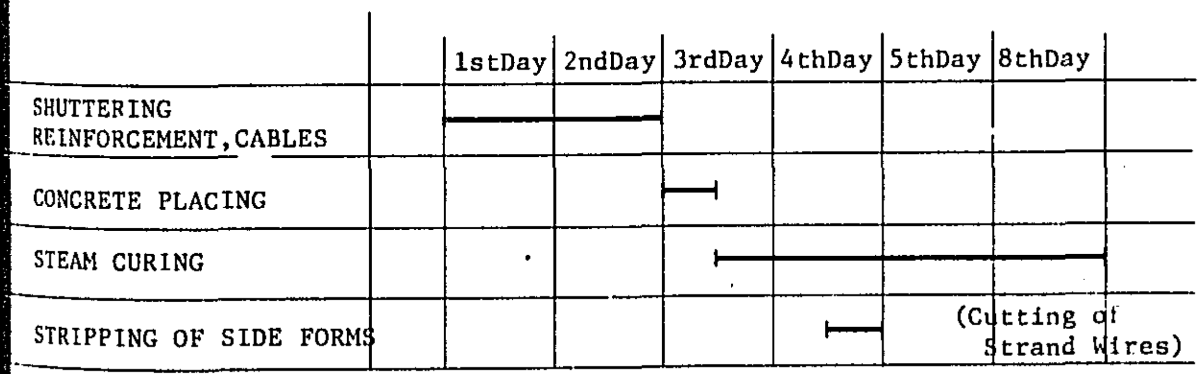
During the construction period, it will be assumed that on an average of 6 days per month, it will be difficult to carry on prefabrication of P.C. beams due to unfavorable weather conditions, cut-off of electric and water supply, mechanical trouble, holidays, and other causes. In order to complete the construction works in the specified period, it will be necessary to prefabricate 5 beams per day on the average in the plant. Therefore, the concrete plant will be equipped with facilities to prefabricate 5 P.C. beams per day.

The prefabrication of P.C. beams will be carried out in the following sequence.

Sequence of Prefabrication




A typical progress schedule for the prefabrication of P.C. beams according to the above sequence will be as follows.



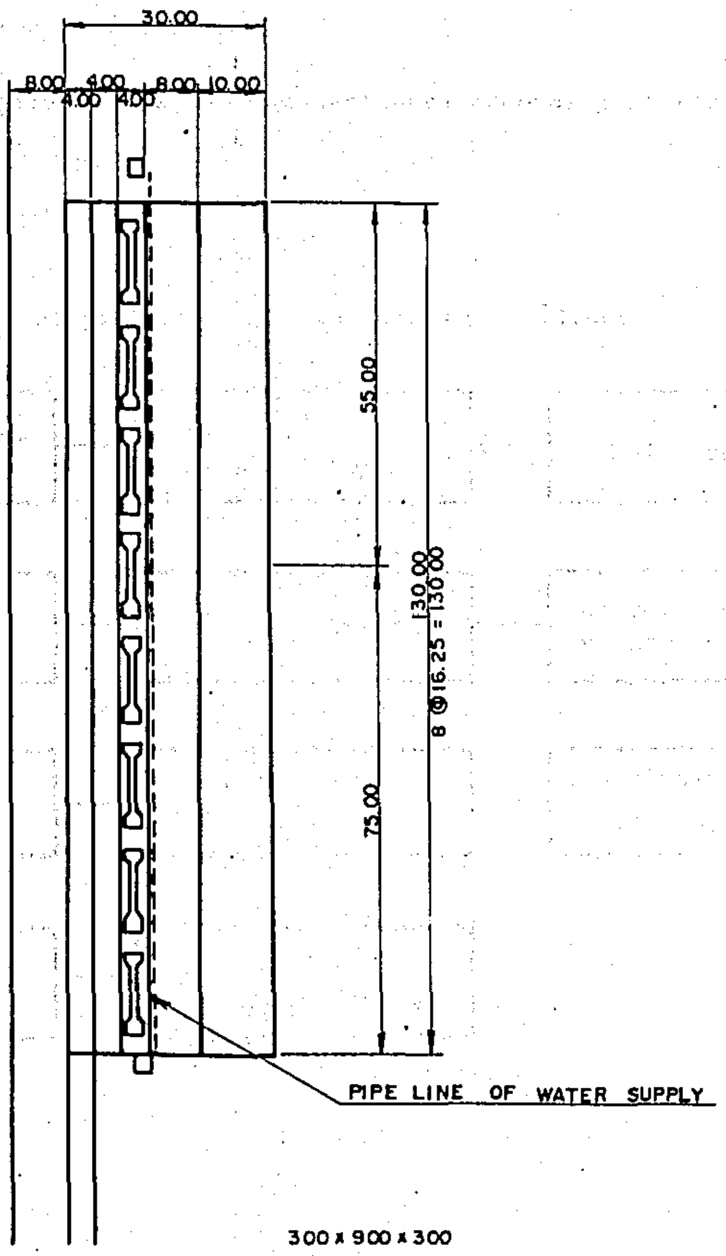
2) P.C. Crane Girder

Prestressed concrete beams will be used for the crane girder to be installed between the pier studs of the unloader pier. The beams will be of 12m in length, and approximately 70ton/beam in weight. The section of the beam will be as follows.

The demand for the precast concrete beams will be 28 beams according to design drawings. The beams will be prefabricated in the specified yard within the construction base.

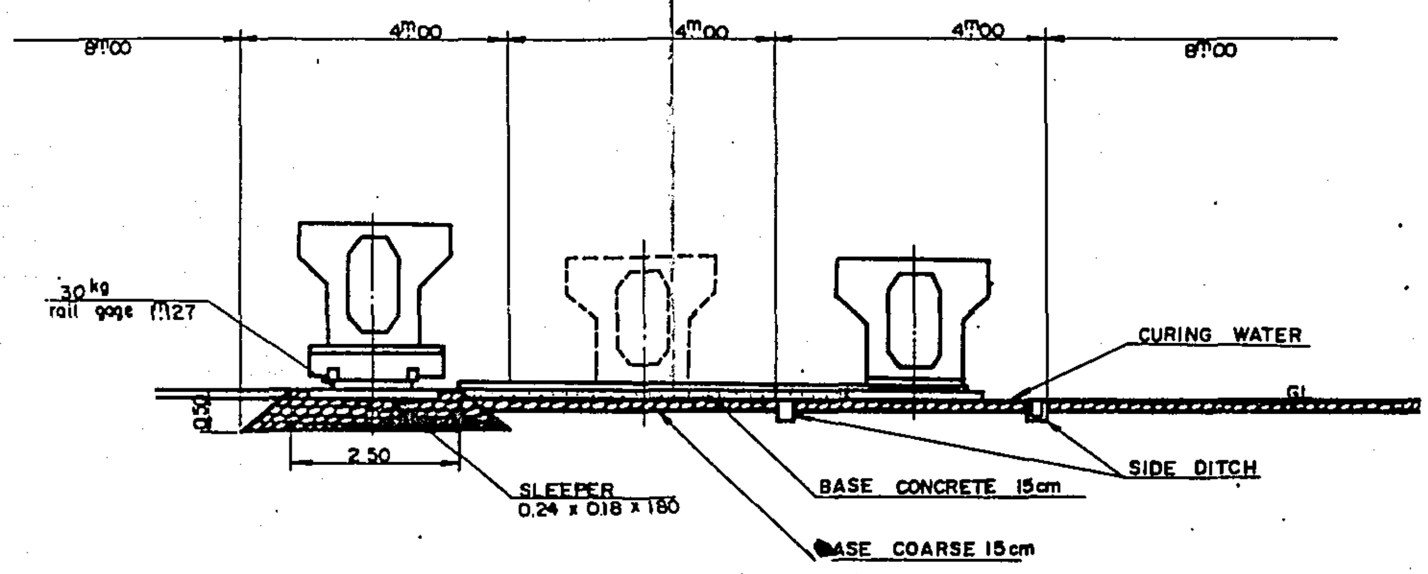
The beams will be conveyed on rails to the temporary jetty and transported by a floating crane to the construction site to be installed.

As the beams are of heavy weight of approximately 70 tons per beam, the prefabrication yard will be located near the temporary jetty for loading. The arrangement within the yard will be as follows.




P.C. GIRDER PREFABRICATION YARD

S = 1/100

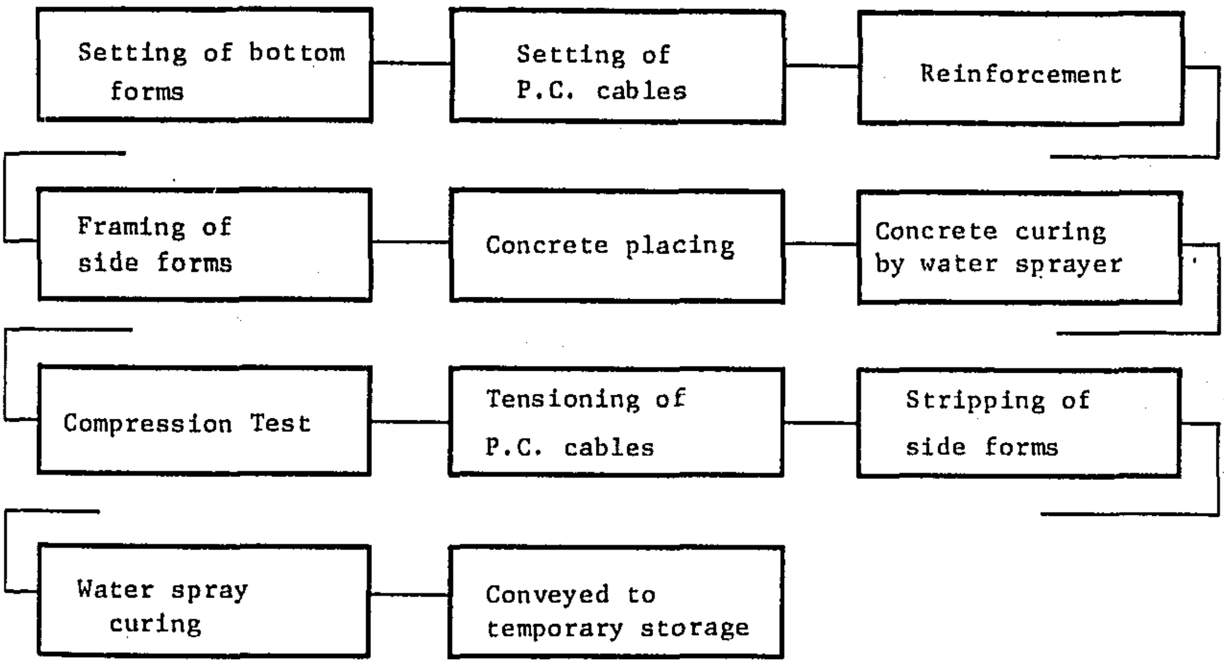



PREFABRICATION : P.C GIRDER

As 7 days will be required on the average for the prefabrication of one beam, in order to complete the construction works within the specified period, facilities will be prepared for the prefabrication of 8 beams.

P.C. crane girders will be prefabricated according to the following process.

Process of prefabrication of P.C. crane girders



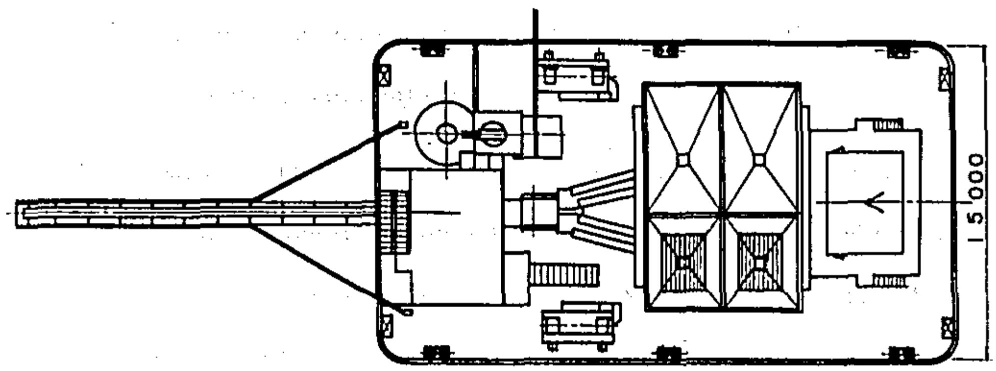
## VII-5 Coping Concrete

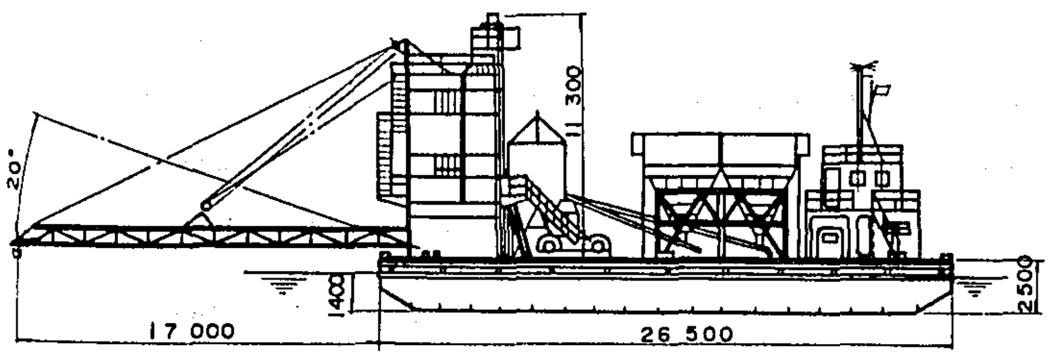
1) Concrete

The volume of concrete required within the site will be approximately 13,000$m^3$.

Construction works will not be completed in time if the concrete is to be mixed in the concrete plant on land and conveyed to site, and a large range in strength will be found in quality tests of concrete. Therefore, the concrete will be mixed in a floating mixing plant and placed in site.

Floating Mixing Plant




CONCRETE MIXING PLANT

· MIXING CAPACITY 25 $m^3$/ HOUR

Floating mixing plant

For the unloader pier requiring approximately 590m$^3$ of concrete, concrete placing will be carried out in two steps. The placing of the coping concrete for the pier stud on the west end of the approach trestle will be carried out continuously without interruption.

During the summer, the temperature exceeds 40°C in the daytime, with frequent unfavorable days for concrete placing. For the main structures, concrete placing will be carried out during the night (7:00 P.M. ∿ 8:00 A.M.). As one round of concrete placing will be approximately 13 hours, the minimum concrete mixing capacity of the floating concrete mixer in demand will be 21m$^3$/H per hour. As the volume required for the approach trestle is 27m$^3$, it will be possible to complete the concrete placing in one night.

Curing

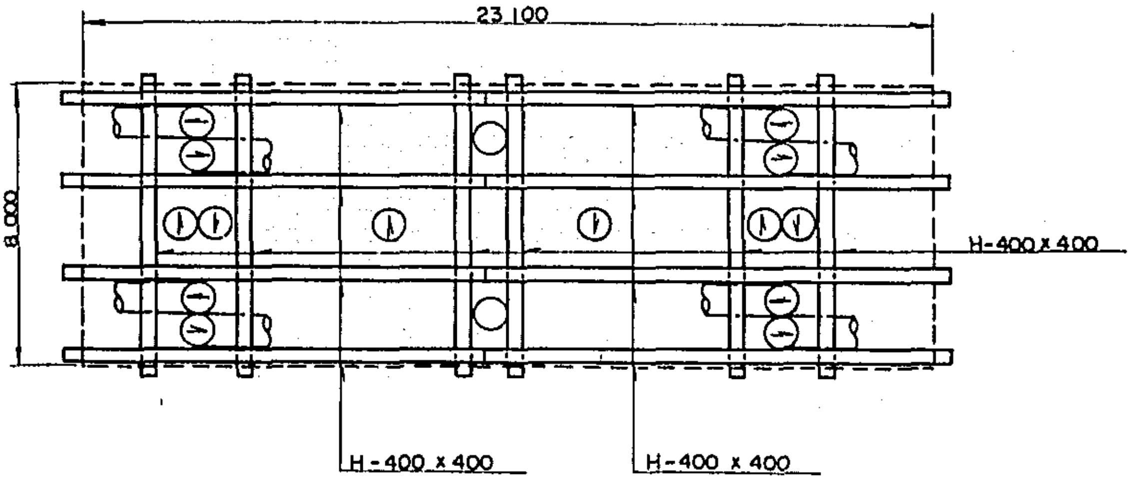
Sea water will be sprayed for the curing of the slab concrete placed-in-situ for the superstructure of the pier studs.
A small pump and sprayer will be provided.

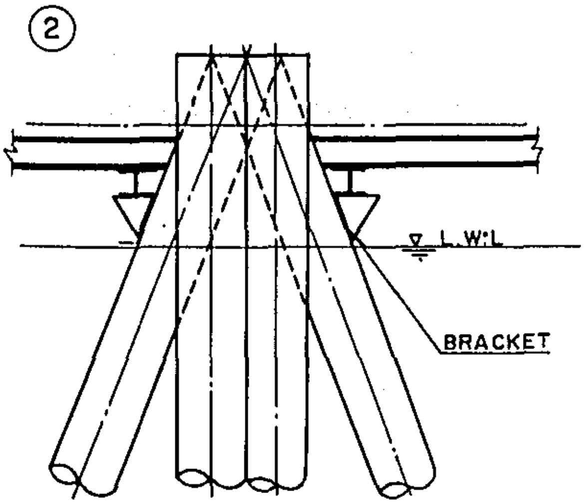
Stripping of shuttering

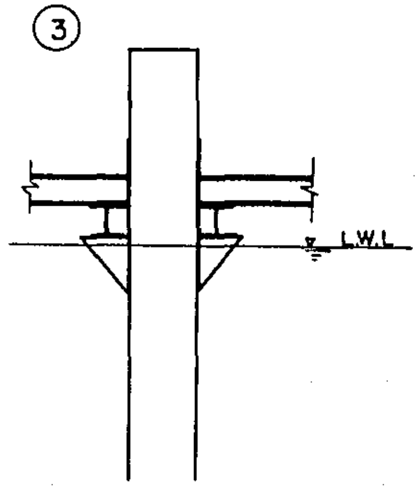
The sides of the shuttering will be stripped after 2 ∿ 3 days from the concrete placing. The bottom and supports will be stripped after it has been confirmed by compression tests that the concrete has acquired the specified compressive strength.

2) Supports

Supports will be composed of H-shaped steel beams with brackets attached to the steel pipe piles by welding as shown in the following diagram.

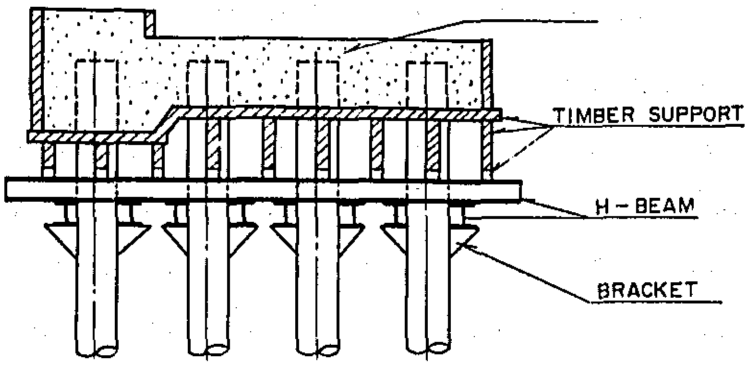


3) Shuttering

Binding members for the shuttering will be fixed by bolts or steel bars. Separating material will be applied to the shuttering boards so that the binding members may be completely stripped from the concrete surface.




4) Reinforcement

The total volume of reinforcement in demand at the site will be approximately 630 tons. After processing as specified in the design drawings in the shop on land, the reinforcement will be conveyed to site by pontoons.

In the shop, electric bar benders and bar cutters will be provided for the processing of reinforcement.

## VII-6 Miscellaneous Works

### 1) Utility Ducts

Precast concrete will be used for ducts for water and oil supply and cables.

The duct will be of approximately 12m ∿ 18m in length and approximately 38t ∿ 73t in weight per duct. The section of the duct is given in detail in the design drawing.

With thin sections in part, it will be advisable to reduce the number of times that the ducts will be moved as far as possible. Therefore, the ducts will be cast on the main berth. When the P.C. slab between the pier studs of the unloader pier or the P.C. slab of the approach trestle has been completed, prefabrication tables will be installed on the floor system in number to meet the requirement of the construction schedule. The tables will be shifted with the completion of the P.C. slab so that the ducts may be cast in places as near the place of installation as possible.

### 2) Cathodic Protection of Steel Pipe Piles

Cathodic protection by galvanic anodes will be applied.

Metals such as zinc, aluminum, and magnesium with lower electric potential than the body to be protected, or compounds of positive metals will be attached to the piles to be protected by electric wires.

Welding will be carried out in water by divers. The effect of protection will be confirmed by galvanometry.

3) Rubber Fenders

Nine sets of rubber fenders of C, 1700H, will be lifted by the wheel crane on the pier or a floating crane, and installed on the bolts embedded in the concrete in advance.

4) Capstans and Bollards

Nine sets of 100 ton bollards, and 8 sets of 70 ton bollards will be installed in the specified locations by a crane on the pier or a floating crane.

## VII-7 Related Facilities

### 7-1 Land Reclamation Works

1) Dredging

Dredging of the area to be reclaimed will be carried out under a separate contract for naviation channel dredging. The area will be dredged to a depth of -5m below chart datum, and the dredging will be completed by the end of October, 1975.

2) Sand Filling

Material for sand filling will be collected from the borrow area at the daily rate of 920m$^3$, and will be transported to the working yard through the temporary construction road with an effective width of 6m and crushed stone pavement of approximately 2 km in length.

At the working yard, material will be dumped directly into a barge of 500 tons from the elevated lumpway on the E-shaped temporary pier. The barges will be towed to the site by tugboats of 500HP. The material will be picked up and dumped into the area to be reclaimed by a tractor shovel with a capacity of 1.4m$^3$ or a clamshell of 1.2m$^3$.

The ground reclaimed to El. +0.5m above the chart datum will graded and levelled by a special bulldozer of the 16 ton class which will be capable of maneuvering on soft and wet filling material.

3) Riprapping

Material for riprapping will be obtained from an approved quarry and will be handled and transported to the site by the same method as the case of sand filling material. The material will be picked up and dumped in the area by a clamshell of 1.2m which will be mounted on a pontoon of 300 tons.

The mounting and diking of ripraps will be carried out in two steps. The first step up to El. +0.5m will be carried out in parallel to works for sand filling. The second step above El. +0.5m will be carried out after the sand filling has been completed.

The surface of riprapping submerged in water will be smoothed out by divers and the surface exposed out of water will be smoothed by ordinary laborers.

4) Armour Stones

Armour stones will be transported, handled and pitched in the same manner as the case of riprapping.

Placing and pitching of armour stones will be carried out in areas where riprapping and sand filling have been completed. The surface of the stones above El. +0.5m from the chart datum will be smoothed out by pitching.

2. Small Craft Trestle

1) Piling

Piles to be supplied by P.Q.A. will be brought into the stock yard and will be prepared for piling in specified lengths.

Piling works will be carried out under similar sequence and methods of working as piling works for the main berth structure and approach trestle.

2) Pile Jackets

Pile jackets will be cast in-situ with metal forms.

3) Pile Caps and Beams

Staging and scaffolding work of steel members will be installed to support forms for the preparation of pile caps and beams of cast in-situ concrete works.

The general sequence of concrete works will be similar to works for the main berth structure and approach trestle.

4) Precast Concrete Slab

Precast concrete slabs will be prefabricated in the prefabrication yard.

Slabs will be handled and transported to the temporary pier by a 40 ton truck crane and a heavy truck of 15 tons.

Prefabricated slabs of a maximum weight of 15 tons will be loaded on 100 ton barges and towed to site by tugboats of 250HP.

A 40 ton crawler crane mounted on a 300 ton barge will be provided at the site for the placing and pitching of the slabs.

# WORKING SCHEDULE

PROJECT

Year: 1976 (Jan–Dec), 1977 (Jan–Dec), 1978 (Jan–Jun)

| Item No. | Description | | Q'ty | |
|---|---|---|---|---|
| | PREPARATION | | | |
| | CONCRETE PLANT | | | ERECTION — TRIAL MIXING |
| | YARD | | | POWER & WATER SUPPLY |
| | | | | GRADING WORKS — CONCRETE PAVEMENT |
| | TEMPORARY JETTY | | | PILING AND CAUSEWAY — DREDGING |
| | SURVEYING PLATFORM | | | SURVEY — PILING |
| | NAVITATION TRANSPORT OF WORKING CRAFT AND EQUIPMENT | | | |
| | DREDGING AND LAND RECLAMATION | | | DREDGING LAND RECLAMATION & BERTH AREAS — LAND RECLAMATION AND SLOPE PROTECTION |
| | APPROACH TRESTLE | PILING | (60 PILES) | |
| | | COPING CONCRETE | (8 PIER STUDS) | |
| | | P C BEAM | (252 BEAMS) | PREFABRICATION P.C. BEAM |
| | | UTILITY DUCT & COVER | (16 DUCTS) | PREFABRICATION |
| | UNLOADER PIER | PILING & DRILLING | (266 PILES & 50 DRILLINGS) | DRIVING & DRILLING |
| | | COPING CONCRETE | (16 PIER STUDS) | |
| | | PC BEAM | (507 BEAMS) | PREFABRICATION & INSTALLATION |
| | | UNLOADER GIRDER | (28 GIRDERS) | PREFABRICATION & INSTALLATION |
| | | UTILITY DUCT & COVER | (29 DUCTS) | PREFABRICATION & INSTALLATION |
| | MOORING DOLPHINE | PILING & DRILLING | (30 PILES & 30 DRILLINGS) | |
| | | COPING CONCRETE | (2 PIER STUDS) | |
| | MAINTENANCE PLATFORM | PILING | (20 PILES) | |
| | | COPING CONCRETE | (2 DECKS) | |
| | SMALL CRAFT TRESTLE | PILING | (72 PILES) | |
| | | CONCRETE SLAB | | |
| | CATHODIC PROTECTION OF STEEL PIPE PILE | | | |
| | RUBBER FENDER, BITT, GUARD RAIL & OTHERS | | | |
| | CONCRETE PAVEMENT | | | |
| | POWER SUPPLY & TELEPHONE SYSTEM | | | SHOP WORK — SHIPPING |
| | BUNKERING & OIL FACILITIES | | ON THE TERMINAL<br>ON THE BERTH | |
| | WATER SUPPLY | PIPING & MECHANICAL | WORK ON THE TERMINAL AREA | |
| | | PIPING & MECHANICAL | WORK ON THE BERTH | |
| | BUILDING | TERMINAL OFFICE<br>SUB-STATION HOUSE<br>PUMP HOUSE<br>GATE & GATE HOUSE | | PILING |
| | ROAD & FENCE | | | ROAD & FENCE |
| | REMARKS | | | May Jun Jul Aug Sep MONSOON SEASON |

# APPENDIX

## Structural Analysis and Study of Caisson Type Unloader Pier

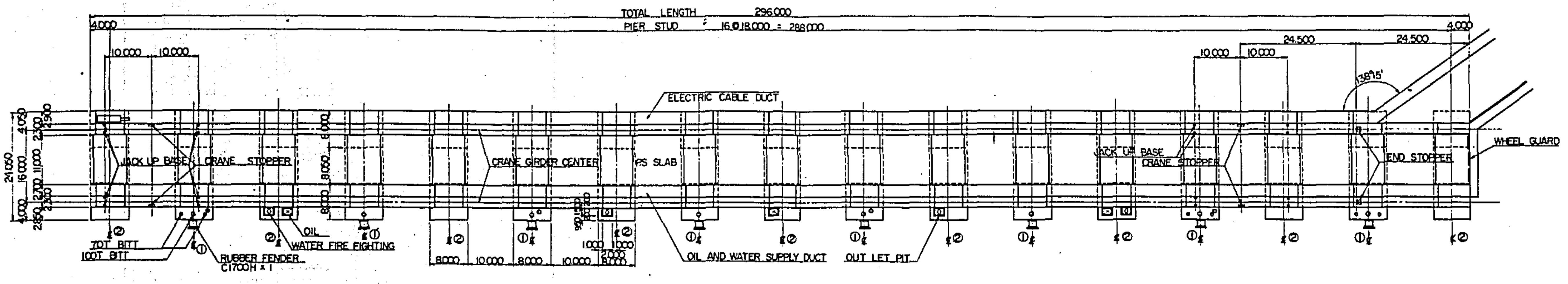
## 1. SUMMARY

This report relates an alternative plan of an Unloader Pier of a caisson type structure, the design of which are shown in the following drawings. It has been made as a comparative study to the steel pipe pile type structures.

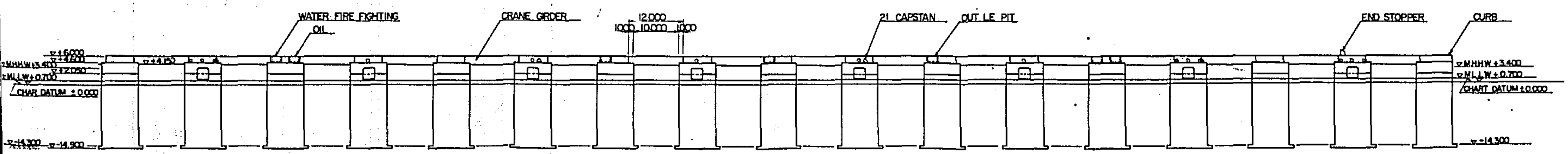
The result of findings has proved that the caisson type structure is not practical for this Project on the following grounds.

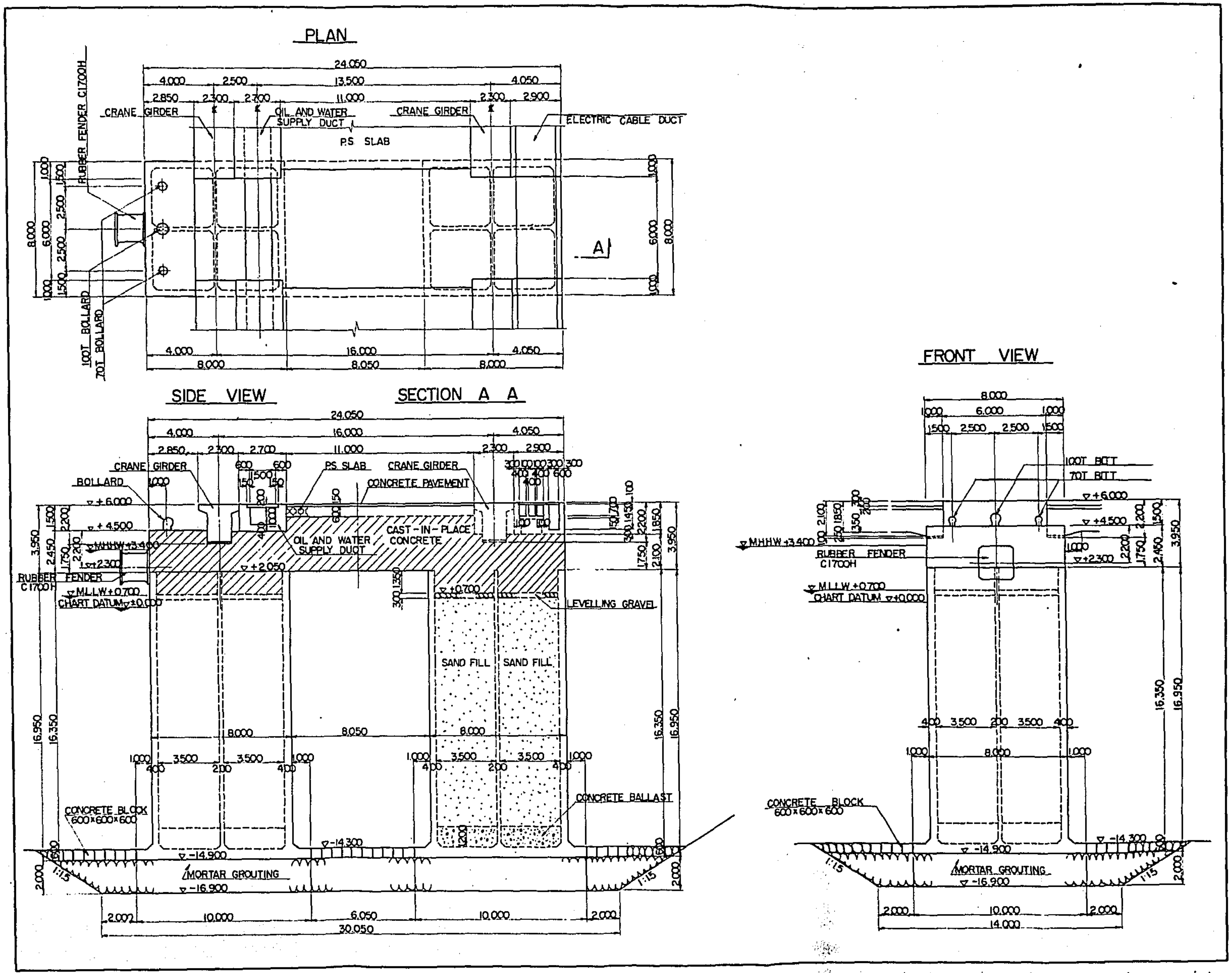
1. The overall construction schedule for the particular type of caisson exceeds the proposed construction period as embodied in the plan of this Project.
2. Theoretical structural calculations may be sufficient as shown in this report. However, a slight deviation in the subbase will cause unstability of the tall structure.
3. The construction period shall be prolonged due to difficulty of construction such as concrete placing at sea and installation of the structure considering the 4 knot current speed.
4. Since the height of the structure exceeds the standard height, a floating dock cannot be employed for construction.
5. Blocking of the flows of current will be undesirable.

## GENERAL PLAN scale 1:300




## ELEVATION scale 1:300




PLAN
SIDE VIEW
SECTION A A
FRONT VIEW
CRANE GIRDER
OIL AND WATER SUPPLY DUCT
P.S SLAB
ELECTRIC CABLE DUCT
RUBBER FENDER C1700H
100T BOLLARD
70T BOLLARD
BOLLARD
CONCRETE PAVEMENT
CAST-IN-PLACE CONCRETE
MHHW +3.400
M.L.L.W +0.700
CHART DATUM ±0.000
LEVELLING GRAVEL
SAND FILL
CONCRETE BALLAST
CONCRETE BLOCK 600×600×600
MORTAR GROUTING
100T BITT
70T BITT
24.050
30.050
14.000

## 2. CONSTRUCTION METHOD AND CONSTRUCTION SCHEDULE

### 2-1 By Construction of Caisson Yard

(1) Prefabrication of caisson on CAISSON YARD up to 9m in height.

(2) Towing the prefabricated caisson to the temporary site in sea.

(3) Completing the caisson up to the designed height by cast-in-place concrete.

(4) Installation of caisson at designated site.

### 2-2 By Employment of Floating Dock

(1) Prefabrication of caisson on FLOATING DOCK up to 9m in height.

(2) Towing the prefabricated caisson to the temporary site in sea.

(3) Completing the caisson up to the designed height by cast-in-place concrete.

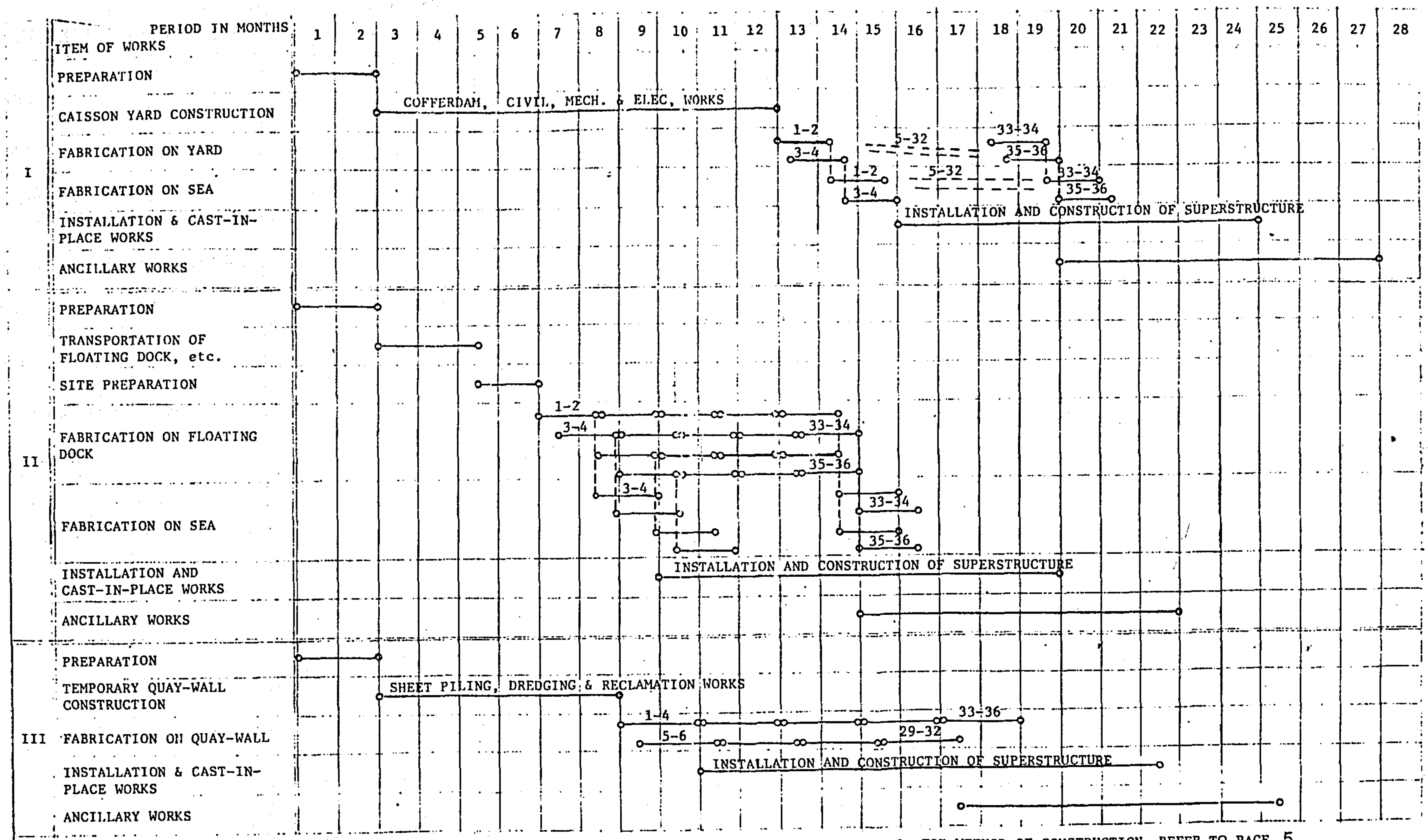
(4) Installation of caisson at designated site.

### 2-3 By Construction of Temporary Quay Wall and Utilization of Floating Crane

(1) Inland prefabrication of caisson on TEMPORARY QUAY WALL up to the designed height.

(2) Lifting by FLOATING CRANE and towing the completed caisson to the designated site for installation.

CONSTRUCTION SCHEDULE FOR CAISSON TYPE ORE/COAL BERTH (FOR ALTERNATIVE STUDY)




I: SLIPWAY AND 600t-FLOATING CRANE
II: 4-2,000t FLOATING DOCK AND 600t-FLOATING CRANE
III: TEMPORARY QUAY WALL AND 1,000t FLOAING CRANE

NOTE: 1. FOR METHOD OF CONSTRUCTION, REFER TO PAGE 5
2. THE NUMBERS SPECIFIED ABOVE INDICATE THE NO. OF CAISSONS TO BE FABRICATED.

| Item No. | Pay Item | Unit | Quantity (Approx.) | Schedule Rate in PAK. RS | | Item Price in PAK. RS | |
|---|---|---|---|---|---|---|---|
| | | | | RS. | Paisa | RS. | Paisa |
| | COST BY CONSTRUCTION METHOD 2-1 | | | | | 1,000 Rupies | |
| | SUMMARY | | | | | | |
| 1 | Preparation Work | Sum | 1 | | | 7,600 | |
| 2 | Transportation & Hires of Floating Crane | Sum | 1 | | | 29,000 | |
| 3 | Temporary Work (Caisson Yard,etc.) | Sum | 1 | | | 23,870 | |
| 44 | Caisson Construction | Pcs | 34 | 1,590.5 | | 54.077 | |
| 5 | Foundation Works | Sum | 1 | | | 19,356 | |
| 6 | Installation of Caissons | Sum | 1 | | | 7,600 | |
| 7 | Superstructure | Sum | 1 | | | 31,900 | |
| | Total : | | | | | 173,403 | |

| Item No. | Pay Item | Unit | Quantity (Approx.) | Schedule Rate in PAK. RS | | Item Price in PAK. RS | |
|---|---|---|---|---|---|---|---|
| | | | | RS. | Paisa | RS. | Paisa |
| | <u>COST BY CONSTRUCTION METHOD 2-2</u> | | | | | 1,000 Rupies | |
| | <u>SUMMARY</u> | | | | | | |
| 1 | Preparation Works | Sum | 1 | | | 7,600 | |
| 2 | Transportation and Hires of Floating Dock | Sum | 1 | | | 42,200 | |
| 3 | Temporary Works | Sum | 1 | | | 485 | |
| 4 | Caisson Construction | Pcs | 34 | 1,410.5 | | 47,957 | |
| 5 | Foundation Works | Sum | 1 | | | 19,356 | |
| 6 | Installation of Caissons | Sum | 1 | | | 7,600 | |
| 7 | Superstructure | Sum | 1 | | | 31,900 | |
| | Total : | | | | | 157,098 | |

| Item No. | Pay Item | Unit | Quantity (Approx.) | Schedule Rate in PAK. RS | | Item Price in PAK. RS | |
|---|---|---|---|---|---|---|---|
| | | | | RS. | Paisa | RS. | Paisa |
| | <u>COST BY CONSTRUCTION METHOD 2-3</u> | | | | | 1,000 Rupies | |
| | <u>SUMMARY</u> | | | | | | |
| 1 | Preparation Works | Sum | 1 | | | 7,6000 | |
| 2 | Transporation & Hires of Floating Crane | Sum | 1 | | | 48,000 | |
| 3 | Temporary Quay Wall Construction | Sum | 1 | | | 16,550 | |
| 4 | Caisson Construction | Pcs | 34 | 1,682.8 | | 57,215.2 | |
| 5 | Foundation Works | Sum | 1 | | | 19,356 | |
| 6 | Installation of Caissons | Sum | 1 | | | 7,600 | |
| 7 | Superstructure | Sum | 1 | | | 31,900 | |
| | Total : | | | | | 188,221.2 | |