CHAPTER 5 DESIGN OF BREAKWATER

5. DESIGN OF BREAKWATER

5.1 Existing Breakwater

5.1.1 History of Existing Breakwater Development

Historically, the breakwaters of Tanjung Priok have been constructed stage wisely as briefed below.

1) **Stage-1**: Dam Barat and Dam Timur, the first breakwaters for Tanjung Priok Port, were constructed between 1877 and 1882 under the Dutch rule to protect inner harbor. The opening between offshore tips of these breakwaters is served as west gate to the port. The breakwater construction continued to expand the port basin in east direction.

2) **Stage-2**: Dam Tengah was constructed between 1915 and 1920 under Dutch rule, to connect Dam Timur in front of Port Basin I and II.

3) **Stage-3**: Dam Citra was constructed in 1961 about 100 m offshore from alignment line of Dam Tengah and in front of the current International Container Terminals.

4) **Stage-4**: Dam Pertamina was constructed in 1972 on the east side of the port area, and the opening between offshore tips of Dam Pertamina is served as a east gate to the port. After construction of Dam Pertamina, no further breakwater development has been implemented in the port and the breakwater alignment remains the same.

The general plan of the breakwaters is shown in Figure 5.1.1 and the cross sections of the individual breakwaters are shown in Figure 5.1.2 to Figure 5.1.6 (all the data on the existing breakwaters were supplied by IPC-II). The relocation plan of existing breakwaters is mainly on such parts as were constructed during the stage-1 and stage-2, which is the initial part of Tanjung Priok Port Development.

Photograph of Existing Dam Timur toward offshore side

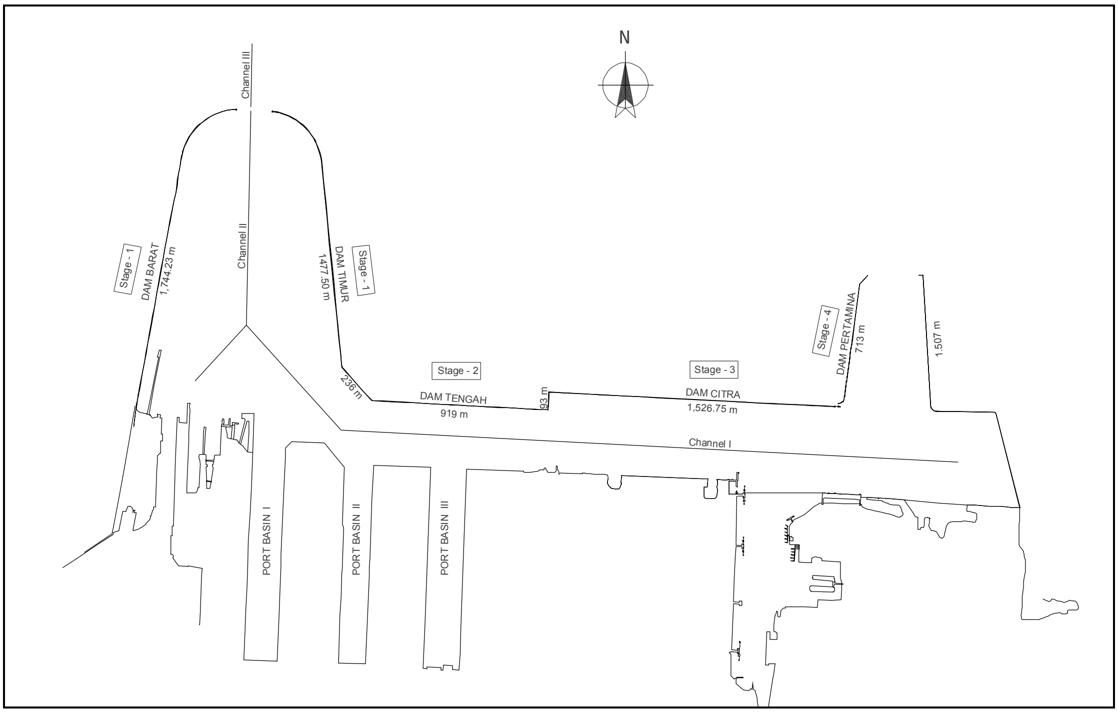


Figure 5.1.1 Existing Port Protection

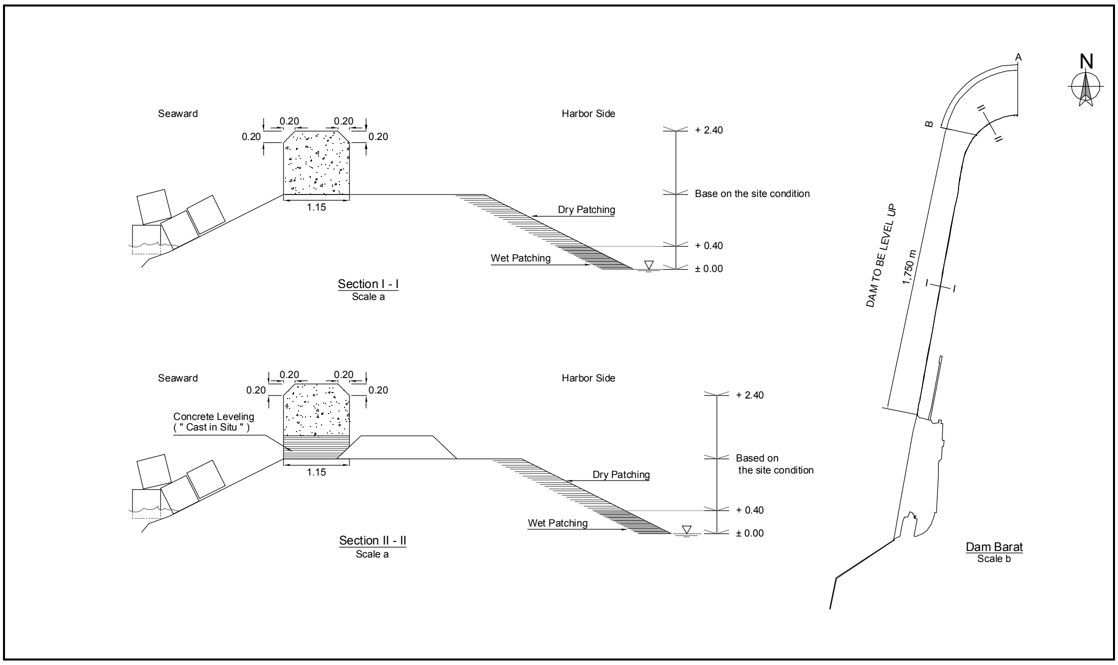


Figure 5.1.2 Plan and Cross Section of Dam Barat

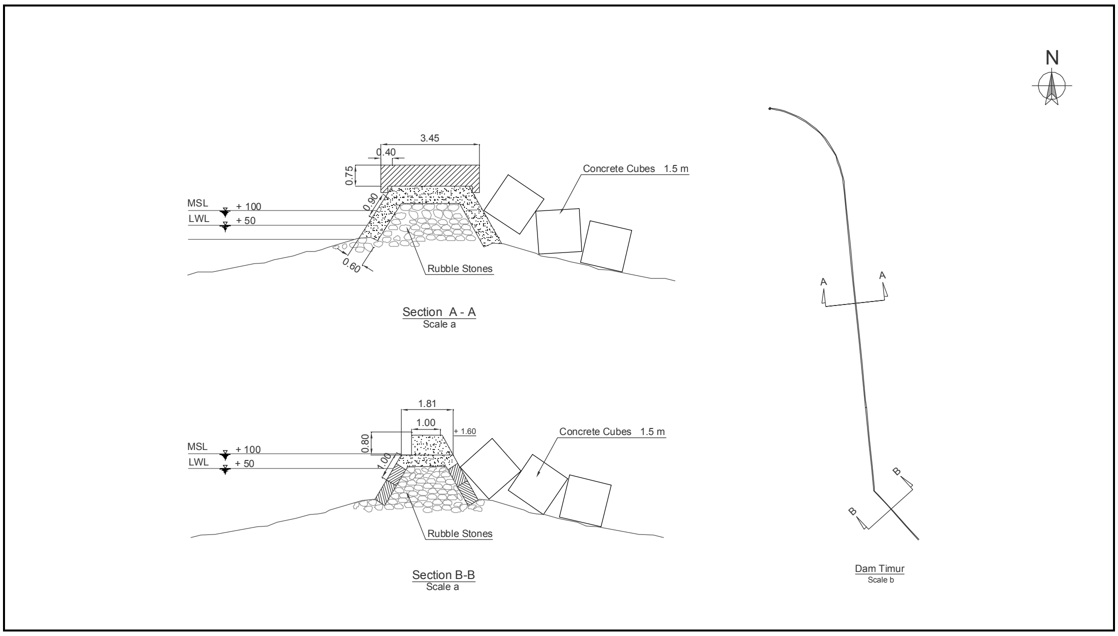


Figure 5.1.3 Plan and Cross Section of Dam Timur

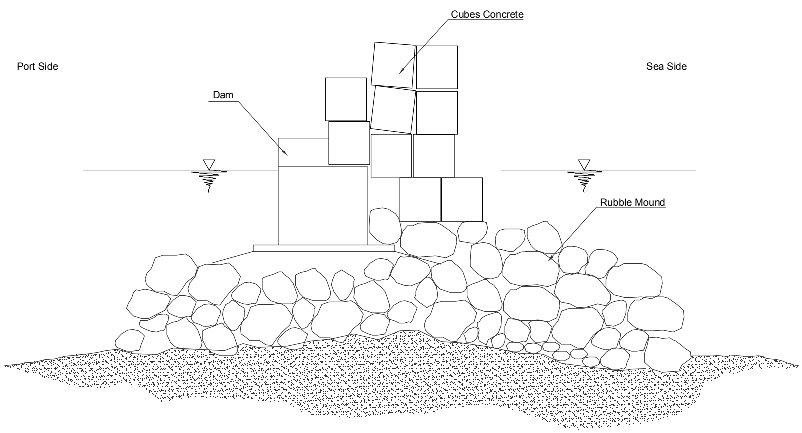


Figure 5.1.4 Cross Section of Dam Tengah

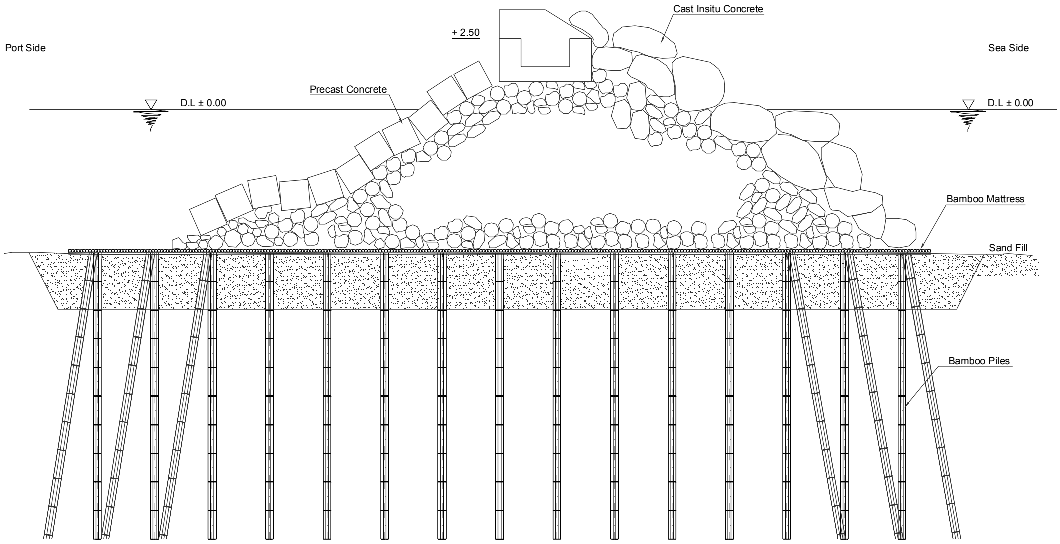


Figure 5.1.5 Cross Section of Dam Citra

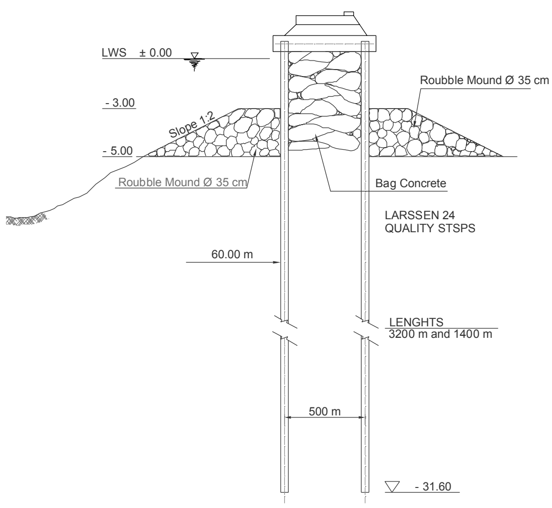


Figure 5.1.6 Cross Section of Dam Pertamina

5.1.2 Review of Existing Breakwater Structures

To review and supplement the above historical information of the existing breakwater structures, the JICA Study Team has conducted the detailed structural investigation by use of seismic tomography. A typical tomography result is shown in Figure 5.1.7 below (Detailed data is included in the Drawings).

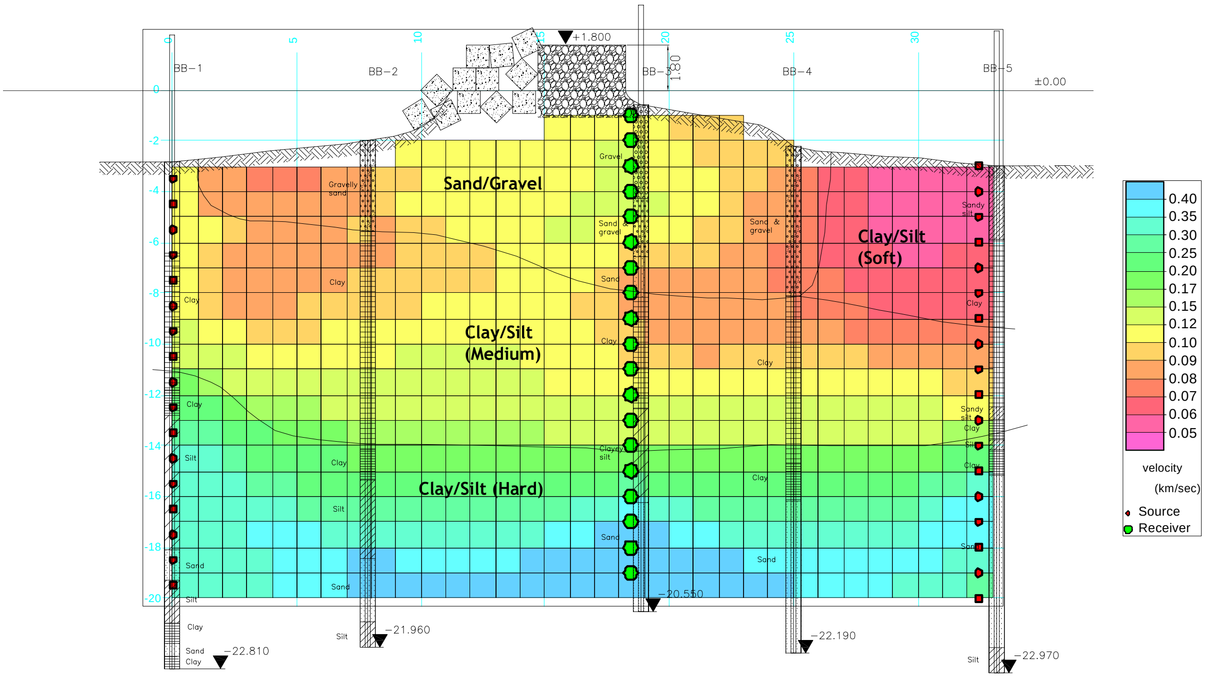


Figure 5.1.7 Tomography at Section T-B (Dam Timur)

Basically, the existing breakwaters are made of two parts, namely sub-structure (foundation) and upper-structure. It is understood that the foundation of the existing breakwater was formed by dumping selected materials like sand and stone that gradually replaced poor in-situ soils. The upper-structure is made of a rubble-mound type. In addition to this structural combination, bamboo-piled foundation with bamboo mattress was also used to support a rubble-mound structure. This bamboo structure, which was applied more recently than natural soil replacement plan, has been serving the role of port protection.

The tomography investigation has disclosed that the naturally replaced foundation materials penetrating down to about 2-3 m above the hard clay/silt layer. The subsoil underlying this foundation zone has been historically consolidated and structurally reinforced, contributing to minimizing further consolidation settlement presently.

Even so, in some parts a little consolidation settlement seems to have continued, requiring continuing maintenance on the top portion of the breakwater by supplying armor stones and concrete blocks, thereby maintaining the planned crest elevation of the breakwater, which is +2.5 m D.L. These site situations suggests that existing breakwater structures, though requiring further improvement and modification, would be good alternative plans for the new breakwater structure, and should be fully considered in the succeeding design process.

5.2 Design Principle of New Breakwater

5.2.1 New Breakwater

This chapter handles the structure design of breakwaters that will be incorporated into the Project. The names and dimensions of the breakwaters are mentioned below and their layouts have been determined in the preceding chapter as shown in Figure 4.4.1.

Breakwater "New Dam Tengah"
New Dam Tengah-1 1,243m
New Dam Tengah-2 318m
Breakwater "New Dam Tengah"100m

5.2.2 Design Standard

"Technical Standards and Commentaries for Port and Harbor Facilities in Japan" has been used for the design of breakwater structures. The design criteria have been taken from the data of the JICA Feasibility Study and the field investigation data collected on site and analyzed by the JICA Study Team. "The Standard Design Criteria for Ports in Indonesia, 1984" has been used to determine the design seismic coefficient for the design of breakwater foundation.

5.2.3 Maximum Usage of Existing Breakwaters Materials

With the relocation of the existing breakwaters, a lot of demolished materials will be generated, including sand, gravel and concrete blocks. It is economical and environment-friendly to reuse these valuable resources.

The tomographic and soil surveys (Refer to 2.6 Geology) were conducted to study the possibility or reuse of existing breakwater materials in construction of new breakwater. The survey results show clear distinction of existing breakwater materials, showing the mixture of sand and stones. These materials are considered reusable as replacement sand. Existing concrete blocks will also be reusable as armor stones in shore side slope protection. Shown below are the estimated volumes of reusable breakwater materials.

Coping Concrete (m3)	Sand and Gravel (m3)	Concrete Block (pcs)
15,300	170,600	20,700

5.2.4 Engineering Site Conditions

(1) Water Depth

For the design of the breakwater, the water depth has been assumed at -4.0m.

(2) Tide

Tides in Tanjung Priok Port are as shown below.

HHWL (Highest High Water Level)	+1.05 m
MHWS (Mean High Water Spring)	+0.91 m
MSL (Mean Sea Level)	+0.48 m
MLWS (Mean Low Water Spring)	+0.09 m
DL (Datum Level = LLWL: Lowest Low Water Level)	0.00 m

(3) Wave

In the JICA Feasibility Study (2003), the design wave heights in front of the new breakwater construction sites were determined based on the wave forecast and wave transformation calculations. (Refer to APPENDIX 5-A) In this study, the design waves have been reviewed based on the offshore wave data obtained from the Royal Meteorological Institute (Refer to APPENDIX 2-B).

Based on the above review work, the design waves of high-and low-frequencies have been determined as shown below. The high-frequency waves are ambient waves. The low-frequency waves are abnormal waves to be used as design waves for structural analysis. The bathymetry around the new breakwater has no significant change, including the tip of the New Dam Barat, so that the design wave has been determined on the condition that the water depth at the both construction sites at Dam Barat and Dam Tengah is almost same, about - 4.0 m.

1) High-frequency wave height is applied as the design wave height to decide the breakwater crest elevation, with consideration given to the permissible rate of wave over-topping for port operations. The high-frequency design wave was determined as follows.

   H1/3 = 1.5 m, T = 6.0s, Direction of Incident Wave: North (1 year return)

2) Low-frequency wave height is applied to examine the breakwater stability. The low-frequency design wave was determined as follows:

   H1/3 = 2.5 m, T = 7.5s, Direction of Incident Wave: North (50 years return)

   H1/3 = 1.5 m, T = 6.0s, Direction of Incident Wave: North-East (10 years return)

(4) Earthquake

Design seismic coefficient was determined as shown below in accordance with the “Standard Design Criteria for Ports in Indonesia” published in 1984 by the Directorate General of Sea Communications, Ministry of Communications, Indonesian Government shown in Figure 5.2.1

Horizontal Coefficient (kh) = Kr x Ki = 0.05 x 1.5 = 0.075 = 0.10

where,

Kr = Regional seismic coefficient = 0.05 (Zone IV, Soft soil)
Ki = Coefficient of importance = 1.5 (Special Class)

When the foundation of breakwater structure consists of loose sand layers, sand liquefaction that occurred in the case of earthquake should be considered.

As shown in Figure 5.2.2, large-scaled earthquake is predicted to occur on the southern coastal areas (facing the Indian Ocean) of Java Island. Tanjung Priok Port is located on the northern coast of Java Island about 200 km away from the seismic–active zone, and the influence of such earthquake is considered small for the seismic damping and subsoil of port area consists of silty clay layer. Under these conditions, sand liquefaction has not been considered in the foundation design.

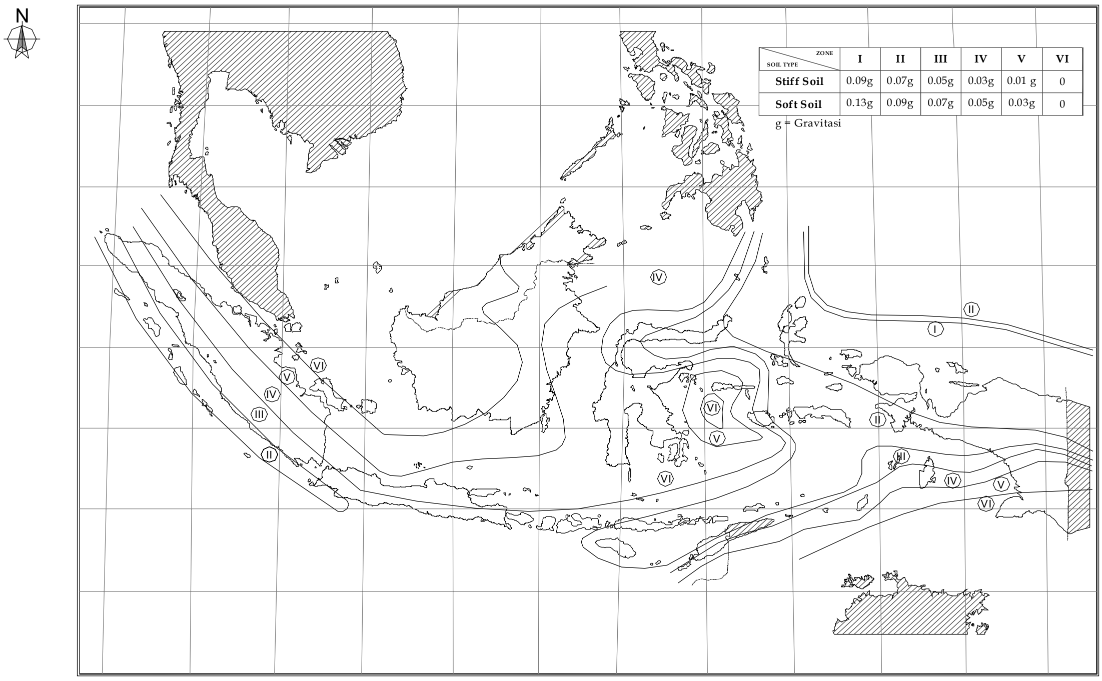


Figure 5.2.1 Distribution Map of Seismic Coefficient

(5) Soil Conditon

Soil conditions for the breakwaters have been set up as follows and more detailed data are shown in APPENDIX 5-C.

Soil Condition of Dam Barat

<u>Elevation -4.00 m</u>

Silty Clay N = 0 $\Phi = 0^\circ$
γt = 1.43 tf/m3 γ' = 0.43 tf/m3 (in water)
C = 0.60 tf/m2
Cc = 0.98
Cv = 0.0028 cm2 / sec.
$m_v = 0.1182P^{-0.897}$

<u>Elevation -15.00 m</u>

Silt N = 44 $\Phi = 0^\circ$
γt = 1.69 tf/m3 γ' = 0.69 tf/m3 (in water)
C = 44 tf/m2

Soil Condition of Dam Tengah

<u>Elevation -4.00 m</u>

Silty Clay N = 0 $\Phi = 0^\circ$
γt = 1.45 tf/m3 γ' = 0.45 tf/m3 (in water)
C = 1.0 tf/m2
Cc = 1.12
Cv = 0.0028 cm2 / sec.
$m_v = 0.1182P^{-0.897}$

<u>Elevation -12.00 m (- 13.00 m)</u>

Silty Clay N = 4 $\Phi = 0^\circ$
γt = 1.69 tf/m3 γ' = 0.69 tf/m3 (in water)
C = 4 tf/m2
Cc = 0.42
Cv = 0.03 cm2 / sec.

<u>Elevation -15.00 m</u>

Silty Clay N = 12 $\Phi = 0^\circ$
γt = 1.69 tf/m3 γ' = 0.69 tf/m3 (in water)
C = 12 tf/m2
Cc = 0.42
Cv = 0.03 cm2 / sec.

5.3 Preliminary Study of New Breakwater Structure

5.3.1 Approach to Breakwaters Design

The approach to the breakwater design is illustrated in Figure 5.3.1. The design starts from the set-up of "Engineering Site Condition", where the design parameters of hydraulic, oceanographic, geological and seismographic conditions have been determined.

On the basis of these design conditions, Comparative Study on Breakwater Structures has been conducted.

As suggested by the existence of poor soil condition, foundation improvement would be imperative for structural requirements of breakwater except for the piled foundation. So, firstly the optimum foundation type has been studied.

Then, "Upper-Structure Type" has been discussed to couple with the selected foundation types, except for the piled structure, which has been designed as an integrated structure without splitting upper-and sub-structures.

Through the above comparative study, the best combination of upper/sub-structure has been chosen for the main section of breakwater which is New Dam Tengah-1, which is not physically restricted in shaping the optimum formation of the structure. In the meantime, the construction of New Dam Barat will be restrained by the closeness to the dredged channel section, so that somewhat modified structure (with Bamboo piles) has been proposed for this part of the breakwater.

The breakwater sections of New Dam Tengah-1 and New Dam Barat have been designed with the life time of 50 years, while the breakwater, which will connect the New Dam Tengah-1 and the existing Dam Citra, has been designed as a temporary structure with its design philosophy slightly modified with a life time of 10 years.

On the basis of the above design concept, the preliminary design was worked out for such structural type as supposes from the existing breakwaters in accordance with the flow chart below.

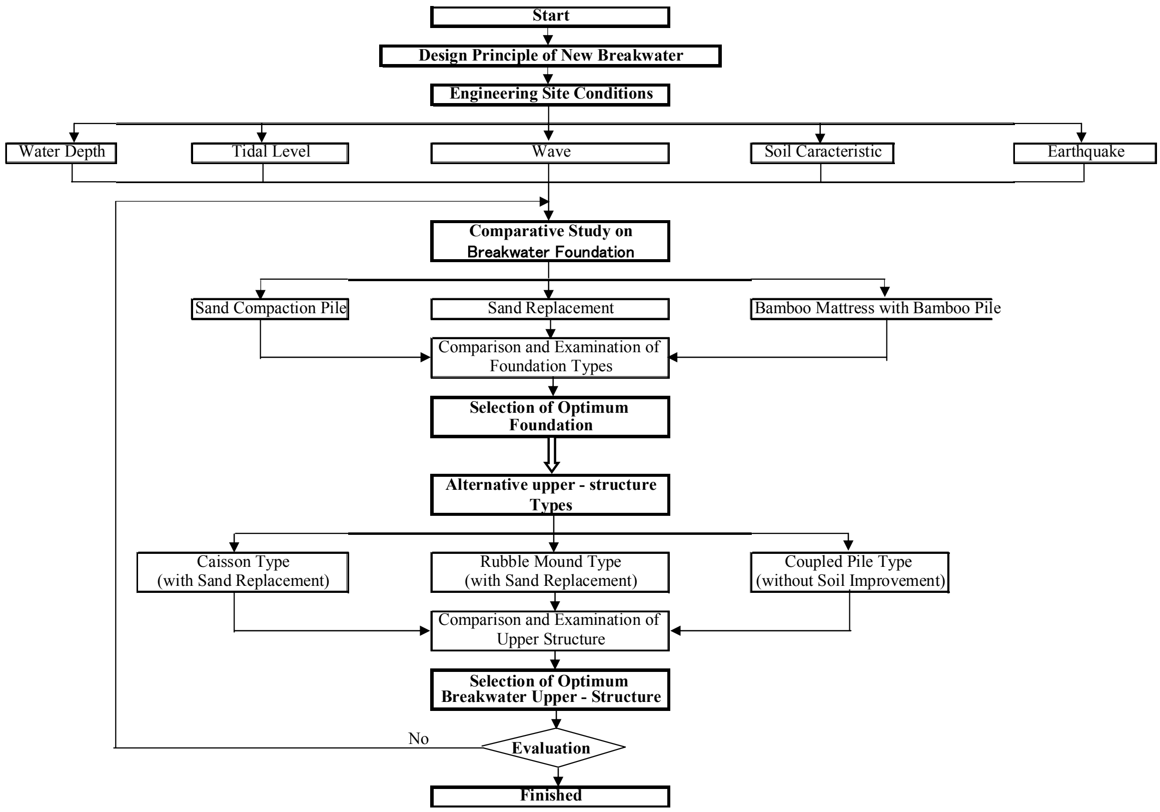


Figure 5.3.1 Flow Chart for Preliminary Design of Breakwater

5.3.2 Selection of Optimum Foundation Types

As explained earlier, the existing soft soil cannot be used as the foundation of the breakwater, so that several soil improvement plans have been evaluated.

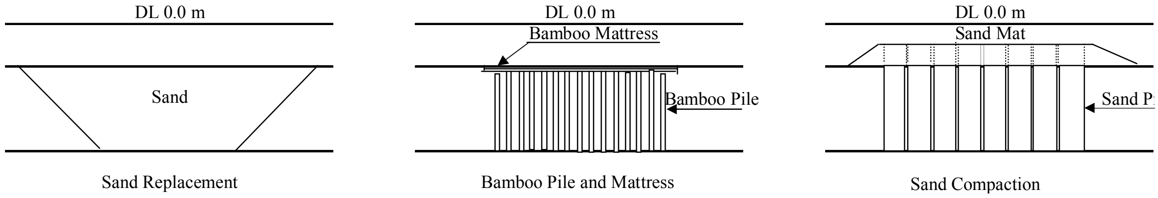
The existing subsoil at New Dam Tengah is composed of very soft silty clay layers (N-value : 0) from seabed to the elevation of –12.0 m or -13.0m, soft silty clay layers (N-value: 4) from -12.0 m or -13.0m to the elevation of -15.0 m. To improve such poor subsoil conditions, three (3) alternative methods are compared and evaluated hereunder. Those alternatives are listed below and comparison results are shown in the Table 5.3.1 and 5.3.2.

1) Sand Replacement Method
Replacement width and depth are determined based on the stability of whole structure through a circular failure analysis and considering the consolidation settlement of the armor layer slope at both sides of breakwater.

2) Bamboo Pile and Mattress Method
Bamboo mattress is used for distributing more uniformly upper load to the bamboo pile. Bamboo pile is acting as a friction pile for preventing the buckling of pile.

3) Sand Compaction Method.
The width and depth of sand compaction are determined based on the stability of whole structure through a circular failure analysis and considering the consolidation settlement of the armor layer slope at both sides of breakwater.

For the design of the breakwater, the water depth has been assumed at -4.0m.



The evaluation of alternative methods for the soil improvement is shown in Table 5.3.1.

Table 5.3.1 Comparison of Alternative Foundation Type (Soil Improvement)

Description	Sand Replacement Method	Bamboo Pile and Mattress Method	Sand Compaction Method
Material	Sand and Reuse of Removal Material	Bamboo for Mattress and Pile, Replacement Sand	Good Quality Sand for Pile and Matt
Equipment	Dredger, Hopper Barge, Tug Boat, Clamshell	Bamboo Pile Driving Barge and Crane Barge, Tug Boat, Dredger, Hopper Barge	Sand Compaction Barge, Hopper Barge, Crane Barge, Tug Boat
Settlement	Small (Depending of Replacement Depth)	Small (Depending on Pile Length) but Big in future	Small (Depending on Improvement Ratio)
Construction Period	Short	Long	Long
Environmental Impact	Bad	Good	Bad
Construction Cost	Low	Low	High

Each of the alternative methods is evaluated with scoring system as shown in Table 5.3.2.

Table 5.3.2 Evaluation of Alternative Foundation Type (Soil Improvement)

Description	Sand Replacement Method	Bamboo Pile and Mattress	Sand Compaction Method
Material	3	2	1
Equipment	2	3	1
Settlement	3	1	2
Construction Period	3	1	2
Construction Cost	3	2	1
Environmental Impact	1	3	2
Total Score	**15 Points**	**12 Points**	**9 Points**
Evaluation Ranking	**1st**	**2nd**	**3rd**

Score given : 3 point for Best, 2 point for Better and 1 point for Good

The above evaluation table indicates that the Sand-Replacement Plan would be the best choice, though, Bambbo-Pile Plan could also be considered the second-best.

The typical cross sections of alternative soil improvement methods are as shown in Figure 5.3.2 to Figure 5.3.4. The cross sections studied has been prepared only for comparison purpose, particularly to compare resulting construction costs and periods on the condition that upper-structure is the same for all alternative foundation plans.

(Sea Side) (Land Side)

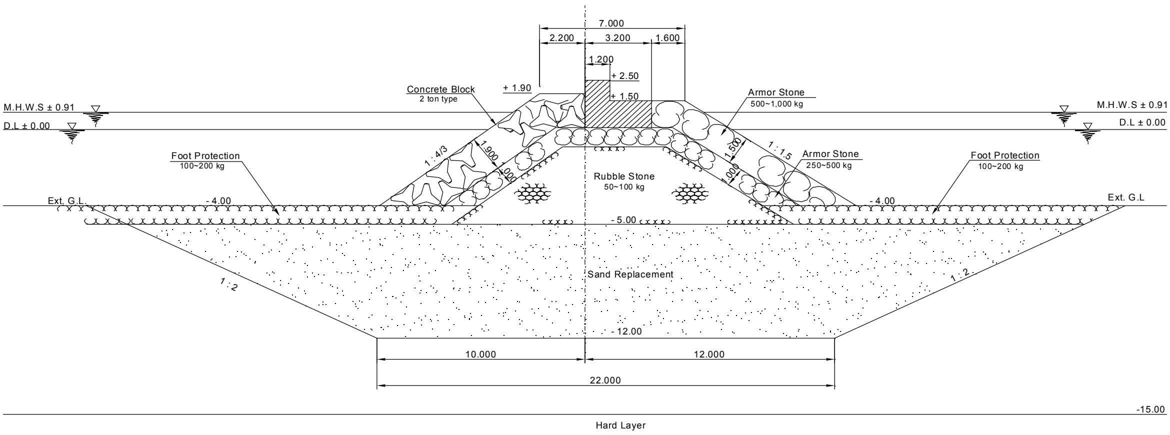


Figure 5.3.2 Sand Replacement

(Sea Side) (Land Side)

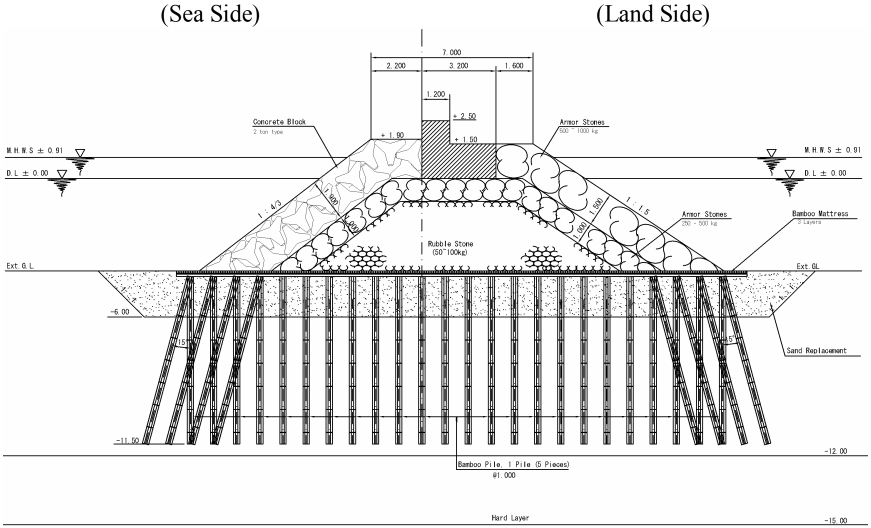


Figure 5.3.3 Bamboo Pile and Mattress

(Sea Side) (Land Side)

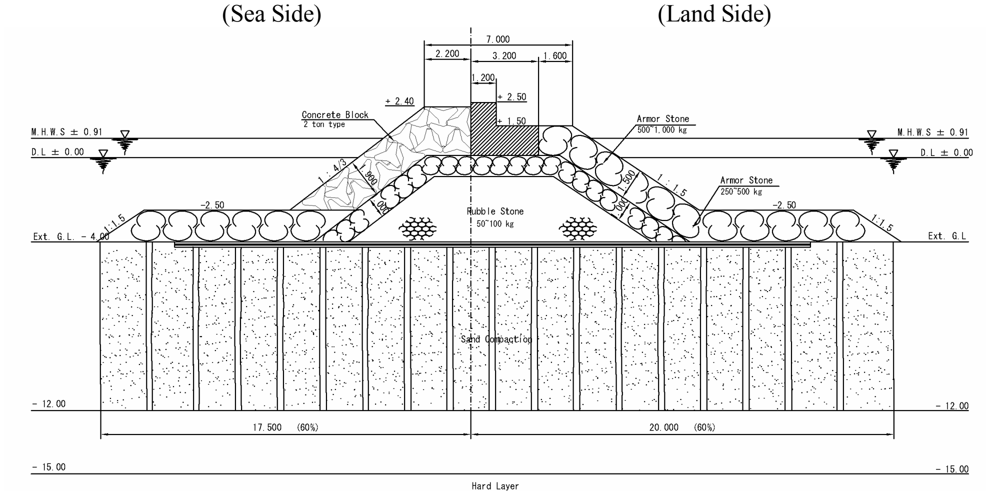


Figure 5.3.4 Sand Compaction

5.3.3 Comparative Study of Breakwater Upper-Structures

The following upper-structural types of the new breakwater have been selected for comparison purpose:

1. Rubble Mound Type with Sand Replacement
2. Caisson Type with Sand Replacement
3. Coupled Pile Type with Sheet Piles

(1) Rubble Mound Type with Sand Replacement

A preliminary structural plan of rubble-mound breakwater with sand replacement has been prepared as shown in Figure 5.3.4. To set up the major dimensions of the structure, the crest elevation and the size of armor stone that govern the crest width, and the total stability of structure, including the soil improvement portion have been analyzed as detailed below.

(Sea Side) (Land Side)

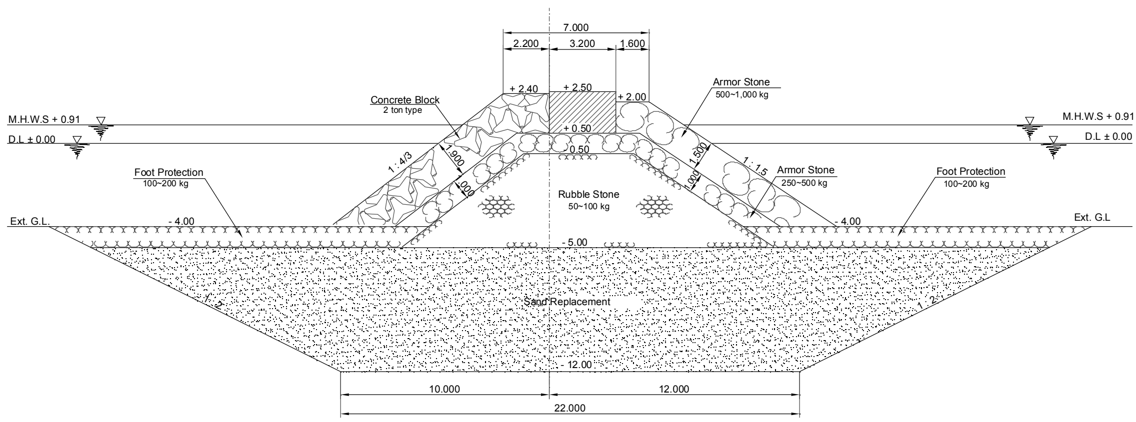


Figure 5.3.5 Typical Cross Section of Rubble Mound Type

<u>**Crest Elevation**</u>

Breakwater crest elevation should be not less than 0.6 times the design significant wave height above the mean monthly-highest water level (MHSL).

$H = M\ H.W.S + 0.6\ H_{1/3} + \alpha$ (allowance)

$= 0.91\ m + 0.6 \times 2.5\ m + \alpha$

$= 2.41\ m + \alpha$

$= 2.50\ m$ (D.L.)

<u>**Concrete Blocks and Armor Stones**</u>

The mass (weight) of concrete blocks required to cover the front slope of the breakwaters is calculated by the following formula:

$M = \rho\gamma H^3/[Ns^3(S\tau - 1)^3]$

Where,

M: minimum mass (weight) of concrete blocks (ton/unit),
$\rho\gamma$: density of concrete blocks (tons/m^3),
H: design wave height (m),
Ns: stability number,
$S\tau$: specific gravity of concrete blocks relative to seawater.

Assuming that the front slope is 1:4/3 and concrete blocks (KD = 8.3, no breaking wave) are used as armor layer of the breakwaters, the parameters in the above formula were determined as follows. The design wave height with a return period of 1/50-year is applied to this calculation.

$P\gamma$ = 2.3 tons/m^3,

$H_{1/3}$ = 2.5 m,

Ns^3 = K_D*cot α = 8.3*4/3 = 11.07,

$S\tau$ = 2.3/1.03 = 2.23

M = $2.3*2.5^3/[11.07(2.23-1)^3]$ = 1.75 (ton)

Concrete blocks (2-ton type) will be used for armor layer of the breakwaters. Armor stones, each weighing from 500 kg to 1,000 kg, will be used for shore side slope protection.

Stability of Rubble Mound Structure

Stability of rubble mound structure has been calculated for circular failure, and the minimum safety factor = 1.382 > safety factor = 1.30 has been obtained as shown in Figure 5.3.6.

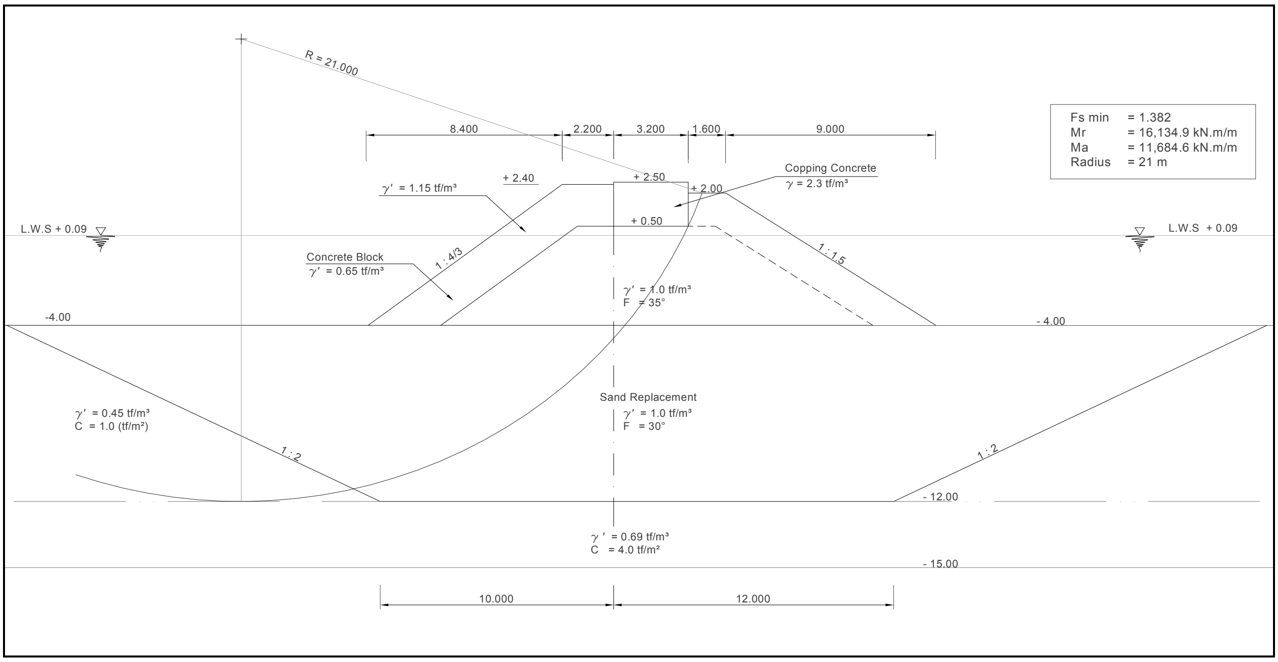


Figure 5.3.6 Circular Failure Diagram of Rubble Mound

(2) Caisson Type with Sand Replacement

A preliminary structural plan of caisson breakwater (5.5 m wide and 5.0 m high, including coping concrete) is shown in Figure 5.3.7, and their structural basis are described as below, including stability for circular failure, sliding and bearing of its foundation.

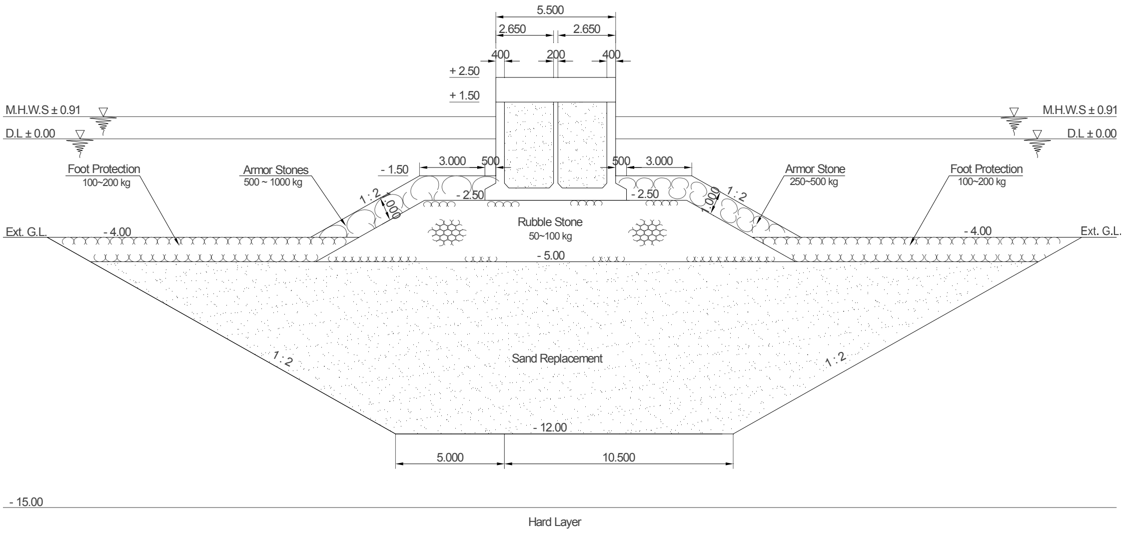


Figure 5.3.7 Typical Cross Section of Caisson Type

<u>Stability of Caisson Structure for circular failure.</u>

The circular failure diagram is shown in Figure 5.3.8.

Result of circular failure is as below.
Minimum safety factor = 1.367 > safety factor = 1.30

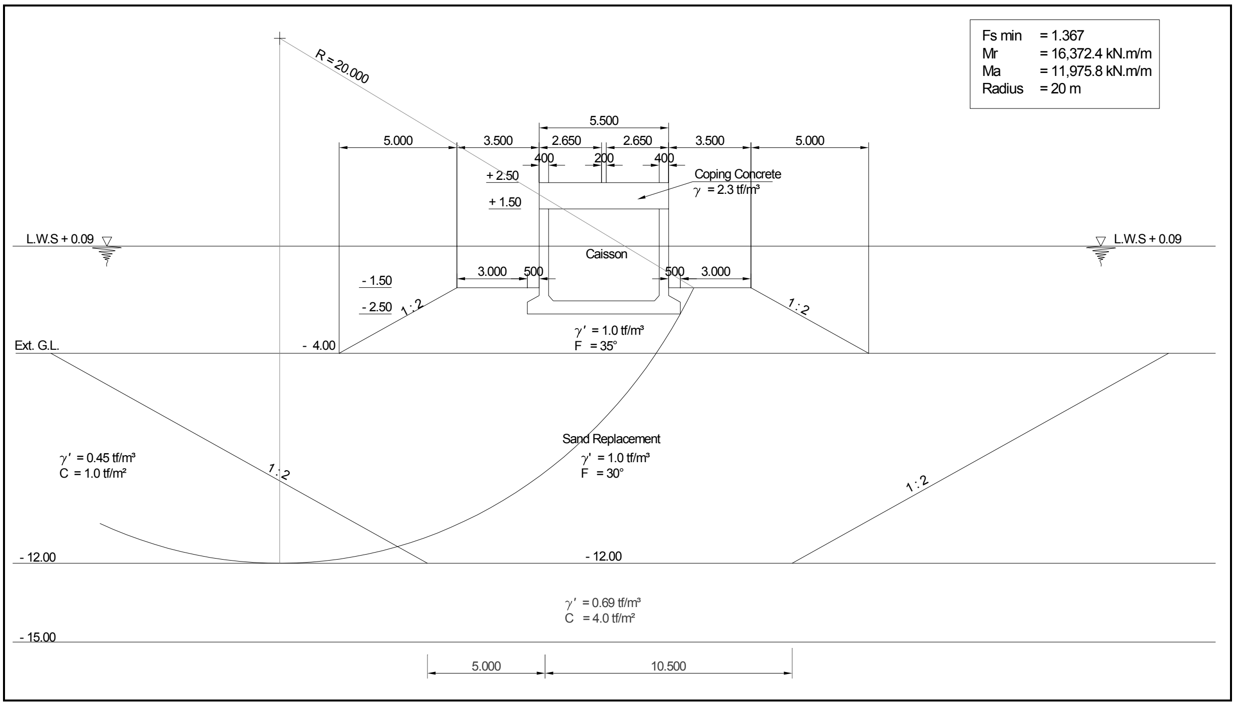


Figure 5.3.8 Circular Failure Diagram of Caisson.

Stability for Sliding, Turning and Bearing

The calculation process for the following outputs are detailed in Appendix 5.D.

Stability	Self Wight (KN/m)	Wave Pressure (KN/m)	Uplift (KN/m)	Mr (KN·m/m)	Ma (KN·m/m)	Mr (KN·m/m) Uplift	Safety Factor	Allowable Safety Factor
Sliding	415.29	171.52	66.88	-	-	-	1.22	> 1.20
Turning	-	-	-	1,349.7	400.21	- 278.67	2.68	> 1.20

Ground reaction (surface of mound)

Ra = 154.85 KN/m2 < 617.5 KN/m2

Ground reaction (surface of seabed)

Ra = 106.44 KN/m^2<389.75 KN/m^2 (with soil improvement)

Stability of caisson during earthquake is not studied because of seismic force (H = 41.53 KN/m) is smaller than intensive wave pressure (171.52 KN/m).

Required Weight of Rubble Stones for Toe Protection

The required weight of rubble stones for the toe protection is as follow.

H1/3	H (m)	D (m)	d/h (m)	Weight (kg)
2.5	5.91	3.41	0.58	500

Armor stones, each weighing from 500 kg to 1,000 kg, will be used for toe protection of the breakwater.

(3) Coupled Pile Type with Sheet Pile

P.C. Sheet-Pile supported by combination piles (vertical + batter) withstands the wave force. The pile-head is capped by reinforced concrete. To prevent erosion at the foundation, armor stones are placed on the bamboo-mattress on the seabed. The preliminary structural plan of coupled-piled breakwater is shown in Figure 5.3.9 and their structural basis are summarized below, and the detailed calculation are attached in APPENDIX 5-E.

(Sea Side) (Land Side)

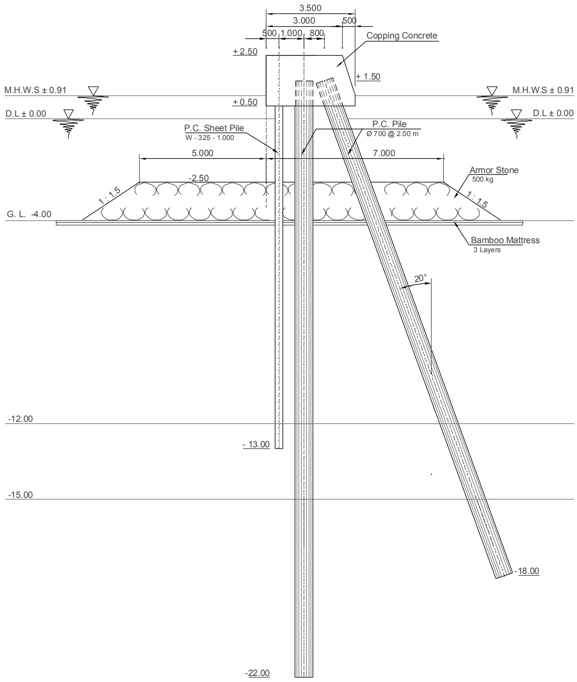


Figure 5.3.9 Typical Cross Section of Coupled Pile Type

Breakwater structure of coupled pile type consists of combined vertical and batter PC piles with PC sheet piles.

Analysis of Section Force

Wave pressures and intensity wave pressures refer to APPENDIX 5-B.

Analysis results of section force are shown below.

Pile	H (KN/m)	V (KN/m)	Axial Force (KN/m) (KN/Pile)	Bending Moment (KN-m/m)
PC Pile（Batter）	120.75	144.65	390.78 (781.56)	-
PC Pile (Vertical)	120.75	144.65	-226.59 (-453.18)	-
PC sheet pile	120.75	-	-	86.89

Section force during earthquake is not studied because seismic force (H = 14.5 KN/m) is smaller than intensive wave pressure (120.05 KN/m).

Selection of PC Pile and PC Sheet Pile

Required PC Pile: φ700 A Type @ 2.0m (allowable axial force is 1,260.0 KN/Pile > 781.56 kN/m).

Required PC Sheet Pile: W-325-A-1000 (allowable cracking moment is 110.82 KN-m/m > 86.89 kN-m/m).

Embedded Pile Length

Embedded pile lengths are as follows.

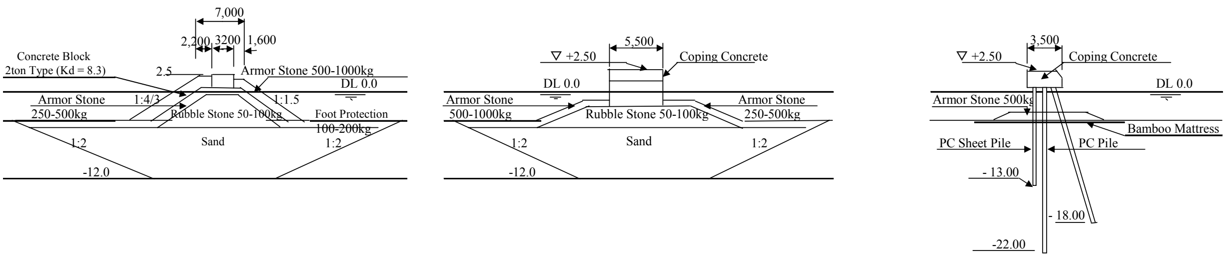
	Elevation of Pile Tip (m)	Bearing Capacity (KN/pile)	Axial Forth (KN/pile)	Safety Factor (F)	Allowable (Fa)
Batter Pile	-18.00	3,559.6	781.56	4.55	>2.50
Vertical Pile	-22.00	1,474.0	- 453.18	*3.24	> 3.00

* critical factor

5.3.4 Selection of Optimum Breakwater Upper-Structure

(1) Comparison of Alternative Upper-Structural Types

The comparison of alternative structural types for the new breakwaters is shown below and summarized in Table 5.3.3.



Rubble Mound Type   Caisson Type   Coupled Type

Table 5.3.3 Comparison of Alternative Upper- Structural Types

Description	Rubble Mound Type	Caisson Type	Coupled Pile Type
Structure Type	Simple Structure Good Permanence Popular in Indonesia	Complicated Structure Good Permanence Scarce in Indonesia	Complicated Structure Meager Permanence Scarce in Indonesia Problem to Analysis
Required Construction Materials	Concrete Block, Rubble Stone, Armor Stone, Reusable of Demolished Material	Caisson, Rubble Stone, Armor Stone	PC Piles, PC Sheet Pile, Armor Stone
Required Construction Equipment	Floating Crane, Barge, Clamshell, Tug Boat, Diving Boat	Floating Dock, Floating Crane, Barge, Clamshell, Tug Boat, Diving Boat	Pile Driving Barge and Vibro Hammer , Floating Crane, Barge, Clamshell, Tug Boat, Diving Boat
Construction Method	Easy (Equipment And Material)	Difficult (Equipment and Manufacturing and installation of Caisson)	Difficult (Piling Barge and Treatment Pile Head)
Correctness	Not Much Required	Required	Required
Construction Period	Long	Long	Short
Countermeasures for Settlement	Easy	Easy	Not Necessary
Maintenance	Easy	Easy	Difficult
Construction Cost	1.800 (Million \/m)	2.031 (Million \/m)	1.985 (Million \/m)

(2) Evaluation of Alternative Structure Types

Each of the alternative structural types is evaluated with scoring system as shown in Table 5.3.4.

Table 5.3.4 Evaluation of Alternative Upper-Structural Types

Description	Rubble Mound Type	Caisson Type	Coupled Pile Type
Structure Type	3	2	1
Required Construction Material	3	2	1
Required Construction Equipment	3	1	2
Reuse of Existing Breakwater Materials	3	2	1
Construction Method	3	2	1
Correctness	3	2	1
Construction Period	1	2	3
Countermeasure of Settlement	2	2	3
Maintenance	3	3	1
Construction Cost	3	1	2
Total Score	**27 Points**	**19 Points**	**16 Points**
Evaluation Ranking	**1st**	**2nd**	**3rd**

Score given : 3 point for Best, 2 point for Better and 1 point for Good

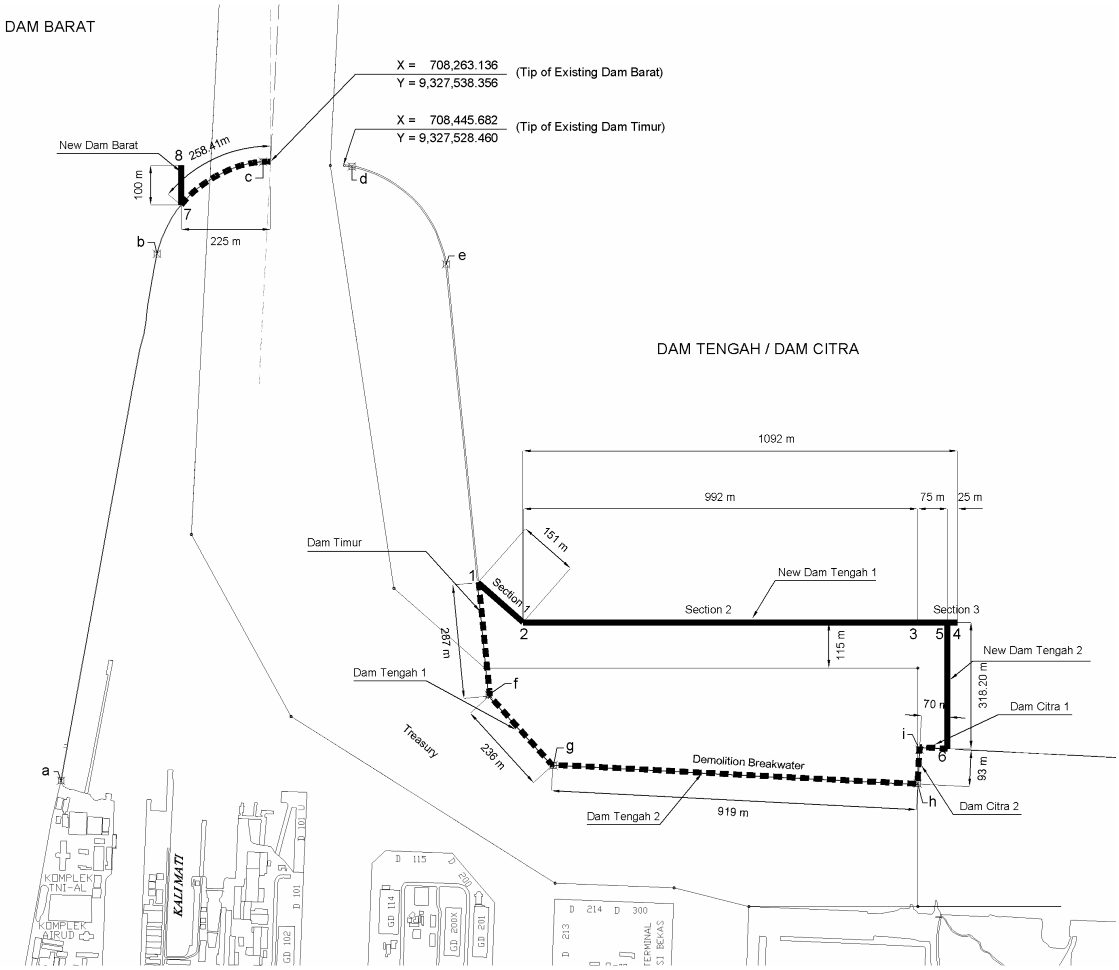
Based on the above comparison and evaluation results, the Rubble Mound Type was selected as the optimum upper-structural type for the new breakwaters.
In the succeeding sections, more detailed structural analysis has been made on a breakwater section basis, considering its structural requirements.

5.4 Basic Design of New Dam Tengah-1

5.4.1 Layout of New Breakwater

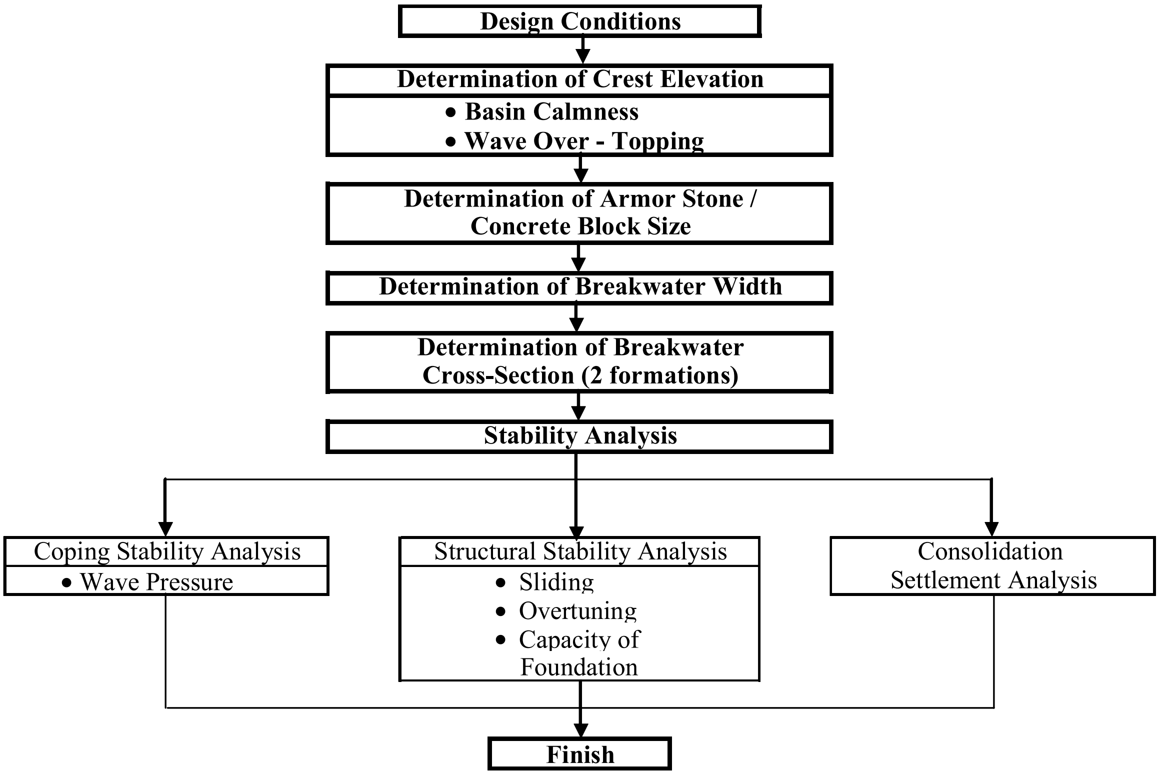
New Dam Tengah-1 consists of three sections, namely "Tengah-1-Section-1", "Tengah-1-Section-2" and "Tengah-1-Section-3". Section-3 will be located on the eastern end, forming a head portion of New Dam Tengah-1. These sections should be a permanent structure. Unlike New Dam Barat, they can be constructed without any physical constraints, so that the design philosophy determined in 5.3.4 'Selection of Optimum Breakwater Structure" has been followed, which means that a rubble-mound structure with sand replacement has been selected for the detailed design.

Layout of new breakwaters is as shown below.



In the stage of the detailed design, the stability of the breakwater has been examined for two cross sections, one for the replacement down to -12 m and another down to -13m in order to reflect the varying elevations of the assumed bearing layer. The cross-sections of breakwater have two formations with/without reuse materials.

The detailed design sequence of rubble-mound-type breakwater is presented as below.



5.4.2 Design Conditions

The design conditions for New Dam Tengah-1 are the same as provided in Preliminary Design of Breakwater.

5.4.3 Determination of Crest Elevation

(1) Crest Elevation Required

1) Crest Elevation
Breakwater crest elevation should be not less than 0.6 times the design significant wave height above the mean monthly-highest water level (MHWL).

Dimension of the design wave can be determined as shown below.

$H_{1/3}$ = 1.5m, T = 6.0 s, Wave Incident : N (High frequency wave)

$H_{1/3}$ = 2.5m, T = 7.5 s, Wave Incident : N (Low frequency wave)

Crest elevation of the new breakwaters is calculated by the following formula and crest elevation required is shown in Table 5.4.1.

$H = MH.W.S + 0.6\ H_{1/3} + \alpha$

Table 5.4.1 Crest Elevation Required

MHWS or HHWS	Wave Height (m)	Crest Elevation (m)
+0.91	1.5	1.81 +α
+0.91	2.5	2.41+α
+1.70	1.5	2.60+α

2) Permissible Rate of Wave Over-Topping
High-frequency higher wave is applied as the design wave height to decide the breakwater crest elevation, considering the permissible rate of wave over-topping for port operations.

The permissible rate of over-topping depends on such factors as the structural type of breakwater and land use requirements behind seawall. The following table gives a standard of the permissible rate of wave over-topping, considering the importance of the facilities behind the seawall.

Importance of Land Use behind Seawall	Rate of Over-topping (m3/s/m)
Areas where there is a high connection of houses, public facilities, etc. behind the seawall, and so it is anticipated that flooding due to over-topping or spray would cause particularly serious damages	About 0.01
Other important areas	About 0.02
Other areas	0.02 – 0.06

Source : Technical Standards and Commentaries for Port and Harbour Facilities in Japan, January 2002

The crest elevation (+ 2.50 m) for the breakwaters is given below, considering the permissible rate of wave over-topping as 0.02 – 0.06 m3/s/m. Seabed elevation in front of the breakwaters is determined as - 4.0 m.

Breakwater Crest Elevation	MHWS +0.91m	HHWS +1.70 m
	DL +2.50	DL +2.50m
Crest Height above MHWS	1.59	0.80
Rate of Wave Over-topping (m3/s/m)	0.004	0.035
Elevation of Wave Over-topping	Acceptable	Acceptable

The above-mentioned crest elevation (DL + 2.50m) has been calculated from the rate of wave over-topping, which is based on the high frequent wave and MHWS (+0.91m) and HHWS (+1.70m), respectively.

3) Planned Crest Elevation
In determining design crest elevation of the new breakwaters, also fully considered is the original design crest elevation of the existing Dam Barat breakwater.

On the basis of this design review, the crest elevation of the breakwater has been set at + 2.50 m.

Generally, wave height at the concave corner of the breakwater is greater than trunk part of breakwater. However, it is not considered for protecting the existing and new breakwaters by the concrete blocks.

(2) Armor Stones and Concrete Blocks

The mass of concrete blocks required to cover the front slope of the breakwaters is calculated by the following formula :

$M = \rho\gamma H3/[Ns3(S\tau - 1)3]$

Where,

M: minimum mass of concrete blocks,
$\rho\gamma$: density of concrete blocks,
H: design wave height,
Ns: stability number,
$S\tau$: specific gravity of concrete blocks relative to seawater.

Assuming that the front slope is 1:4/3 and concrete blocks (KD = 8.3, no breaking wave) are used as armor layer of the breakwaters, the parameters in the above formula were determined as follows. Design wave height of return period of 1/50-year is applied to this calculation.

$P\gamma$= 2.3 tons/m3, H1/3 = 2.5 m, Ns3 = KD*cot α = 8.3*4/3 = 11.07, $S\tau$ = 2.3/1.03 = 2.23

M = 2.3*2.53/[11.07(2.23-1)3] = 1.75 (ton)

Concrete blocks (2-ton type) will be used for armor layer, but head treatment of the breakwater will be used 3.2 type on the side for reinforcement. Armor stones, (500 kg to 1,000 kg) or existing concrete blocks (1.17 ton) to be demolished will be used for port side slope protection.

(3) Crest Width

The crest structure of the new breakwater is designed to have two (2) units of 2-ton type concrete blocks, coping concrete and armor stones. The concrete blocks are intended to achieve sufficient wave-dissipating effect, and the coping concrete is to secure the calmness in the port basins. A total crest width will be 7.0 m, consisting of 2.20 m of concrete blocks, 3.2 m of coping concrete and armor stones based on the sliding stability calculation results, and 1.6 m of armor stones, each weighing from 500 kg to 1,000 kg.

(4) Coping

Design wave pressure acting on the coping concrete covered with wave dissipating concrete block is calculated as shown follows.

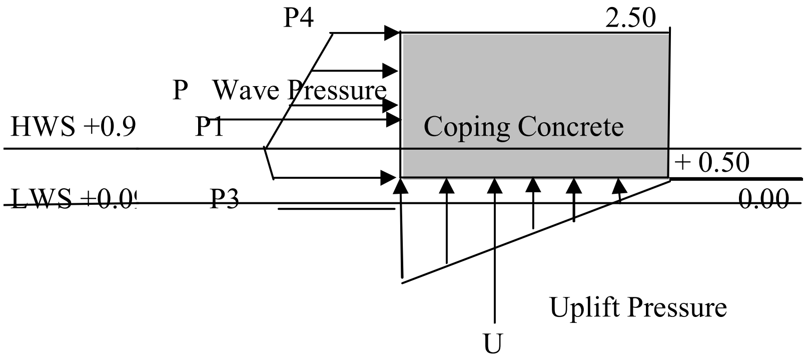


Table 5.4.2 Design Conditions and Calculation Results of Wave Pressure

Design Conditions	Ho' = 2.5 m	LWS = +0.09m MHWS = +0.91m	Lo = 87.75m	Ho'/L0 = 0.0285

Tidal Level	P1 (KN/m2)	P3 (KN/m2)	P4 (KN/m2)	P (KN/m)	U (KN/m)
+0.09m	22.64	20.08	7.56	33.22	32.13
+0.91m	28.04	24.09	18.37	47.58	38.54

Coping concrete will be provided at the breakwater crest. Results of sliding stability calculation are shown in Table 5.4.3.

Table 5.4.3 Results of Sliding Stability Calculation

Tide Level (m)	Wave Pressure (KN/m)	Uplift (KN/m)	Weight (KN/m)	Safety Factor (F)	Allowable Safety Factor (Fa)
DL +0.09	33.22	32.13	144.35	2.03	1.20
DL +0.91	47.58	38.54	144.35	1.33	1.20

(5) Proposed Cross Section

Typical cross section of New Dam Tengah-1 is shown in Figure 5.4.1.

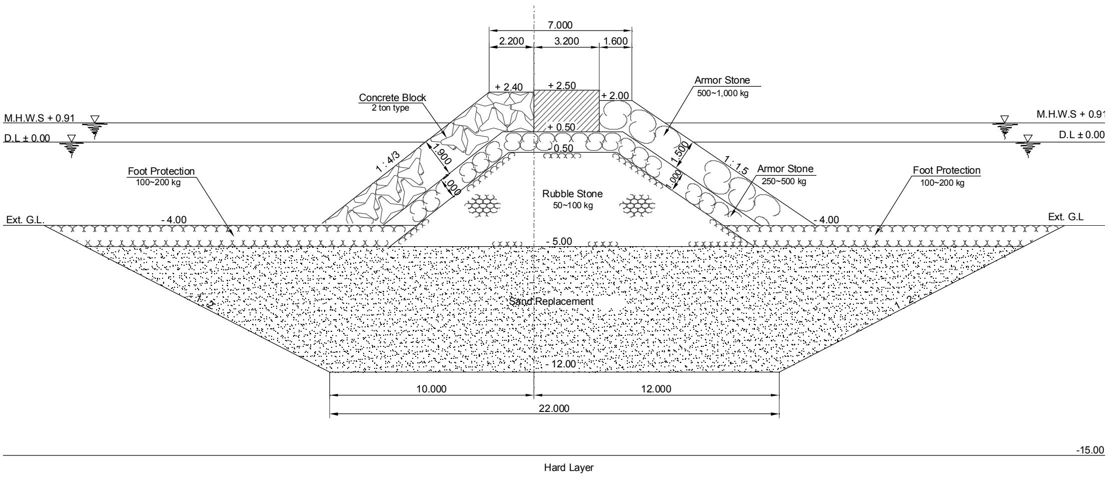


Figure 5.4.1 Typical Cross Section of New Dam Tengah-1

5.4.4 Structural Stability Analysis

Stability of the New Dam Tengah-1 was analyzed for circular failure. Results of the circular failure are as follows in Figure 5.4.2 and 5.4.3.

a) Replacement Depth -12.0m
Minimum safety factor = 1.382 > Fa = 1.30

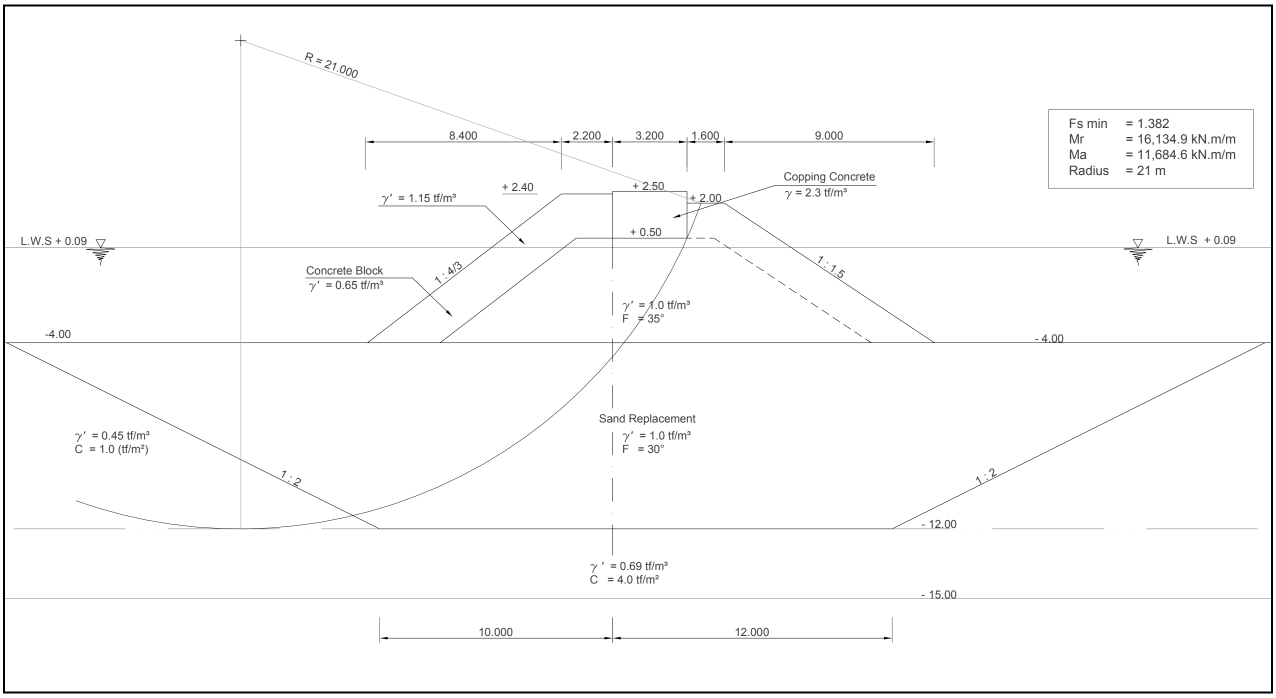


Figure 5.4.2 Circular Failure Diagram of Rubble Mound (-12.0m)

b) Replacement Depth -13.0m
Minimum safety factor = 1.379 > Fa = 1.30

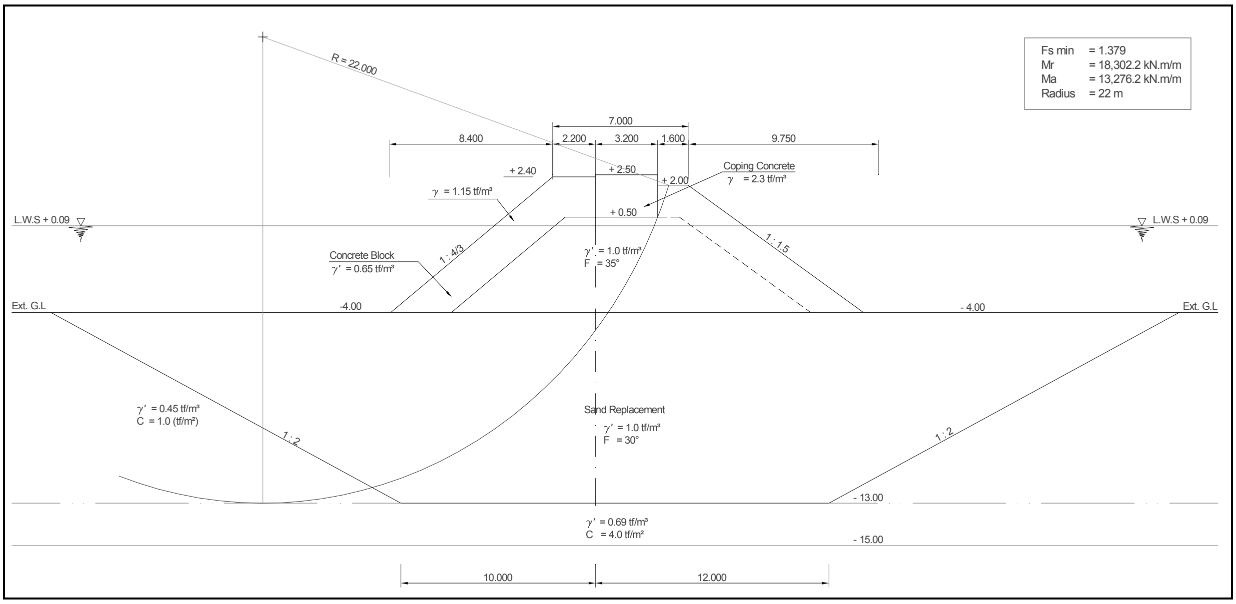


Figure 5.4.3 Circular Failure Diagram of Rubble Mound (-13.0m)

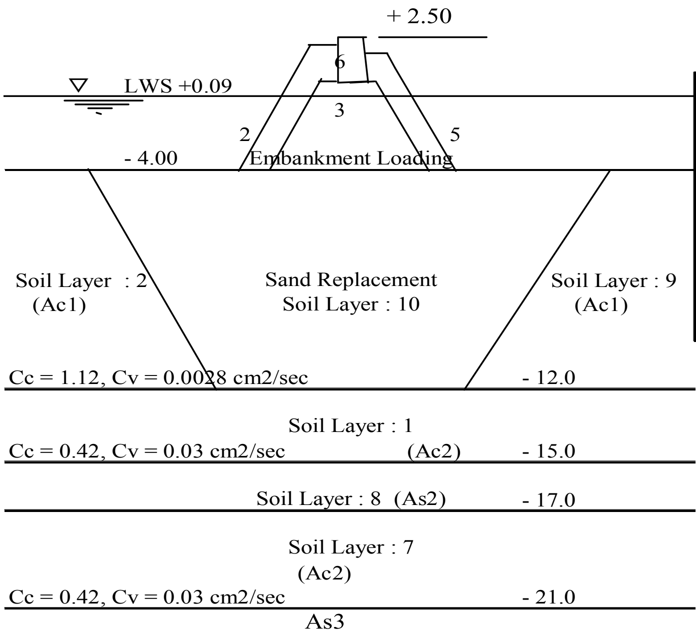
5.5 Detailed Design on New Dam Tengah-1

5.5.1 Consolidation Settlement Analysis

Consolidation settlement with sand replacement was analyzed based on the assumed soft layers on site as follows.

Each layers of calculation method are as follows.

a) Replacement Depth -12.0ma) Replacement Depth -12.0m

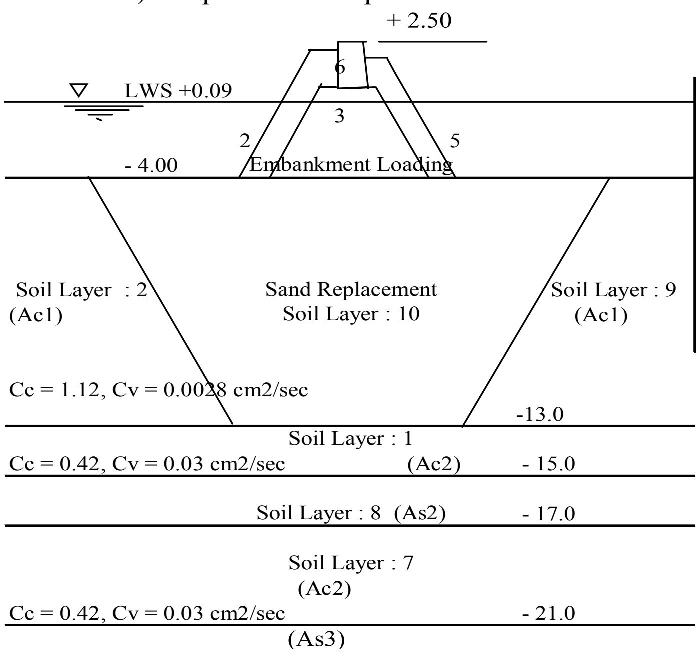


Conditions of Calculated Consolidation Settlement

Layer No.	γ_t (tf/m3)	Po (kg/cm2)	ΔP (kgf/cm2)
1	0.69	0.904	0.607
2	0.45	-	-
3	1.00	-	-
4	0.65	-	-
5	1.00	-	-
6	2.40	-	-
7	0.69	1.345	0.460
8	1.00	-	-
9	0.45	-	-
10	1.00	0.400	0.787

Degree (%)	10	20	30	40	50	60	70	80	90	100
Time (day)	8	13	18	24	30	33	48	71	90	
Settlement (cm)	1.62	3.24	4.86	6.48	8.10	9.72	11.34	12.96	14.58	16.20

b) Replacement Depth -13.0m



Conditions of Calculated Consolidation Settlement

Layer No.	γ_t (tf/m3)	Po (kg/cm2)	ΔP (kgf/cm2)
1	0.69	0.969	0.602
2	0.45	-	-
3	1.00	-	-
4	0.65	-	-
5	1.00	-	-
6	2.40	-	-
7	0.69	1.376	0.460
8	1.00	-	-
9	0.45	-	-
10	1.00	0.450	0.778

Degree (%)	10	20	30	40	50	60	70	80	90	100
Time (day)	8	14	21	26	32	37	54	80	102	
Settlement (cm)	1.56	3.12	4.68	6.24	7.80	9.36	10.93	12.49	14.05	15.61

Consolidation settlement is around 14 ~ 15 cm and consolidation time is 102 days which is 90 % of consolidation degree.

5.5.2 Usage of Demolished Materials of Existing Breakwater

With demolition of the existing breakwater, such demolished materials as coping concrete, sand and gravel will be reused for the new breakwaters as a part of the replacement sand at the level of -5.0 m to -8.0m. The demolished concrete blocks will also be reused as armor stones for shore side slope protection of the new breakwaters. Figure 2.5.3 shows the detailed sections of the new breakwaters by use of these demolished materials.

5.5.3 Reinforcement of Breakwater Head

Breakwater head will be reinforced with larger concrete blocks, each weighing 3.2 tons that is 1.5 times the weight of concrete block (2.0 tons) to be placed on main section of the breakwater, and the breakwater head will be of round shape.

5.5.4 Concave Corner Portion

Generally, wave height at the concave corner of the breakwater is greater than trunk part of breakwater. However, it is not considered for protecting the existing and new breakwaters by the concrete blocks.

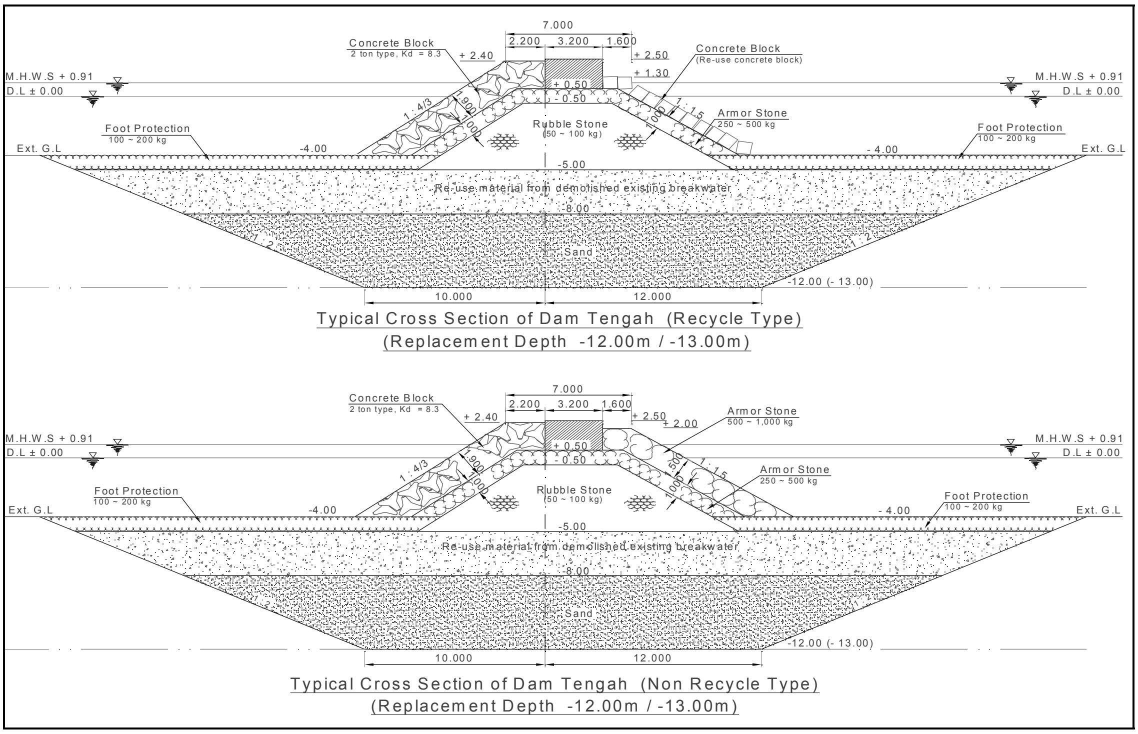


Figure 5.5.1 Typical Cross Section of New Dam Tengah-1

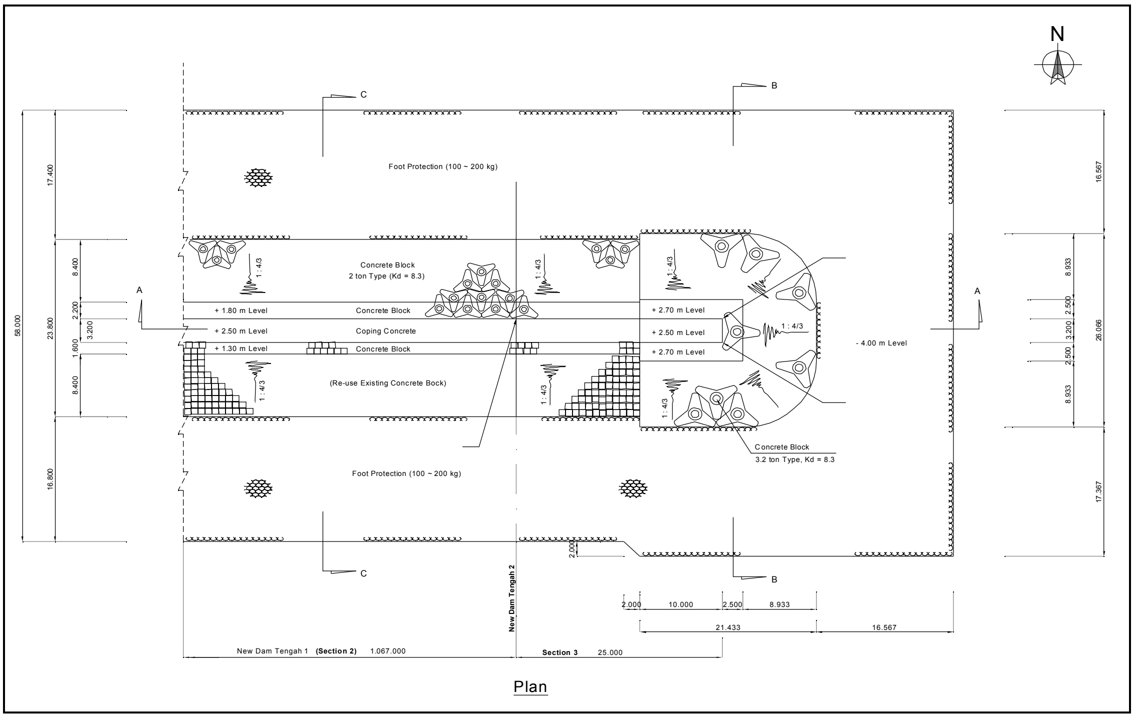


Figure 5.5.2 (1) Head Treatment of New Dam Tengah-1 (Plan)

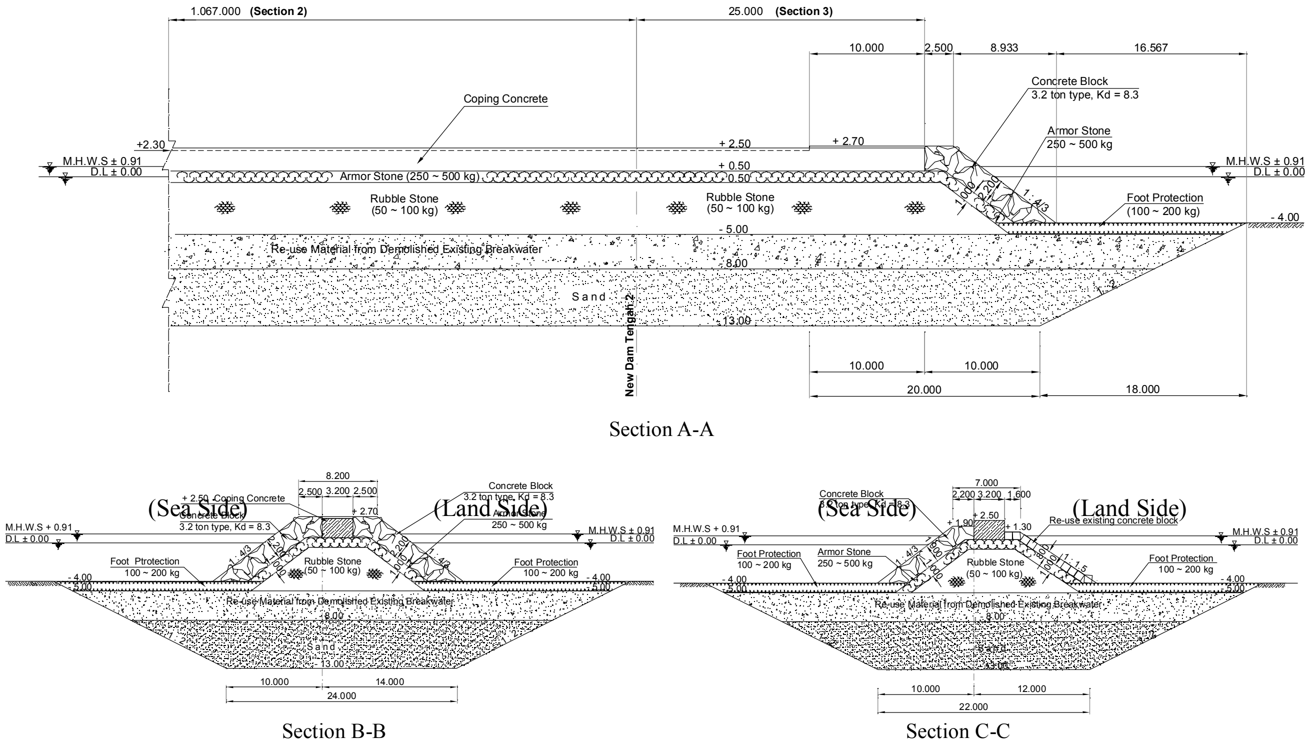


Figure 5.5.2 (2) Head Treatment of New Dam Tengah-1 (Section)

5.6 Basic Design of New Dam Tengah-2

5.6.1 Design Conditions

The design conditions for New Dam Tengah-2 follows the same criteria used for New Dam Tengah-1 except for the followings.

(1) Life-time

Life-time has been changed from 50 years to 10 years.

(2) Waves

Breakwater is aligned in the north direction facing the east.

Considering the local bathymetry and the existence of cape named "Tanjung Karawang", the wave from the east is limited to N ~ NE.

The design wave for New Dam Tengah-2 has been determined as the waves with a return period of 10 years. Considering the JICA Feasibility study, the wave direction NE for 10 years period, has been determined as below.

Direction	NE
Height (m)	1.45
Period (s)	6.3

(3) Soil Condition

Soil conditions has been taken from the nearest boring data executed in New Dam Tengah 2 area.

5.6.2 Selection of Structure Type

Rubble mound type is selected for New Dam Tengah-2, and the selected type is the same adopted for New Dam Tengah-1. Considering that New Dam Tengah-2 is of a temporary.

5.6.3 Determination of Basic Cross Section

(1) Crest Elevation

Crest elevation of the new breakwaters is calculated by the following formula and crest elevation has been decided as +2.00m.

$H = MH.W.S + 0.6\ H1/3 + \alpha$

MHWS or HHWS	Wave Height (m)	Crest Elevation (m)
+0.91	1.45	1.78 +α

(2) Armor Stones

The mass (unit weight) of armor stones required to cover the front slope of the breakwater is calculated by the formula.

Assuming that the front slope is 1:2 and armor stone (KD = 2.4, no breaking wave) are used as armor layer of the breakwaters, the parameters in the formula were determined as follows. Design wave height of return period of 1/10-year is applied to this calculation.

$P\gamma$ = 2.65 tons/m^3, $H_{1/3}$ = 1.45 m, $Ns^3 = K_D$*cot α = 2.4*2 = 4.8, $S\tau$ = 2.65/1.03 = 2.57

M = 2.65*1.45^3/[4.8(2.57-1)^3] = 0.43(ton)

Armor stones will be used 500 ~ 1,000kg for armor layer.

(3) Proposed Cross Section

The cross section of New Dam Tengah-2 is shown in Figure 5.6.1.

(Land Side) (Sea Side)

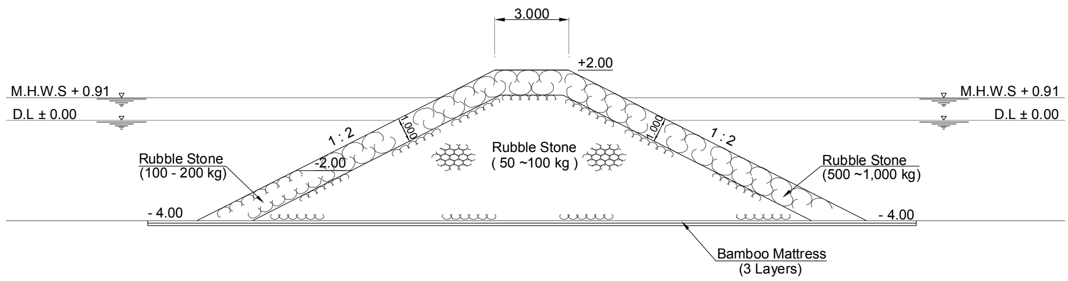


Figure 5.6.1 Typical Cross Section of New Dam Tengah-2

5.6.4 Structural Stability Analysis

The stability analysis for New Dam Tengah-2 has covered settlement analysis and circular failure analysis.

(1) Circular Failure

Stability of the New Dam Tengah-2 was analyzed for circular failure.

It is considered the circular arc does not pass through the Bamboo Mattress, as the Bamboo Mattress is rigged by three layers of bamboo (tip end diameter 8 cm) and the each joint of bamboo mattress is reinforced by the joint mattress.

Circular failure was analyzed with the water depth of – 3.50 m for the front area which is on the slope of New Basin while the area with water depth of – 4.0 m is formed flat which is not dredged for the New Basin.

Results of the circular failure analysis are shown in Figure 5.5.1 below.

Minimum safety factor = 1.756 > Fa = 1.30

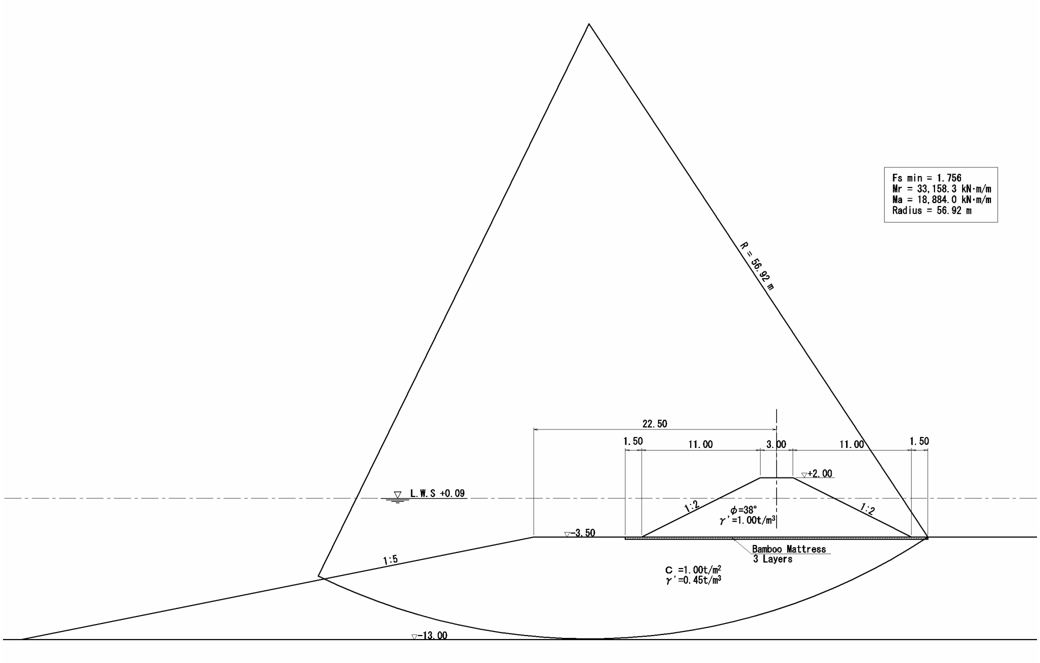


Figure 5.6.2 Circular Failure Diagram of New Dam Tengah-2

5.7 Detailed Design of New Dam Tengah-2

(1) Settlement Analysis

Consolidation settlement is considered to occur by the Ac1 layer (9.0 m) because settlement of Ac2 layer was only 4cm by the previous analyzed (New Dam Tengah 1).

Consolidation settlement is used below equation.S = mv.ΔP.h

Where,

S : final consolidation settlement (cm)
Mv : coefficient of volume compressibility (cm2/kg)
Po : effective overburden pressure at the center of a sub-layer before consolidation (kg/cm2)
ΔP : increase in consolidation pressure (kg/cm2)
h : thickness of layer (cm)

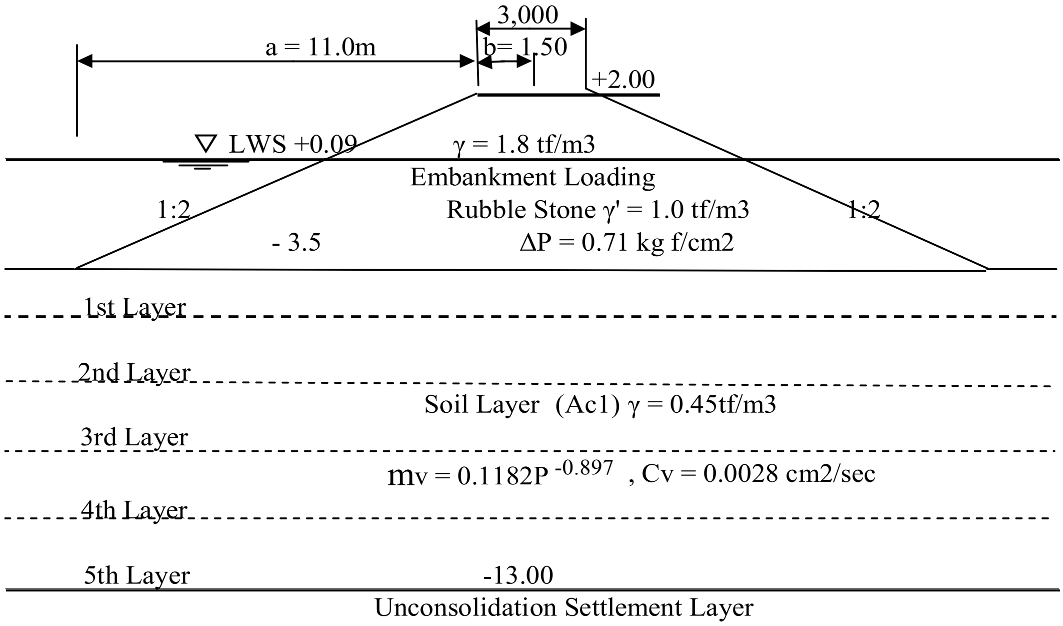
Increase in consolidation pressure (ΔP kg/cm2) is estimated by the Osterberg influence value refers to "Chapter 5 Foundation Settlement of Technical Standards and Commentaries for Port and Harbor Facilities in Japan".

$Tv = Cv.t / h^2$

Where,

Tv : time factor
Cv : Coefficient of consolidation (cm2/d)
T : time after the consolidation starts (d)
h : maximum drainage distance (cm)

Log P- mv Curve refer to APPENDIX 5-C



Layer	ΔP (kgf/cm2)	mv (kgf/cm2)	S = mv.ΔP.Hn (cm)
1	0.71	0.28	29.8
2	0.68	0.24	32.6
3	0.61	0.22	26.8
4	0.53	0.20	21.2
5	0.51	0.18	18.4
Total			128.8

Degree (%)	10	20	30	40	50	60	70	80	90	100
Time (day)	30	116	265	470	735	1,071	1,522	2,115	3,163	=
Settlement (cm)	12.8	25.5	38.3	51.0	63.8	76.6	89.3	102.8	114.9	127.6

The consolidation settlement after 1year, 5 years, 10 years is estimated at around 0.45m, 0.95m and 1.15 m respectively.

The crest height of Dam Citra is maintained at the elevation of + 1.50m. Therefore, the crest height of New Dam Tengah 2 should also be maintained at + 1.50 m

(2) Detail of Bamboo Mattress

Bamboo mattress is composed of three (3) layers of bamboos (length of each bamboo : 10 m to 12 m) tied up with hemp ropes at a spacing of 50 cm. Each bamboo mattress is unified with joint bamboo mattress. In installing the bamboo mattress, rubble stone should be laid evenly to avoid the unbalanced load on the mattress.

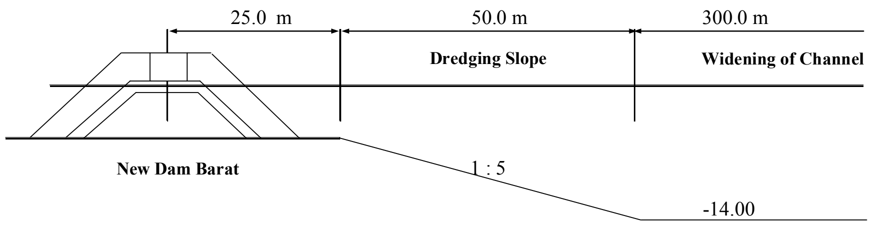
5.8 Basic Design of New Dam Barat

5.8.1 Design Conditions

The design conditions for New Dam Barat follows the same criteria used for New Dam Tengah-1 except for the followings.

(1) Construction Site Condition

In accordance with the relocation plan of breakwater (See Figure 2.4.1). New Dam Barat is to be aligned parallel to the new navigation channel and the distance in-between is 75m (25+50) as shown below.



(2) Subsoil

The soil conditions applied to the design of New Dam Barat are given in Table 5.8.1.

It can be said that the soil conditions at the construction of New Dam Barat are unfavourable; thickness of first silty clay layer with N-value of 0 (ac1) is 2 m to 3 m larger than that of New Dam Tengah-1, and the shearing strength is lower as compared with that of New Dam Tengah-1.

Table 5.8.1 Soil Condition of New Dam Barat

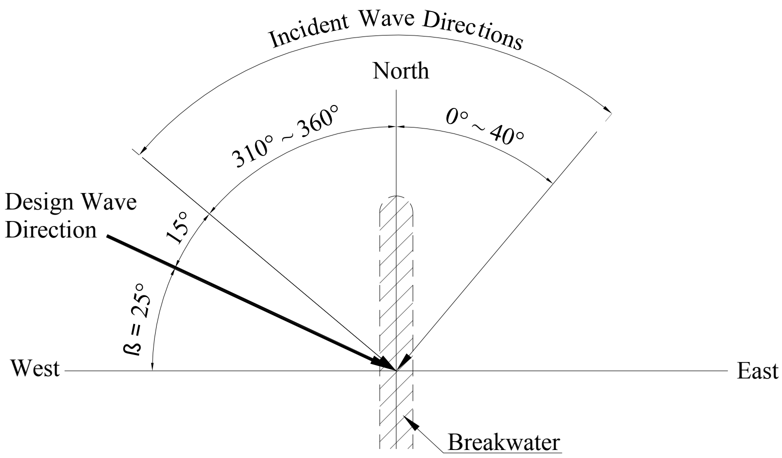
Elevation -4.00 m Silty Clay N = 0 $\Phi = 0°$ $\gamma t = 1.43$ t/m3 C = 0.6 (tf/m2) Cc = 0.98 Cv = 0.0028 cm2 / sec. $m_v = 0.1182P^{-0.897}$
Elevation -15.00 m Silt N = 44 $\Phi = 0°$ $\gamma t = 1.69$ t/m3 C = 44 tf/m2

The subsoil at the port entrance is composed of soft clay (Ac1) down to – 15.0 m.

(3) Waves

The design wave has been determined, considering the wave direction, not perpendicular to the breakwater face, but with the angle of β = 25o as illustrated below.

Direction	310 ~ 340°	340 ~ 360°	10 ~ 40°
Wave Height (m)	2.50	2.50	2.50
Wave Period (s)	7.4	7.5	7.6



5.8.2 Comparison of Optimum Structure

For the design of New Dam Barat, its upper-structure follows the rubble-mound structure adopted for New Dam Tengah-1. Only the foundation type has been additionally discussed as follows.

In consideration of the above site conditions, two alternative breakwater structures have been studied as follows:

Alternative 1 : Replacement by sand until – 15.0 m as shown in the Figure 5.8.1 (Same as New Dam Tengah-1)

Alternative 2 : Replacement by sand until – 6.0 m, driving the bamboo pile until – 14.5 m and laying the bamboo mattress on the replaced sand as shown in the Figure 5.8.2

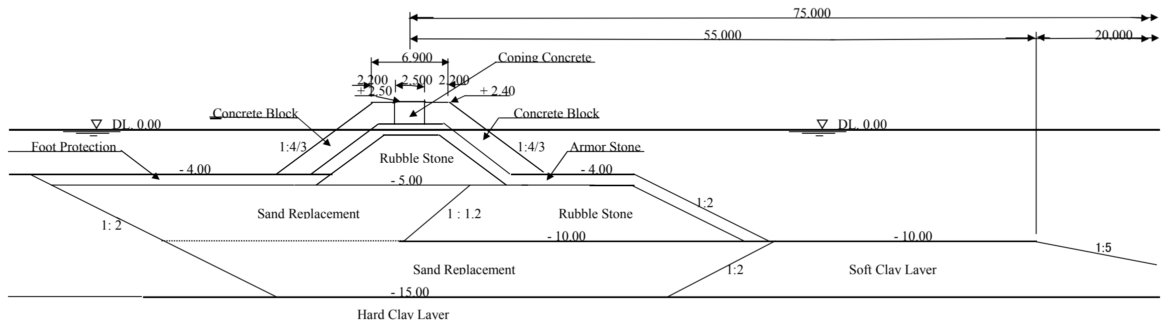


Figure 5.8.1 Alternative 1 : Sand Replacement up to – 15.0 m

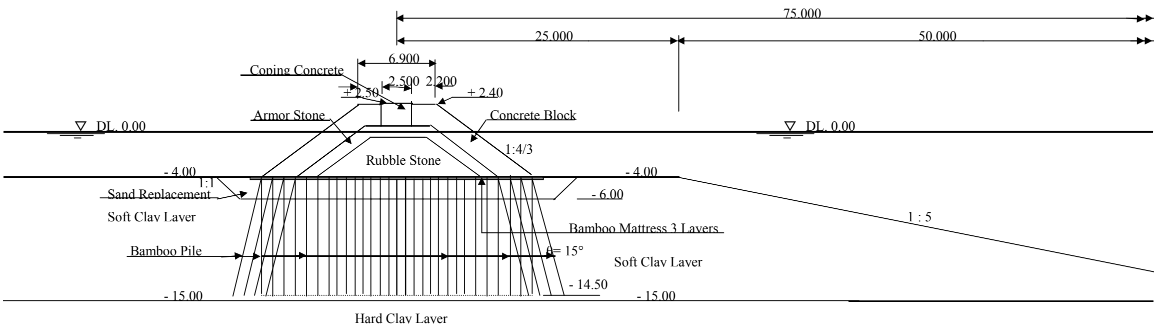


Figure 5.8.2 Alternative 2 : Sand Replacement up to – 6.0 m with Bamboo Pile & Mattress

(1) Alternative-1

Sand replacement foundation width is designed larger to avoid the above problem. Horizontal mound is provided, by dredging to the depth of – 10.0m, between the side slope of the replaced sand foundation and that of the channel, to maintain the stability of the side slope of the channel.

When analyzing the stability of Alternative-1, Safety Factor of circular failure has been calculated as 1.308 shown in the Figure 5.8.3, which is more than the required figure of 1.30.

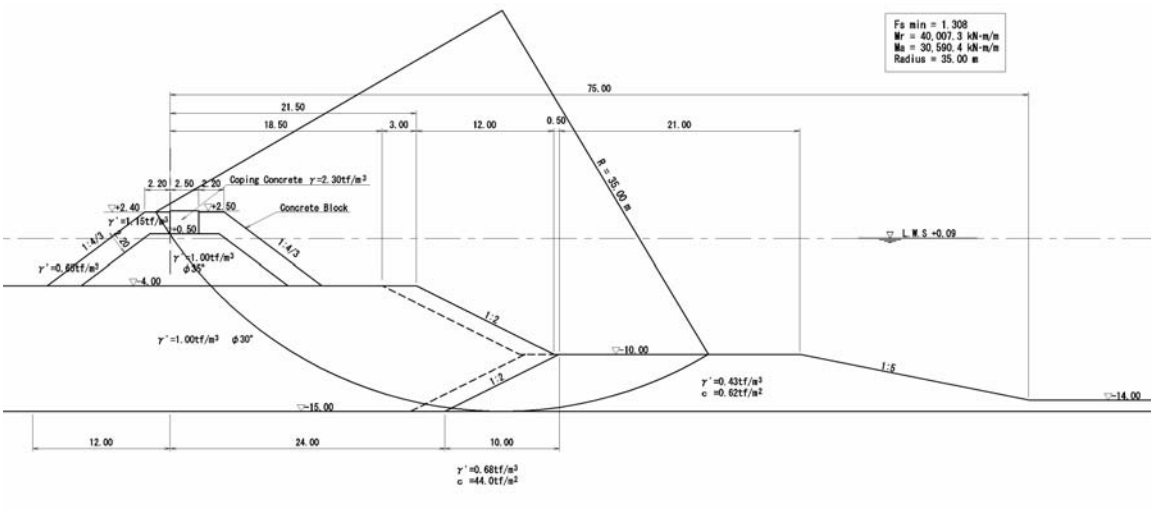


Figure 5.8.3 Circular Failure Diagram of New Dam Barat (Alternative - 1)

(2) Alternative-2

The bamboo piles bear the loads, which are distributed and reduce the upper structure weights to be supported by the bamboo mattress on the ground. No significant influence will occur to the dredging slope by using the bamboo piles supporting breakwater.

Problem which may arise on using bamboo is its lifespan (20~30 years). This concern, however, would be diminished, because the breakwater could be demolished in the future, when the westward port development plan is fully-matured.

The above discussion favors Alternative-2 for development of New Dam Barat. As such, Bamboo piled supported rubble-mound structures with sand replacement have been adopted for New Dam Barat.

(3) Selection of Optimum Structure

Replaced sand foundation in Alternative 1 is the foundation structure adopted for New Dam Tengah - 1. This type of foundation stability, but it requires the lager sand replacement width, implies the increased volume of dredging and replacement sand and stones of rubble and armor, resulting into higher construction cost (more than 2 times of Alternative-2).

Replaced sand foundation with bamboo pile and mattress in Alternative-2 is a piled foundation that avoids the influence on the side slope of the channel, by dispersing the surcharge to the deeper foundation ground through the bamboo piles driven. Accordingly, the type is selected as the optimum foundation structure of New Dam Barat.

5.8.3 Determination of Basic Cross Section

(1) Crest Elevation

Breakwater crest elevation should be not less than 0.6 times the design significant wave height above the mean monthly-highest water level (MHSL).

H = M H.W.S + 0.6 H1/3 + α (allowance)

= 0.91 m + 0.6 x 2.5 m + α
= 2.41 m + α
= 2.50 m (D.L.)

(2) Armor Stone and Concrete Block

Concrete blocks (2-ton type) will be used for armor layer, but head treatment of the breakwater will be used 3.2 type on the side for reinforcement same as Dam Tengah-1.

(3) Coping Concrete

Coping concrete will be provided at the breakwater crest. Results of Sliding stability calculation are as follows.

1) Design Wave Pressures

Design wave pressure acting on the coping concrete covered with wave dissipating concrete block is calculated as shown follows.

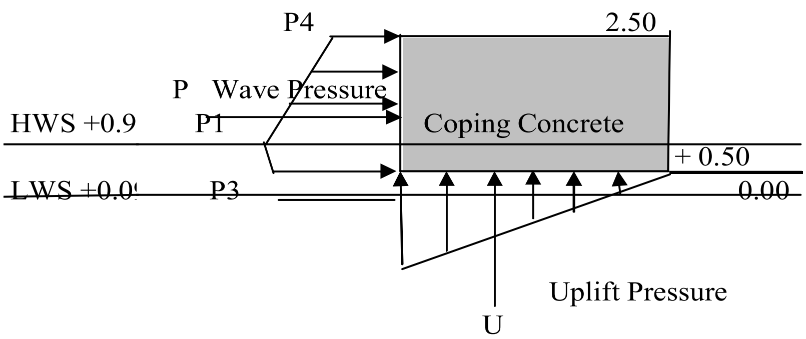


Table 5.8.2 Design Conditions and Calculation Results of Wave Pressure

Design Conditions	Ho' = 2.5 m β = 25°	LWS = +0.09m MHWS = +0.91m		Lo = 87.75m	Ho'/L0 = 0.0285
Tidal Level	P1 (KN/m^2)	P3 (KN/m^2)	P4 (KN/m^2)	P (KN/m)	U (KN/m)
+0.09m	20.89	18.53	6.96	30.65	23.16
+0.91m	25.88	22.23	16.95	43.91	27.79

Tide Level (m)	Wave Pressure (kN/m)	Uplift (kN/m)	Weight (kN/m)	Safety Factor (F)	Allowable S.F (Fa)
DL. + 0.09	30.66	23.16	115.23	1.66	1.20
DL. + 0.91	43.91	27.79	115.23	1.20	1.20

(4) Width and Thickness of Sand Replacement

In determining the sand replacement width, allowance of 2 m from the center of the pile driven at the toe of breakwater slope is given so that the batter and vertical piles of the foundation may bear the horizontal load. Accordingly, the sand replacement width determined is 28 m (24m of pile driving width and 2 m of allowance at each side of the breakwater). Sand replacement thickness is determined to be 2.0 m.

(5) Use of Bamboo pile and Mattress

Supporting capacity of bamboo pile against the vertical load is determined to be 4.5 ft / pile, based on the data obtained from the past experience, and the pile pitch is determined to be 0.75 m at the center and 1.0 m on both sides.

In determining the bamboo pile length, friction pile (not bearing pile) is adopted to avoid the stress concentration on the bamboo piles and to secure the stress dispersion to silty clay layers. Accordingly, the bamboo pile tip is designed 0.5 m above the bearing stratum.

Batter piles are used to reinforce the toe of the breakwater slope.

Bamboo mattress is used to distribute the surcharge evenly to the bamboo piles. Buoyancy of bamboo mattress is expected, but the buoyancy is not considered in the design of the breakwaters. Shearing strength of bamboo (170 kg/ cm2) is expected to resist the circular failure.

Circular failure is calculated, considering the shearing strength (170 kg/cm2) of the bamboo.

(6) Proposed Cross Section

Typical cross section of the New Dam Barat is as shown in Figure 5.8.4.

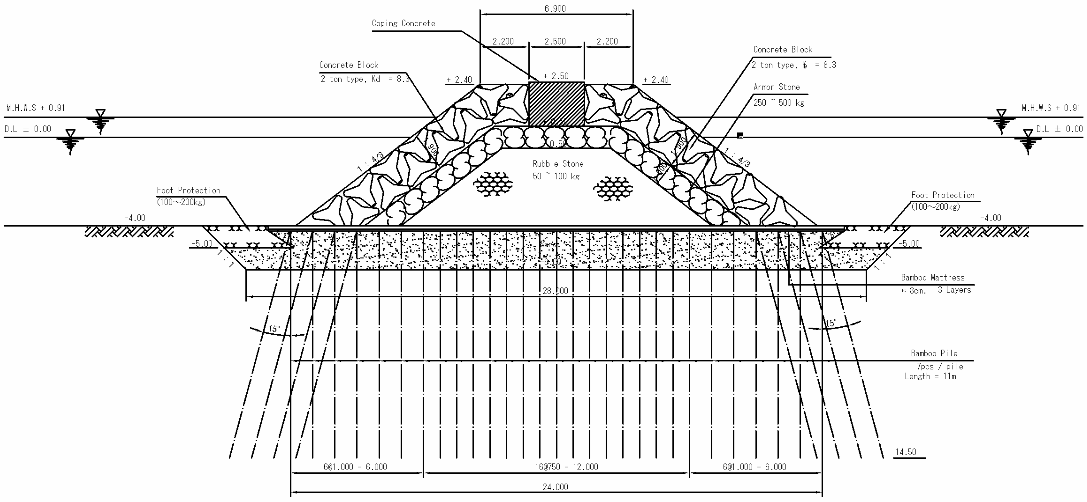


Figure 5.8.4 Typical Cross Section of New Dam Barat

5.8.4 Structural Stability Analysis

(1) Bearing Capacity and Circular Failure

A bamboo pile consisting of seven bamboos each with diameter of 10 cm bounded together (30cm of pile diameter).

Bamboo pile pitch was determined as shown below, based on the study results of bearing capacity of the bamboo piles and also on the data obtained from the past experience.

Center Part : 0.75 m

End parts : 1.00 m

The upper structure is supported by the bamboo mattress and bamboo pile which are subject to the future consolidation settlement. Pile embedment length of – 14.50 m was determined to allow for the future consolidation settlement and to avoid the concentration of stress on the piles.

Battered piles are used at the end part of sloped foundation for its reinforcement.

Circular failure is calculated, considering the shearing strength (170 kg/cm2) of the bamboo.

Taking into consideration of the effect of bamboo mattress and bamboo pile foundation, it is confirmed that the foundation will be fully stable against the circular failure.

1) Width and Thickness of Sand Replacement
In determining the sand replacement width, allowance of 2 m from the center of the pile driven at the toe of breakwater slope is given so that the batter and vertical piles of the foundation may bear the horizontal load. Accordingly, the sand replacement width determined is 28 m (24m of pile driving width and 2 m of allowance at each side of the breakwater). Sand replacement thickness is determined to be 2.0 m.

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Batter piles are used to reinforce the toe of the breakwater slope.

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Taking into consideration of the effect of bamboo mattress and bamboo pile foundation, it is confirmed that the foundation will be fully stable against the circular failure.

5.9 Detailed Design of Dam Barat

(1) Consolidation Settlement

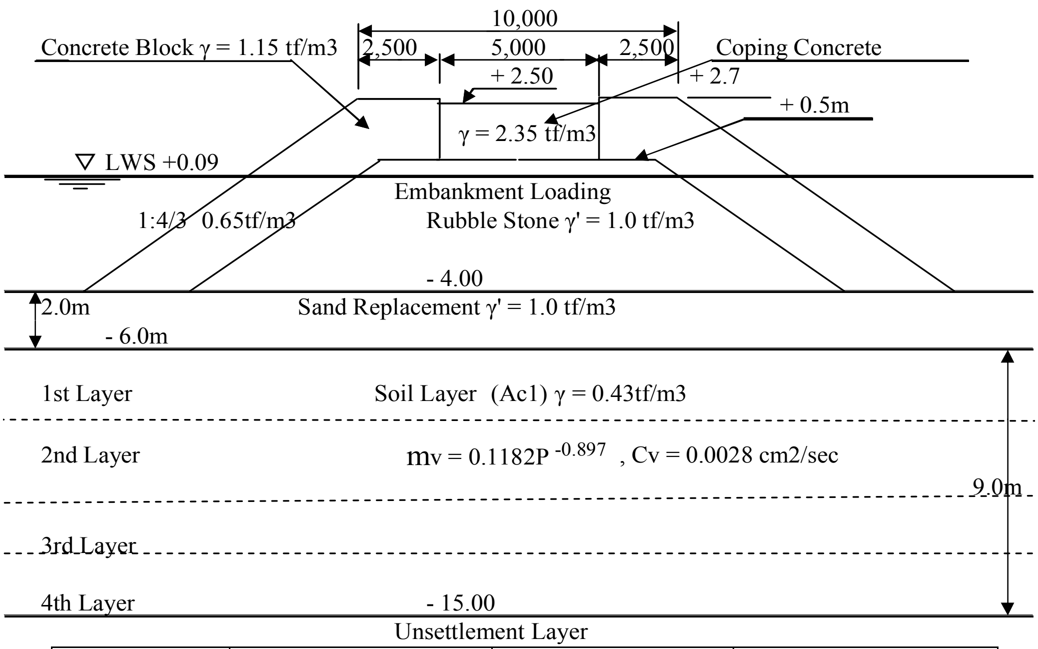
Case 1 : Thickness length (H) is under the sand replacement bottom.

Foundation of The New Barat is supported by the bamboo pile through the bamboo mattress.

To minimize the future consolidation settlement, the load is to be shared by the bamboo pile and ground at the ratio of 1:3 for sand compaction and 3:7 (Related data: Construction of Breakwater by Bamboo Pile and Bamboo Mattress Foundation, Civil Engineering Journal, March 1986), and the discharge distance is determined to be 9.0 m.

Case 2 : Thickness length (H) is below the lowest one-third of the embedded pile under the sand replacement bottom.

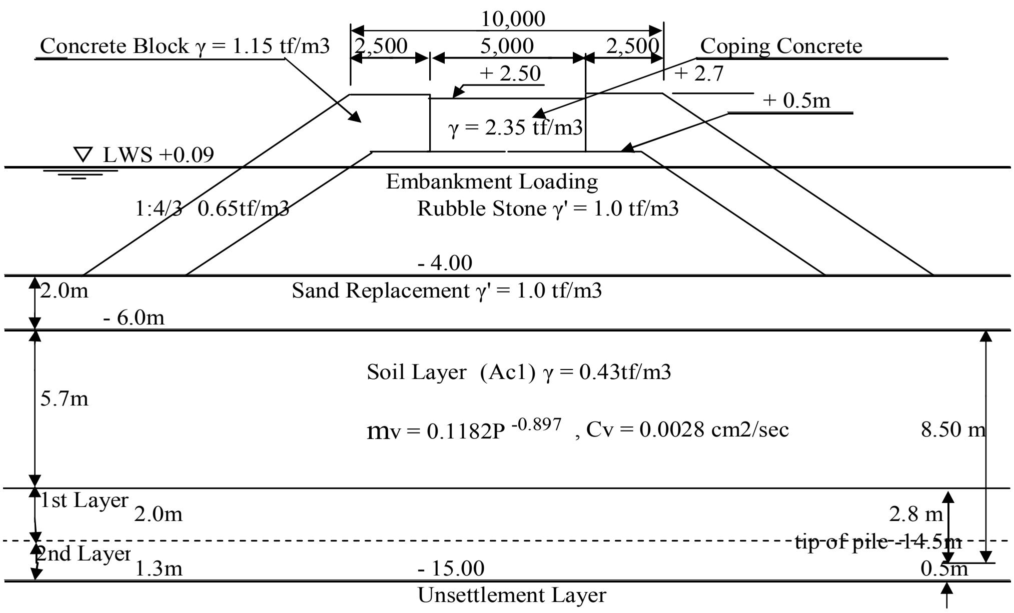
Case 1 : Thickness length (H) is under the sand replacement bottom.



Layer	ΔP (kgf/cm2)	mv (kgf/cm2)	S = mv.ΔP.Hn (cm)
1	0.24	0.40	24.0
2	0.23	0.30	17.3
3	0.21	0.24	10.1
4	0.20	0.21	8.9
Total			60.3

	10	20	30	40	50	60	70	80	90	100
Time (day)	27	104	238	422	660	961	1,366	1,898	2,839	-
Settlement (cm)	6.0	12.1	18.0	24.1	30.2	36.2	42.2	48.2	54.3	60.3

Case 2 : Thickness length (H) is below the lowest one-third of the embedded pile under the sand replacement bottom.



Layer	ΔP (kgf/cm2)	mv (kgf/cm2)	S = mv.ΔP.Hn (cm)
1	0.70	0.16	22.4
2	0.67	0.15	13.1
Total			35.5

Degree (%)	10	20	30	40	50	60	70	80	90	100
Time (day)	27	104	238	422	660	961	1,366	1,898	2,839	-
Settlement (cm)	3.6	7.1	10.7	14.2	17.8	21.3	24.9	28.4	32.0	33.7

The results of 2 cases has been obtained as above, final settlement value is considered the Case 1 (60.3cm).

(2) Reinforcement of Breakwater Head

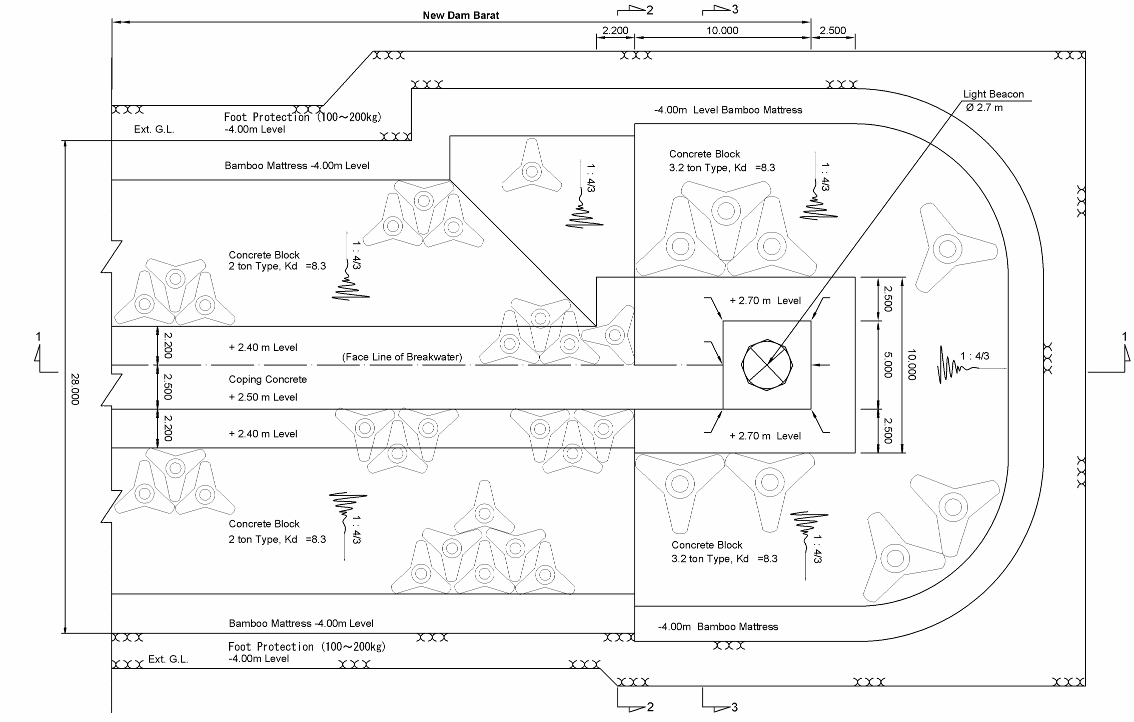
Breakwater head will be reinforced with larger concrete blocks, each weighing 3.2 tons that is 1.5 times the weight of concrete block (2.0 tons) to be placed on main section of the breakwater, and the breakwater head will be of round shape.

(3) Detailed of Bamboo Mattress / Bamboo Pile

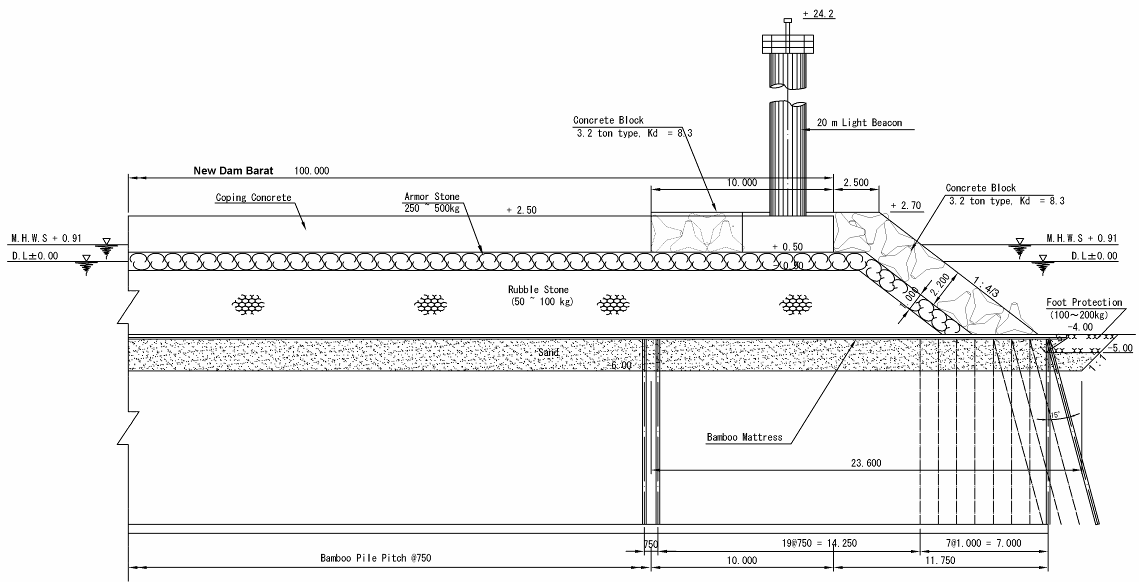
Bamboo mattress is composed of three (3) layers of bamboos (length of each bamboo : 10 m to 12 m) tied up with hemp ropes at a spacing of 50 cm. Each bamboo mattress is unified with joint bamboo mattress. In installing the bamboo mattress, rubble stone should be laid evenly to avoid the unbalanced load on the mattress.

A bamboo pile is composed of seven (7) pieces of bamboo bound together with hemp ropes at a spacing of 50 cm, and its length will be 10.5 to 12.0 m.

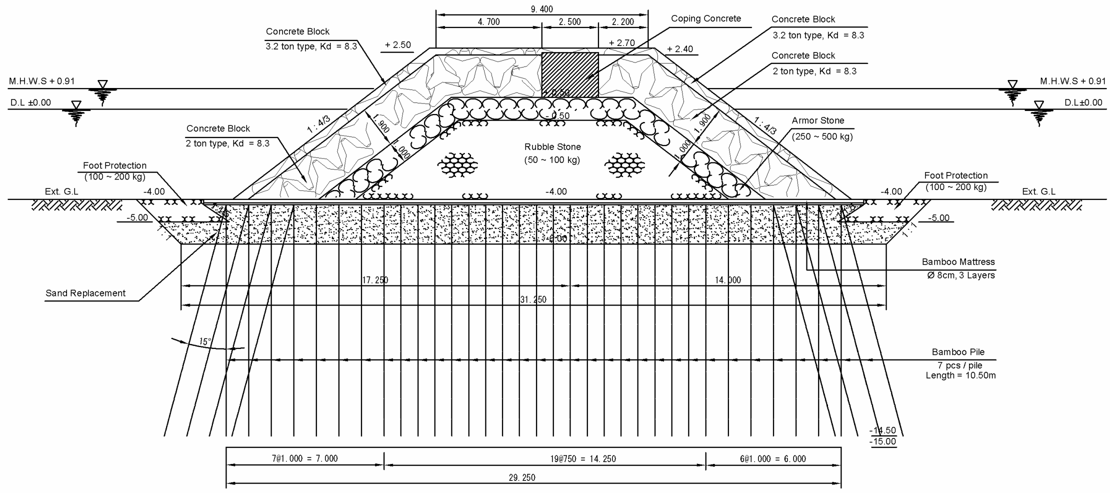
Head treatment of the New Dam Barat is as shown in Figure 5.8.5.



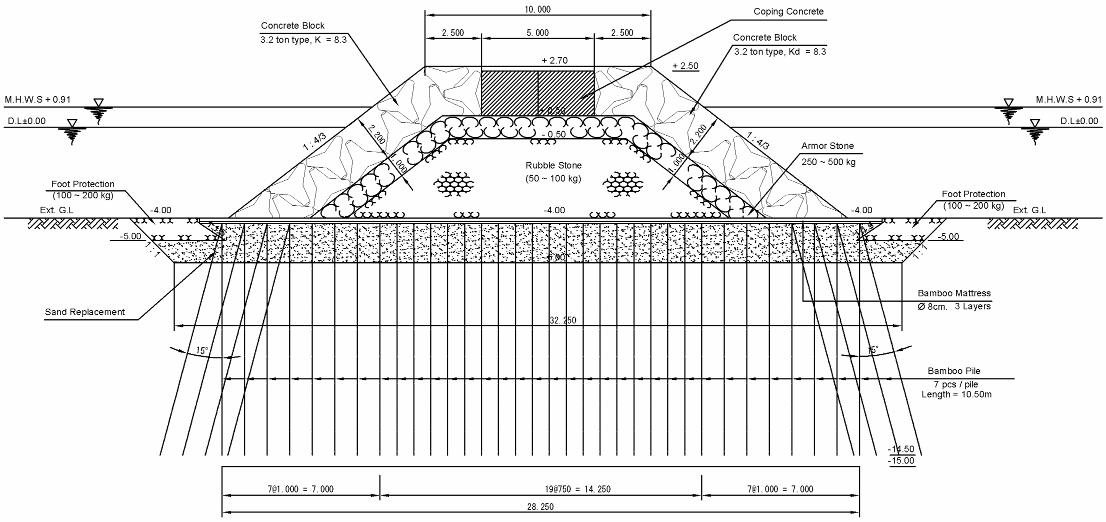
Plan



Section 1-1



Section 2-2



Section 3-3

Figure 5.8.5 Head Treatment of New Dam Barat

CHAPTER 6 BREAKWATER RELOCATION, DREDGING AND DUMPING WORKS

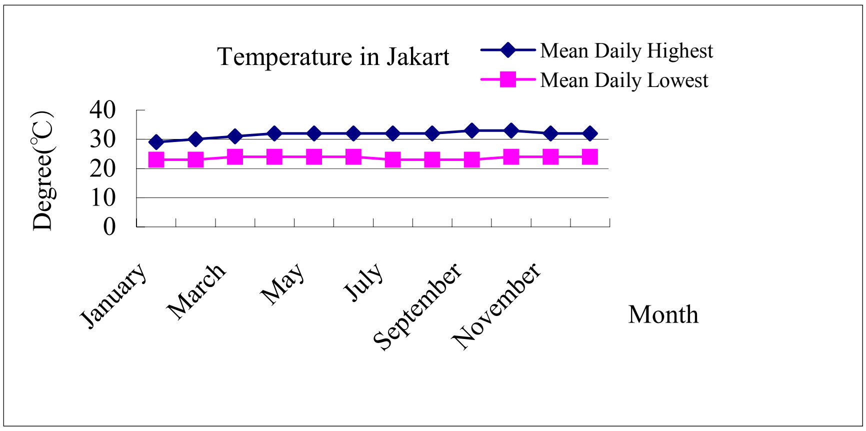
6. BREAKWATER RELOCATION, DREDGING AND DUMPING WORKS

6.1 Construction Site Conditions

(1) Meteorological and Wave Conditions

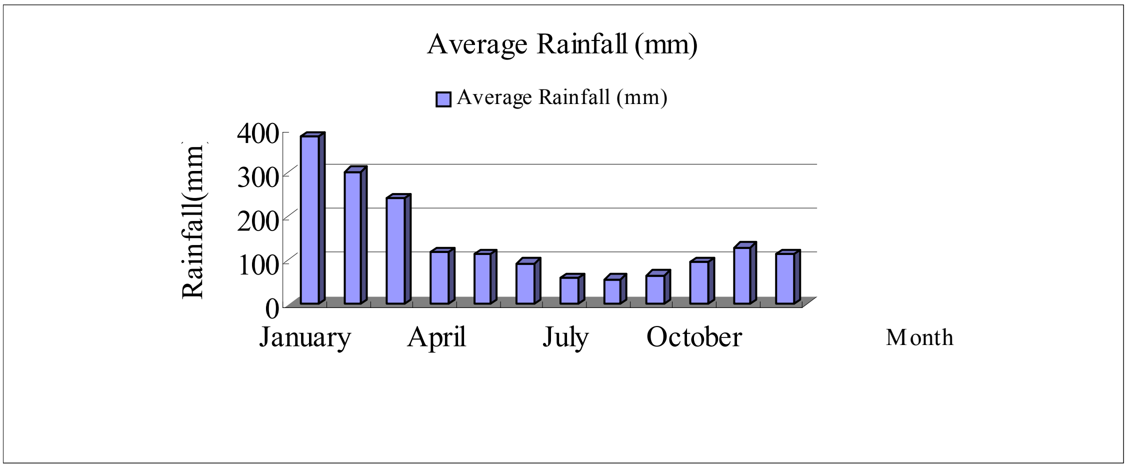
In order to decide possible working days per year on the sea, meteorological and wave conditions have been reviewed and examined. The region covering the construction site has the tropical monsoon climate, which has two seasons namely, northwest monsoon from November to March next year and southeast monsoon from April to November. During the northwest monsoon season average rainfall is comparatively much more than southeast monsoon season. So it can be said that the wet season starts from November or December to March and the dry season begins from April or May to November.

Figure 6.1.1 and Figure 6.1.2 show average temperature with small seasonal variation and rainfall by month.



Source: The Study for Development of Greater Jakarta Metropolitan Ports Final Report, JICA, December 2003

Figure 6.1.1 Average Temperature by Month (1934-1994)



Source: The Study for Development of Greater Jakarta Metropolitan Ports Final Report, JICA, December 2003

Figure 6.1.2 Average Rainfall by Month (1934-1994)

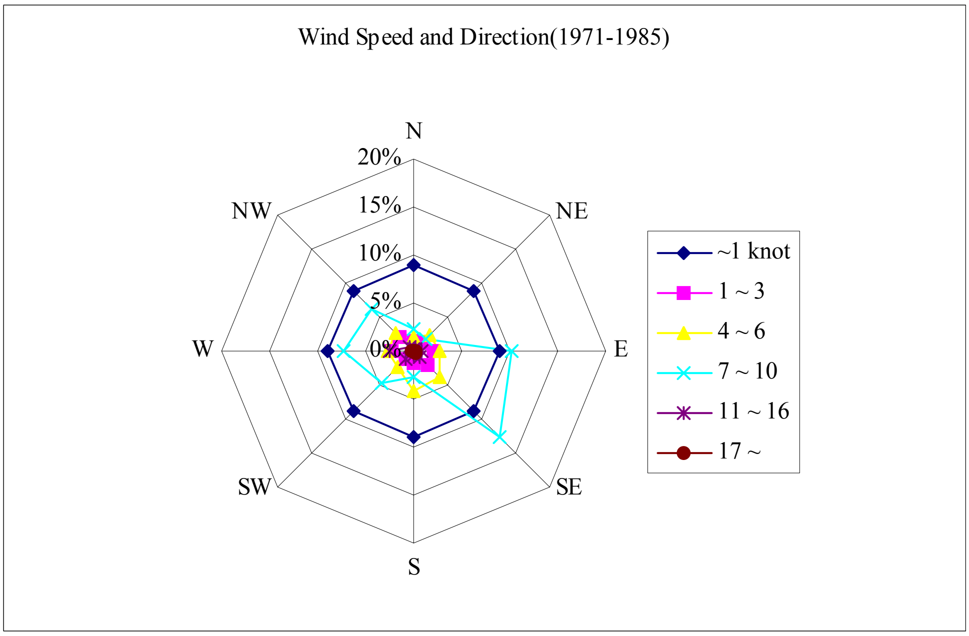
Temperature and rainfall sometimes might affect concrete or other works. Then it should be considered down time for such works but these stopping hours by temperature or rainfall usually in Jakarta are short enough to reconsider and adjust whole working schedule anyway. On the other hand, strong wind e.g. over 10m/sec and high wave exceeding 0.5m would affect the working schedule on the sea to some extent.

Table 6.1.1 shows wind statistic data obtained from Hydro-oceanographic Service Centre of Indonesia Navy in the West Java Sea during past 15years.

Table 6.1.1 Wind Directions and Speed in the West Java Sea (1977~1985)

Knot	N	NE	E	SE	S	SW	W	NW	Total
Calm	-	-	-	-	-	-	-	-	669
1 ~ 3	78	98	162	163	92	71	172	152	988
4 ~ 6	106	182	210	294	310	175	196	205	1678
7 ~ 10	172	129	766	953	209	359	547	468	3603
11 ~ 16	9	16	59	64	26	84	182	43	483
17 ~	2	1	12	11	5	6	15	0	52
Total	367	426	1209	1485	642	695	1112	868	7473
	N	NE	E	SE	S	SW	W	NW	Total
Calm	-	-	-	-	-	-	-	-	0.0895
1 ~ 3	0.0104	0.0131	0.0217	0.0218	0.0123	0.0095	0.023	0.0203	0.1322
4 ~ 6	0.0142	0.0244	0.0281	0.0393	0.0415	0.0234	0.0262	0.0274	0.2245
7 ~ 10	0.023	0.0173	0.1025	0.1275	0.028	0.048	0.0732	0.0626	0.4821
11 ~ 16	0.0012	0.0021	0.0079	0.0086	0.0035	0.0112	0.0244	0.0058	0.0646
17 ~	0.0003	0.0001	0.0016	0.0015	0.0007	0.0008	0.002	0	0.0070
Total	0.0491	0.057	0.1618	0.1987	0.0859	0.093	0.1488	0.1162	1.0000

Source: Weather Data Indonesian Water, Hydro-oceanographic Service Centre of Indonesia Navy 1995



Source: Compiled and drawn from Weather Data Indonesian Water, Hydro-oceanographic Service Centre of Indonesia Navy 1995

Figure 6.1.3 Wind Rose in the West Java Sea

Wind speed 17knots corresponds to 8.7m/sec and the occurrence of over 17knots is 0.7% in equivalent to 2.6days per year. Dominant wind directions were SE to E and W to NW as and dominant wind speed were 7~10knots(3.6m/sec~5.1m/sec) as shown in the Figure 6.1.3.

According to another wind data (1997~2002 cf. Table 6.1.2) obtained at the Cengkareng meteorological station (Soekarno-Hatt Airport) and compiled by JICA Study Team for development of Greater Jakarta Metropolitan Ports, more than 10m/sec wind occurred only 0.5%(1.8days per year). Gales are reported in April, May and December. However, they are rarely happened by the description of above said study report.

Table 6.1.2 Wind Observation Data at Cengkareng (1997-2001)

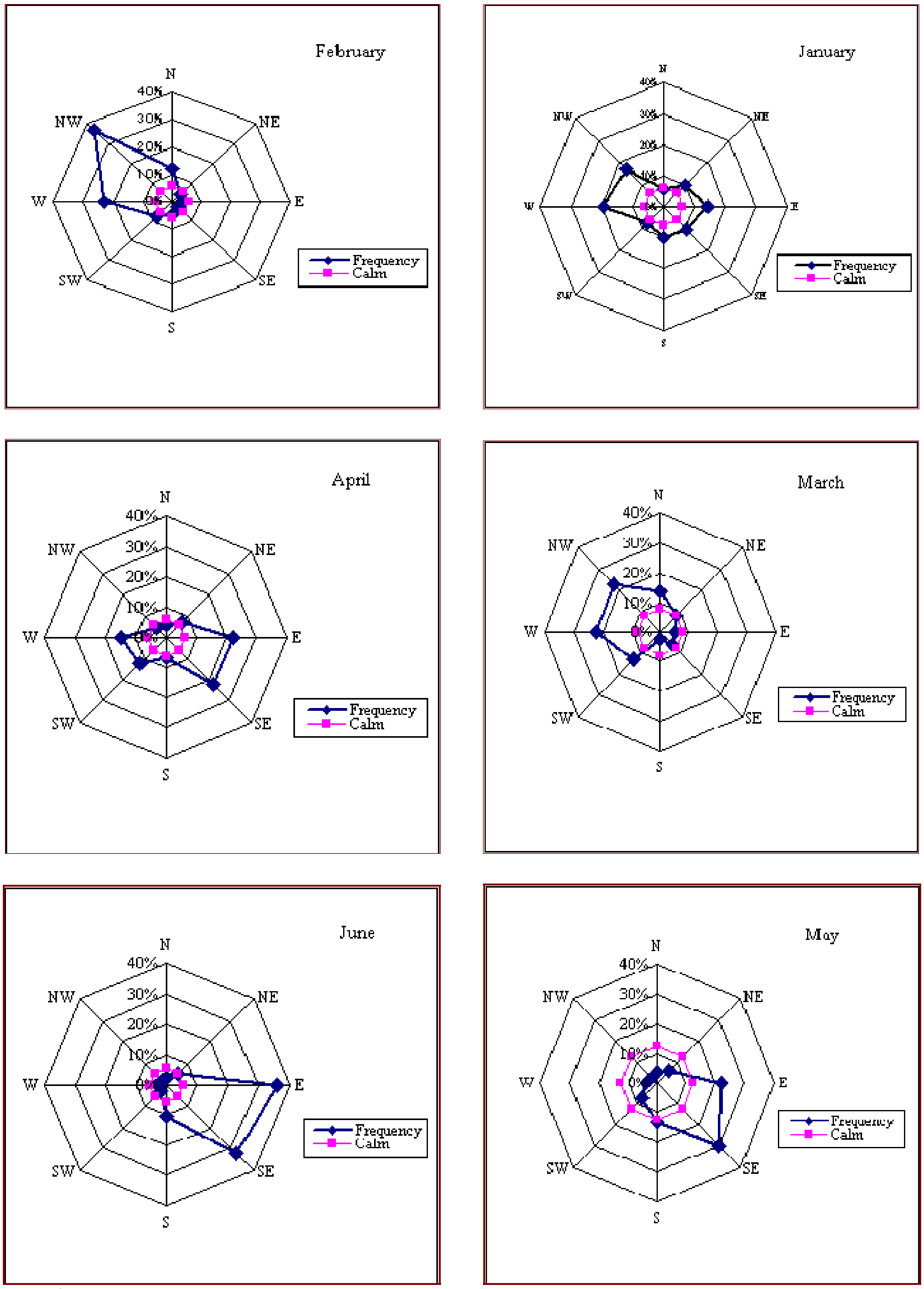
Direction / Speed (m/s)	N	NNE	NE	ENE	E	ESE	SE	SSE	S	SSW	SW	WSW	W	WNW	NW	NNW	calm	TOTAL
0.0 ~																	6,120	
1.0 ~	265	172	163	143	384	368	633	1,054	2,191	1,223	932	804	605	218	177	188		9,520
2.0 ~	494	390	287	247	348	312	527	893	1,538	860	1,083	1,355	1025	340	325	418		10,442
3.0 ~	596	492	384	247	262	103	166	233	270	252	592	1,103	839	289	346	476		6,650
4.0 ~	565	581	540	324	189	73	44	76	84	127	305	789	643	272	322	413		5,347
5.0 ~	310	461	562	332	137	29	21	16	36	66	267	616	626	163	275	384		4,301
6.0 ~	86	175	351	247	70	13	11	14	15	16	121	383	412	75	106	111		2,206
7.0 ~	48	63	240	158	39	11	9	7	8	7	55	242	286	48	34	61		1,316
8.0 ~	33	17	106	98	17	2	2	1	6	5	38	93	121	23	11	29		602
19.0 ~	16	10	50	61	6	1	2	1	4	3	5	37	71	18	11	17		313
10.0 ~	10	5	14	25	1	2		1	1		6	14	31	6	7	4		127
11.0 ~	1	3	9	8	1			2		1	3	3	9	4	1	1		46
12.0 ~		1	5	6	1				1		1	2	10	2				29
13.0 ~	1		1	5	1				1			1	5	3	2			20
14.0 ~			1			1							7					9
15.0 ~				1									5	1				7
16. ~													1					1
TOTAL	2,425	2,370	2,713	1,902	1,456	915	1,415	2,298	4,155	2,560	3,408	5,442	4,696	1,462	1,617	2,102	6,120	47,056

Source: Supporting Report of Engineering Study for development of Greater Lakarta Metropolitan Ports, 12. 2003 JICA

Figure 6.1.4 (1) and Figure 6.1.4 (2) show wind direction and occurrence in the West Java Sea by month. These data cover the 5 degree (longitude and latitude) square area of the West Java Sea during the past 15years from1971 to 1985 and were obtained by Hydro-Oceanographic Service Indonesian Navy in Tanjung Priok.(Source: [Weather Data Indonesian Water] Published by Hydro-Oceanographic Service Indonesian Navy in 1995)

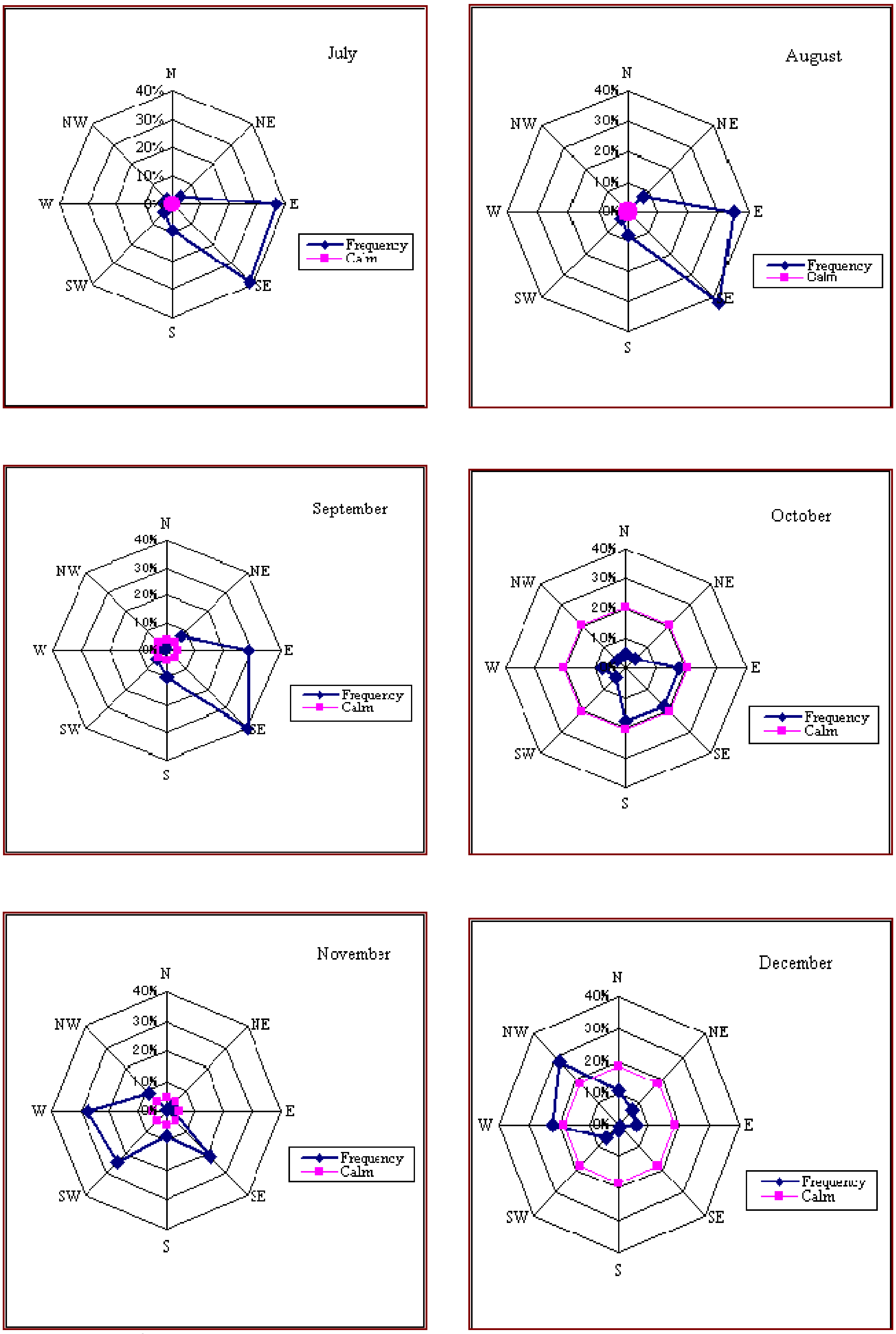
It is well depicted particularity of tropical monsoon wind, the NW to W winds were dominant during the wet season (from November to next year March or April) and dry season (after April to October) the SE to E winds usually blew.

Usual works on the sea are particularly affected by wind and wave condition. Dredging work will be carried out at inner and outer of the harbour and the planned dumping site is located far from dredging areas approximately 15 to 16 sea miles. However, it is located within the Java Bay where the climate is rather moderate than West Java offshore Sea, winds and waves are not strong throughout the year. Based on the data shown in the Table 6.1.2, JICA Study Team in 2003 had been conducted the wave forecast used SMB method. Combined occurrence of wave height and direction is presented in the Table 6.1.3. The waves generate offshore area of Tanjung Priok Port and the frequency of wave height exceeding 0.5m is about 13%.



Source: Figured from the data [Weather Data Indonesian Sea] ,Hydro-Oceanography Service Indonesian Navy,1995

Figure 6.1.4 (1) Wind Direction by Month in West Java Sea (1971-1985)



Source: Figured from the data [Weather Data Indonesian Sea], Hydro-Oceanography Service Indonesian Navy,1995
Source: Figured from the data [Weather Data Indonesian Sea], Hydro-Oceanography Service Indonesian Navy,1995

Figure 6.1.4 (2) Wind Direction by Month in West Java Sea (1971-1985)

However, the construction site of new breakwater and demolishing breakwaters are located in the shallow water and also protected by Tanjung Karawang from NE~E waves and by Tanjung Pasir from NW~W waves which means that wave force becomes weaker than that one in offshore. Furthermore existing Dam Timur protects effectively eastward working areas from W to NW direction waves. Therefore, except extreme case, we can suppose that winds and waves would not affect so much nor stop continuing works on the sea.

Table 6.1.3 Combined Occurrences of Wave Height and Direction (%)

Height \ Direction	W	WNW	NW	NNW	N	NNE	NE	ENE	E	Total	Cumulative
Calm										68.55	68.55
0≦H<0.25	2.15	0.33	0.31	0.39	0.54	0.54	0.46	0.28	0.37	5.36	73.92
0.25≦H<0.5	3.79	0.88	0.92	1.15	1.30	1.49	1.67	0.85	0.64	12.70	86.61
0.5≦H<0.75	2.07	0.47	0.45	0.51	0.43	0.71	1.11	0.81	0.33	6.89	93.50
0.75≦H<1.0	1.32	0.40	0.16	0.13	0.09	0.24	0.51	0.44	0.15	3.43	96.93
1.0≦H<1.25	0.67	0.20	0.09	0.04	0.02	0.10	0.24	0.20	0.08	1.64	98.58
1.25≦H<1.5	0.29	0.16	0.02	0.01	0.01	0.03	0.08	0.12	0.03	0.76	99.34
1.5≦H<1.75	0.13	0.08	0.01	0.02	0.00	0.01	0.04	0.05	0.01	0.36	99.69
1.75≦H<2.0	0.06	0.02	0.01	0.01		0.01	0.03	0.04	0.00	0.18	99.87
2.0≦H<2.5	0.04	0.05	0.01					0.02	0.00	0.11	99.99
2.5≦H<3.0	0.01							0.00		0.01	100.00
3.0≦H<3.5											
Total	10.52	2.59	1.97	2.26	2.40	3.14	4.14	2.81	1.62	100.00	
(0.5≦H)	(4.58)	(1.38)	(0.74)	(0.72)	(0.56)	(1.11)	(2.01)	(1.68)	(0.61)	(13.39)	

Source: Modified the data from Supporting Report of Engineering Study for development of Greater Jakarta MetropolitanPorts,12.2003JICA
Note: Estimation point is at boarder line of the Jakarta Bay and the West Java Sea.

Possible working condition of marine works differs kind of works, particularity of sea conditions especially wave and wind speed or current. Table 6.1.4 shows a limit of various sea works in general.

Table 6.1.4 Working Limit of Sea Condition

Item	Kind of Works	Significant Wave Height (m)	Average Wind Speed (m/sec)	Rainfall (mm/hour)	Maximum Current (knot)
Navigation and Transportation	Self- propelled	1.0m	15m/sec	-	2
	Towing or Pushing	0.6m	10m/sec	9	2.5
Dredging	Pump	0.6m	11m/sec	10	2
	Grab Bucket	0.6m	11m/sec	10	2
Various Works	Anchoring	0.5m	10m/sec	8	2
	Sinking or settlement	0.4m	6m/sec	2	1
	Hanging	0.5m	7m/sec	5	1
	Pile driving	0.4m	8m/sec	4	1
	Filling , Throwing	0.7m	10m/sec	10	2
	Concrete	0.5m	9m/sec	5	2
	Diving, Levelling	0.5m	8m/sec	10	1

Source: Port Facilities Construction published by Sankaido in 1980

Referring to Table 6.1.4 the limit of works on the sea of the Project will be determined.

Down time of the Project for usual construction works on the sea condition would be assumed as followings. (Except trailing suction hopper dredger)

Due to strong wind (over 17knots including gale) ----------------------------**3** days per year

Due to high wave(over 0.5m NNW~NNE) (0.72+0.56+1.11)%×365=**9** days per year

Total 12 days

As for demolishing Dam Barat, it is better to choose dry season when wind blows from SE-E direction which less affect the works.

(2) Traffic Condition

Traffic conditions around Tanjung Priok Port are described precisely in Chapter3.1 Nautical and Operational Aspects of Existing Port and Chapter3.2 Ship Maneuvering Patterns. However, it is noteworthy from the point-view of implementation of sea works of the Project to point out related critical matters.

Main channel passes through between Dam Barat and Dam Timur and the most dangerous area is around the entrance water area (150m wide and -14.0m deep), due to there in no enough space to avoid collision of ships on a miss manoeuvring.

The monitoring control tower is located at the left corner of Pier No 2, which has three (3) floors. The operating control room itself takes the uppermost of the monitoring control tower where staffs from the pilot office communicate with ships going out and coming in and conduct traffic control 24hours by 2 shifts through VHF radio communication system only. They can use a double lens telescope auxiliary in the daytime but there is no radar system, which put difficult to control night time navigation extraordinarily. More than one hundred ships call every day and number of ship calls don’t differ day and night according to the staffs.

Over 500 GRT ships are compulsory maneuvered by pilots. However, the responsibility on a secure ship maneuvering shall be borne by the ship captain (skipper) complying with the Government Decree. The number of tugboats accompanying a ship within the Port is determined as follows:

LOA ≧150m 3 tugboats

LOA < 150m 2 tugboats

Ship waiting area is located seaside of the breakwaters and at both side of the channel. An out going ship has the priority to navigate first and the entering ships have to wait for out going ships pass through the entrance.

The west entrance in the only operational entrance, accepts commercial ships mainly. East entrance (-5.0m) is used freely only for small ships such as fishing boats, tugboats etc. Therefore, it is recommended that work vessels use the east entrance as much as possible to avoid traffic congestion and sea accidents.

(3) Environmental Condition

According to the Government Regulation “Port Affairs of Republic of Indonesia” Decree No.69 promulgated in 2001, Port areas are divided by “Port Working Area” and “Port Interest Area”. In case of Tanjung Priok Port, “Port Working Area” is water and land areas of inner breakwaters where port commercial activities are performed daily. "Port Interest Area." covers mostly surrounding water area of outer breakwaters area including offshore anchorage or waiting water areas from east Ancol (near RUKINDO work base) to Kalibaru port area.

Water quality in inner harbour was comparatively mal-graded especially levels of nutrient salts (Ammonia T-N, T-P) exceed Indonesian Standard. Sediment quality in inner harbour also is likely affected by human and industrial contaminated discharge as well. It means that contractors should pay much attention not to give impact to water areas during dredging or other works especially in the harbour.

On the surface of seabed in both inner and out of the harbour fluid mud layers are prevailing. These fluid mud layers are supposed less than 0.5m thick. However, during dredging works it is requested to prevent diffusing high SS water discharge around dredging points because high SS water provably gives certain impact to ecology in the seawater.

Dredging soil should be transported and dumped in the proposed new dumping area located northwest area of Tanjung Karawang indicated in Figure 6.3.2. JICA team should conduct turbidity analysis during dredging and dumping then evaluate the impact. If the analysis suggests moderating impact, countermeasures should be considered.

Along the eastern Jakarta Bay, there are many fishing stands and nets as shown in the Figure 10.1.1 and 10.3.14. Usually, fishermen come to these fishing stands and nets at night time and catch fishes gathered by lights attached to the fishing stands. Dredging works, particularly by trailing suction hopper dredger (TSHD) is planed to carry out dredging through night. Therefore, it is necessary for a TSHD to prevent collision with fishing boats or fishing stands and nets.

(4) Others

According to the data of infrastructure research group, which complies contractors from Japan, experts from DGSC and others, there were seventy five thousand contractors in Indonesia at the end of the year 2001 and out of them, four thousand belong to Corporative of Indonesian Contractors (GAPENSI), one hundred twenty belong to the Association of Contractors in Indonesia (AKI). It means that Indonesian contractors have sufficient experiences to carry out various kinds of construction works. Among them, there are major state-owned construction companies with enough capacity to the Project shown in the Table 6.1.5 and RUKINDO is a sole state-owned dredging company. (cf. 6.3.2)

Table 6.1.5 Major State-owned Construction Company

Name of Company	Share Holder			Field			
	State	Employee & Manage	Public	Construction	Building	Mechanical &Electrical	Engineering
PT. Adhi Karya(Persero)	51%	24.5%	24.5%	○	○	○	○
PT.Brantas Abipraya (Persero)	100%	-	-	○			
PT.Hutama Karaya (Persero)	100%	-	-	○			
PT.Istaka Karaya (Persero)	100%	-	-	○			
PT Nindoya Karaya (Persero)	100%	-	-	○			
PT.PP（Persero)	51%	49%		○	○		
PT.Waskita Karya(Persero)	100%	-	-	○			
PT.Wijya Karya(Persero)	100%	-	-	○			

Source: Directory of Construction Company 2004

6.2 Breakwater Construction and Demolition

6.2.1 Principle of Construction/Demolition Plan

Relocation works comprise demolishing the existing breakwaters and construct new breakwaters in front of water areas of the existing breakwater alignment. Work plan that contains work methods, equipments and stage-wise schedules should be considered with basic issues or promises rely on the following principles.

(1) Basic Promises

1) Not interrupt or interfere present ship navigation nor port operation.

Tanjung Priok Port is the busiest port in Indonesia with that the number of ship calls was over one hundred per day, or the average interval time of fourteen minutes per ship (cf. Appendix6.A). And even liner service ships entering port were sometimes compelled to wait in offshore water area because of one way traffic control. Therefore, the works should not disturb nor interfere usual ship movement or navigation no matter how the work cost would increase. It can be avoided considering work method, order of works or schedule and limiting working area etc.

2) Prevent waves coming into the channel and Basin areas and keep tranquility as much as possible.

Demolishing breakwater without new wave prevention facilities means that waves easily intrude into the harbour and disturb cargo handling at the berths, if the waves would be large enough to affect berthing ship movement.

According to **"Technical Standards and Commentaries for Port and Harbour Facilities in Japan published by OCDI in January 2002", Calmness of Basin** (Notification Article 22, Clause 4) is stipulated as follows.

The calmness of specified level shall be achieved for 97.5% or more days of the year, except for those cases where the use of mooring facilities or the area in front of mooring facilities is categorized as a special use. And the commentary said that the threshold wave heights for cargo handling for the basins in front of mooring facilities should be determined appropriately in consideration of the type, size, and cargo handling characteristics of the vessels. For this purpose, values listed in the bellow Table may be adopted. Calmness of basin is usually evaluated by the wave height of the basin, but it is desirable to consider as necessary the effects of the wave direction and period, which affect the motion of the moored vessels as well.

Threshold of Wave Height for Cargo Handling

Ship Size	Threshold Wave Height for Cargo Handling ($H_{1/3}$)
Small-sized ships	0.3m
Medium-and large-sized vessels	0.5m
Very large vessels	0.7~1.5m

Note Small-size ships are the vessels smaller than about 500GT that mainly use the basin for small crafts and very large ships are the vessels larger then about 50,000GT that mainly use large dolphins and offshore berths. Medium-and –large ships are the vessels that do not belong to the small-sized and very large ship categories.

As discussed and concluded in the Appendices Chapter 4A **"Operational Downtime Assessment"**, based on a down time criteria of H1/3=0.25m(more sever than Japanese Standard above mentioned), 1.37%=5days, Optimum length of overlap in Dam Tengah, is concluded 160m. Its 160m-overlap length can be used as a principle that new breakwater preceding length is to be followed 160m ahead of demolishing Dam Tengah.

3) Avoid increasing sedimentation as much as possible.

As same as wave penetration issue it should be remained not to accelerate sedimentation during relocation works of breakwaters. The source of sedimentation is likely seabed soil transferred from offshore to inner harbour by wave or current actions. These actions will be weakened by breakwater overlapping length. The other large sources are originated from Kali Sunterbaru, Terusan Lagoa and Kali Japat drainage channels. The survey done by JICA team and reported in the" Supporting Report of Engineering Study for Development of the Greater Jakarta Metropolitan Ports" shows that estimated yearly seabed variation becomes +7.0m, +3m~+4m and +0.4m, at the outlet of Kali Sunterbaru, Terusan Lagoa and Kali Japat respectively. Total volume flowed into the harbour has not been assessed. However, over half of maintenance dredging(about 200,000M3 per year) should be needed in the basin from in front of pier to Dermaga TPK Koja. This fact verified large quantity of soil flowed into harbour through these drainage channels. Therefore, relocation works of Dam Tengah itself is not directly to increase sedimentation caused by these drainage channels flows.

4) Reuse materials of the existing breakwaters to maximum extent.

Demolishing part of Dam Timur and Tengah are rubble mound composite type of breakwaters, which consist of concrete cubic blocks, crown concrete, rubble stones and sand as shown figure 6.2.1. New breakwater structure is decided also similar rubble mound composite type with replacement foundation. (cf. Figure 6.2.2) So that almost of all demolishing materials can be reused for new breakwater construction. Considering nearness of demolishing site and new breakwater construction site and quality of demolishing materials, it is suitable to use demolishing materials for replacement works of new breakwater except concrete cubic blocks. N value of the replacement part is expected over 10 and it means that internal friction degree is to be over 30°. From the past experience and testing of replacement part of crashing crown concrete that are mixed with gravel and sand can be anticipated more than 10 N-value foundations. As for concrete cubic block they can be reused for new breakwater inner slope protection material. As a conclusion, from point view of cost saving and recycling, demolished material should be reused to maximum extent to adjust the speed of demolishing and new construction.

(2) Certain Ideas of Making Relocation Plan

Based on the above-mentioned basic premises, there comes up certain basic ideas for relocation plan.

1) Construction of new breakwaters and removal works of existing breakwaters should be simultaneously started from the view-point of reusing existing materials to maximum extent.

If we would follow strictly 160m-overlap plan between new breakwater and the existing breakwater, it is natural to construct new breakwater first, then demolishing works follow. The first work of new breakwater is excavating soft soil and dumping new sand purchased and transported to construct the foundation for upper structures. It means that at least 160m long we cannot use demolishing materials for replacement. Therefore, removal works of existing breakwater should be started at the same time with excavation works for new breakwater.

2) Removal works and works for new breakwaters will be commenced from the foot position of Dam Timur toward east direction because of less influence part against wave action.

Conveniently, we have to remove the foot of Dam Timur 290m, segment 1 and 234m, segment 2. These parts extend nearly parallel to dominant wave direction from the North. Furthermore, these areas are sheltered from strong waves of W – NW direction. Considering these circumstances, it is very suitable for removal works to start at the north end of Segment 1 to southward. After demolishing segment 1 the open length to the north direction is only about 30m.

3) Set a temporal barrier to prevent wave and decrease sedimentation before the new breakwater become effective.

JICA Study Team is now conducting wave and sedimentation analysis during construction period when the 50m wide open-entrance exists. If the result of the analysis would be fatal, a temporal wave preventing measures might be introduced. A barrier might be constructed with submerged dike using removal materials, old scraped ships sank down or floating barges loaded with removed concrete blocks. Without barrier case removal of existing breakwaters desirably will be remained at –1.0m level before the new breakwater becomes effective to expect preventing high wave action and decreasing sedimentation.

4) Using existing concrete blocks a small jetty will be formed at the foot of Dam Timur for dissipating running waves along the breakwater.

The first removal works is to take existing concrete cubic blocks and stock in some place till the new breakwater rubble stone body will be completed. So, it takes at least half year till reusing these concrete blocks. According to the soil boring along the section of Dam Timur, within 15m area of seaward, there exists rather stiff layers' consisting gravel, sand with gravel and sand. These places (shallow than –2.5m) are suitable not only to construct a small jetty reusing concrete cubic blocks without soil improvement works but also can be used for temporal stock places for concrete cubic blocks removed from the existing breakwaters.

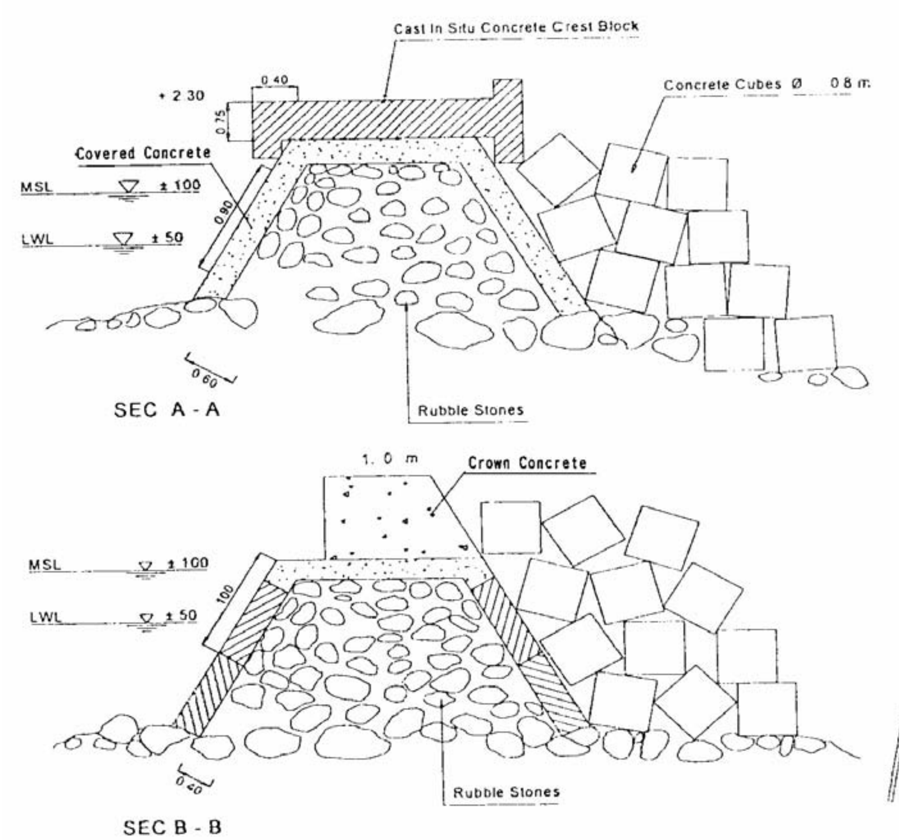
Based on these ideas Alternative plans simply can be summarized and evaluated in Table 6.2.1.

Table 6.2.1 Comparison of Alternative Work Plan for Breakwater Relocation

Plan	Alternative 1	Alternative 2	Alternative 3		
Evaluation Item	Same time start removal and new construction	After new construction removal start	Make barrier beforehand of removal work		
			Option 1	Option2	Option3
			Submerged dike of removed material	Sank Ship	Barge loaded with removed Concrete block
Tranquility	○	●	◊	●	○
Sedimentation	○	○	◊	○	◊
Reuse of Material.	●	◊	●	◊	◊
Easiness &Speed of Works	○	◊	▲	▲	●
Environment	▲	▲	▲	○	○
Cost	●	○	▲	▲	▲
Total Evaluation	●	○	▲	○	○

●: Best ○: Better ◊: Fair ▲: Inferior

Comparing each evaluation by item and considering total evaluation, alternative1 can be selected as the optimum plan. Alternative 3 is good as the idea itself and there were examples in past marine works but they are not practical from the point-view of urgency or necessity of preventing wave, sedimentation and cost saving. Alternative 2 has an advantage to wave action but it is not economical from reuse of existing material.



Dam Timur

Dam Tengah

Source: IPC-II

Figure 6.2.1 Typical Section of Dam Timur and Dam Tengah

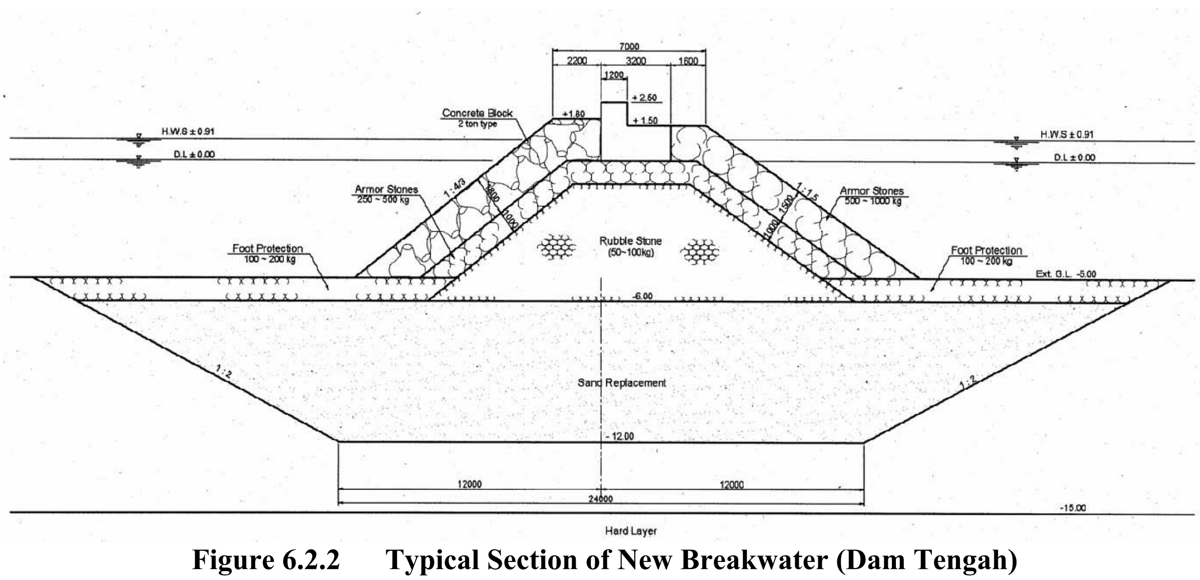


Figure 6.2.2 Typical Section of New Breakwater (Dam Tengah)

6.2.2 Work Quantities of Construction/Demolition of Breakwater

(1) Order of Remove and Demolishing Works

The first step of works is to remove concrete cubic blocks and transport and put them on the stock places. The storing places can be chosen along the north-side of seaward area of Dam Timur as discussed in the previous chapter (2) 4). Necessary length might be about 200m. This work will be done by clamshell on the flat barge from seaward area. Starting point is as already mentioned at the foot of Dam Timur near the tip of new breakwater.

Second works is to crash and demolish crown concrete and covered concrete, then transport and dump to the excavated area for new breakwater foundation. Reusable materials including gravel and sand under crashed and demolished part will be estimated approximately one hundred twenty six thousand cubic meter as shown in Table 6.2.3, that is, less than 40% of necessary replacement volume. It means that we can transport and dump these materials directly to the excavated site without storing. We will need excavators with giant breakers on a flat barge and other work vessels for this works.

The third work is to excavate core bodies consisting of mostly gravel stones of the existing breakwaters down to -4.0m ~ -5.0m. This work needs big crawler cranes with clamshell or orange peal type buckets on the flat barges.

The fourth works is dredging works by large grab bucket dredgers with more than -4.0m draft equipped by heavy type grab buckets, if necessary. These grab bucket dredgers will engage excavating the remained body material and rather hard clayey layers stiffed by upper load of existing breakwater structures. This works continue to the down level of -14.0m because almost all of removing areas will be used as basins -14.0m deep. Excavating soil and small portion of the remained materials by the third works will be transported to new designate dumping areas as disposals.

Table 6.2.2 Gravel and Sand Volume of Dam Timur and Dam Tengah

Kind of Layer	Segment1	Segment 2	Segment 3-1	Segment3-2	Segment3-3	Segment4,5	Total (m3)
Length (m)	287m	236m	200m	300m	419m	163m	1,605m
Gravel (m3)	12,230	10,050	0	0	6,620	2,580	31,480
Sand & Gravel (m3)	23,220	19,090	20,780	24,000	24,470	9,520	121,080
Sand (1)(m3)	5,220	4,300	8,920	8,190	26,440	10,290	63,360
Sand (2)(m3)	0	0	3,260	4,950	2,260	880	11,350
Total	40,670	33,440	32,960	37,140	59,790	23,270	227,270

Source: JICA D/D Team
Note: Except Dam Barat

Table 6.2.3 Summary of Re-usable Materials Volume

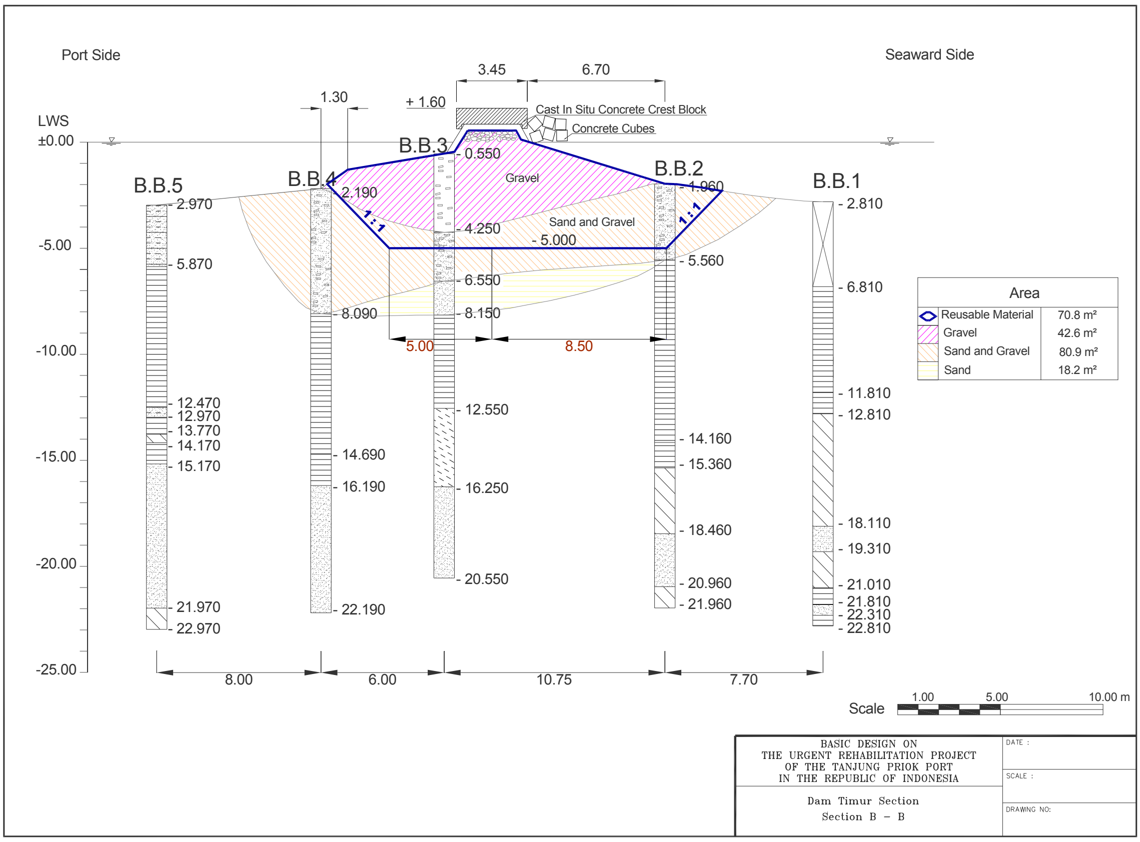
No.	Description	Area (m2)	Length (m)	Volume (m3)
1	Segment 1	70.8	287.0	20,320
2	Segment 2	80.1	236.0	18,900
3	Segment 3	79.8	919.0	73,320
4	Segment 4	82.0	93.0	7,630
5	Segment 5	82.0	70.0	5,740
	Total	-	1,605.0	125,910

Source: JICA D/D Team
Note: Segment 6 (Dam Barat) is 25,400m3 Volume is estimated from only boring profile

(2) Reusable Quantities

Along the existing breakwaters Dam Timur and Dam Tengah, 5 sections with five bore-log surveys have been executed. Using these bore-log observation reusable materials and its quantities are estimated. The bore-log observation reveals that just under the upper structure covered by concrete there exist gravels but size of these gravel is rather small. Most of them pass through boring-rod with the diameter 89mm.

By these boring-logs, gravel, gravel & sand and sand can be calculated as shown Figure 6.2.3. Total volume amounts 227,270m3 as shown in Table 6.2.2. Considering checking criteria of quality of reusable materials in supervision and actual excavation parts (bottom level, seabed width of excavation and alignment), reusable volume has been calculated based on the section shown in Figure 6.2.3(1)~(5). Total number resulted 125,910 m3 that is 55% of estimated existing reusable volume. (cf. Table 6.2.3). Reusable section sets 1.0m shallow level of the lines estimated by boring- log as a general rule and bottom width of excavation is 13.5m at-5.0m level. This excavation line is straight from the starting point of Dam Timur to the corner of segment 2 and 3, then, again takes straight line to the end of segment 4.



Source: JICA D/D Team

Figure 6.2.3 (1) Excavation Section shown with Boring-Log in the Segment 1

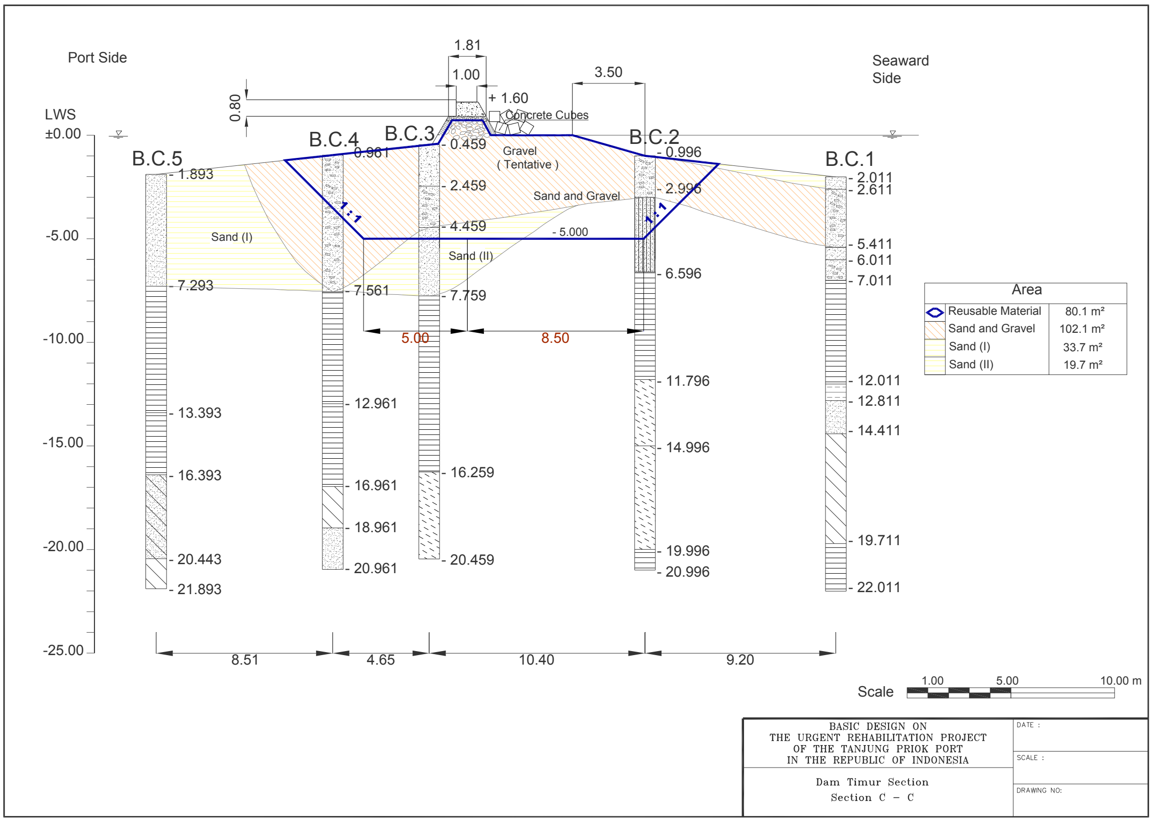


Figure 6.2.3 (2) Excavation Section shown with Boring-Log in the Segment 2

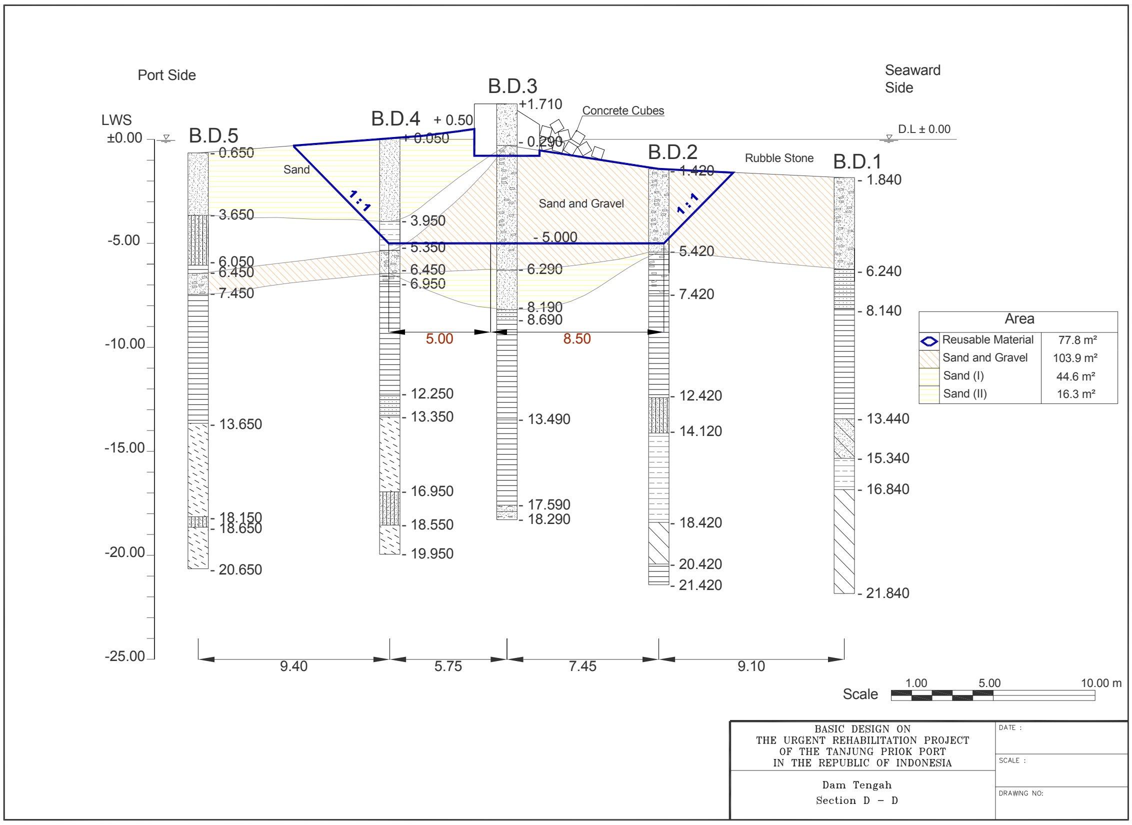


Figure 6.2.3 (3) Excavation Section shown with Boring-Log in the Segment 3

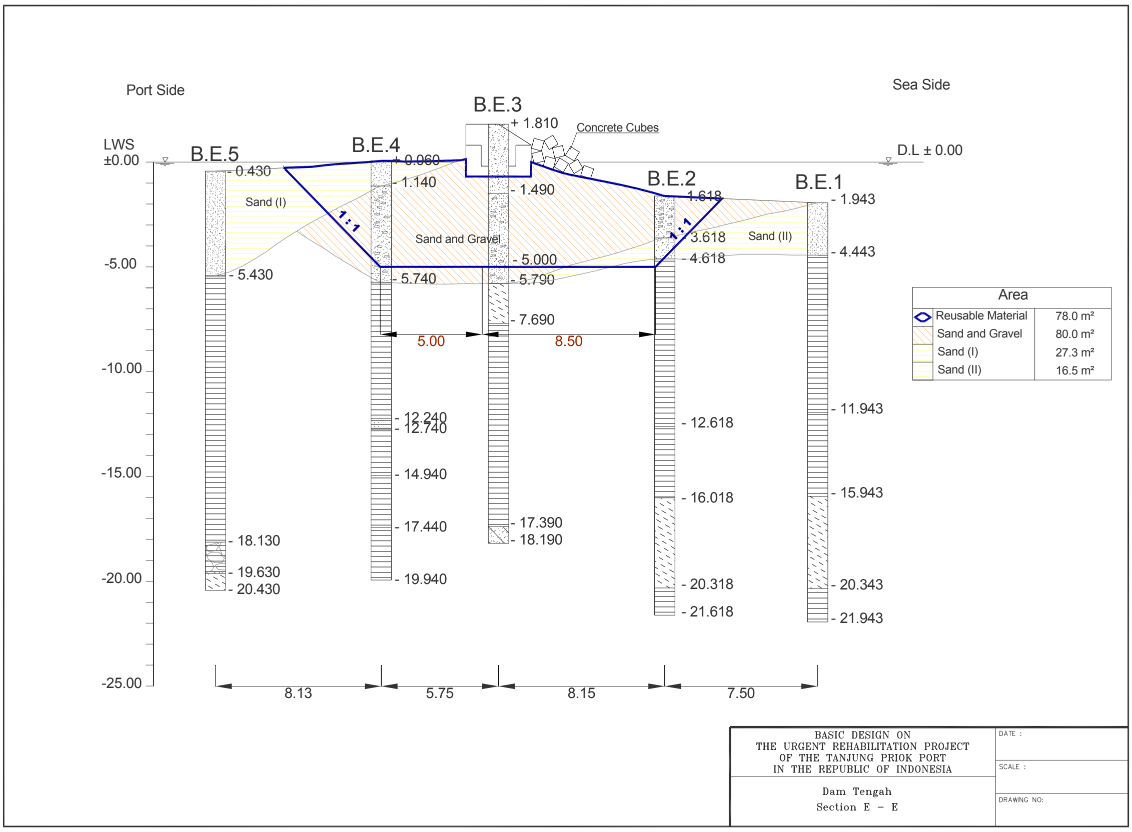


Figure 6.2.3 (4) Excavation Section shown with Boring-Log in the Segment 4

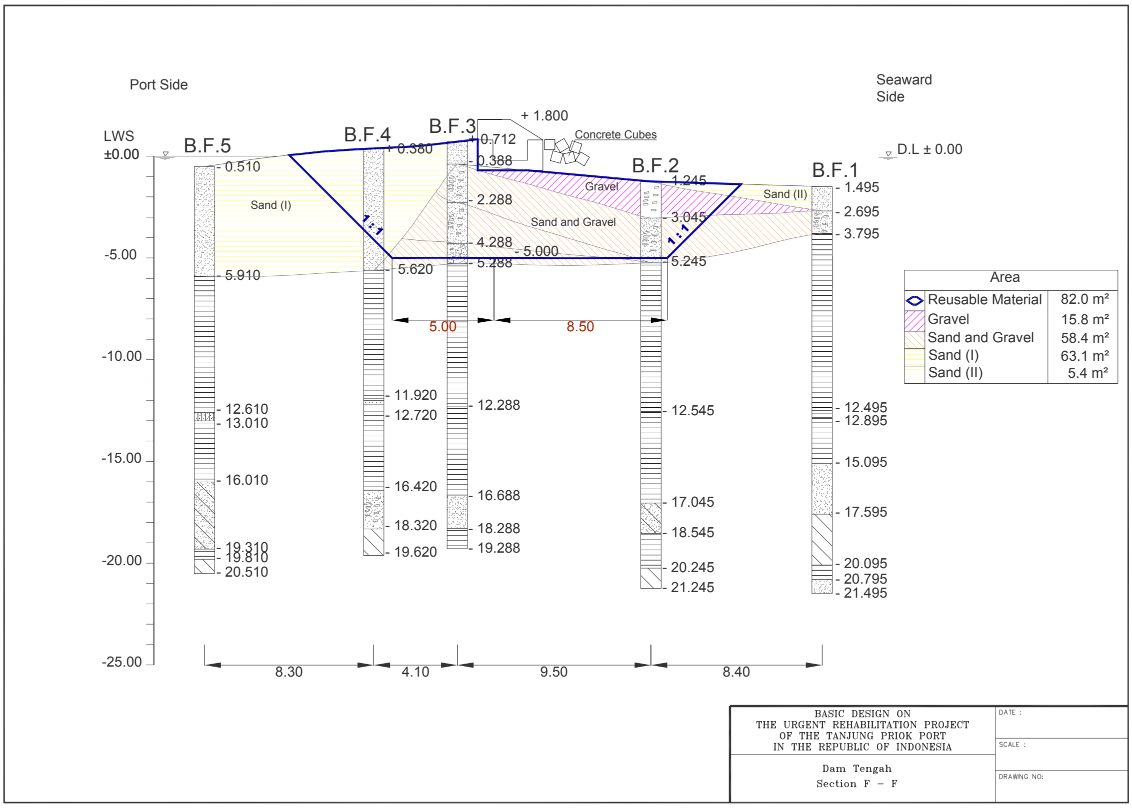


Figure 6.2.3 (5) Excavation Section shown with Boring-Log in the Segment 4,5

(3) Order and Work Quantities of New Construction of Breakwaters

The first step of new construction works of breakwaters is excavating soft soil for replacement. Considering wave effect and reusing materials demolished as replacement, the starting point is naturally decided at the west end corner of straight line of the central breakwater segment B. The straight alignment west to east is the most effective part to prevent the wave penetration from North. Then from west to east excavation work should proceed and this work should be followed as early as possible to throw or dump new sand or the reused materials because of preventing surrounding mud not to flow into the excavated section. Segment B is 1,092m long so that we should divide 2 or 3 sections to adjust the excavation and replacement speed. Segment C 318.2m long is the closing length of new breakwater so, next priority should be given to Segment A with 151m length. So the order of works will proceed Segment B, A, C. (pls. compare in the next page)

The most suitable equipment is a grab bucket dredger with necessary work vessels, tug boat, hopper barges, anchor and work boat. Excavated soft soil should be transported and dump in to the designated dumping area about 27km far from the excavation site. Cycle time of excavation and transportation will be discussed latter.

The second work is filling with reusable materials and new sand. It Attention should be paid for no remain of soft soil and fluid mud on the excavated seabed. Reusable materials are taken by clamshell on the flat barge. Its speed is not so fast so that it needs to stop excavating soft soil and wait till filling works finish.

The third work is throwing rubble stones (50~100kg per piece) on the surface filled with sand to form breakwater core body. This work is also done by clamshell on the flat barge loaded with rubble stones. Rubble stones will be transported directly from Bojonegara or Merak quarry sites by barge equipped with excavators. Levelling and forming slope of rubble stone works will be executed by clamshell.

The fourth work is constructing armour layer by more large rubble stones 250kg~500kg and 100kg ~200kg per piece. This work will proceed as same as form rubble stone mound.

The fifth works is to set pre-cast concrete block of 22.1ton (3.2 W×1.0H×3.0L) for upper structure. Then, it is followed by placing concrete blocks both reused cubic concrete blocks (0.8×0.8×0.8m, 1.26ton/piece) and new deformed concrete blocks (2ton, 3ton). For these works crawler cranes on the flat barge are suitable.

The last works is to cast in situ concrete for upper crown body. This work will be done with ready mixed concrete loaded and agitated by concrete mixer truck on flat barge and skip hanged by crawler crane. Instead of skips small concrete pumping vehicles can be adopted.

Quantities for new breakwater is summarised in Table 6.2.4.

Table 6.2.4 Quantities for New Breakwater (New Dam Tengah, New Dam Barat)

Description	Unit	Segment A (151m)	Segment B (1,092m)	Segment C (318.2m)	Dam Barat (100m)	Total (1,661.2m)
Excavation for Replacement	m3	38,950	366,730	-	5,370	411,050
Replacement Materials	m3	40,740	396,940	-	6,980	451,640
Rubble Stones	m3	5,540	130,840	20,060	3,300	159,740
Armour Stone	m3	7,100	28,170	10,180	3,300	48,750
Pre-cast Coping	pcs	50	364	-	34	448
Cast in situ concrete	m3	537	3,495	-	302	4,334
Concrete cubic Block	pcs	-	17,092	-	-	17,092
Concrete deformed Block	pcs	1,809	13,660	-	2,516	17,985

Source: JICA Team Note: Rubble and Amour Stone is included 30% increase ratio in actual works. Segment C (New Dam Tengah-2) needs bamboo mattress 7,800m^2 . New Dam Barat needs bamboo piles 3,248 nr.and mattress 2,328 m^2

Demolishing the existing breakwater will start from the cross point of new breakwater segment A and Dam Timur and works proceeds Segment1→2→3→4→5. New construction will commence west end of segment B to eastward and when the works will reach the end of segment B, Work vessel fleets will shift to segment A, then, segment C. Demolishing speed might be slower than new construction breakwater speed. Then, it makes possible to reuse demolishing materials for replacement works of new breakwater construction.

One of big reason why the segment A will be latter stage is that the open length to the north is approximately 100m and less affects tranquility of inner harbour than segment B and approach section of segment A should be needed complicated structure. Furthermore, work vessels should need the entrance during relocating works and new construction works as well.

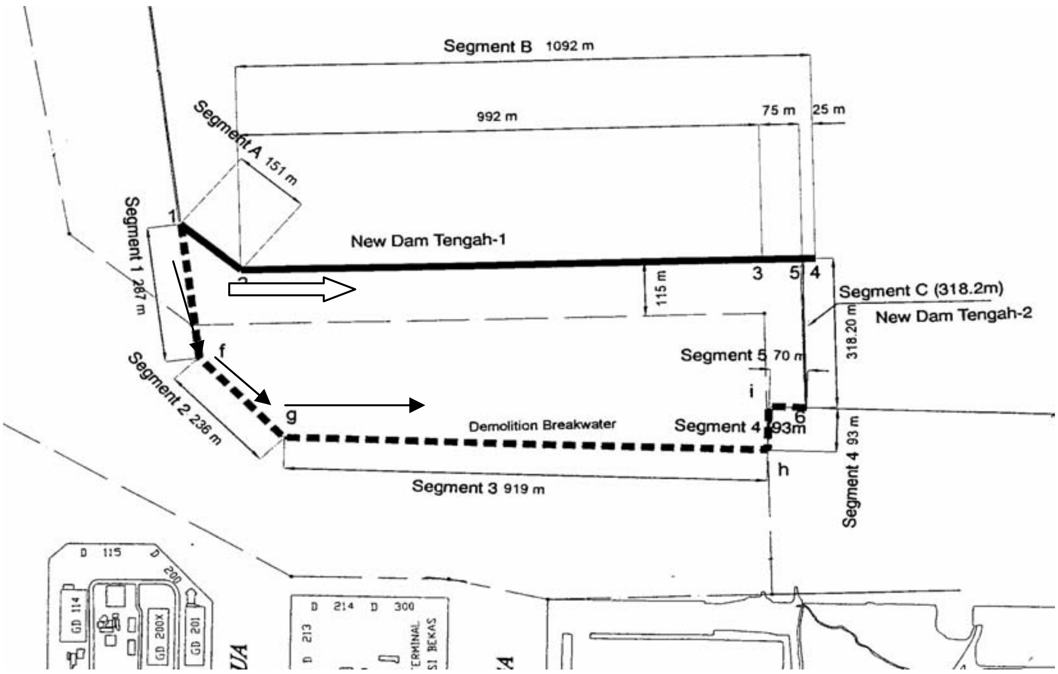


Figure 6.2.4 Work Process between Demolishing and New Construction Works

However, even if a 100m open part will affect tranquility of inner harbour or sedimentation. So that the shooter is the open period, the less influence is anticipated.

According to the work schedule and period discussed in 6.2.4. (5), the duration of opening period of Segment A will last for less than 8 month. Figure 6.2.5 shows the relation between demolishing speed of segment 1～3 and completion speed of rubble stone mound (-0.5m) of new breakwater segment B.

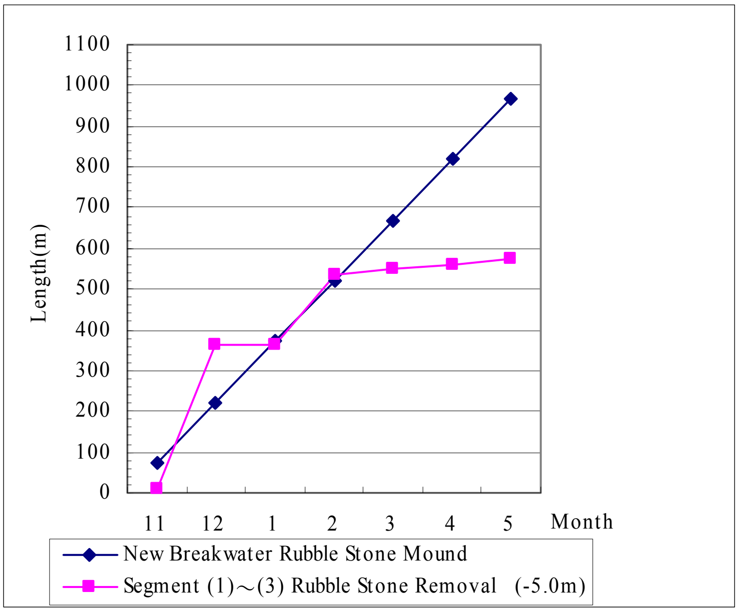


Figure 6.2.5 New Breakwater Construction Speed and Demolition Speed

After five months since the commencement of new construction works and demolition works, the rubble stone mound of segment B will be completed 510m long. While demolition works will progress about 560m. It means that behind areas of about 510m section can be protected by rubble mound (-0.5m) from open-sea current and wave coming into the harbour except the behind area of segment A (151m). But after five month the existing rubble mound of Dam Tengah (segment 3,4) will be removed faster than the construction speed of new rubble mound. Therefore, the gap (open length) will increase till consecutive 3 month later. But 7 months after the commencement of works the gap will become decreasing then, 10 months later the gap will be closed except the section Segment C (Dam Tengah-2-- 318.2m). (cf. Appendix6 B Stage Wise Figure)

The open section of the segment C gives less affection to inner harbour because of weak wave from eastward comparing ones from northward. It is desirable open to keep navigation course for trailing suction hopper dredger, which will engage with dredging works in area 5 (cf. Figure 6.3.1) without passing west entrance to avoid sea traffic congestion. However, it is recommended that careful observation for sedimentation in the vicinity of the open area should be carried out and taken an appropriate measure if tranquillity would decrease and sedimentation would become much.

6.2.3 Construction Works Detailed

(1) Construction Work Base

As for construction works of new breakwaters, construction work bases are needed for accommodating work vessels, manufacturing pre-cast concrete blocks, transporting rubble stones, sand and concrete blocks and other works. Especially, for manufacturing concrete blocks and transporting them to the site on the sea, we need rather wide yards because number of deformed 2-ton or 3.2-ton blocks for wave dissipating is estimated 17,985 pieces totally. The location of needed work base is the nearer, the better the construction site. In this sense, if there are some places could be found in Tanjung Priok Port area and arrowable by IPC-II, it is convenient for the Project. However, considering severe traffic congestions in and around Tanjung Priok Port and environmental impact caused by the construction work, it should be decided to find outer area of the Port.

The JICA Team carried out a reconnaissance survey around Tanjung Priok Port and found three candidate places for construction work bases shown in Figure 6.2.5.

In East Ancol area near P.T. (Persero) PENGERUKAN INDONESIA (RUKINDO) base, there are berths for lighters or barges about 150m long sheltered by a breakwater. Behind these berths, there are about 4ha land spaces owned by P.T. Manggala and Ekahuri where actually are used for a stockyard of gravel stones or concrete piles etc. Sea distance from this place to the new breakwater construction site is about 6.5km.

In Cilincing east of Kalibaru Port about 1.5km there are berths for small boats, stock yard for sand, other materials and bamboos. A few construction material suppliers, for example P.T.Citra Urip Langgeng, use these facilities. Behind berths there are small open yards. The scale of this place is rather small but is available for the work base of the project. Sea distance from this place to the new breakwater construction site is about 5.5km.

Marunda port located about 7km from the new breakwater construction site is under control of IPC-II. There are long berths about 1,100m with −3m deep. These berths are owned by 4 private companies and used as mostly unloading berths for large barges transporting vast amount of sand from Bangka Island in south Sumatra. These sand are supplied as mostly concrete fine aggregate or construction use. Behind these berth, there are wide aprons and open stock yards. Very near to these berths there is a wide open sand stockyard owned and operated by big sand supplier P.T. Karya Technik Pasir Indo. This company supplies to some concrete ready-mixed factories, for example, P.T. Pionirbeton Industri located in Taman Mini about 20km from Marunda Port.

For conducting the works the Contractor should establish the actual work base, but it might be impossible to use inner areas of Tanjung Priok port as work base. Therefore, these three places are candidates for JICA Team to consider in the work plan especially to set cycle time of transporting concrete blocks to the site or others.

Figure 6.2.6 Candidates for Work Base

(2) Selection of Work Vessels and Equipment

Site conditions (space for working area, depth, approach distance etc.), suitability for works, availability, capacity and cost have been examined, in order to select work vessels, fleets and equipments. And through the interview to a few marine Contractors in Jakarta and reconnaissance survey of the working site of surrounding Tanjung Priok port adding to the past experiences and references for work vessels selection, the following work vessels and equipments by each kind of works are selected to prepare working plan.

a. Excavation for Replacement

- Grab Dredger Capacity 8 m³
- Hopper Barge Capacity 1000 m³
- Anchor Boat/ Work Boat (1x250 Hp)
- Tug Boat (2 x 600 Hp)

b. Replacement as Removal Material or Sand

- Crane Capacity 50 t
- Flat Barge (180 ft)
- Anchor Boat/ Work Boat (1x250 Hp)
- Tug Boat (2 x 500 Hp)

C. Rubble Stone Mound

- Crane Capacity 80 t
- Flat Barge (180 ft)
- Anchor Boat/ Work Boat (1x250 Hp)
- Tug Boat (2 x 500 Hp)
- Grab Bucket Cap. 2 m³
- Wheel Loader 3m3

d. Amour Stone

- Crane Capacity 80 t
- Flat Barge (180 ft)
- Anchor Boat/ Work Boat (1x250 Hp)
- Tug Boat (2 x 500 Hp)
- Grab Bucket Cap. 2 m³
- Wheel Loader 3m3

e. Crown Concrete

-Manufacture pre-cast coping block (22.1t/3m/piece)

- Crane 50 ton
- Steel Form

- Transport and place coping block

- Crane 50 ton
- Flat Barge (130 ft)
- Trailer Truck
- Anchor Boat/ Work Boat (1x250 Hp)
- Tug Boat (2 x 500 Hp)

- Crown concrete cast in-situ (3.55m3/m)

- Flat Barge (130 ft)
- Anchor Boat/ Work Boat (1x250 Hp)
- Tug Boat (2 x 500 Hp)
- Agitator truck 4.5m3 Concrete Pump Truck

f. Concrete Block (Inner Harbour)

- Flat Barge (130 ft)
- Anchor Boat/ Work Boat (1x250 Hp)
- Tug Boat (2 x 500 Hp)
- Grab Bucket 2 m³

g. Concrete Block(Outer Harbour)

- Anchor Boat/ Work Boat (1x250 Hp)
- Tug Boat (2 x 500 Hp)
- Crane Capacity 50

-Manufacture 2t deformed concrete block placing

- Crane 50ton
- Steel Form

(3) Capacity of Work Vessels or Equipment

Capacity of a Grab bucket dredger usually can be figured out by the following formula.

$q = q' \times f \times K \times 60^2 / Cm$

Here: q: capacity of excavating (dredging) volume per hour (m3/h)

q': Nominal capacity of grab bucket

f: Changing ratio of excavated soil

K: Excavation efficiency of grab bucket

Cm: Cycle time of grab bucket

Table 6.2.5 Changing Ratio of Excavated Soil (f)

Kind of Soil		Changing Ratio of Excavated volume
Classification	N-Value	
Clayey Soil	Under 10	**0.95**
	10~30	0.90
	30~50	0.85
Sandy Soil	Under 10	0.90
	10~30	0.85
	30~50	0.80
Soil mixed with Gravel	30	0.85
	30~50	0.75
Rock	Soft	0.75
	Medium	0.65
	Hard	0.65

Source: Cost Estimation Standard of Port and Harbour Works in Japan, Ministry of Land, Infrastructure and Transport in 2004

Table 6.2.6 Excavation Efficiency (K) and Cycle Time (Cm)

Kind of soil		Garb Dredger Standard (for Ordinal Foundation)									
Classification	N-Value	D2.5 m^3		D5.0 m^3		D9 m^3		D15 m^3		D23 m^3	
		K	Cm	K_3	Cm	K	Cm	K	Cm	K	Cm
Clayey Soil	Under 10	1.05	75 (sec)	1.10	80 (sec)	**1.15**	**90** (sec)	1.20	110 (sec)	1.20	130 (sec)
	10~30	0.70		0.75		0.90		1.00		1.00	
Sandy Soil	Under 10	0.90		0.95		1.05		1.10		1.10	
	10~30	0.65		0.70		0.75		0.85		0.90	
Soil mixed with Gravel	30	0.35		0.45		0.55		0.60		0.65	

Source: Cost Estimation Standard of Port and Harbour Works in Japan, Ministry of Land, Infrastructure and Transport in 2004

In case of excavation of soft clayey layer N- value under 10 by 8m3(in the Table 6.2.6 instead of 8m3, D9m3can be used) grab bucket dredger (f), (K), (Cm) can be chosen 0.95, 1.15 and 90 respectively. Then, capacity per hour can be calculated bellow.

q=8.0×0.95×1.15×60×60/ 90=**349.6m3**/hour

Necessary number of tugboat and hopper barge with bottom door open per day can be calculated by following formula.

Tugboat

N=qo/f×(1/5+d/v1+d/v2)/B

Hopper Barge

N+1

Here:

qo: capacity of grab bucket dredger per hour (m3/h)---349.6 m3/hour

f : changing ratio of excavated volume------------------0.95

d : towing distance(km)------------------------------------27km

v1: towing speed going (km/h)-----------------------4.5knot(8.325km/h)

v2: towing speed returning (km/h)-------------------5.5knot(10.175km/h)

B: nominal capacity of hopper barge---------------1,000 m3

Tugboat N=349.6/0.95×(0.2+27/8.325+27/10.175)/1000=368×5.896/1000=2.17=3

Hopper barge N+1=3+1=**4**

(4) Productivity and Work Period of Excavation

Excavation productivity can be estimated by capacity of grab dredger and cycle time of dumping as shown in Table 6.2.7

Table 6.2.7 Excavation Productivity and Work Period

Description	Figure and Calculation
Excavation Performance per Cycle	1,000m3 per cycle
1.Working Cycle Time	8.8 hour
Time to Fill 1,000m3 barge	2.7 hour (1000/349.6)
Sailing Time (loaded)	3.2 hour (27km/4.5knot=27/8.325)
Dumping Time	0.2 hour
Sailing Time (empty)	2.7 hour (27km/5.5knot=27/10.175)
2.Effective Working Time per Day	8~10hour
3.Excavation Cycle per Day	3cycle (8h/2.7h=2.96≒3)
4.Production per Day and per Month	3,000 m3 3,000×24 days/month=72,000 m3
5.Excavation Volume	366,730 m3 Segment B=1,092m
6.Work Period	366,730 m3 /72,000 m3=5.1month

Effective working time per day and working day per month are supposed 8hours and 24days respectively in general. As already mentioned before, down time by natural condition is 12days per year. It means that the average down time is one day per month. By assuming one day per week for off work, workdays are 24days per month. According to "the study on reinforcement of dredging capacity in Indonesia" March 2003 and other studies, monthly operation days for grab bucket dredger are set 24days as well.

The calculation done results work period of 5.1 month for segment B 1,092m long. However, excavation works cannot proceed continually to meet the supply speed of reusable materials and new sand (cf.**6.2.4**), although replacement works should follow excavated section as early as possible for preventing fluid mud sedimentation. The total excavation work length of segment B will be divided into 2 or 3 sections (one length would be 500m or 330m through segment B). At least a150m length space should be needed for work vessels entering into an excavated section for dumping replacement materials.

(5) Other Works

1) Small Wrecks Identified by Diver's Survey

JICA Team conducted a magnetometer survey, side-scan sonar survey and diver survey to find and identify anomaly points, wrecks or obstacles for dredging or other marine works around the project sea areas. (Cf. Report, Marine Architecture, Bathymetric, Seismic Profiling, Magnetometer, Side Scan Sonar and Diver Surveys for Urgent Rehabilitation Project of The Tangjung Priok Port, Volume II, August 2005 by PT. Diagram Triproporisi Engineering Consultant in consortium with PT. Carsurin and Marmoya Barunapersada.)

There were found by side-scan sonar survey and identified by diver 4 points of small wrecks in the vicinity area of along new breakwater construction site. 7 points of metal or metal cluster debris were recognized by side scan sonar survey as well. Figure 6.2.6 shows the location of 4 wrecks and 7 metal debris (including 2 metal cluster debris) along the new breakwater alignment and 13 metal debris (including 1 metal cluster debris) in the planned dredging areas. Hatching area in the Figure 6.2.7 means the excavation area including the slope for replacement, in which there are 2 points of wrecks, 1 metal debris and 2 metal cluster debris.

Figure 6.2.8 shows the side scan images of wreck No.W005 and No.W007. No.W005 was estimated approximately 12.0m long and 8.0m wide and 0.6m high from seabed. The material was made of steel and it was so corroded that the diver took off a part of steel plate easily by his hand. Wreck No. W007 was the same condition as No.W005, which had 11.0m long, 6.3 wide and 0.6m high above seabed. Whether the core bodies of the wrecks, for example like engine, were buried and remained or not, could not be confirmed by side scan sonar survey nor diver survey.

However, it will not need to stop dredging work and prepare an additional work fleet, because JICA Team has planned to utilize a rather large grab dredger with 20 m3 bucket for excavation work to meet the speed of supplying reusable materials from demolishing the existing breakwaters as already mentioned. A large 20m3 grab bucket is probably able to grasp more than 15.0m long corroded hulls or mass of the wreck. It takes probably 2 or 3 days to clear one wreck during dredging work by the grab bucket dredger.

If dredging work would difficult to continue at the worst, dismantling the core body of the wreck shall be examined according to the condition of the wreck. JICA team supposes metal or metal cluster debris disturbing dredging works so much to stop or slow down without special treatment.

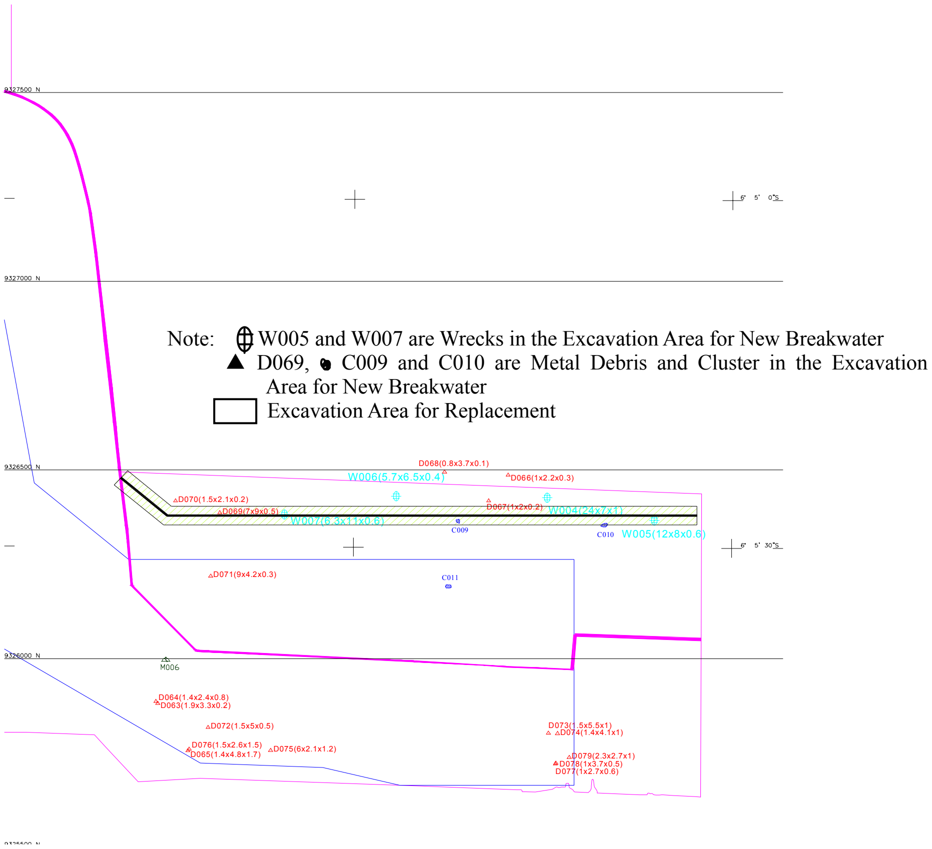


Figure 6.2.7 Location of Wrecks and Metal Debris near New Breakwater

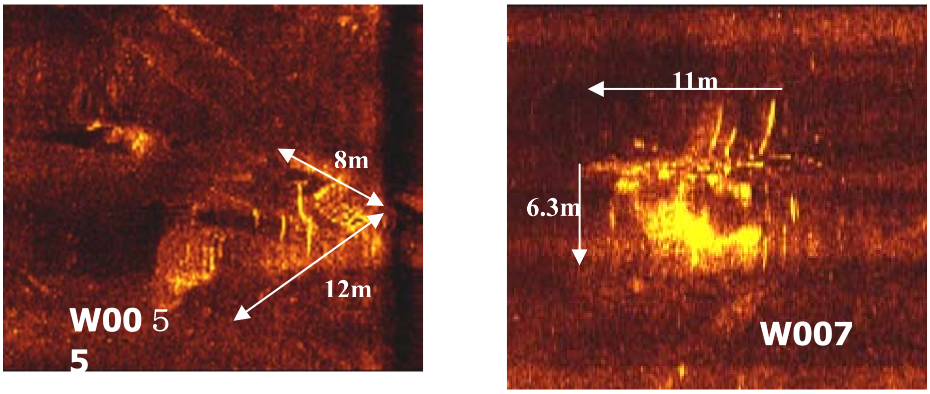


Figure 6.2.8 Side Scan Image of Wreck No.W005 and No.W007

2) Rubble Stone and Armour Stone

Clamshell work vessel fleet will form rubble stone and armour stones for new breakwater bodies. The fleet consists of a crawler crane attached with 2.0m^3 grab bucket and a 180 feet flat barge, a tugboat with horse power 1,000 and an anchor & work boat with horse power 250. A crawler crane will be fixed on a 180 ft. flat barge loaded about 800m^3 rubble or armour stones at berths of the work base by wheel loader. Unloading cycle time is estimated 2.0 minutes and by assuming the working hour of 8 hours per day, productivity per day could be estimated 360 m^3 in both case of rubble stone and armour stone. It is assumed that 2 minutes cycle time includes levelling and forming works as well.

3) Concrete Crown

It is assumed two processes for constructing the concrete crown. First, manufacture and place the concrete block on the surface of rubble mound at elevation ±0.0m. The size of pre-cast concrete coping blocks is 3.2 m wide, 3.0m long and 1.0m thick and weighing 22.1ton per piece to avoid casting fresh concrete in water as designed position. Second is to cast ready mixed concrete for remained part of crown concrete. Manufacturing the blocks needs rather wide areas adjacent to water front lines easy to load on a flat barge for placing on the sea. Manufacturing and placing speed for pre-cast coping blocks can be estimated 10 pieces and 20 pieces per day respectively.

4) Concrete Block

Concrete blocks on amour stones are designed as protective layers for both offshore side and harbour side. Reused concrete cubic block of 0.8m×0.8m×0.8m weighing 1.2 ton should be placed on the slope of harbour side. Necessary number of these blocks will amount 17,092 pieces for segment B. Incidentally, these number are less than the removal number of the existing breakwater. However, some blocks will be damaged to certain extent during removal and placing works. Estimation of concrete blocks damage ratio is very difficult, nevertheless, it will not be needed to manufacture new one. On the other hand, for offshore side slope, deformed concrete blocks are needed. Number of these new blocks is calculated 13,660 pieces for segment B. JICA Team estimates the manufacturing productivity of these blocks to be 100pieces per day (necessary concrete volume will be 87m3.) This work is critical due to wide concrete block manufacturing yards and the work itself takes long period. Practically two or three sites should be prepared for manufacturing yard and loading site to place blocks on the sea. In addition to necessary yards and berths, quite a big number of steel forms are necessary. By assuming three days for curing after casting, necessary number of steel forms would be amount of 300sets.

6.2.4 Demolition Plan Details

(1) Work Quantities of Demolition

Work quantities of demolishing the existing breakwaters are tabulated as bellow.

Table 6.2.8 Quantities of Demolition

Description	Unit	Segment 1.2 (523m)	Segment3, 4, 5 (1,082m)	Segment6 (258m)	Total (1,863m)
Remove concrete block	pcs	6,018	11,630	3,063	20,711
Demolish crown and cover concrete	m3	3,510	9,590	2,215	15,315
Remove rubble stone down to –5.0m	m3	38,470	93,220	38,930	170,620
Dredge under –5.0m to –14.0m	m3	93,240	192,890	45,990	332,120

Note: net volume calculated based on section drawings

(2) Selection of Work Vessels and Equipment

As mentioned in **6.2.3(2)** suitable work vessels and equipments can be supposed as follows. These work vessels fleets shall be engaged independently from the other party of work vessels and equipments for new construction work of breakwaters.

a. Remove concrete block

- Crane Capacity 50 t
- Flat Barge (180 ft)
- Anchor Boat/ Work Boat (1x250 Hp)
- Tug Boat (2 x 500 Hp)

b. Demolish crown and cover concrete

- Flat Barge (130 ft)
- Crane Capacity 50 t
- Excavator with Breaker
- Flat Barge (180 ft)
- Anchor Boat/ Work Boat (1x250 Hp)
- Tug Boat (2 x 500 Hp)

c. Remove Rubble Stone down to elevation -5.00 m

- Flat Barge (130 ft)
- Crane Capacity 50 t
- Grab Bucket Cap. 5,50 m^3
- Flat Barge (180 ft)
- Anchor Boat/ Work Boat (1x250 Hp)
- Tug Boat (2 x 500 Hp)

d. Dredge under -5.0m to -14.0m

- Grab Dredger Capacity 20 m^3
- Hoper Barge Capacity 1000 m^3
- Anchor Boat/ Work Boat (1x250 Hp)
- Tug Boat (2 x 600 Hp)

(3) Critical Work

Demolish works have four steps as said above,

a. remove concrete block,

b. demolish crown concrete and covered concrete,

c. remove rubble stone down to –5.0m and

d. dredge under –5.0m to –14m.

Dredging volume to the planned elevation –14m after excavation of reused materials amounts 332,120m3. According to the borings and seismic tomographic survey, soil dredged is classified as clay but rather stiff one. Considering dredging volume and stiffness of soil, a large grab bucket dredger is suitable for this works. Therefore, the grab bucket dredger equipped with 20m3 grab is selected to utilize. However, the dredging work is critical to the total demolition work period.

(4) Capacity and Productivity

Capacity and productivity can be calculated in the same way as descried in **6.2.3 (3)**

$q = q' \times f \times K \times 60^2 / Cm = \mathbf{301.4} m^3/hour$

Here: q: capacity of excavating (dredging) volume per hour (m^3/h)

q': Nominal capacity of grab bucket=**20**m^3

f: Changing ratio of excavated soil =**0.85**(See in Table6.2.5 clayey soil N=30-50)

K: Excavation efficiency of grab bucket=**0.65**(See in Table 6.2.6 D23m3, soil mixed with gravel)

Cm: Cycle time of grab bucket=130sec=**2.2**minute

Table 6.2.9 Dredging Productivity and Work Period

Description	Figure and Calculation
Dredging Performance per Cycle	1,000m^3 per cycle
1.Working Cycle Time	9.4 hour
Time to Fill 1,000m3 Barge	3.3 hour (1000/301.4)
Sailing Time (loaded)	3.2 hour (27km/4.5knot=27/8.325)
Dumping Time	0.2 hour
Sailing Time (empty)	2.7 hour (27km/5.5knot=27/10.175)
2.Effective Working Time per Day	8~10hour
3.Dredging Cycle per Day	3cycle (10h/3.3h=3.03≒3)
4.Production per Day and per Month	3,000 m^3 ×24 days/month=72,000 m^3
5.Dredging Volume	286,130 m^3 Segment 1~5=1,605m
6.Work Period	286,130 m^3 /72,000 m^3=4.0month

(5) Work Schedule and Period

The tentative construction schedule and work period is prepared based on detailed work plan described above. Total works period is 22.6month as shown in Figure 6.2.9. Demolition works Segment 1~5, Dam Timur and Dam Tengah will take 9.3 months. However, these works should carry out parallel with new breakwater construction. Segment C, temporally will close Dam Tengah–1 is planned to be done latter stage after the dredging works by trailer suction hopper dredger.

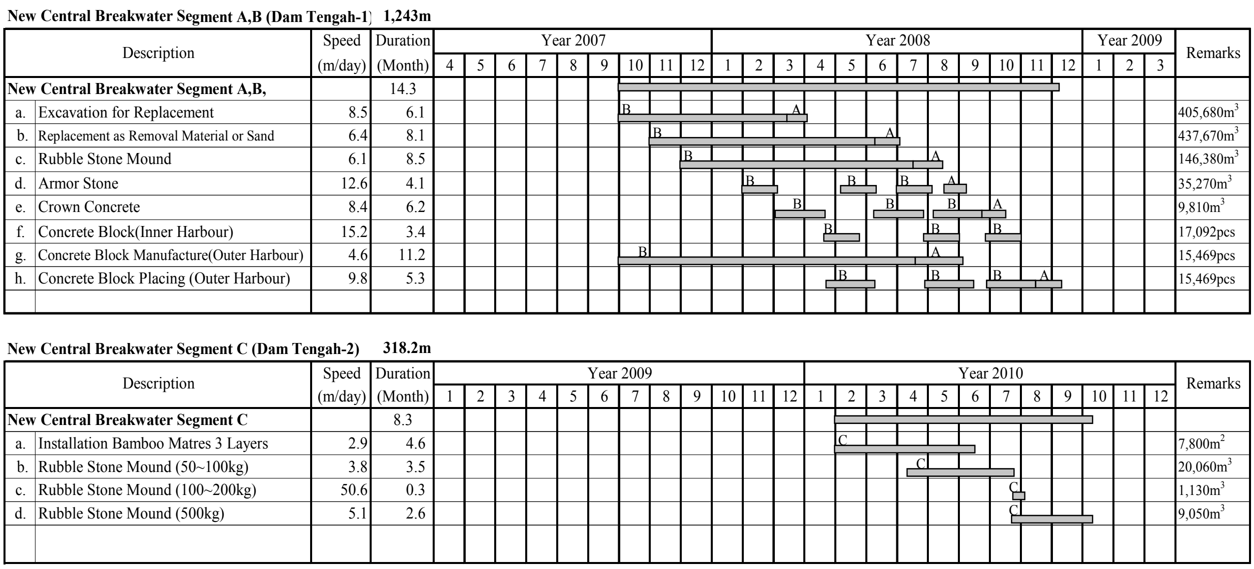


Figure 6.2.9 Tentative Construction Schedule and Work Period