CHAPTER 4 DESIGN OF CIVIL WORK

CHAPTER 4 DESIGN OF CIVIL WORK

This chapter deals with the design of marine civil works and onland civil works which include the following facilities

1 Quaywall

1.1 Container Berth (340 m long)

1.2 Multi-purpose Berth (220 m long)

1.3 Passenger Berth (240 m long)

2 Revetment

2.1 Revetment of the Resolution Area

(East Revetment 250 m long, West Revetment 340m long)

2.2 Temporary Revetment of the Onshore Dumping Area

(A-North 515 m long, A-West 440 m long, B 530 m long)

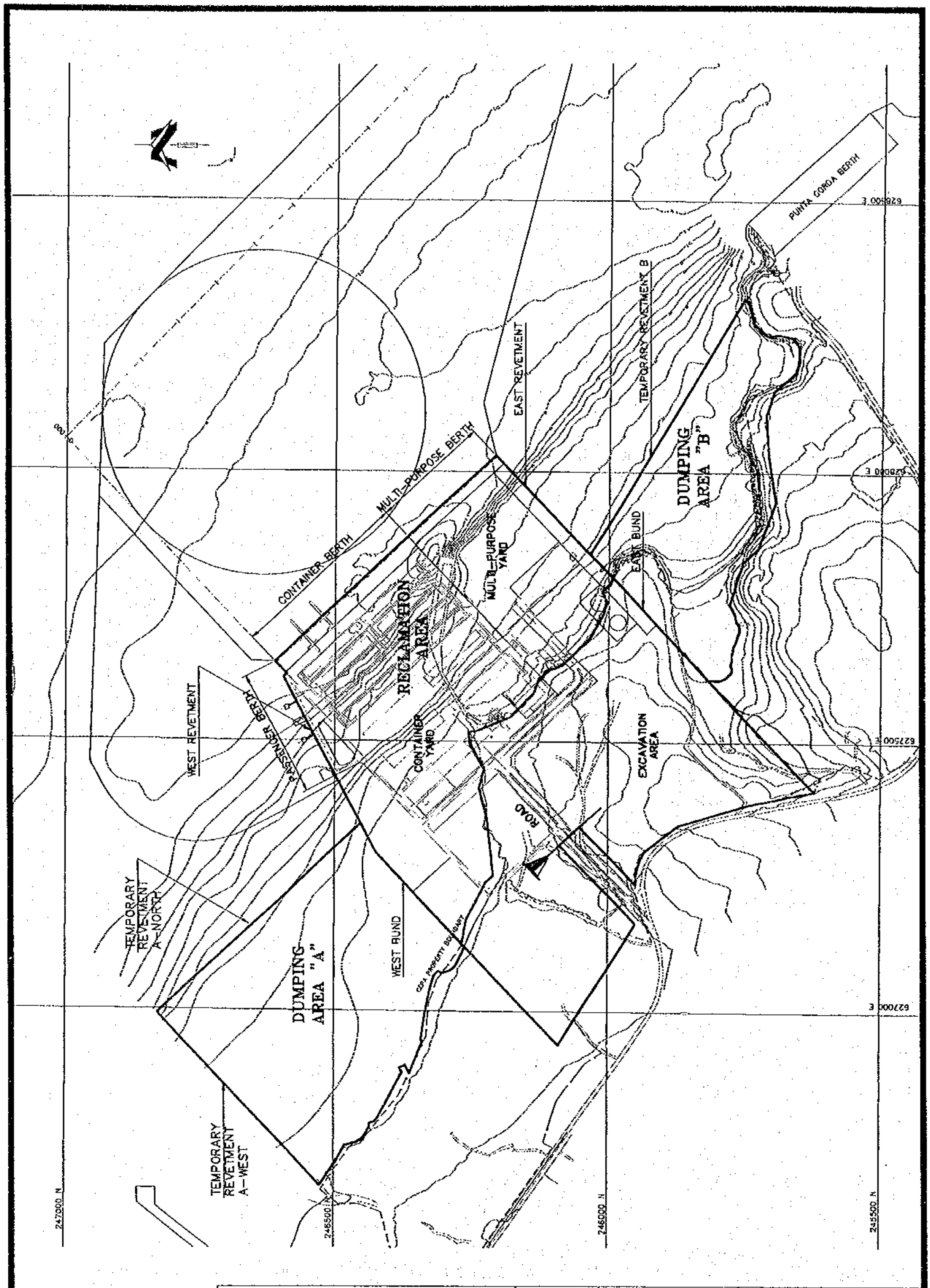
2.3 Temporary Bund between the Reclamation Area and the Temporary Onshore Dumping Area (West 260 m long, East 50 m long)

3 Earthworks and Reclamation

4 Road and Pavement

5 Storm Drainage

Figure 4.4.1 shows the locations of major Onshore Civil Works.



Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.1.1	LOCATIONS OF MAJOR CIVIL WORKS

4.1 Design Criteria

4.1.1 Codes and Standards

There are no available any Salvadorean design standards, codes and/or manuals for the design of ports and harbor facilities. In the past the Design Manual of the U.S. Army was used for designing the Acajutla, Cutuco and Punta Gorda Ports. On the other hand, El Salvador has developed local codes and manuals for building works, which are available and legally applied.

The following design standards, code and manual were basically adopted to design the port facilities such as berth, channel, revetment, dredging and reclamation for the Project.

1) Technical Standards for Port and Harbor Facilities in Japan, edition 1999, issued by the Ports and Harbors Bureau, Ministry of Transport of Japan.

2) Shore Protection Manual: U.S. Army Coastal Engineering Research Center, 1984.

3) British Standard Code of Practice for Maritime Structures, BS6349.

4) Regulation for Structural Safety of Constructions, 1997, by ASIA " Asociación Salvadoreña de Ingenieros y Arquitectos", edition 1997, issued by the Ministry of Public Works of El Salvador.

4.1.2 Design Criteria

(1) Natural Conditions

1) Meteorological and Oceanographic Conditions

The meteorological and oceanographic conditions used for the design are summarized in Table 4.1.1.

Table 4.1.1 Natural Meteorological and Oceanographic Conditions

Meteorological Conditions	Temperature	Highest Maximum	38.7℃
		Lowest Minimum	20.2℃
	Rainfall	Annual Precipitation (1997)	1,272 mm 2,123 mm
		10-year Probable Rainfall Intensity	70 mm/hr
	Wind	Stormy Condition Maximum Wind Velocity	31 m/sec
Oceanographic Conditions	Tide	H.W.L	+3.37 m
		L.W.L	-0.13 m
	Current	Maximum Velocity	1.78 m/s
		Direction	SE
	Wave	Operational Condition (H1/3)	1.0 m
		(T1/3)	3.4 s
		Stormy Condition (H1/3)	2.2 m
		(T1/3)	4.5 s

2) Geological and Geotechnical Conditions

The geological profiles of main port structure s (berth and revetments) along face lines are shown in Figures 4.1.2 to 4.1.4.

a) Design parameters for port structures

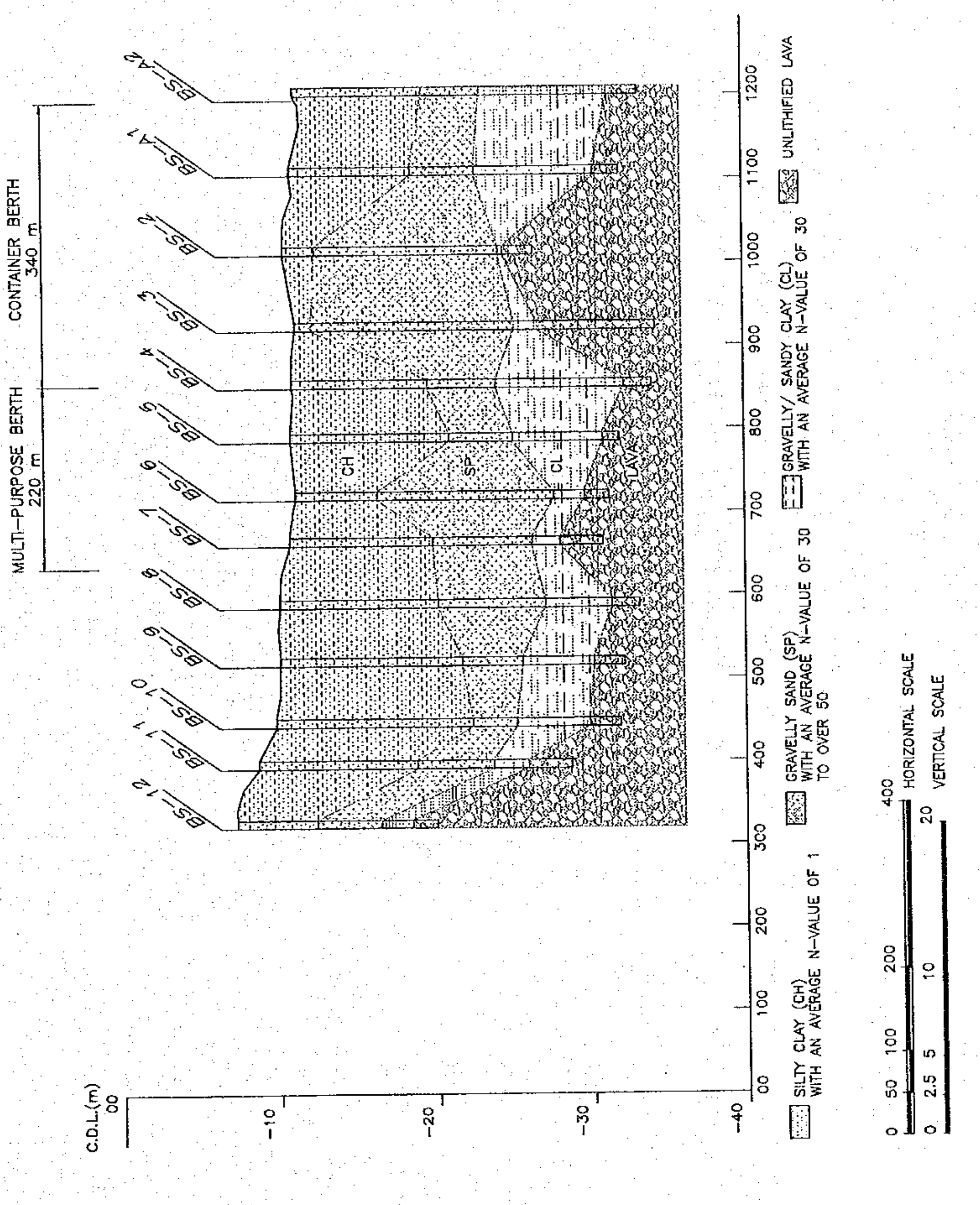
The subsoil at the berth front is composed of:

- Layer 1: Silty clay (CH), 5.0 to 10.0 m deep
- Layer 2: Gravelly sand (SG), 5.0 to 10.0 m deep
- Layer 3: Gravelly/sandy clay (CL), 2.0 to 5.0 m deep

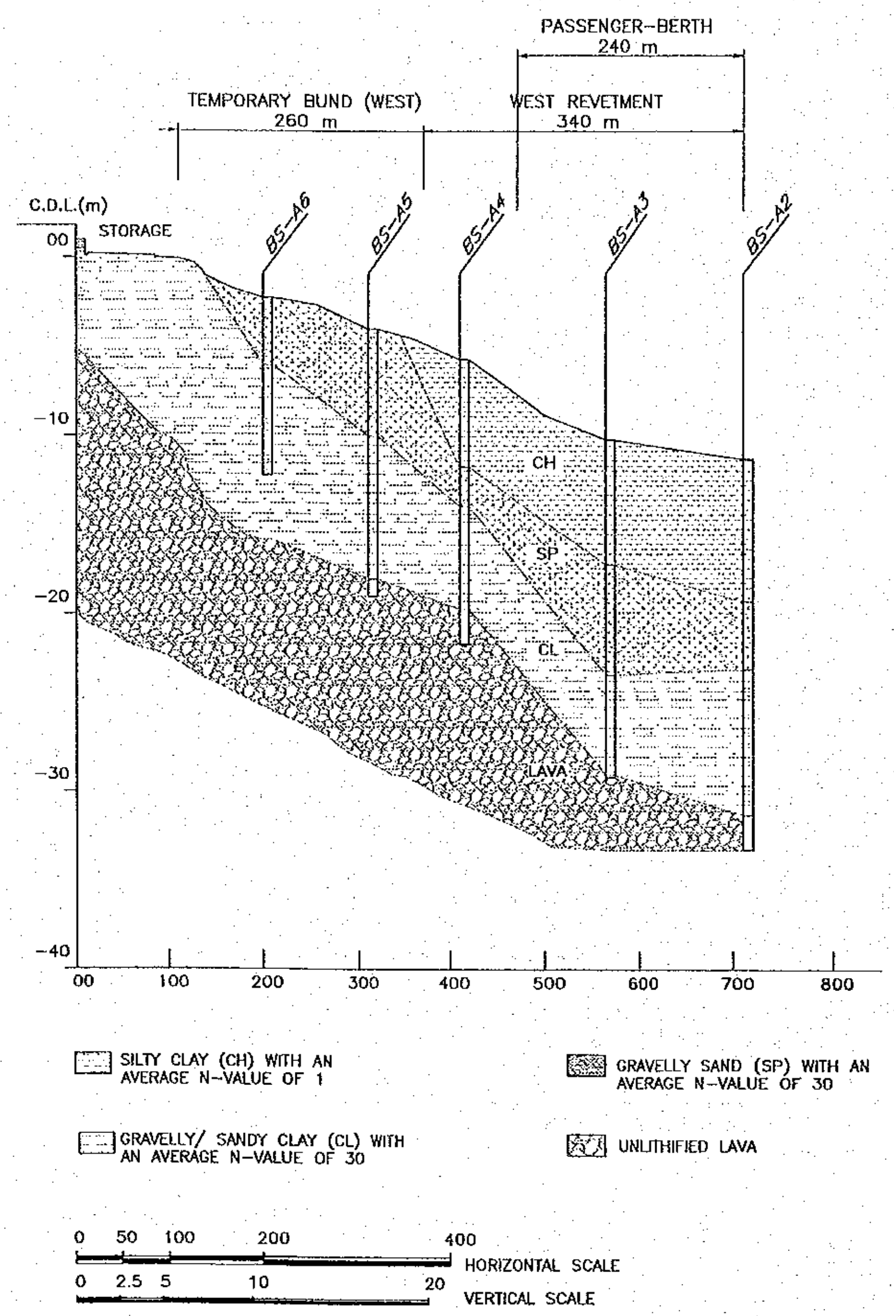
The design parameters for port structures are summarized in Table 4.1.2

Table 4.1.2 Proposed Design Parameters for Port Structures

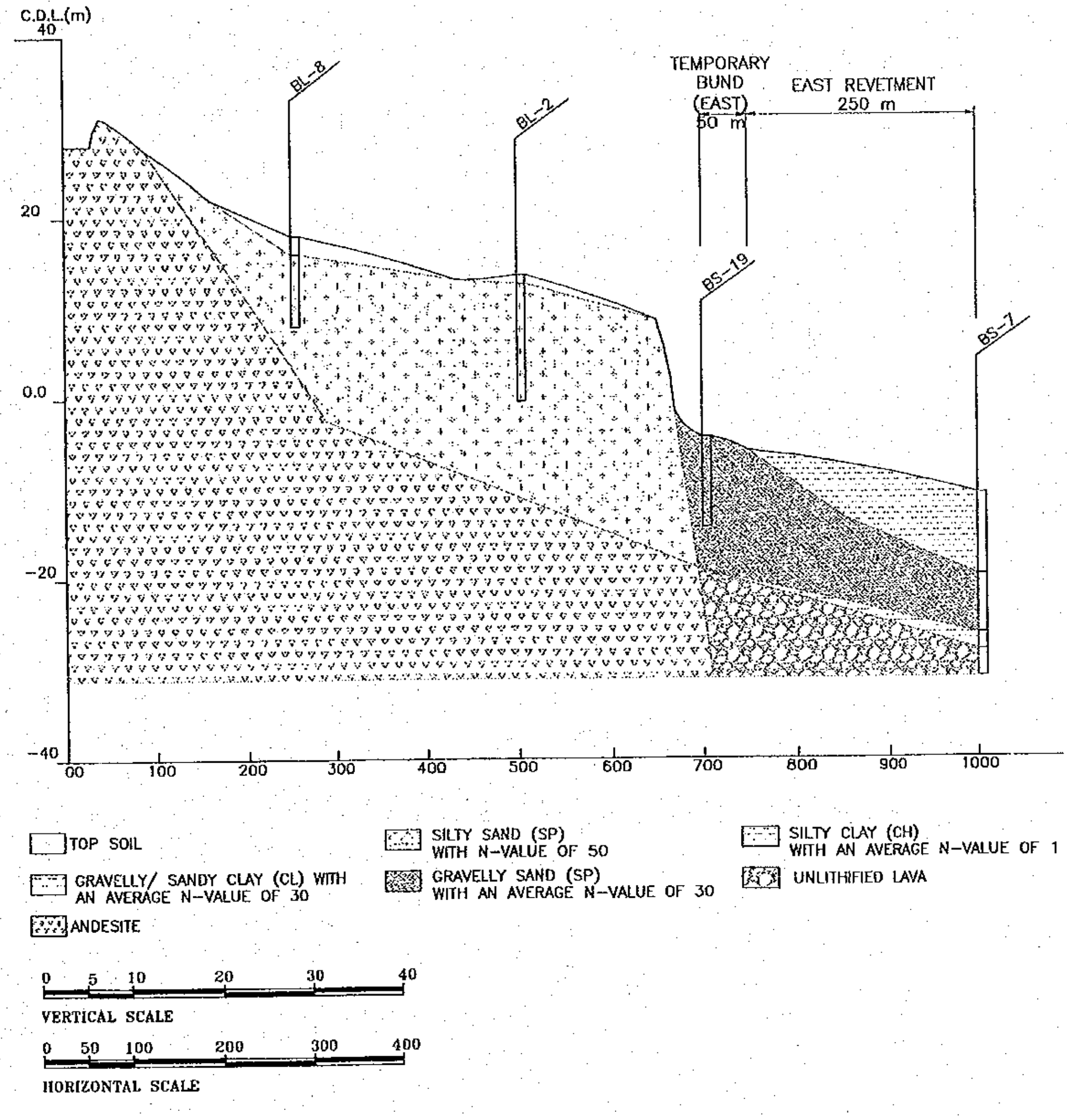
LAYER 1 : Silty Clay $\gamma' = 4$ kN/m^3 $\gamma = 14$ kN/m^3 $c_u = 5.0$ kN/m^2
LAYER 2 : Gravelly Sand $\gamma' = 10.0$ kN/m^3 $\gamma = 18.0$ kN/m^3 $\phi = 35$ degree
LAYER 3 : Sandy Clay $\gamma' = 10.0$ kN/m^3 $\gamma = 18.0$ kN/m^3 $\phi = 35$ degree



Japan International Cooperation Agency (JICA)	Figure	Title
Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.1.2	Geological Profile at Container and Multi-purpose Berths



Japan International Cooperation Agency (JICA)	Figure	Title
Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.1.3	Geological Profile at Passenger Berth & West Revetment



Japan International Cooperation Agency (JICA)	Figure	Title
Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.1.4	Geological Profile at East Revetment

b) Design parameters for reclamation and soil improvement

The silty clay layer in the reclamation has high water content, high plasticity and high compression, and classified as soft clay.

The mechanical properties of the layer are as follows:

- N-value: 1 - 5
- Moisture content: 30% – 200%
- Mean unit weight: 14 kN/m^3
- Shear strength c: 5.0 kN/m^2
- Mean initial void ratio e_o: 3.46
- Compression index C_c: 0.2 to 2.0
- Coefficient of consolidation C_v: 0.1162 cm^2/min
- Coefficient of volume compression m_v: 0.9×10^{-4} to 15.4×10^{-4}kN/m^2

3) Seismic Condition

The design seismic coefficient was determined considering three sources of data as follows.

- Building Standards of El Salvador
- Mercalli Seismic Intensity of Past Large Earthquakes
- Past Earthquake Data in El Salvador for the last 32 years

Detailed discussion of Seismic Conditions is presented in Appendix F.

Based on the above three data and information, it was concluded that the design seismic coefficient to be adopted in the design should be 0.20 as shown in Table 4.1.3.

Table 4.1.3 Design Seismic Coefficient

	Seismic Coefficient for General Seismic Resistance
1) Building Standard	0.20
2) Mercalli Seismic Intensity	0.08
3) Estimate based on the Past Earthquake Data	0.20
Design Seismic Coefficient	0.20

4.2 Berth Structure

4.2.1 Berth Requirements

(1) Design Vessels

The design vessels for each berth were determined as the Panamax size as shown in Table 4.2.1. The characteristics of vessels are summarized in Appendix E.

Table 4.2.1 Design Vessels

Berth	Design Vessels					
	Type	DWT	LOA (m)	Breadth (m)	Draft (m) Max.	Draft (m) Sailing
1. Container Berth	Container Ship	55,000	294.0	32.2	13.1	13.1
2. Multi-purpose Berth	Bulk Carrier	43,000 ~ 50,000	185.0	32.2	12.0	11.8
3. Passenger Berth	Passenger Ship	25,000 GT	195.0	27.0	8.5	8.0
	Car Carrier	25,000	200.0	32.2	10.0	8.5

(2) Berth Dimensions

The berth dimensions were determined considering the following points

a) The length of the berth is determined based on the overall length of the design vessels including a necessary length for their mooring.

b) The infront water depth of the berth is determined based on the full draft of the design vessels as discussed in Section 3.2.

c) The crown height of the berth is determined based on the following factors:

i) The tide level and the maximum vessel size.

ii) The crown height of the existing berths Punta Gorda and Cutuco Port.

For the passenger berth, the ramp dimension of car carriers was taken into consideration.

A detailed analysis to determine the berth dimensions is illustrated in Appendix F and the results are summarized in Table 4.2.2

Table 4.2.2 Berth Dimensions

	Length (m)	Water Depth (m)	Crown Height (m)
Container Berth	340.0	-14.0	+5.0
Multi-Purpose Berth	220.0	-14.0	+5.0
Passenger / Car Carrier Berth	240.0	-9.5	+5.0

(3) Design Considerations

1) The Container Berth and Multi-purpose Berth will be constructed along the same berth alignment continuously. Considering the Multi-Purpose berth would be utilized by container vessels partially in future, container crane rails are to be expanded into the Multi-purpose berth for more efficient cargo handling operation. This concept is reflected to the Design of berths.

2) The Passenger Berth will be located on the northwest side of the Container Berth with an angle of 75° from the container berth alignment. The passenger berth will be utilized to handle passengers and car cargo. The ship type of these cargo will not be required the continuous apron along the berth, because these cargo will be handled in and out at the particular point of exists of passenger ships and ramp positions of car carriers. So the passenger berth is designed so as to connect by the bridges between ships and berth.

4.2.2 Design Conditions

(1) Natural Conditions

1) Meteorological and Oceanographic Conditions

Meteorological and oceanographic conditions to be taken into account in the basic design of berth structures are shown in Table 4.2.3.

Table 4.2.3 Meteorological and Oceanographic Conditions

Meteorological Condition	Rainfall	10-Year Probable Rainfall Intensity	70 mm/hr
	Wind	Stormy condition Maximum wind velocity	31 m/s
Oceanographic Condition	Tide	H.W.L	+3.37 m
		L.W.L	-0.13 m
	Wave	Operational condition (H1/3)	1.0 m
		(T1/3)	3.4 s
		Stormy Condition (H1/3)	2.2 m
		(T1/3)	4.5 s

(2) Geological Conditions

The following typical section of geological conditions corresponding to berth locations were determined based on the results of geological investigations.

1) Container Berth

a) Case I

Layer	Depth	Unit weight (kN/m³)	Submerged unit weight (kN/m³)	Angle of internal friction (Degree)	Cohesion (kN/m²)
Silty Clay	-14.0m - -20.0m	14.0	4.0	-	5.0
Gravelly Sand	-20.0m - -26.0m	18.0	10.0	35	-
Gravelly Clay	-26.0m - -30.0m	18.0	10.0	35	-
Lava	-30.0m -	-	-	-	-

b) Case II

Layer	Depth	Unit weight (kN/m³)	Submerged unit weight (kN/m³)	Angle of internal friction (Degree)	Cohesion (kN/m²)
Gravelly Sand	-14.0m - -26.0m	18.0	10.0	35	-
Gravelly Clay	-26.0m - -27.0m	18.0	10.0	35	-
Lava	-27.0m -	-	-	-	-

2) Multi-Purpose Berth

Layer	Depth	Unit weight (kN/m³)	Submerged unit weight (kN/m³)	Angle of internal friction (Degree)	Cohesion (kN/m²)
Silty Clay	-14.0m - -20.0m	14.0	4.0	-	5.0
Gravelly Sand	-20.0m - -26.0m	18.0	10.0	35	-
Gravelly Clay	-26.0m - -30.0m	18	10.0	35	-
Lava	-30.0m -	-	-	-	-

3) Passenger/Car Carrier Berth

Layer	Depth	Unit weight (kN/m³)	Submerged unit weight (kN/m³)	Angle of internal friction (Degree)	Cohesion (kN/m²)
Silty Clay	-10.0m - -19.0m	14.0	4.0	-	5.0
Gravelly Sand	-19.0m - -26.0m	18.0	10.0	35	-
Gravelly Clay	-26.0m - -30.0m	18.0	10.0	35	-
Lava	-30.0m -	-	-	-	-

(3) Berthing Conditions

In accordance with the Technical Standard for Port and Harbor Facilities in Japan, the following berthing conditions were applied for tug-assisted operation.

- Berthing speed: 0.1 m/s
- Approach angle: max.10 degrees

(4) Design Loads

The following loads are to be considered in the design;

1) Dead Loads

[Self weight]

reinforced concrete a unit weight: 24.0 kN / m^3

concrete a unit weight : 22.6 kN / m^3

reinforced concrete for piled platform, a specific weight: 20.6 kN/m^2

2) Live Loads

The following Uniformly Distributed Live loads (UDL) were determined applying the following formula:

$$UDL = \frac{Wdc}{Ac} \times Nn \times Cf$$

Where,

Wdc: Design Container Weight (kN/TEU)
Use 70% of Max container weight of 250 kN

Ac: Effective area of container box (15.0m²/TEU)

Nn: Design stacking numbers (=4)

Cf: Ratio of effective container stacking area
Cf= 0.4 at Apron
= 0.8 at Yard

Table 4.2.4 Design Live Loads (UDL)

Location	Design Live Loads	
	Normal Condition	Seismic Condition
Container Berth		
Apron	20.0 kN/m²	10.0 kN/ m²
Yard	40.0 kN/ m²	20.0 kN/ m²
Multi-purpose Berth		
Apron	20.0 kN/ m²	10.0 kN/ m²
Yard	20.0 kN/ m²	10.0 kN/ m²
Passenger Berth		
Platform	20.0 kN/ m²	10.0 kN/ m²
Yard	10.0 - 20.0 kN/ m²	5.0 – 10.0 kN/ m²

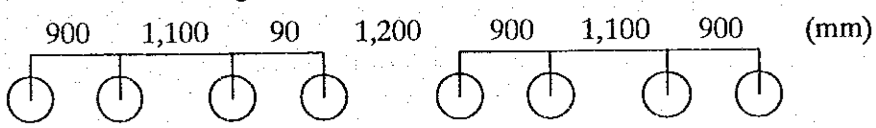
Notes: UDL at Container Berth is estimated from the following equation

3) Container Crane Loads

Principal dimensions of Panamax type Gantry Crane are shown below.

Lifting capacity	450 kN
Total load of the crane:	7,900 kN
Maximum corner load:	306.4 kN
Seaside rail from the face line:	3.0 m
Rail span:	25 m
Distance between buffers:	28 m

Wheel arrangement:



The design wheel loads of container crane were computed as indicated in Table 4.2.5.

Table 4.2.5 Wheel Load of Container Crane

Condition		Wheel load		Note
		Seaside wheel (kN/wheel)	Landside wheel (kN/wheel)	
Normal condition	Vertical	385	305	
	Horizontal	23	19	
Seismic condition	Vertical	580	450	Kh=0.2
	Horizontal	55	50	

4) Loads Imposed by Ships

a) Tractive Force

In accordance with Technical Standards for Port and Harbor Facilities in Japan, the following tractive forces were applied.

Table 4.2.6 Tractive Force

Gross Tonnage(GT)	Tractive Force on Bitt (kN)	Tractive Force on Bollard (kN)
5,000 – 10,000	7000	500
10,000 – 20,000	1,000	700
20,000 – 50,000	1,500	1,000

b) Berthing Force

The fender reaction force was estimated from the required absorbing ties of the effective berthing energy caused by the design vessels.

The effective berthing energy was calculated from the following formula stipulated in the Technical Standards for Port and Harbour Facilities in Japan.

$$E = \frac{Ms \text{ x } V\text{\textasciicircum}2}{2} \text{ x } Ce \text{ x } Cm \text{ x } Cs \text{ x } Cc$$

Where:

E: Effective berthing energy (kN-m)
Ms: Displacement tonnage (tons)
V: Berthing velocity (m/sec)
Ce: Eccentricity factor
Cm: Hydrodynamic mass coefficient
Cs: Softness coefficient (generally 1.0 is used)
Cc: Berth configuration coefficient (generally 1.0 is used)

$$Cm = 1 + \frac{\pi}{2 \text{ x } Cb} \text{ x } \frac{d}{B}$$

Where:

Cb: Block coefficient of vessel
d: Draft of vessel (m)

B: Breadth of vessel (m)

$$Cb = \frac{Ms}{Dd \times Lpp \times B \times W0}$$

Where:

Dd: Draft (m)
LPP: Length between perpendiculars(m)
B: Breadth (m)
W0: Unit weight of seawater (= 1030 kN/m2)

$$Ce = \frac{1}{1 + (L/r)^2}$$

Where:

Ce: Eccentricity coefficient (generally accepted value: 0.5 at 1/4 point contact)
L: Distance alongside the waterline of the quaywall from the center of gravity of the vessel to the berthing point
Rr: Turning radius (m) around the vessel's center of gravity on a level surface.

The berthing energy and fender reaction force were computed as shown in Table 4.2.7.

Table 4.2.7 Reaction Force and Absorption Energy of Fender

Item	Container Berth	Multi-purpose Berth	Passenger Berth	
Design Ship	Container Ship	Bulk Carrier	Passenger Ship	Car carrier
	55,000 DWT	43,000 ~ 50,000 DWT	25,000 GT	25,000 DWT
Length : LOA(m)	294	185	195	200
: LPP (m)	278	173	173	187
Breadth : B (m)	32.2	32.2	27.0	32.2
Draft: D(m)	13.1	11.8	8.0	8.5
1. Displacement Ms(ton)	76,300	63,000	18,200	40,000
2. Effective Berthing energy: E V=0.1m/s	380 kN-m	259 kN-m	91 kN-m	200 kN-m
3.Fender Energy absoption Reaction force (cell type fender)	393 kN-m Max 870 kN	285 kN-m Max 700 kN	107 kN-m Max 340 kN	224 kN-m Max 550 kN

Note: Energy absorption is 90% of catalog value, reaction force is 110% of catalog value

4.2.3 Alternative Study on Structural Types

(1) Alternatives of Structure Type

Generally the berth structures can be divided into two major types i.e. the open faced berth and the closed faced berth.

Open faced structures are represented by a deck type supported with piles, and an earth slope underneath the deck to bridge the level difference between the port area level and the infront water depth. Vertical loads are transferred to the subsoil by piles while the horizontal loads by battered piles or anchors.

Closed vertical faced structures generally act as an earth retaining structure and are sub-divided into gravity type retaining structure and flexible retaining structure.

Flexible retaining structures are based on the principle that the horizontal loads acting on the structure are taken by passive earth pressure and anchors. Loads acting on gravity type retaining structures transmit the loads at foundation level to the subsoil by friction and vertical reaction forces.

Categorizing into the gravity type berth, generate caisson type, cellular block type, concrete block type, L-type block type are considered.

The design infront water depth is –14.0 m for the Container and Multi-purpose Berths and –9.5 m for the Passenger Berth.

Depending on the infront water depth, the L-shaped block type is more suitable for small scale structures. Hence L-type block types and concrete block type are not included as study alternative.

(2) Container and Multi-purpose Berths

1) Alternative Structures

Based on the above general considerations, the following structural types were selected for further study and evaluation:

Table 4.2.8 Structural Type Alternatives

Alternative	Structural Type	Basic Type
Alternative 1	Concrete Caisson	Gravity Type
Alternative 2	Vertical Steel Pile	Piled Platform Type
Alternative 3	Combined Steel Pile	
Alternative 4	Steel Sheet Pile	
Alternative 5	Steel Sheet Pile Cellular cofferdam	Gravity Type

Alternative 1: Concrete Caisson Type (see Figure 4.2.1)

Concrete Caisson is cast in onland or on floating dock, and placed on the rubble mound foundation to make the berth front in the planned alignment. Taking account economical point of view, floating dock method is proposed for the Project.

Alternative 2: Vertical Steel Pile Type (see Figure 4.2.2)

The wharf platforms are founded on the vertical steel pile piles in order to make a stability against external fances.

Alternative 3: Combined Steel Pile Type (see Figure 4.2.3)

This is similar to Alternative 2, but to support the wharf platforms combined steel pipe piles are used to minimize a horizontal displacement.

Alternative 4: Steel Sheet Pile Type (see Figure 4.2.4)

The steel sheet piles and driven into seabed to form the berth front. Generally this steel sheet pile wall is anchored by tie-rod toward landside.

Alternative 5: Steel Sheet Pile Cellular Cofferdam Type (see Figure 4.2.5)

The structure consists of a circular cell formed by straight line steel sheet piles and its stability depends on the cell section and steel sheet pile.

The typical section of alternatives are shown in Figures 4.2.1 to Figure 4.2.5, respectively.

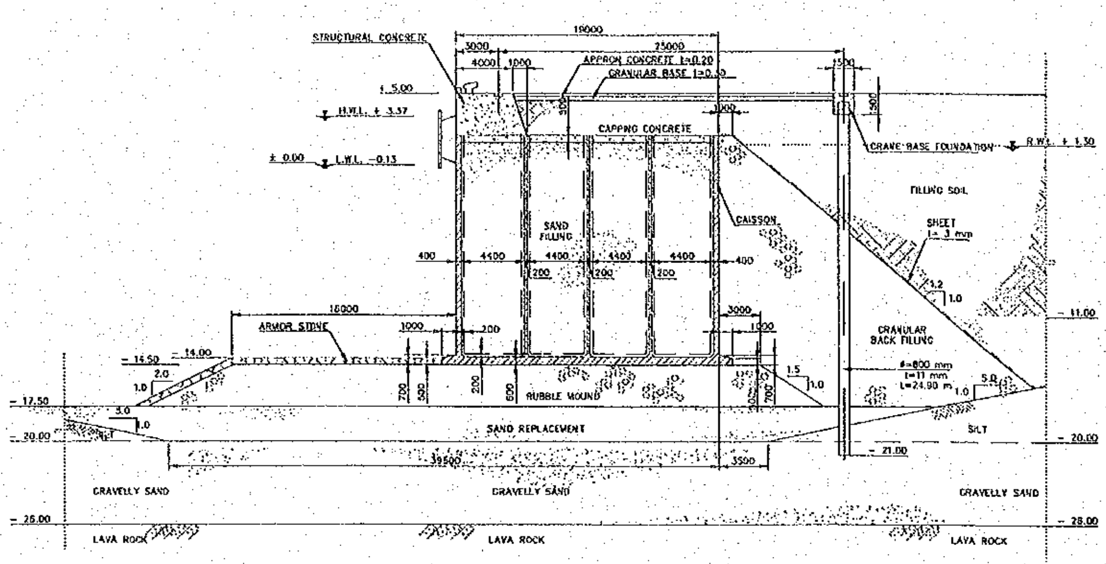


Figure 4.2.1 Alternative 1 Concrete Caisson

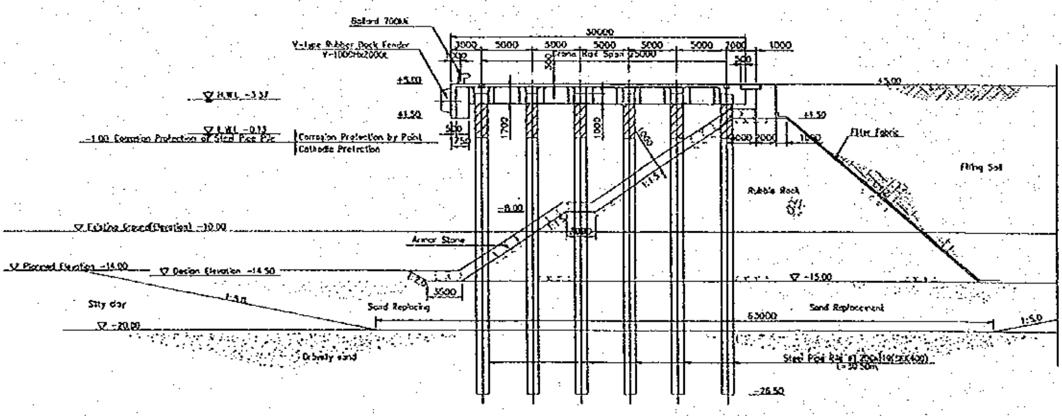


Figure 4.2.2 Alternative 2 Vertical Steel Pile

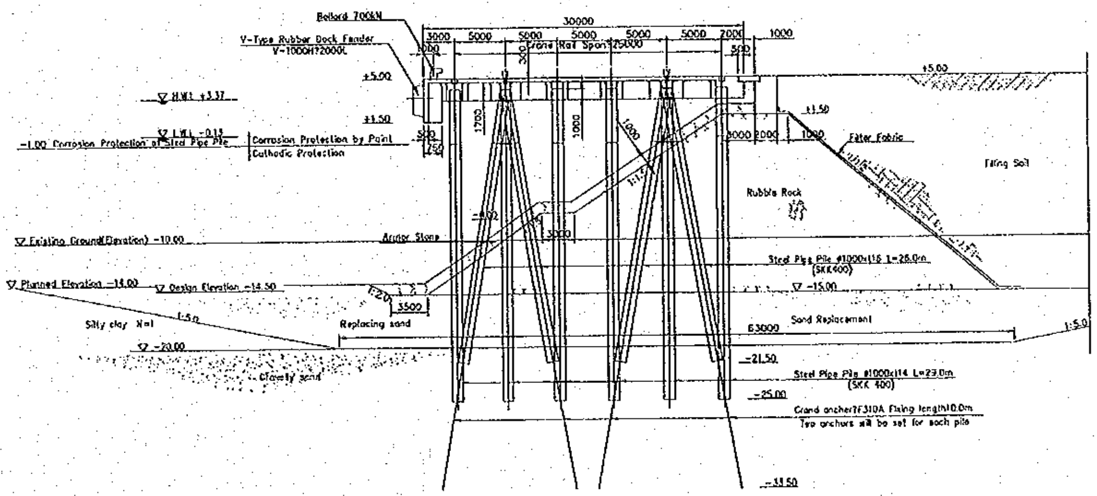


Figure 4.2.3 Alternative 3 Combined Steel Pile

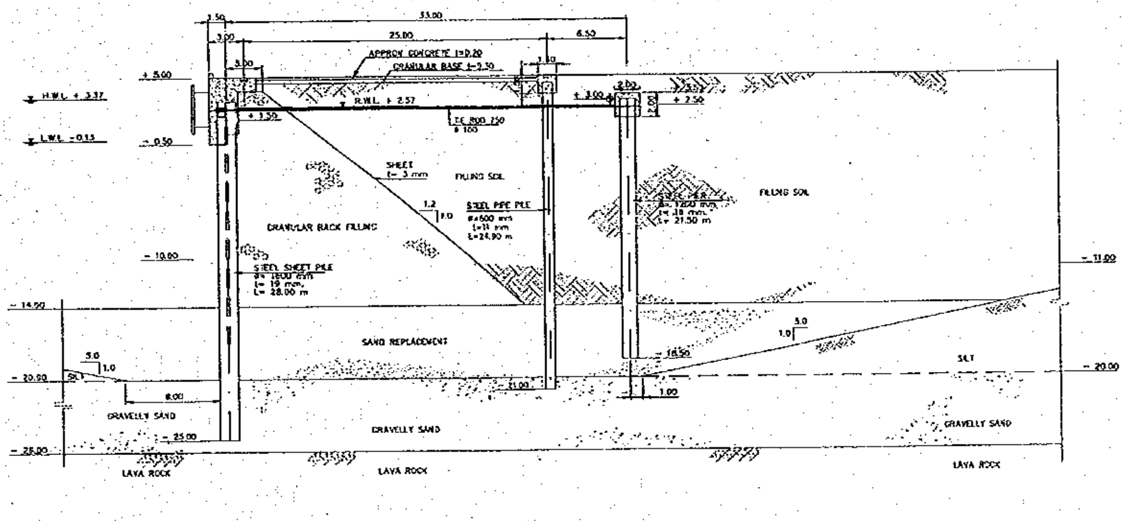


Figure 4.2.4 Alternative 4 Steel Sheet Pile

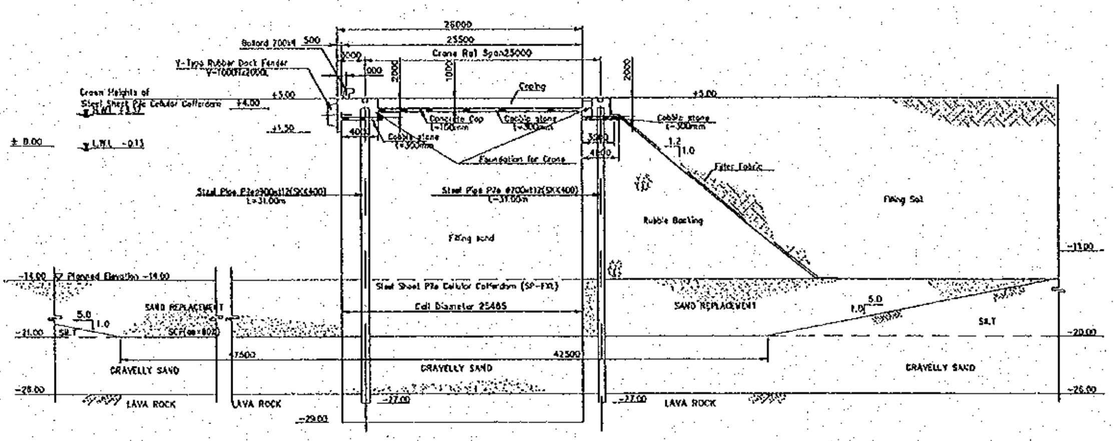
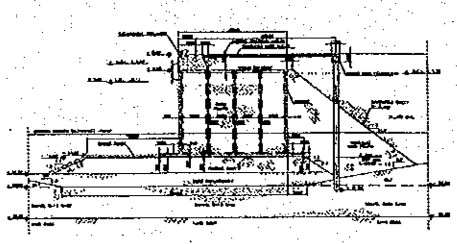
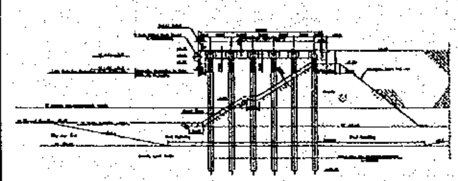
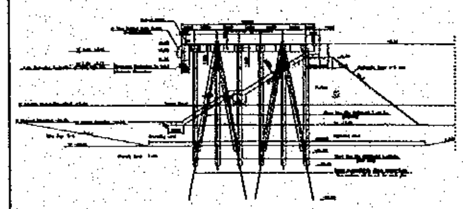
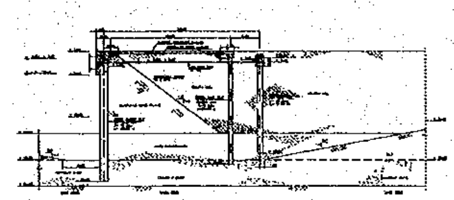
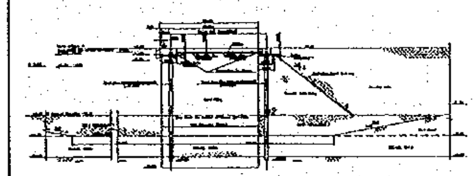


Figure 4.2.5 Alternative 5 Steel Sheet Pile Cellular Cofferdam

Table 4.2.9 Comparison of Structural Alternatives for Container/Multipurpose Berths (-14.0 m)

Type / Description	Alt. 1 Concrete Caisson Type	Alt. 2 Vertical Steel Pile Type	Alt. 3 Combined Steel Pile Type	Alt. 4 Steel Sheet Pile Type	Alt. 5 Sheet Pile Cellular Type
Characteristic of the Structure					
Engineering Aspect	-No corrosion protection required thus longer life time is expected. -Soft material be must replaced with coarse sand to obtain sufficient bearing capacity and stability against seismic force. Asphalt mattress also be provided to increase sliding resistance. -Land side crane foundation is provided. -Most of material can be locally found -Maintenance is easy	-Corrosion protection on piles is required -Soft material must be replaced with coarse sand to obtain sufficient resistance of embedded portion and to improve effective length of piles. -No pulling force occurs -Flexible for seismic force -No crane foundation required independent -All steel piles shall be imported -Maintenance on piles (corrosion) is required.	-Corrosion protection on piles required -Soft material be replaced with coarse sand to improve effective length of piles. -Rock anchors is required to resist pulling force. -Advantageous against seismic force. -No crane foundation required independent -All steel piles be imported -Maintenance on piles required.	-Corrosion protection required -Soft material be replaced to obtain sufficient resistance of embedded portion. -No pulling force occurs. -Land side crane foundation required. -All steel piles and sheet piles be imported -Maintenance on sheet piles required.	-Corrosion protection required -Soft material be replaced to obtain sufficient shear strength of cell structure and resistance of embedded portion. -No pulling force occurs. -both side crane foundation required. -All sheet piles be imported -Maintenance on sheet piles required.
	(B)	(A)	(C)	(C)	(C)
Construction Aspect	-Caisson fabrication yard or floating dock (FD) is required. If floating dock is used construction yard required is less than (11,500 m2). - Floating dock ahall be mobilized from abroad -Construction period is relatively long.	-Relatively large construction yard required (23,500 m2) -Main construction equipment be mobilized from abroad. -Pre-drilling work is required for pile driving -Construction period is relatively short.	-Relatively large construction yard required (23,500 m2) -Main construction equipment be mobilized from abroad. -Pre-drilling work is required for pile driving -Construction period is relatively long. -Pre-drilling of raked piles and reliability of rock anchors be due examined	-Relatively large construction yard required (23,500 m2) -Main construction equipment be mobilized from abroad. -Continuous pre-drilling required for front sheet piles. -Construction period is relatively short.	-Relatively large construction yard required (23,500 m2) -Main construction equipment be mobilized from abroad. -Sheet piles installation is quite difficult due to hard layers and boulders anticipated
	Least Problem (A)	Less Problem (B)	Some problem anticipated (C)	Less problem (B)	Difficult (D)
Cost Index	1.00 (A)	1.16 (B)	1.42 (C)	1.30 (C)	1.42 (C)
Total Evaluation	A (Recommended)	B	C	C	D

2) Examination and Comparison of Structural Alternatives

The selected five (5) alternatives were examined through conducting a preliminary design with the conditions that impose the most critical loading and subsoil conditions to the structure. The construction plan and cost estimate of each alternative were also prepared based on the outputs of the preliminary design.

The results of examination of the alternatives are presented in Table 4.2.9 for comparison.

They show that the caisson type structure (Alternative 1) is recommendable as the most advantageous in terms of construction cost, construction method, etc.

(3) Passenger Berth

A dolphin type berth structure was proposed for the Passenger Berth, which is composed of two (2) working platforms (center and stern) and two (2) breasting dolphins.

1) Layout of Passenger Berth Dolphins

The alignment of the berth is offset at 25 m from the revetment face line behind the berth considering the slop of rubble mound.

a) Breasting Dolphins Arrangement

The breasting dolphins shall be arranged so that the ship can contact with the straight run of the hull during docking. Considering this straight run hull is 3/4 of over all length, the appropriate spacing is given as around 1/3 of the design ships length.

Since the design ship size for the Passenger Berth is in the range of 150 m and 200 m, the preferable dolphin spacing is around 50 ~ 67 m. Them the spacing of the breasting dolphins is determined at 70 m (refer to Figure 4.2.6).

b) Platform Arrangement

The gangway of passenger ships is generally located at about the center of the ships and normally a wider platform space is not required. On the other hand, car carriers require at least a 10 m wide platform for Ro/Ro ramp operation. The Ro/Ro ramp locates at the stern with an angle of 45° to the berthing line, thus a wider platform is to be provided. Taking the above operational conditions into account, the following platforms are arranged:

- Center Platform Min. 10 m wide

- Stern Platform Min. 20 m wide

The layout plan of the Passenger Berth is indicated in Figure 4.2.6.

As to the appropriate berth height for Car Carrier, it should be lower than the ramp height in any tide condition. Normally Car Carrier has three ramps, one for stern and two for middle position. At least two ramps shall be made available for smooth operation. The heights of each ramp for Car Carriers calling Acajutla Port are indicated in Table 4.2.10.

Table 4.2.10 Height of Ramps for Car Carrier

LOA	Draft	Height of Stern Ramp (m)	Height of Side Upper Ramp (m)	Height of Side Lower Ramp (m)
199 m class	9.7 - 10.1 m	5.2 - 5.4 m	5.2 - 5.4 m	1.9 - 3.6 m
185 m class	8.8 - 9.2 m	5.2 - 5.3 m	5.2 - 5.3 m	2.3 - 3.5 m
180 m class	8.5 - 9.0 m	5.2 - 5.5 m	5.2 - 6.8 m	1.1 - 4.2 m

As can be seen above, it is quite difficult to adjust for lower side ramp during low tie. However, the crown height of +5.0 m can allow to use two ramps in the same time.

2) Required Turning Resin of Passenger Berth

Under strong tidal current, the passenger ships and car carriers have to turn about 75° toward the berth line for berthing and deberthing.

Figure 4.2.6 illustrates how ships would be maneuvered for berthing and deberthing. Based on this, the required turning basin is stick out as outlined by in the figure.

3) Alternatives Structural

Considering the study results of the Container Multi-purpose Berths, the following two (2) alternatives were selected for detailed examination and comparison.

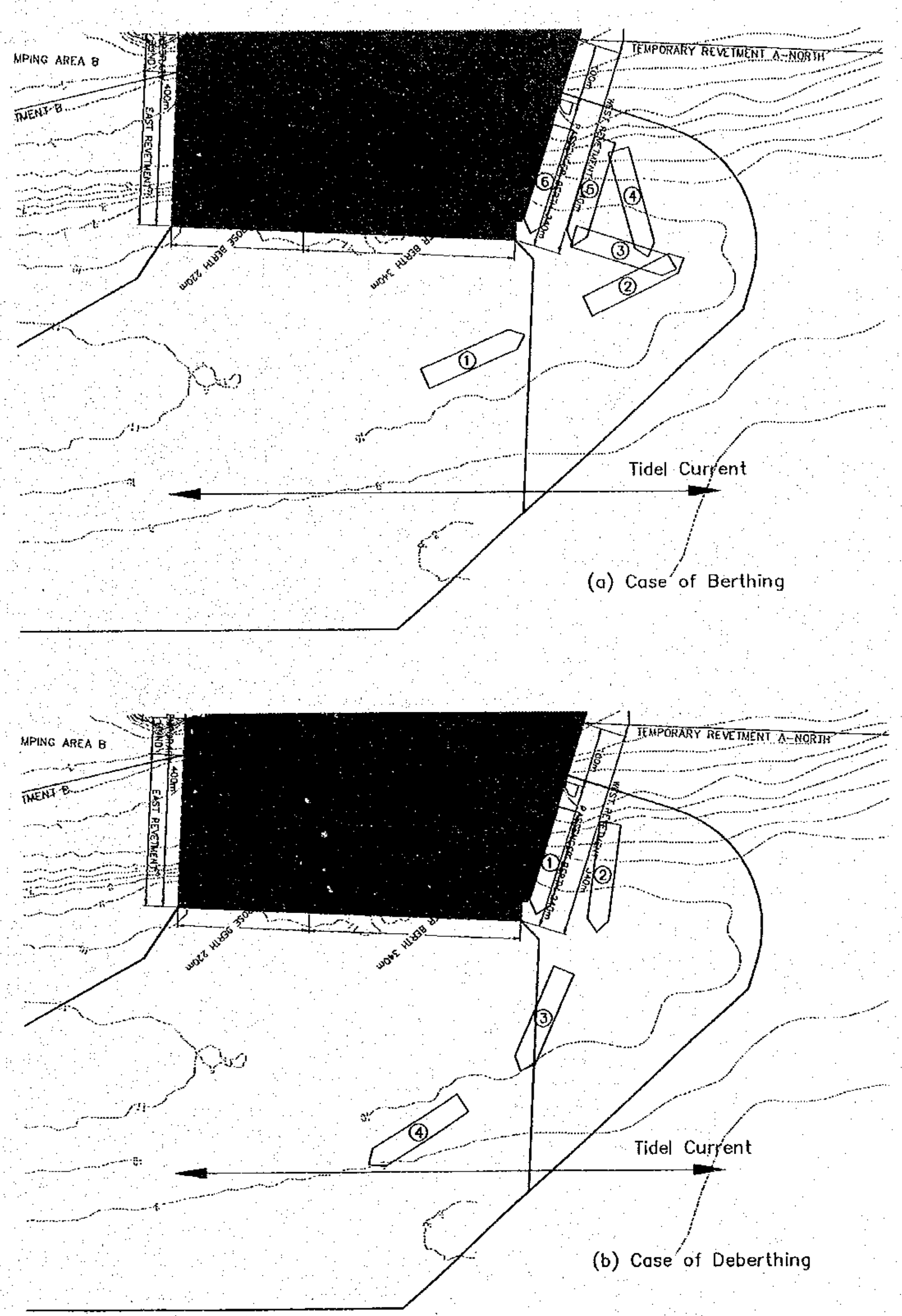
Alternative A: Vertical Steel Pile Type
Alternative B: Concrete Caisson Type

The typical sections of berth structure for each alternative are indicated in Figures 4.2.7 and 4.2.8.

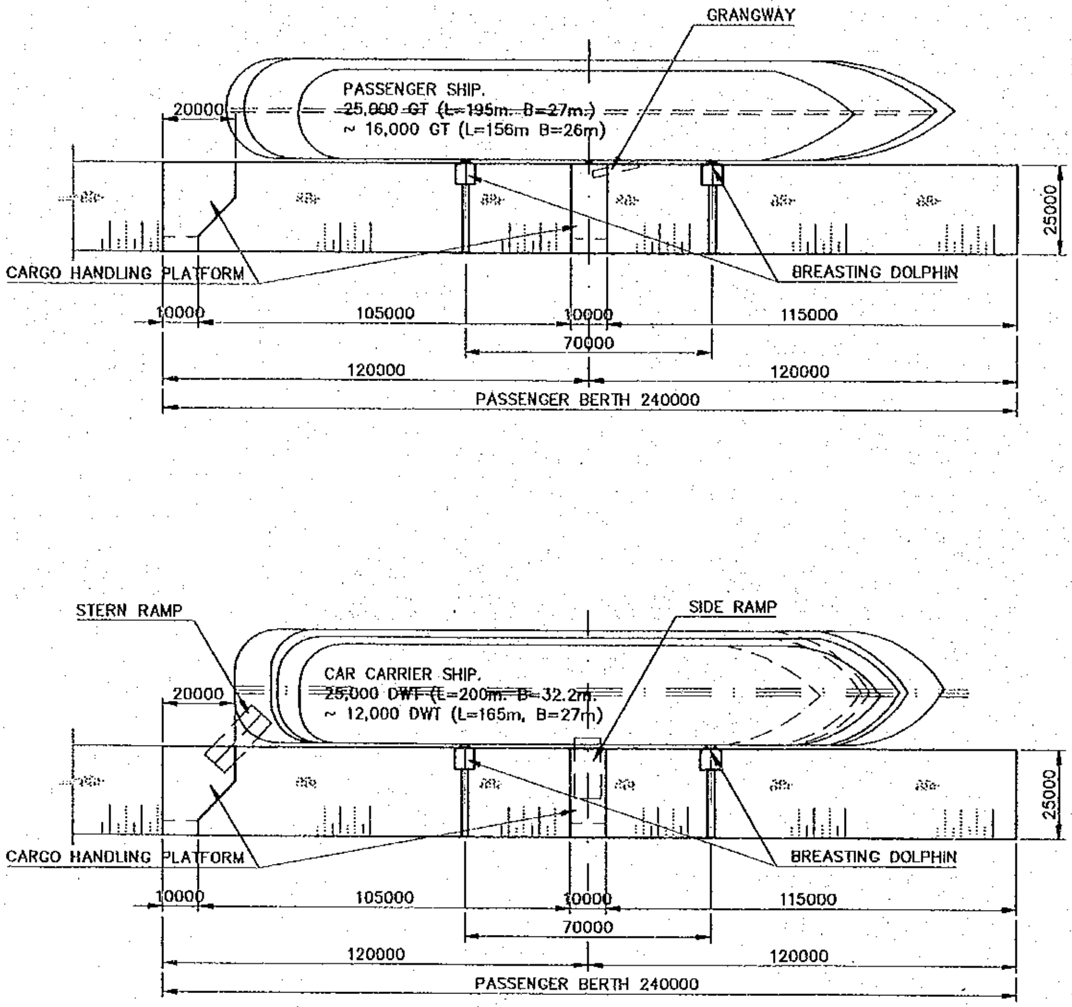
4) Examination and Comparison of Alternative Structures

The two alternatives were examined from engineering, constructional viewpoints and the construction cost.

The examination results are shown in Table 4.2.11, and Alternative A was selected the most suitable one.



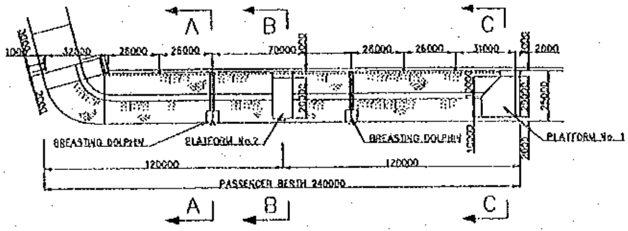
Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.2.6	REQUIRE TURNING BASIN OF PASSENGER BERTH



LAYOUT OF PASSENGER BERTH

SCALE 1:2,000 0 20.0 40.0 60.0 80.0

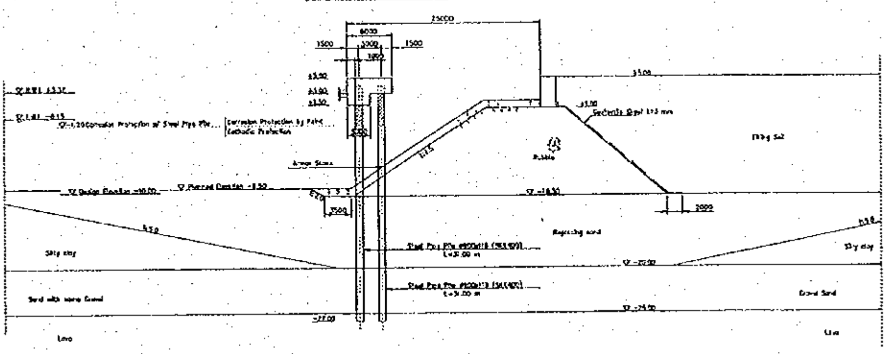
	Japan International Cooperation Agency (JICA)	Figure	Title
	The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.2.7	LAYOUT OF PASSENGER BERTH



KEY PLAN

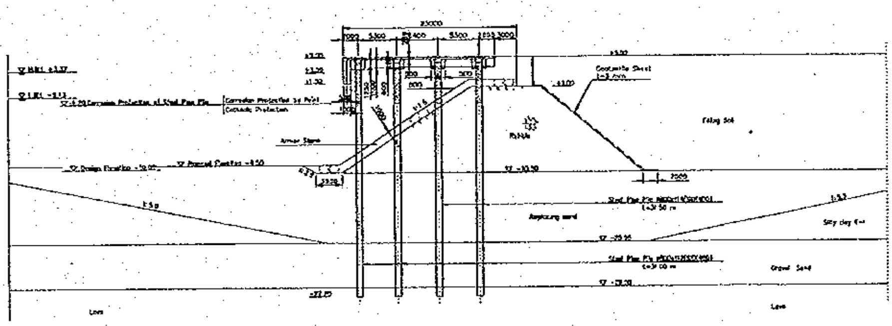
BREASTING DOLPHIN A-A

Typical Cross Section



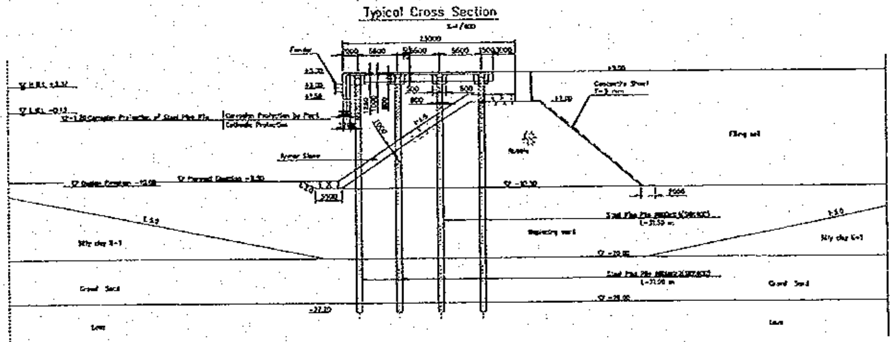
PLATFORM No.1 B-B

Typical Cross Section

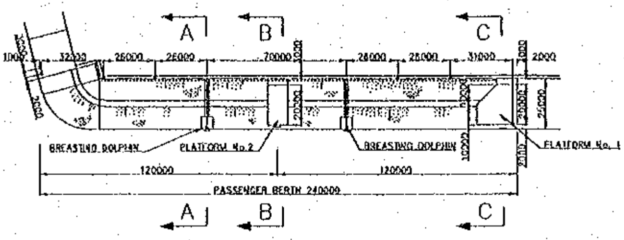


PLATFORM No.2 C-C

Typical Cross Section

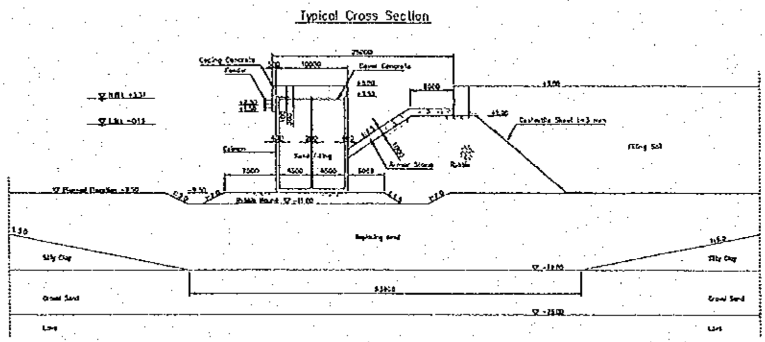


Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.2.8	Piled Dolphin Type

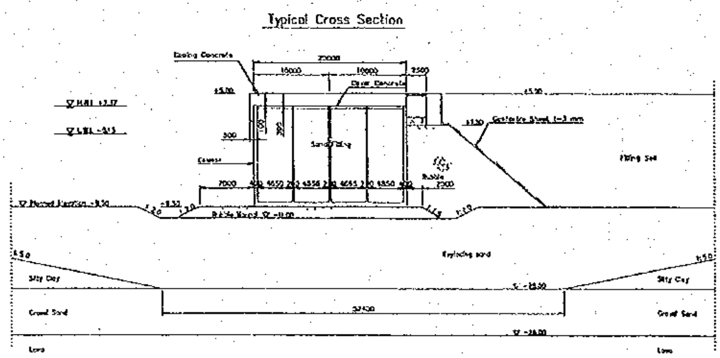


KEY PLAN

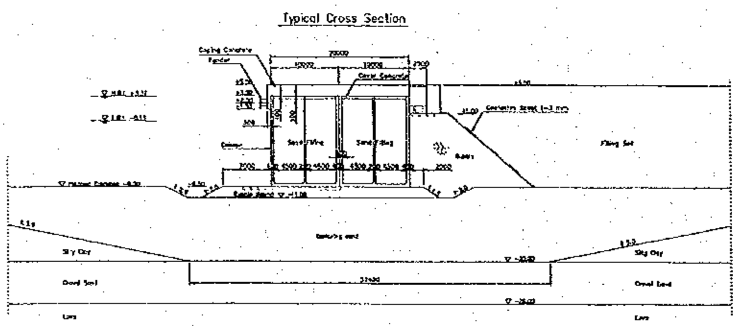
BREASTING DOLPHIN A-A



PLATFORM No.1 B-B

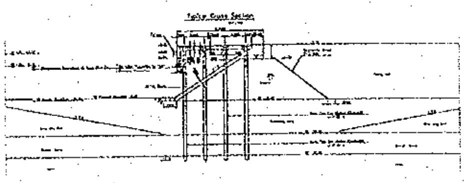
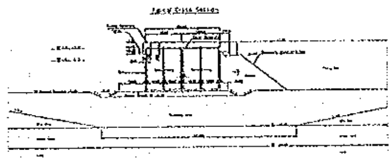
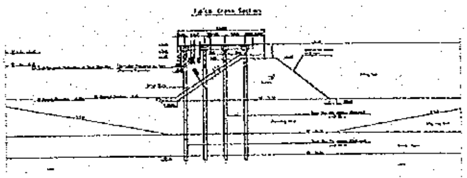
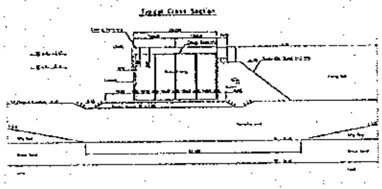
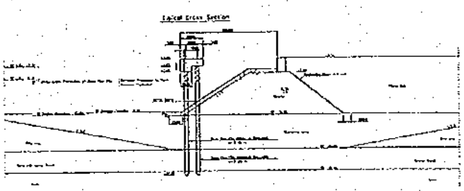
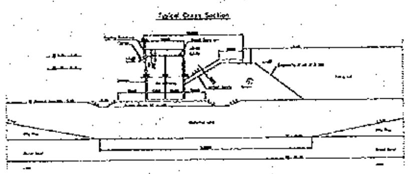


PLATFORM No.2 C-C



Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.2.9	**Concrete Caisson Type**

Table 4.2.10 Comparison of Alternatives Structural for Passenger Berth

Description \ Type	Alt. A Vertical Steel Pile Type	Alt. B Caisson Type
Platform No. 1		
Platform No. 2		
Breasting Dolphin		
Engineering Aspect	- Soil improvement area by sand replacement is less. - More flexibility than alt. B against strong seismic force -Piles be imported	-Wider soil improvement area is required. -Larger size of breasting dolphin required from stability of caisson. -Most of material can be obtained locally.
Evaluation	More advantageous (A)	(B)
Maintenance Aspect	-Necessary to change corrosion Protection after 30 years	-Maintenance free
Evaluation	(B)	(A)
Construction Aspect	-Piling equipment be mobilized in addition to main berths construction -Pre-drilling for pile driving required.	-Same construction equipment can be used with main berth construction
Evaluation	(B)	Less Problem (A)
Environment Aspect	-Lenght of Pile Type Structure shorter than caisson type	-Lenght of caisson Structure is longer than Pile Type
Evaluation	(A)	(B)
Cost Index	1.00 (A) (Included cost of corrosion protection)	1.11 (B)
Total Evaluation	A (Recommended)	B

4.2.4 Proposed Berth Structure

From the result of alternative studies on the berth structure, the following structural types were selected;

- Container / Multi-purpose Berths: R.C. Caisson type
- Passenger Berth: Piled Dolphin type

4.2.5 Examination of Stability

(1) Examination of stability of Container and Multi-purpose Berths (Caisson Type)

In order to increase sliding resistance of caisson, underneath of caisson bottom an asphalt mat is to be placed so that the higher friction coefficient (F = 0.7) could be applied than F = 0.6 without asphalt mat.

1) Stability of Caisson

The safety factor against sliding is calculated using the following formula:

$$F < \frac{f \times W}{P}$$

Where:

W: Vertical resultant force acting on caisson body (kN)
Vertical resultant forces includes weight of caisson body and buoyancy excluding the surcharge.

P: Horizontal resultant force acting on the wall (kN)
Horizontal resultant forces includes the following:
- Horizontal component of earth pressure acting on the rear plane of the virtual wall with surcharge.
- Residual water pressure.
- Buoyancy acting on caisson body
- Seismic force.

f: Coefficient of friction between the bottom of the wall and the foundation.
F = 0.7 (Concrete with asphalt mat against rubble mound)

F: Safety factor
The safety factor should be more than 1.2 under ordinary condition, and 1.0 more than under seismic condition.

The safety factor against overturning as a gravity type quaywall was calculated using the following formula:

$$F < \frac{W \times t}{P \times h}$$

Where :

W:	Vertical resultant force acting on the caisson body (kN)
P:	Horizontal resultant forces acting on the caisson body (kN)
t:	Distance between acting point of the resultant vertical force and the front toe of the caisson body (m)
h:	Height to the acting force of the resultant horizontal force from the wall to the bottom of the caisson body (m)
F:	Safety factor. The safety factor shall be more than 1.2 under ordinary condition and more than 1.1 under seismic condition.

2) Stability of Foundations

Stability of caisson foundations shall be examined against the slope-failure with a circular slip surface and the eccentric load condition.

The safety factor against slope failure with a circular slip surface was calculated using the following formula:

$$F = \frac{R\,\Sigma(\,cl + W'\cos\alpha\tan\phi)}{\Sigma Wx \;+\; \Sigma Ha} = \frac{\Sigma(cb + W'\cos\alpha\tan\phi)\sec\alpha}{\Sigma W\sin\alpha + 1/R\Sigma Ha}$$

Where:

R:	Radius of slip circle (m)
c:	Cohesion of soil (kN/m2)
ø:	Angle of internal friction (degree)
l:	Base length of a slice (m)
b:	Width of a slice (m)
W':	Effective weight of a slice (sum of soil weight and surcharge. For the underwater soil, the underwater unit weight shall be used) (kN/m)
W:	Total weight of a slice (sum of total weight of soil and water and surcharge)(kN/m)
α:	Gradient of the base of a slice (degree)
x:	horizontal distance between the center of gravity of a slice and the center of the slip circle. (m)
H:	Horizontal external force acting on the soil mass in the slip circle (such as hydrostatic pressure, seismic force, water pressure) (kN/m)
Aa:	Length between the elevation of external force H and to the center of the slip circle (m)

F: Safety factor

The safety factor against circular slip under ordinary conditions should be more than 1.3.

The safety factor against slope failure under eccentric axial load condition was calculated using the Bishop method.

$$Fs = \frac{1}{\Sigma W \sin\alpha + 1/R\,\Sigma Ha}\ \Sigma\ \frac{(cb + W'\tan\phi)\sec\alpha}{1+(\tan\alpha\tan\phi)/Fs}$$

Where:

All symbols are the same as the above-mentioned formula.

The factor of safety against circular slip under ordinary conditions should be more than 1.0.

3) Results of Stability Examination

Based on the subsoil conditions described in Section (4.2.2) the stability is examined under the following two cases of subsoil conditions for the Container Berth and one for the Multi-purpose Berth.

a) Case 1: Containers Berth with soft deposit (soil improvement re quired)

The analysis results on the concrete caisson type berth show that the stability of the berth is in all factors satisfactory. It was however found that soft layers shall be replaced by well graded coarse sand to avoid liquefaction by earthquake.

The examination results are summarized in Table 4.2.12.

Table 4.2.12 Results of Structural Stability Examination for Container Berth (Case 1)

Examination Items	Without Crane Load		With Crane Load	
	Ordinary Case	Seismic Case	Ordinary Case	Seismic Case
Sliding (f=0.7)	4.86>1.2	1.19>1.0	5.02>1.2	1.28>1.0
Over-turning	10.2>1.2	2.32>1.1	9.81>1.2	2.32>1.1
Bottom Bearing Pressure (less than 600 kN/m^2)	P=273kN/m^2	P=520kN/m^2	P=321kN/m^2	P=602kN/m2
Slope Failure (Ordinary)	1.59>1.3	-	-	-
Eccentric Load (Bishop's Method)	2.56>1.2	1.04>1.0	2.47>1.2	1.04>1.0

b) Case 2: Container Berth without soft deposit (no soil improvement required)

The same examination of stability as in Case 1 is carried out for this case and it is found that the berth structure can satisfactorily withstand all the design load combination. The thickness of the rubble foundation is estimated at 1.5 m

The examination results are summarized in Table 4.2.13

Table 4.2.13 Results of Structural Stability Examination for Container Berth (Case 2)

Examination Items	Without Crane Load		With Crane Load	
	Ordinary Case	Seismic Case	Ordinary Case	Seismic Case
Sliding (f= 0.7)	4.86>1.2	1.19>1.0	5.02>1.2	1.28>1.0
Over-turning	10.2>1.2	2.32>1.1	9.81>1.2	2.32>1.1
Bottom Bearing Pressure (less than 600 kN/m^2)	P=273N/m^2	P=520kN/m^2	P=321kN/m^2	P=602kN/m^2
Slope Failure (Ordinary)	1.59>1.3	-	-	-
Eccentric Load (Bishop's Method)	3.19>1.2	1.06>1.0	3.06>1.2	1.06>1.0

c) Case 3: Multi- purpose Berth

The same conditions used for Case 1 are also applied for this berth design. Thus, same finding and the results as Case 1 are obtained as indicated in Table 4.2.14.

Table 4.2.14 Results of Structural Stability Examination for Multipurpose Berth

Examination Items	Without Crane Load	
	Ordinary Case	Seismic Case
Sliding (f=0.7)	4.86>1.2	1.19>1.0
Over-turning	10.2>1.2	2.32>1.1
Bottom Bearing Pressure	P=273kN/m	P=520kN/m^2
Slope Failure (Ordinary)	1.86>1.3	-
Eccentric Load (Bishop's Method)	2.56>1.2	1.04>1.0

(2) Examination of Stability of Piles for crane foundations

The stability of land side crane foundations shall be examined by the safety factor of steel pile stress and bearing capacity.

1) Stability of piles

The stress of steel piles was examined by the following equation:

$$F = \frac{\sigma c}{\Sigma \sigma ca} + \frac{\sigma bc}{\sigma ba} < 1.0$$

Where:

σc: Compressive stress by axial compressive force acting on the section (N / mm^2)

σbc: Maximum compressive stress by the bending moment acting on section (N / mm^2)

σca: Allowable axial compressive stress on axis with smallest moment of inertia (N/ mm^2)

σba: Allowable bending compressive stress (N / mm²)

F: Safety factor

The safety factor shall be less than 1.0 in any cases.

2) Axial bearing capacity of piles

A further examination of pile stress of each structure under various loading conditions is carried out. The result indicated that the pile stresses are in any case satisfactory as summarized in Table 4.2.15.

$$F = \frac{Ru}{R}$$

Where :

R: Axial force of pile (kN).

Ru: Ultimate bearninf capacity of pile (kN).

F: Safety factor.

More than 2.5 (Original Case)

More than 1.5 (Seismic Case)

$$Ru = 300\ N\ Ap + 2\ N\ As$$

Where:

Ru : ultimate bearing capacity of pile (kN)

Ap : toe area of pile(m^2)

As : total circumferential area of pile (m^2) (plug effect of open-ended pile = 0.5)

N : N-value of the ground around pile toe

N : mean N-value for total penetration length of pile

As : total circumferential area of pile (m^2)

The N-value is calculated by equation:

$$F = \frac{N_1 + N_2}{2}$$

Where:

N_1 : N-value at the toe of pile

N_2 : mean N-value in the range from the toe of pile to the level 4B above

B : diameter of width of pile (m)

3) Results of stability Examination

The examination results are summarized in table 4.2.15.

Table 4.2.15 Results of stability Examination for crane foundations

Examination Items	Original case	Seismic Case
Safety factor of pile	0.53<1.0	0.91<1.0
Safety factor of bearning capacity	2.89>2.5	1.84>1.5

(3) Examination of Stability of Passenger Berth (steel pile type)

1) Stability of piles

The stress of steel piles was examined by the following equation:

$$F = \frac{\sigma c}{\sigma ca} + \frac{\sigma bc}{\sigma ba} < 1.0$$

Where:

σc: Compressive stress by axial compressive force acting on the section (N / mm^2)

σbc: Maximum compressive stress by the bending moment acting on section (N / mm^2)

σca: Allowable axial compressive stress on axis with smallest moment of inertia (N/ mm^2)

σba: Allowable bending compressive stress (N / mm^2)

F: Safety factor

The safety factor shall be less than 1.0 in any cases.

2) Results of Stability Examination

A further examination of pile stress of each structure under various loading conditions is carried out. The result indicated that the pile stresses are in any case satisfactory as summarized in Table 4.2.16.

Table 4.2.16 Examination Results of Pile Stress

Location	Dimension of Pile	Stress Check of Piles	Loading Case
Breasting Dolphins	D=1,100 mm t=14 mm	0.76 (max)	Mooring case
Pier No1	D=700 mm t=12 mm,	0.75 (max),	Seismic case
Pier No2	D=800 mm t=14 mm	0.77 (max),	Seismic case

4.2.6 Proposed Berth Design

Based on the above examination results, the following proposed berth designs are presented in the following Figures and in Appendix F.

1) Container Berth

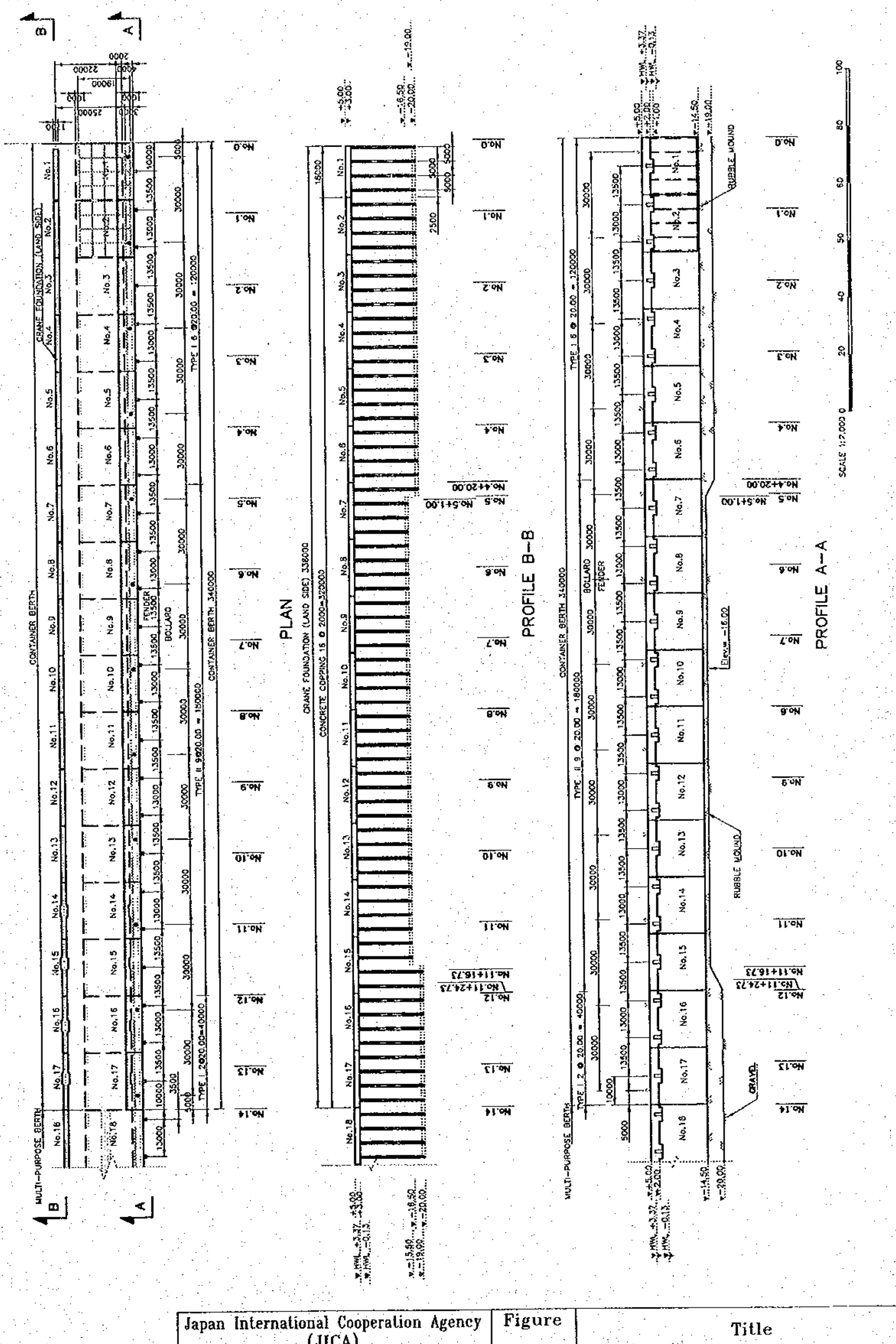
The proposed design of the Container Berth is shown in Figures 4.2.9 and 4.2.10.

2) Multi-purpose Berth

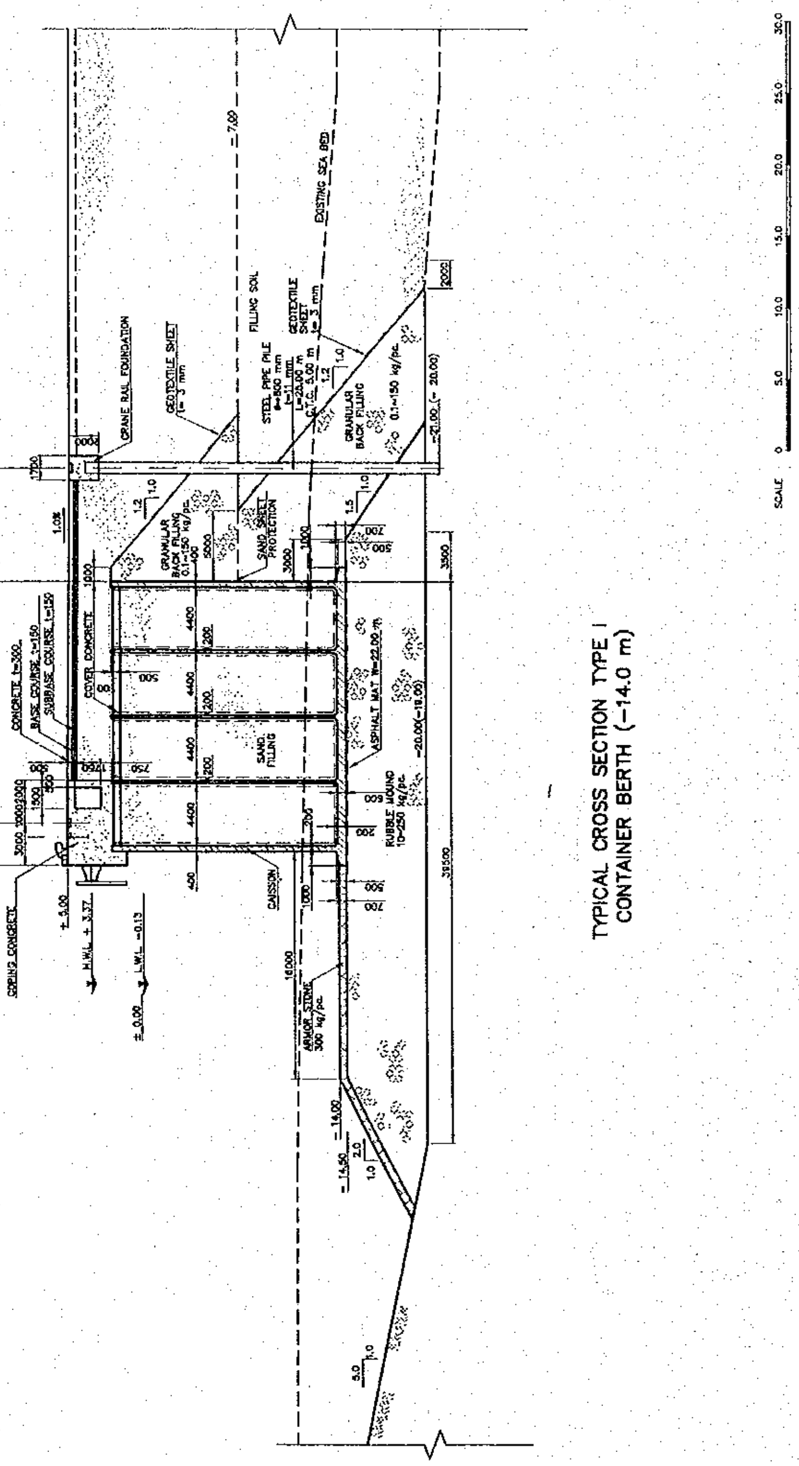
The proposed design of the Multi-purpose berth is shown in Figures 4.2.11 and 4.2.12.

3) Passenger Berth

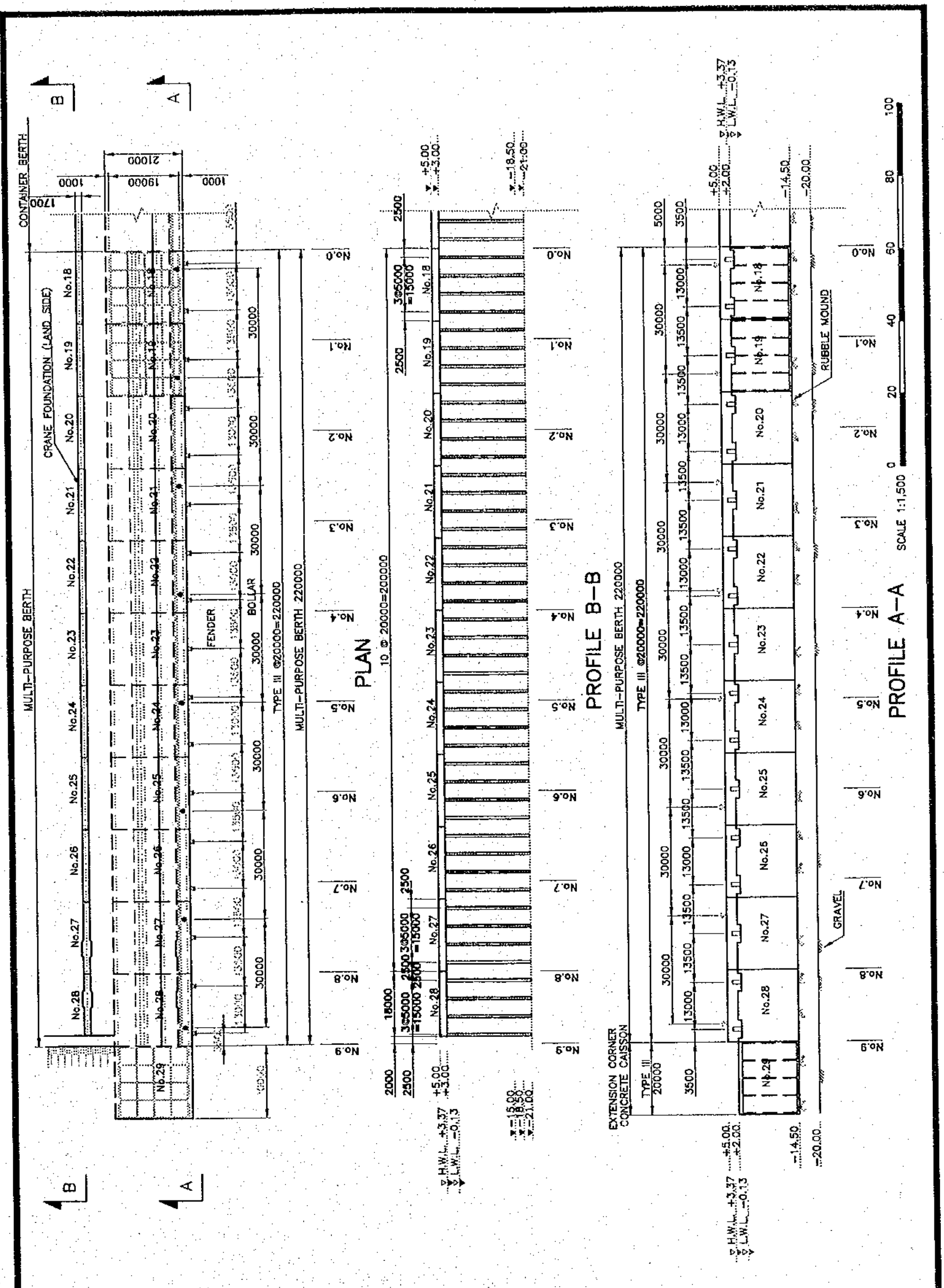
The proposed arrangement of the Passenger Berth is shown in Figures 4.2.13 and 4.2.14.



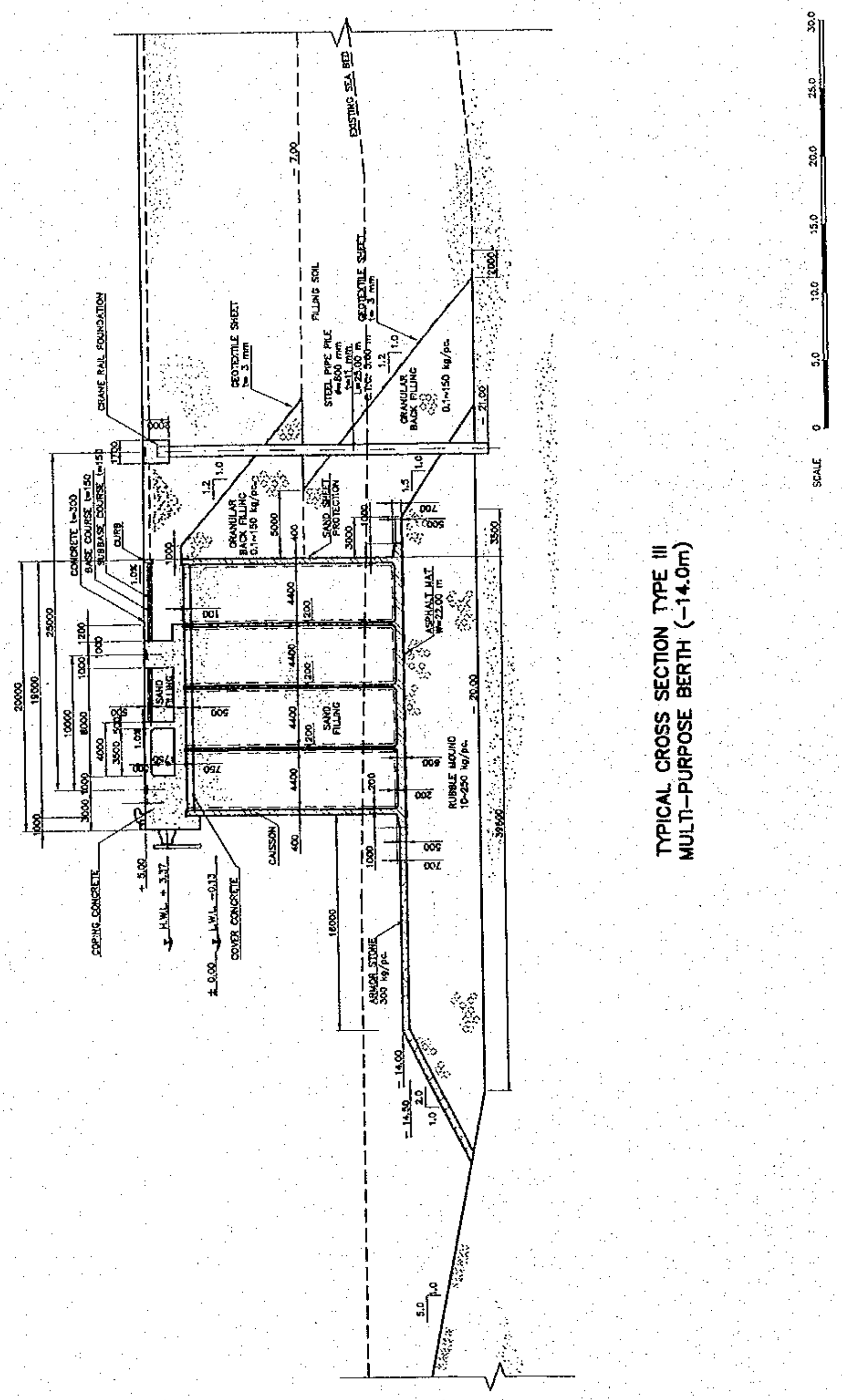
Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.2.10	PLAN AND PROFILE CONTAINER BERTH



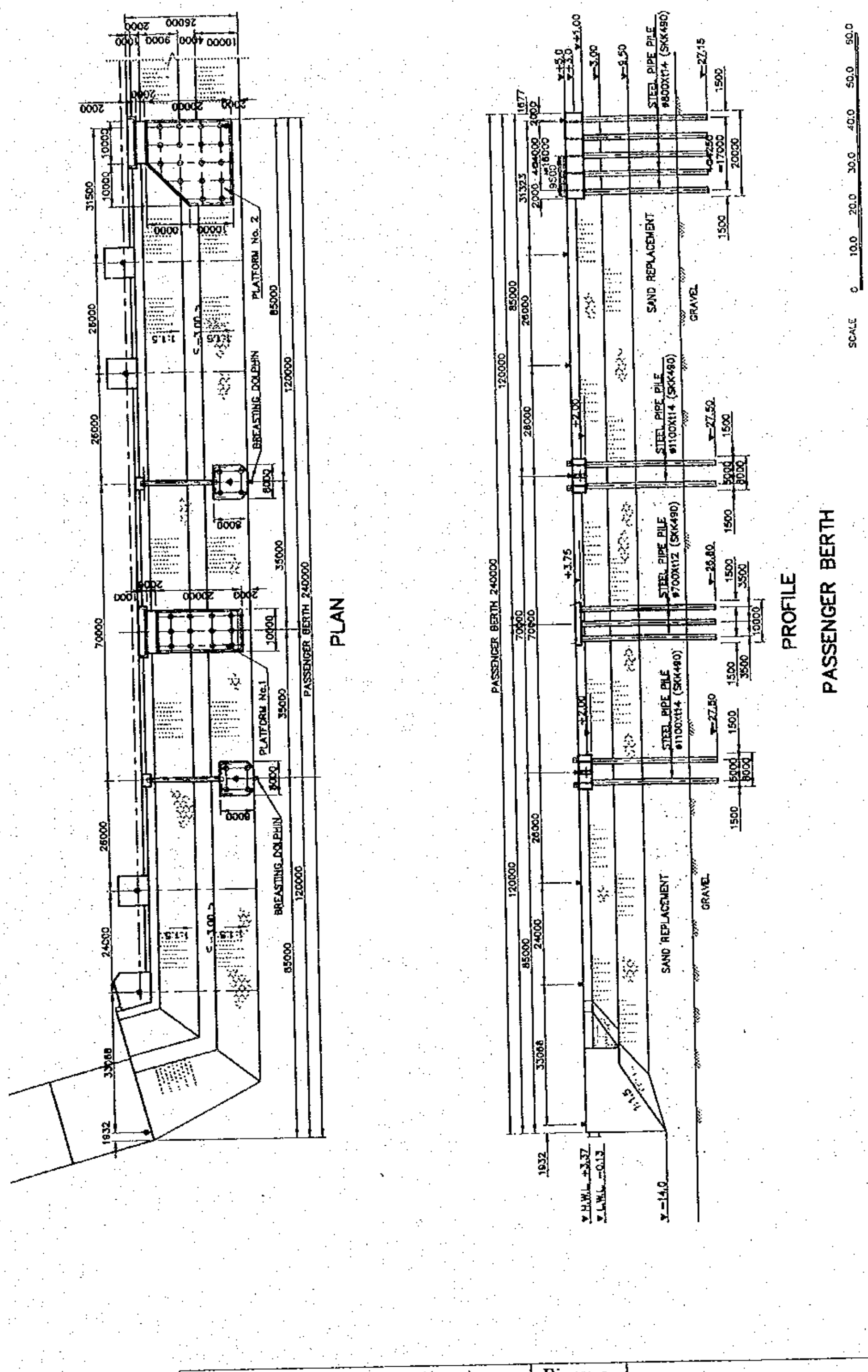
Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.2.11	TYPICAL CROSS SECTION TYPE I CONTAINER BERTH (−14.0 m)



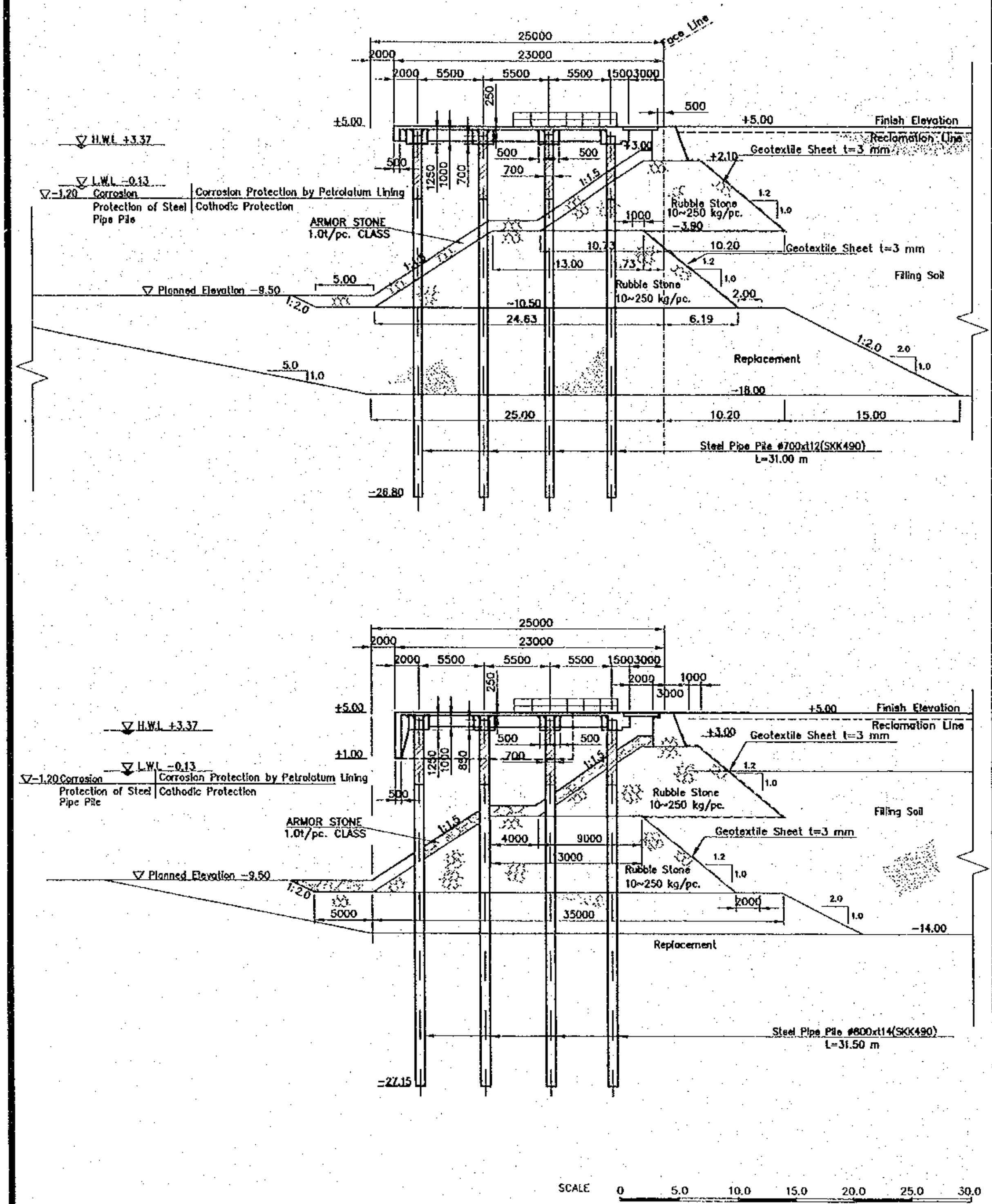
Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.2.12	PLAN AND PROFILE FOR MULTI–PURPOSE BERTH



Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.2.13	TYPICAL CROSS SECTION TYPE III MULTI-PURPOSE BERTH (−14.0m)



Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.2.14	PLAN AND PROFILE OF PASSENGER BERTH



Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.2.15	PIER 1 AND BREASTING DOLPHIN TYPICAL CROSS SECTIONS

4.2.7 Small Craft Basin

The small craft basin is planned for the purpose to moor two tugboats and some small crafts.

(1) Location

The location of the basin is determined in consideration of the following points.

1) To harmonize with futu re port expansion plan
2) To keep calmness for small crafts

From the above points, small craft basin is planned to locate at the corner of west revetment and temporary revetment A-north.

(2) Dimensions of Facilities of Small Craft Basin

The dimensions of the facilities were determined as follows;

1) Design Ship

The maximum design ship is new tugboat to be introduced under the project and the ship sizes are as shown below.

Ship Type	Ship Length	Ship Width	Draft	Number
Tugboat	33.5 m	9.4 m	3.1 m	2

2) Dimension of Facilities

The facilities are designed based on two-abreast berthing.

Wharf Length	50.0 m	The length includes length for mooring lines and ship length.
Depth	4.0 m	The depth was determined considering draft and underkeel allowance.
Crown Height	+5.0 m	The crown height is adjusted to the ground level behind the wharf.

The concrete block type is selected for wharf structure.

3) Basin

The basin area is designed for the small craft enabling to turn in the basin. The turning area is a circle, with a diameter of 100 m (1.5 times of ship length). When the number of small craft increase in the future, the basin area is able to be expanded further inner side of the basin.

Area: 100 m x 90 m Depth: 4.0 m

4) Breakwater

The rubble mound type breakwater, with a length of 95 m, is provided to the northwest side of the basin for the purpose to protect against the northwest waves. The crown height is determined at +6.0 m to prevent the design wave from overtopping on the breakwater.

4.3 Revetment

4.3.1 General

(1) Type of Revetment

Three types of revetment are considered based on the subsoil conditions:

a. Revetment for the port area which is a permanent structure and requires higher durability.

b. Temporary revetment for the onshore dumping area.

c. Temporary bunds for construction at the border of the reclamation area and dumping area which would be filled up.

The type of revetment is illustrated in Figure 4.1.1.

(2) Design Consideration

The revetment of port area is designed as a permanent structure.

As for the temporary revetments and temporary bunds, their details are closely related to the contractor's work sequence. Then, the detailed design is to be prepared by the Contractor and not included in this report.

1) Soft Layer in Port Area

The soft layer has a soft layer of silty clay with high water- contents, high plasticity and low strength. This layer is regarded as not suitable for foundation of the revetment structure which is required to improve its strength or replaced by well graded sand..

2) Soil Improvement Method

As the reclamation area is widely covered by a soft clay layer, it is necessary to carry out foundation improvement work to secure the stable foundation. There are several kinds of soil improvement method such as replacement method, Sand Drain method, Deep Cement Mixing method, Unslaked Lime Pile method and Sand Compaction method.

A comparative study of soil improvement methods is presented in Section 4.4.3.

The replacement method is recommended for the foundation of revetment as according to the result of the comparative study.

It is essential to sufficiently do the pollution prevention measure during this replacement works.

The details of the pollution prevention measure of dredging are described in chapter 8.

4.3.2 Design Conditions

(1) Marine Conditions

1) Tide: Design Tidal Elevations: H.W.L +3.37

M.S.L +1.65

C.D.L ±0.00

L.W.L -0.13

2) Wave: wave height: (H1/3) is 2.2m (probability of 75 year return period)

Period: water period (T 1/3) is 4.5 seconds

3) Slope of sea bed: 1/100

(2) Geological Conditions

This soft layer is generally laid at a water depth of more than –3.0 m and has a thickness ranging from 5.0 m to 10.0 m.

Mechanical properties:

Silty clay:	N Value < 4
Cohesion Cu=	5.0 kN/m^2
Unit weight moistured:	r = 14 kN/m^3
Unit weight submerged:	r' = 4 kN/m^3

(3) Mechanical properties of Imported Materials

1) Replaced Sand Layer (after soil improvement)

N Value:	> 10
Unit weight moistured:	r = 18 kN/m^3
Unit weight submerged:	r' = 10 kN/m^3
Angle of internal friction:	□ = 30°

2) Filling Sand

Unit weight moistured:	r = 18 kN/m^3
Unit weight submerged:	r' = 10 kN/m^3
Angle of internal friction:	□= 30°

3) Rubble stone foundation

Unit weight moistured:	r = 18 kN/m^3
Unit weight submerged:	r' = 10 kN/m^3
Angle of internal friction:	□= 40°

4) Concrete Structure

Unit weight:	rc = 22.6 kN/m^3

(4) Seismic Condition

Seismic coefficient: Kh = 0.20

(5) Loading Condition

Surcharge

Normal condition: 10 kN/m²

Seismic condition: 0 kN/m²Surcharge

(6) Coefficient of Friction

Coefficient of friction for stability examination of sliding (Concrete and rubble foundation): 0.6

4.3.3 Study of Revetment Structure

(1) Structural Types

The structural types of revetment can be generally divided into three types, i.e. vertical type, sloped mound type, and combination type.

The vertical types consist of stone masonry type, concrete block type, concrete wall type, L-shaped concrete block type, caisson type, steel sheet pile type, etc.

On the other hand, sloped mound types consist of stone type, concrete block type, concrete covering type, rubble mound type and, concrete block.

The combined type is a combination with structural types mentioned.

The following five (5) types are selected in the study, as it is widely experienced in past port development projects.

Type 1	Rubble Mound Type
Type 2	Concrete Block Type
Type 3	L-shaped Concrete Block Type
Type 4	Concrete Caisson Type
Type 5	Steel Sheet Pile Type
Type 6	Steel Sheet Pile cellular cofferdam Type

The standard cross sections of these alternatives are shown in Figures 4.3.1 and 4.3.2.

(2) Selection of Proposed Structural Type

The results of comparison of the respective structural types are presented in Table 4.3.1.

For the reasons given below, the rubble mound type was selected as the most recommendable type for the revetment structure.

1) Rubble Mound Type

The rubble mound type is the most rational structure and can be used as a temporary road for filling material transportation even at partial completion of the revetment.

2) Concrete Block Type & L-shaped Concrete Block Type

The concrete block type and L-shaped concrete block type are not recommendable for this project for the following reasons;

a) The concrete block type structures have high possibility of being affected by earthquake as the seismic coefficient used is as big as 0.2.
b) The concrete block type has many sections that are not strongly resistant to earthquake.
c) A large number of concrete blocks have to be manufactured resulting in a longer total construction period.
d) A considerably large area for concrete block production and storage is required.
e) Heavy construction equipment is required for placing and loading of concrete blocks.

3) Concrete Caisson Type

The revetment structure is composed of a main unit made of concrete caissons with a rubble mound foundation.

The manufacturing of caissons requires a construction plant such as dry dock and floating dock, concrete mixing plant, etc. as well as a large material storage yard.

For construction of concrete caissons several special facilities shall be provided at the project site. The main production facilities of concrete caissons shall be brought from abroad. Moreover, the installation of caissons and transportation will require tugboat, crane barge and sand carrier.

The greater part of the revetment is located in shallow areas where the deepest parts are -6 m. More economical alternatives can be applied for such shallow areas. Accordingly, it is not warranted to use this type of structure for the Project.

4) Steel Sheet Pile Type

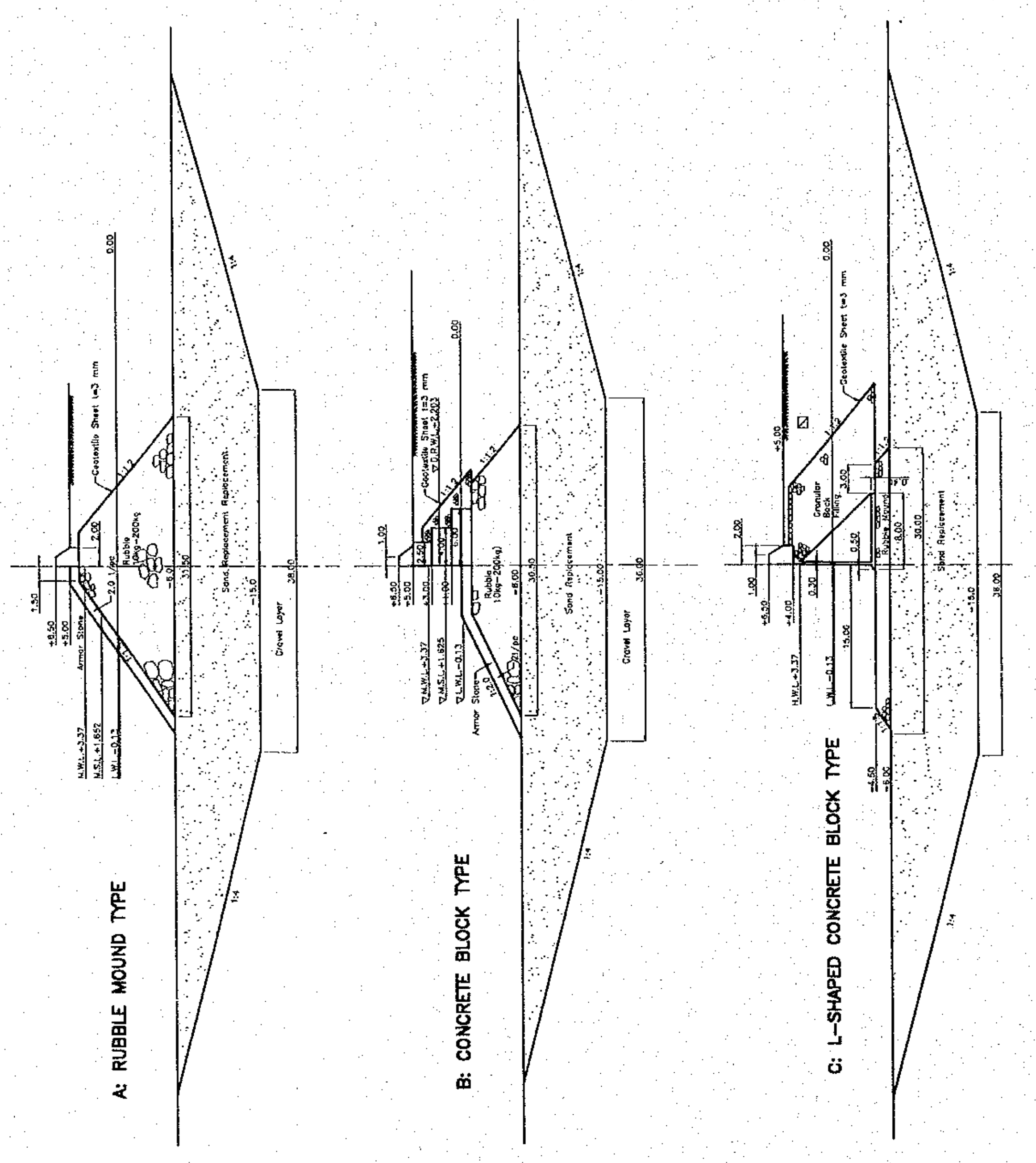
This type has a disadvantage of potential corrosion of the steel sheet piles and most of materials should be imported from abroad. The construction cost will be higher than other revetment types.

The steel sheet pile type is not recommended for the revetment structure of the Project.

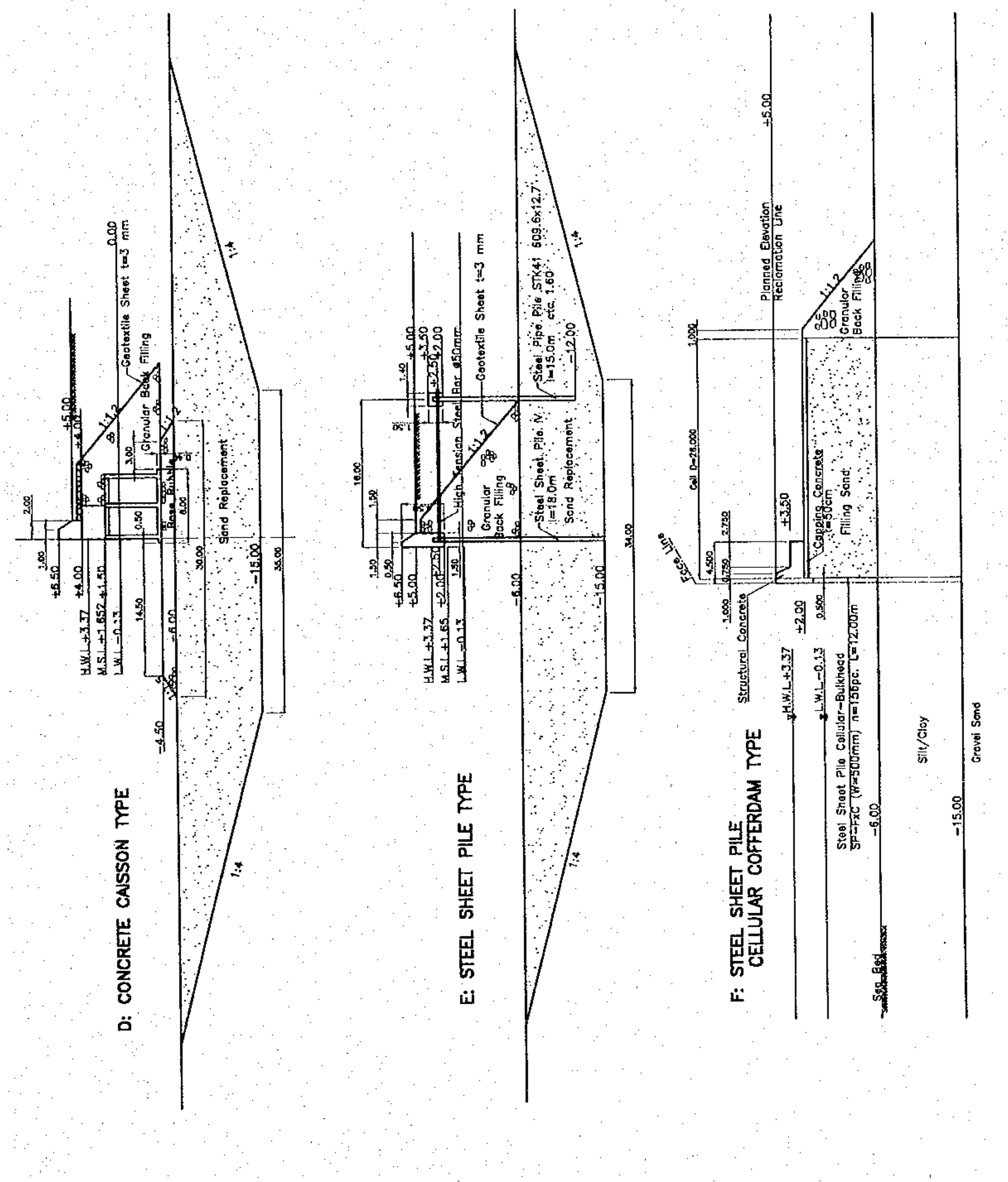
5) Steel Sheet Pile cellular cofferdam Type

This type is not necessary to carry out foundation improvement work to secure the stability of Revetment.

This type has a same disadvantage of the Steel Sheet Pile Type.



Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.3.1	REVETMENT TYPE (1)



Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.3.2	REVETMENT TYPE (2)

Table 4.3.1 Comparison of Revetment Types

Description	Revetment Types					
	A	B	C	D	E	F
Name	Rubble Mound Type	Concrete Block Type	L−shaped Concrete Block Type	Concrete Caisson Type	Steel Sheet Pile Type	Steel Sheet Pile Cellular Cofferdam Typ
Characteristics of the Structur	1) There is no corrosion of the rubbl mound. 2) Whole materials are procured focall 3) Soft soil replacement of the groun(is required to avoid different settlemen 4) After construction of rubble mounc it can be immediatlly used as a road fc fill material transportation o 5) Slope of rubble mound reduce ove topping wave of the revetmen 6) Protection sheet is required to avoi(loss of filled sanc	1) There is no corrosion of th concrete block revetmen 2) Soft soil replacement is requirec to avoid different settlement of th ground. 3) Wave overtopping is large due tc vertical wall under strong gal 4) Rubble foundation is require to avoid different settlement of th structure and circular failur 5) Big size concrete block is requirec to avoid seismic damage 6) Protection sheet is required tc avoid loss of filled sanc	1) There is no corrosion of th L-shaped concrete blocl 2) Soft soil replacement is requirec to avoid different settlement anc circular failurc 3) Rubble foundation is requirec 4) Sand proof screen is reqired fc protection of filled sanc	1) There is no corrosion of the caisso 2) Production plant of caisson is to b mobilized from abroa 3) Soft soil replacement is required tc avoid different settlemen 4) Rubble foundation is requirec 5) Sand proof screen is required fc protection of filled sanc	1) There is corrosion of steel shee pile 2) Steel sheet pile need to be importec 3) Soft soil improvement is requirec for horizontal resistance strengthin 4) There is no settlement 5) Offshore construction equipmer is to be mobilized from abroa 6) Corrosion prevention is required fc maintenance of steel sheet pile	1) There is corrossion of steel sheet pile 2) Steel sheet pile needs to be imported 3) There is no settlement. 4) Soft soil improvement is not necessar 5) Offshore construction equipment is t mobilized from abroad 6) Corrossion prevention is required fc maintenance of steel sheet piles
Construction Property	1) Construction is easy and rapid withou special construction equipmen 2) No fabrication yard is required bu material stockpile yard is require 3) Stability of rubble mound is good during constructior 4) There are many actual result	Fabrication yard of concrete block is requirec 2) Floating crane is required tc install concrete block 3) Construction period is lon compared with rubble mound typ due to construction of concret blocks	1) Fabrication yard of L-shaped concrete block is requirec 2) Floating crane is required tc install L-shaped concrete bloc 3) Construction period is lon compared with rubble mound typ due to construction of L-shape concrete block	1) Stability of the caisson is good during constructior 2) Caisson fabrication yard or floatin dock is requirec 3) Construction period is long due tc preparation of caisson production pla	1) Stability of the wall is unstable until sand filling completior 2) Embedded length of steel sheet pile is upper to layer of gravel sand 3) Material procurement needs preparation period due to steel shee pile productior	1) Stability of the wall is unstable until sand filling completion 2) Embedded length of sheet pile is upper to layer of gravel sand 3) Material procurement period due t steel sheet pile production 4) Fabrication yard of steel sheet pil cellular is required 5) Floating crane is required to install ste sheet pile cellular.
Cost Index	Low 1.00	High 1.48	Middle 1.12	Very High 1.70	Very High 1.91	Very High 2.00
Evaluation	Excellent (Recommended)	Fair	Good	Fair	No Good	No Good

4.3.4 Proposed Structure

The northeast and southwest revetment will be affected by the wave action, thus the slope of revetment surface shall be protected with armor stone.

(1) Computation of Armor Stone

The weight of armor stone is calculated by Hudson 's formula as follows:

$W= (\square r * H^3) / Ns^3 * (Sr - 1)^3$

Where:

W:	Required weight of armor stone (t)
□r:	Unit weight of rocks (t/m^3)
Sr:	Specific gravity against sea water (□r/□o)
(□o)	Unit weight of sea water (1.03 t/m^3)
H:	Wave heigh (m)
	Northeast revetment : 2.2 m (return period 50 years)
	Southwest revetment : 1.9 m (return period 50 years)
Ns:	Coefficient depending on shape, inclination and damage rate of armor stone
	Ns^3= Kd cot□
	(□): Degree (°) between slope face and horizontal line
	Cot□= 1 / tan□= 1.5
	Kd: from the Shore Protection Manual of Coastal Engineering Research Center, Part on sloped mound: 4.0
	Ns^3= 4.0 * 1.5= 6.00
	W (northeast)= $(2.6 * 2.2^3) / 6.00 * (2.6 / 1.03 - 1)^3 = 1.3$ t
	W (southwest)= $(2.6 * 1.9^3) / 6.00 * (2.6 / 1.03 - 1)^3 = 0.8$ t

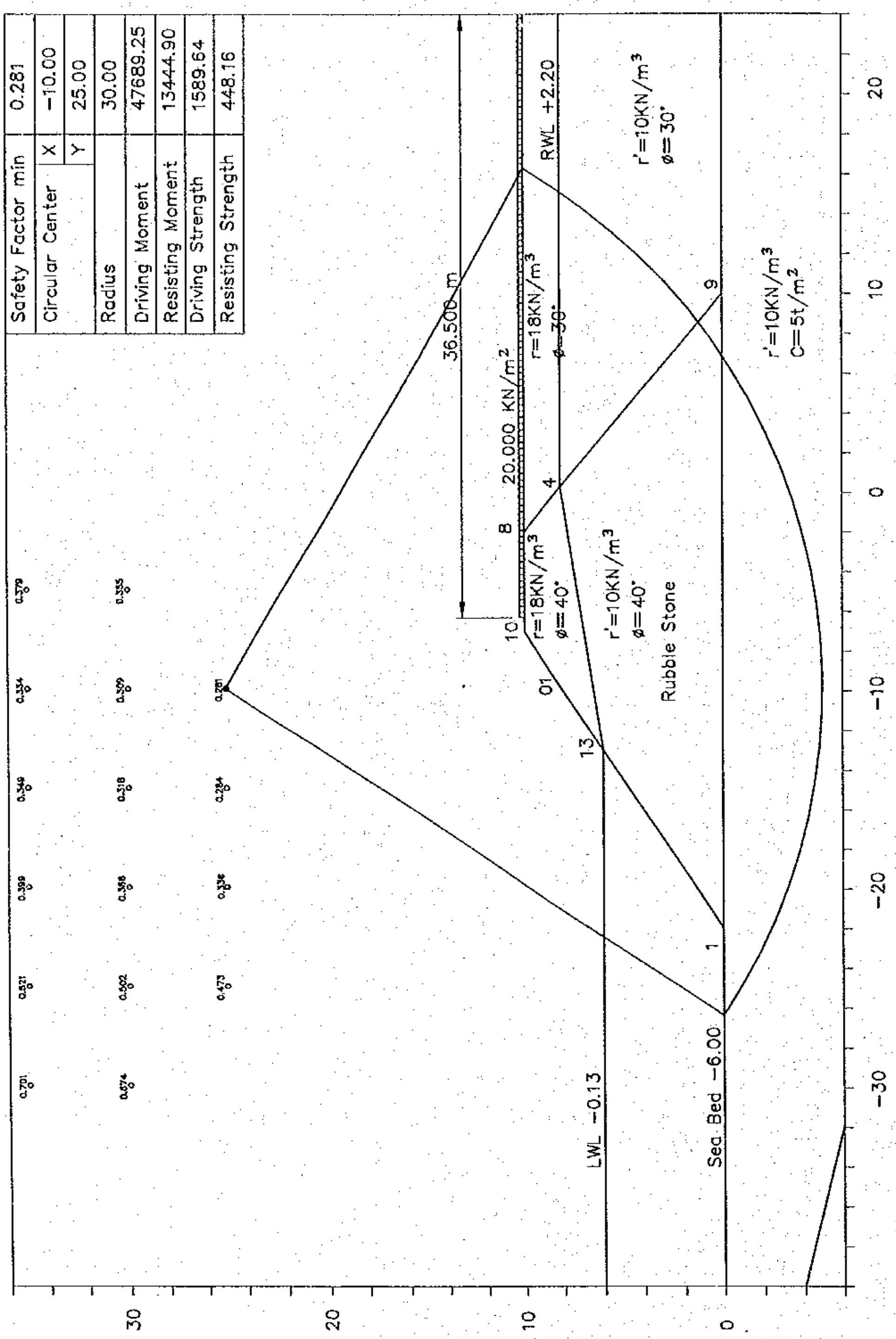
(2) Slope Stability of Rubble Mound Revetment

According to the calculation result of circular failure by Bishop method, the rubble mound revetment slides due to the driving moment of embankment where the clay and silt layer under the seabed as shown in Figure 4.3.3 (Maximum safety factor = 0.281 < 1.0)

On the other hand, when the replacement method is applied to improve the foundation stability as shown in Figure 4.3.4, minimum safety factor is 1.46 and more than 1.0.

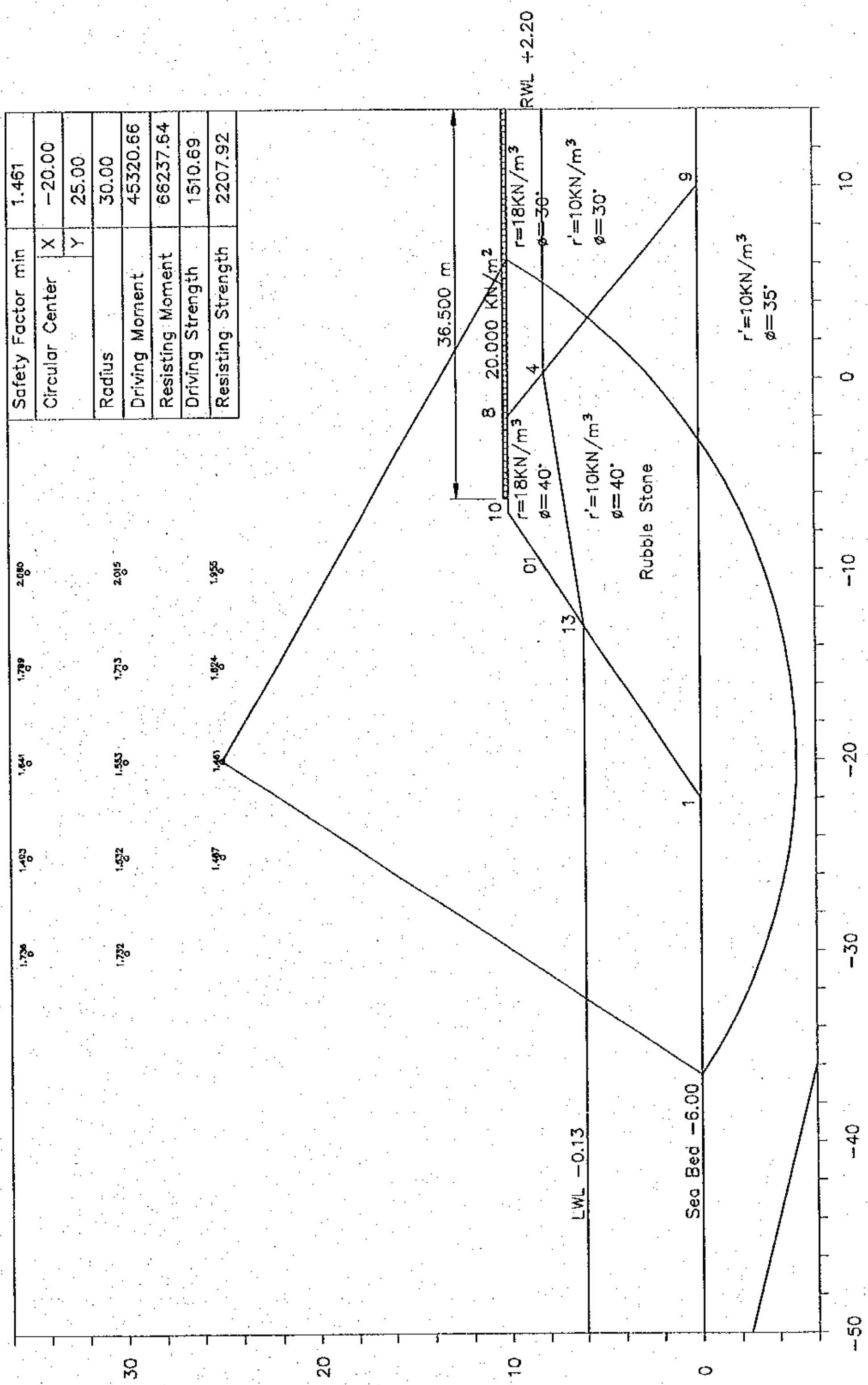
(3) Proposed Structure

Figure 4.3.5 shows typical cross sections of the recommended revetments.

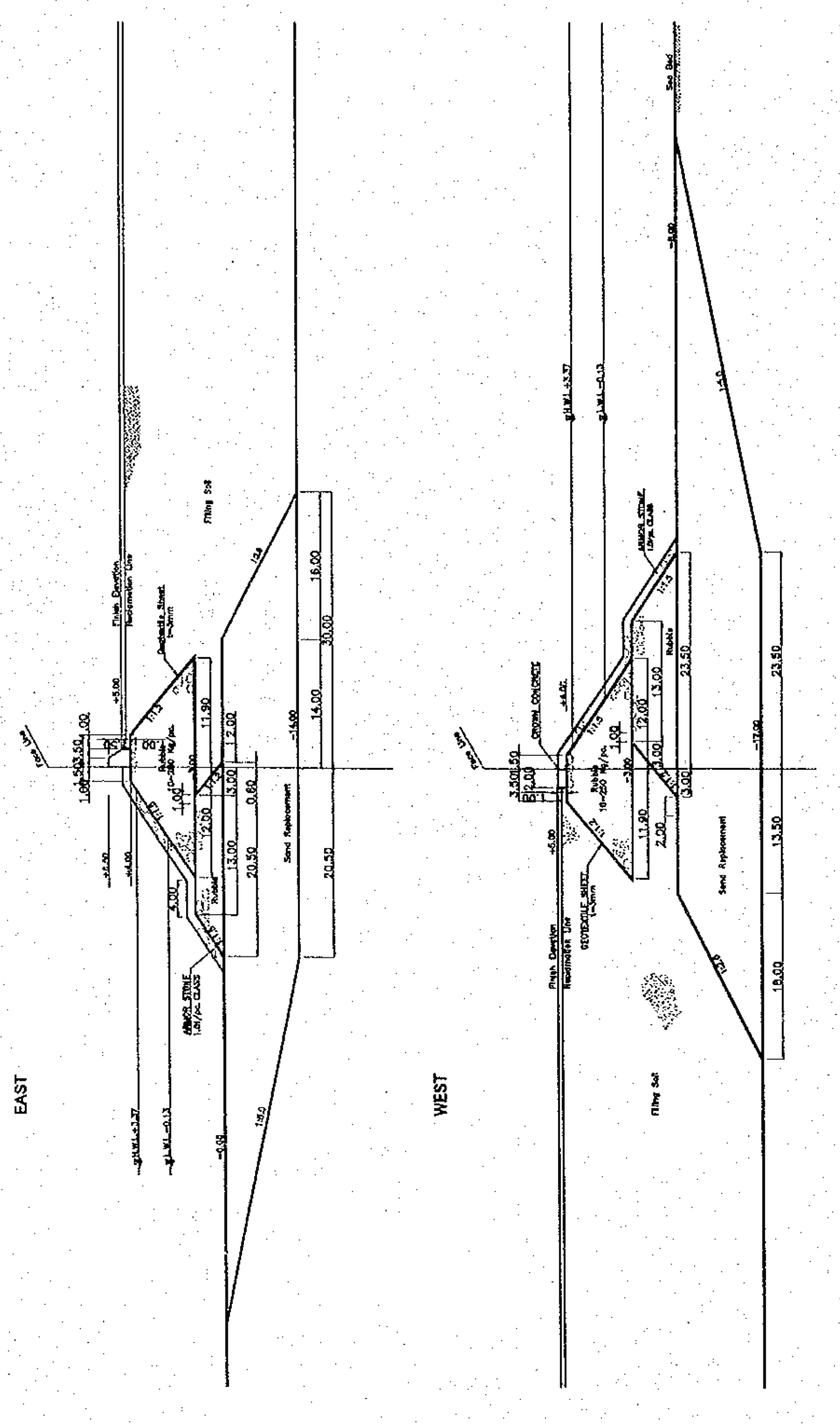


Safety Factor min		0.281
Circular Center	X	-10.00
	Y	25.00
Radius		30.00
Driving Moment		47689.25
Resisting Moment		13444.90
Driving Strength		1589.64
Resisting Strength		448.16

Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.3.3	SLOPE STABILITY OF RUBBLE MOUND REVETMENT(ORIGINAL SOIL)



Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.3.4	SLOPE STABILITY OF RUBBLE MOUND REVETMENT(IMPROVED SOIL)



Japan International Cooperation Agency (JICA)	Figure	Title
The Detailed Design on Port Reactivation Project in La Union Province of the Republic of El Salvador	4.3.5	TYPICAL CROSS SECTIONS OF REVETMENT EEST AND WEST