2.2 Wave Analysis for Outer Port Basin

2.2.1 General

In the master plan study (Part II—Planning of Port Development), the configuration of the outer port has been determined, including the layouts of the breakwaters and revetments for reclamation. The structural analysis for the marine facilities has been carried out in the succeding "Chapter 3 Port Engineering Study". This section assesses the design waves in both extreme and ambient conditions. The former one is mainly used for the design of marine structures, and the latter one for evaluating the port protection provided by the proposed breakwaters alignments. In addition to this wave analysis, influence of long-period waves on mooring vessels has also been studied, because the Port occasionally experience these long-period waves.

2.2.2 Deep Water Wave Analysis

The extreme statistics analysis has been performed by using offshore wave data which has been obtained from long term wind observation in order to estimate offshore design waves. Table III.2.2-1 shows the predicted offshore wave height and period for each return period. The wave height with return period of 50 years has been adopted as the offshore design wave height for structual design.

Table III.2.2-1 Offshore Waves with Return Period of Each Years

Direction / Return Period	SW		WSW		W		WNW		NW	
	H0(m)	T0(s)	H0(m)	T0(s)	H0(m)	T0(s)	H0(m)	T0(s)	H0(m)	T0(s)
1 YEAR	3.8	6.5	4.3	6.5	4.9	8.0	3.6	6.5	3.1	6.5
2 YEARS	4.1	7.5	4.6	6.5	5.3	8.5	4.0	7.0	3.5	6.5
3 YEARS	4.3	7.5	4.8	8.0	5.6	8.5	4.2	7.5	3.7	6.5
5 YEARS	4.6	7.5	5.0	8.0	5.9	8.5	4.5	7.5	3.9	7.5
10 YEARS	4.9	7.5	5.3	8.5	6.3	9.0	4.8	7.5	4.2	7.5
20 YEARS	5.2	8.0	5.6	8.5	6.8	9.0	5.2	8.0	4.5	7.5
50 YEARS	5.6	8.5	5.9	8.5	7.3	9.0	5.6	8.5	4.9	7.5
100 YEARS	5.9	8.5	6.1	9.0	7.7	9.5	5.9	8.5	5.2	7.5

T0 is calculated by using the formula 2.8*SQRT(H0) < T0 < 4.3*SQRT(H0)
Source: British Meteorological Office

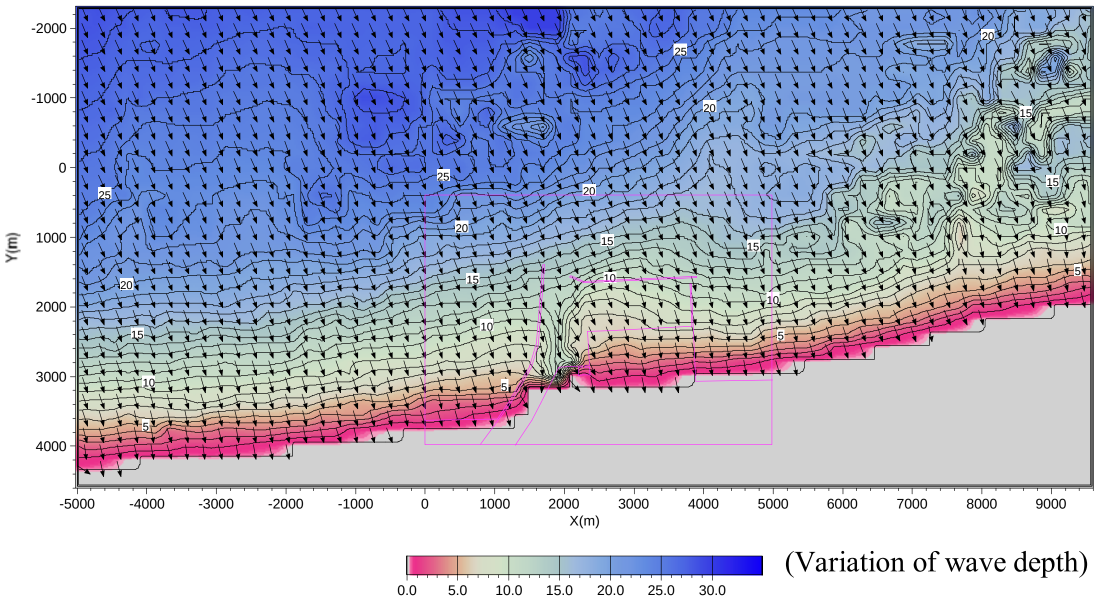
2.2.3 Nearshore Wave Analysis

The offshore design waves have been transformed into nearshore waves to obtain design waves at representative points in the outer port. The nearshore wave heights have been computed using a parabolic approximate model of Mild-Slope Equation which can be applied to the wave refraction and diffraction in an open wave field. Figure III.2.2-1 shows the results of numerical computation of wave deformation for predominant wave direction of WSW. (all computation results are referred to Appendix F 1). The design waves at the point of -20 m depth for each offshore wave direction are shown in Table III.2.2-2.

Table III.2.2-2 Design Wave Characteristics (at -20m Depth)

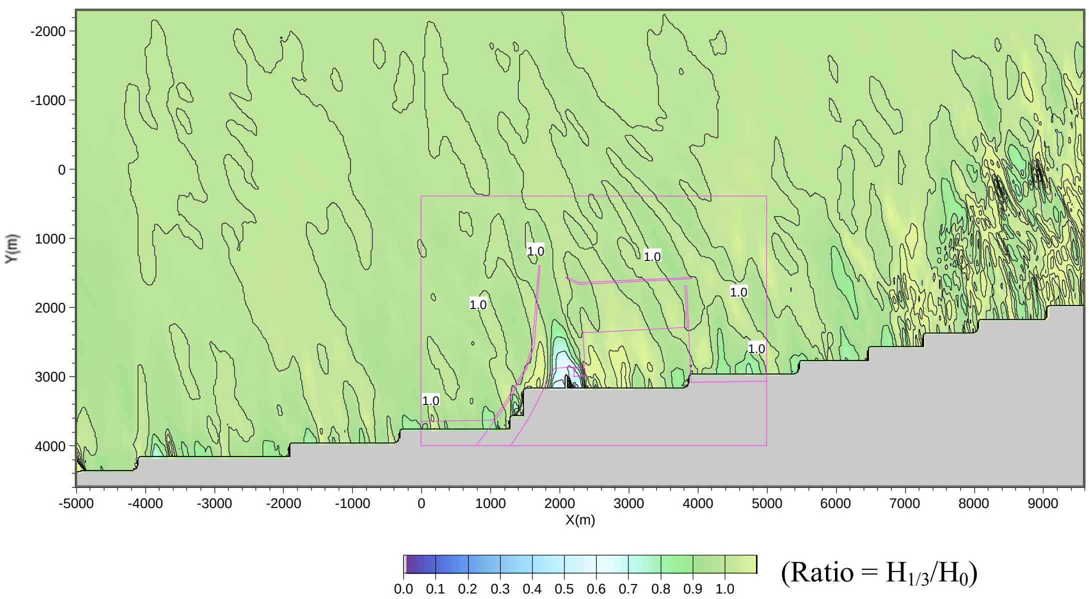
Item	Offshore Wave Direction (degrees)	SW (225)	WSW (247.5)	W (270)	WNW (292.5)	NW (315)
Offshore	H_0 (m)	5.6	5.9	7.3	5.6	4.9
	T_0 (s)	8.5	8.5	9.0	8.5	7.5
-20m Depth	$H_{1/3}$ (m)	5.2	5.5	6.7	5.1	4.4
	H_{max} (m)	9.3	9.8	12.1	9.1	7.8
	$T_{1/3}$ (s)	8.5	8.5	9.0	8.5	7.5
	θ (deg)	227	247	268	289	310

Source: Estimate by the JICA Study Team



(Variation of wave depth)

(1) Distribution of Wave Direction



(Ratio = $H_{1/3}/H_0$)

(2) Distribution of Wave Height Ratio

Figure III.2.2-1 Numerical Computation Result for Wave Deformation (Sample of Incident Wave Direction WSW)

2.2.4 Estimation of Calmness in Outer Port Basin

In order to achieve the optimum layout of the breakwaters for the outer port development, the calmness inside the basin have been projected for ambient waves intruding into the port. In order to evaluate the calmness, six points have been selected as representative areas of basin for the master plan (The results for short term Plan is refered in Appendix F1). The analysis flow of calmness is presented below.

Frequency distribution of incidental wave height (Refer to Section 1.2)
↓
Numerical computation for wave deformation (for each wave direction)
↓
Calculation of frequency distribution of wave height at selected points inside the port
↓
Occurency of waves less than the target wave height

With respect to the frequency distribution of incident waves to the outer port, the wave distribution obtained from a wave observation by JICA Study Team has been adopted.

It is necessary to consider the wave deformations caused by wave refraction, diffraction and reflection in the multi-directional random wave field as well as in actual field condition. To this end, a newly proposed time-dependent mild-slope equation for the multi-directional random wave field (Isobe et.al, 1998) has been applied. The reflection coefficents necessary for this model have been analysed as shown in Figure III.2.2-2, where the reflection effects by structual types of breakwater have been considered.

The desirable levels of calmness have been set up in consideration of ship/cargo cargo handling patterns expected in the basin and the quayside. The required calmness varies depending on the ship characteristics. Generally, it needs to be less than 0.7m for bulk carriers and 0.5m for container ships. In the study, it has been determined that occurence ratio of the wave height less than 0.7m should be secured for the duration of more than 95% of the time all through the year.

Figure III.2.2-3 shows the numerical computation results of wave deformation inside the port for the wave direction of WNW (reffere Appendix F 1 for all directions). Table III.2.2-3 shows the wave occurency less than 0.7 m at each point of the outer port basin. The table presents also the occerence of waves less than 0.5 m as a reference. The major points identified in the computation are breifed as below.

- The occurency of waves less than 0.7 m is secured with more than the target value of 95% in most of the areas of the outer port basin except for the area “I” (Berth No.1) and II, which is located nearest to the port entrance.
- High degree of the wave calmness of almost 100% is secured in area “IV to VI”.

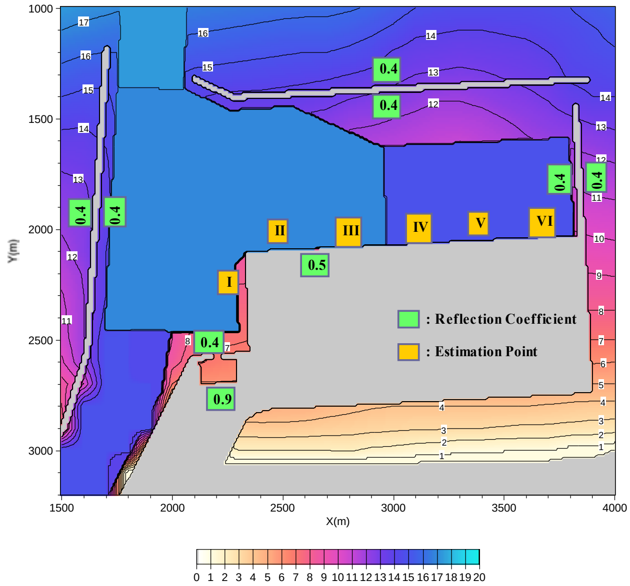


Figure III.2.2-2 Selected Points for Calmness Examination and Assumed Reflection Coefficients

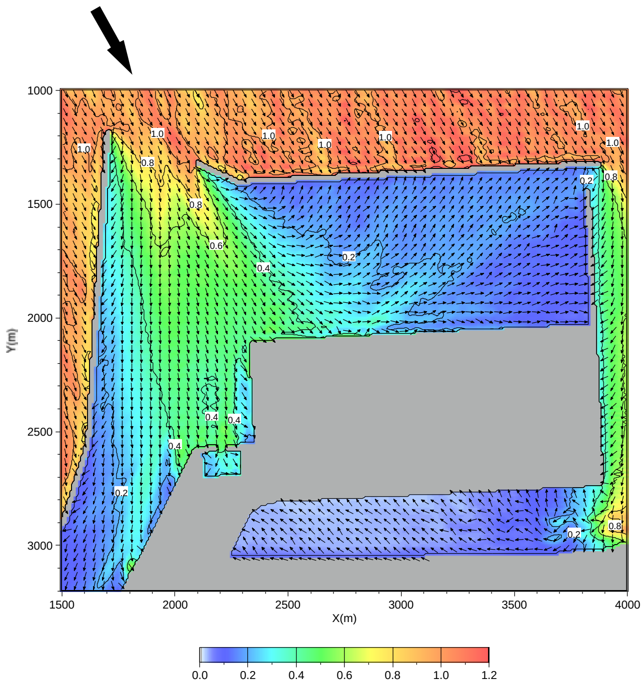


Figure III.2.2-3 Example of Computation Result of Wave Deformation (Incident Wave Direction: WNW)

Table III.2.2-3 Wave Calmness for Each Point

Apperarance Rate of Calmness (%)	Estimation Point					
	I	II	III	IV	V	VI
H1/3 < 0.7m	94	88	96	100	100	100
H1/3 < 0.5m	85	77	92	97	97	100

Source: Estimate by the JICA Study Team

2.2.5 Influence of Long-Period Waves

Among the waves approaching the Port, the component of long-period waves accounts for 5-8% as shown in Section 1.2. The ship motions are generally generated by means of long-period waves. Figure III.2.2-4 shows one example of surge (uprising motion) that happened to a coal carrier (36000GT, L=225m) moored at Tomakomai Port in Japan. The ship motion increases in proportion to the growth of long-period waves. This data shows that the critical height of long-period waves is 0.15m-0.2m to secure safe cargo handling. Furthermore, Delft Hydraulic (1995) Report indicates that the vessels larger than 15,000 DWT mooring in the existing oil berth have long been affected by long-period waves greater than 0.25m in height.

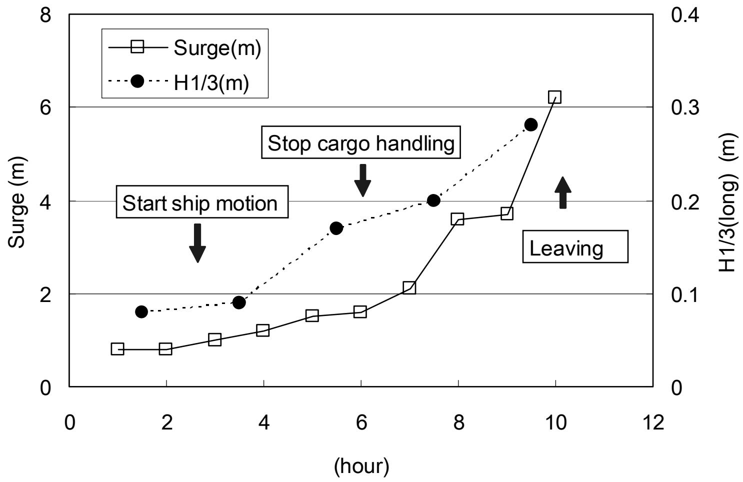


Figure III.2.2-4 Example of Observed Height of Long Waves and Surge Motion

In this section, the influence of long-period waves on the movements of ships mooring in the existing port entrance and the outer port has been assessed from the following points of view. Also, the effects of removing the existing northern breakwater have been studied.

Point-1: How far the removal of the existing northern breakwater will worsen an efficient berth operation in the existing oil berth near the port entrance, and to what extent the newly aligned outer breakwater will recover the aggravation resulting from the above removal work.

Point-2: How far the intrusion of long-period waves will influence the cargo hadling operation in the new oil berth.

Point-3: Is there any possibility of developing long-period waves oscillation in the present and future port areas.

For Point 1 and 2, the numerical computation for wave deformation of long-period waves has been performed for the cases; with and without existing northern breakwater, and outer port development plan. The wave period which was used as input condition for this computation has been adopted as 30 second in consideration of wave observation result. In this computation, it has been assumed the monochromatic free wave as long-period waves and the numerical computation model same as previous for general waves has been adopted.

Point-1

Figure III.2.2-5 shows samples of computation result for the propagation of long-period waves for incident direction W (referee Appendix F 1 for all results). Figure III.2.2-6 shows the relative long-period wave height in direction of existing channel for both cases of with and without existing northern breakwater. In this figure the origin of x axis means the position of port entrance as shown in Figure III.2.2-5. Figure III.2.2-7 shows the result of same line as Figure III.2.2-6 for both cases of present and outer port development layout (Short-Term Plan). The major points identified in the computation are briefed as below.

- Long-period wave height is decreasing as propagating to the inner part of port area.
- The relative wave height at the point of existing oil berth (x = 800m) is very small and approximately 0.15 for both cases of with and without existing northern breakwater.
- From the result shown in Figure III.2.2-7, the relative wave height for outer port development plan is approximately 0.1 and is less than that for present case.

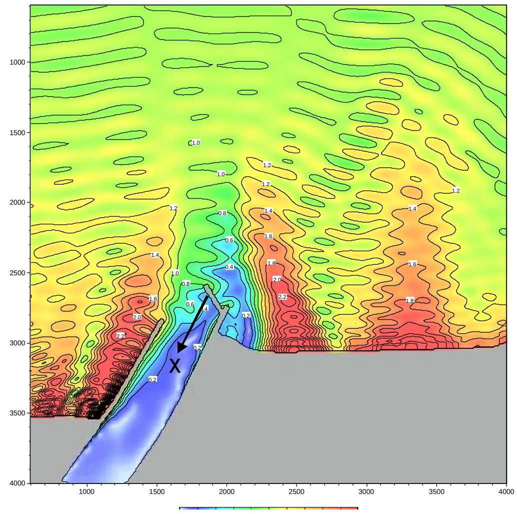
As the result, it was understood that the removal of existing northern breakwater is no negative impact on the characteristics of long-period wave propagation and the outer port development plan is effective to reduce the long-period wave height at the position of existing oil berth.

Point-2

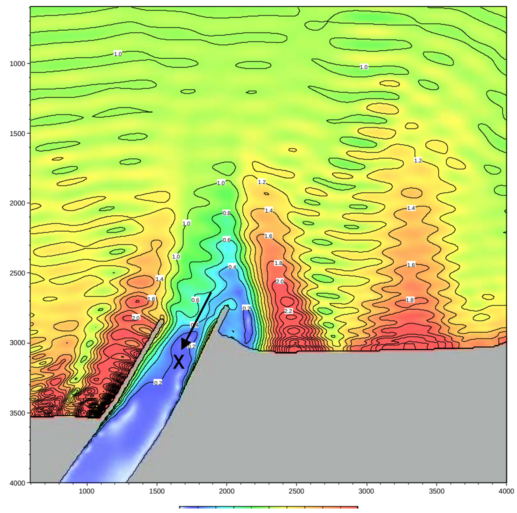
From the computation result for outer port development plan as shown in III.2.2-5(3), the relative wave height at each berth position is presented in Table III.2.2-4. The relative wave height at the position of new oil berth (I) is estimated as 0.3 to 0.4. This value is greater than that at the position of existing oil berth. As a result, there is a possibility that the long-period waves with same degree or more will intrude into the new port basin compared to the existing port area. If the offshore wave height is assumed 4 m same as before, the long-period wave height at new oil berth position is estimated approximately 10 cm. This value is less than the critical wave height for cargo handling. (For the detailed discussion of influence of long-period waves it is necessary to accumulate further data for waves and cargo handling conditions.)

Point-3

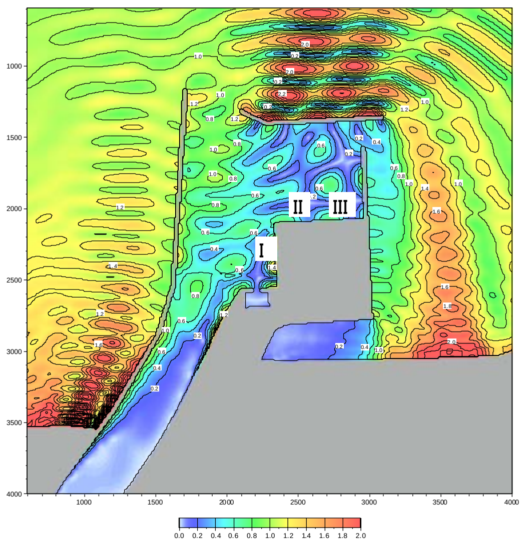
The component of long-period waves, after intruding the port entrance, might generate standing waves with the formation of multi-reflection effects caused by vertical wall. As a result, the height of waves might further increase. It was confirmed by the interview with KSSA that the ship motion problem due to long-period waves appears only at the existing oil berth. In addition, there is no remarkable reflection boundary in the existing port area. From these facts, it is thought that there are no serious problem about the long-period wave oscillation due to multiple reflections inside the port area, and these conditions will not change, even if the outer port is developed as proposed.



(1) Present (with Existing Northern Breakwater)



(2) Without Existing Northern Breakwater



(3) Outer Port Plan (Short-Term)

Figure III.2.2-5 Example of Computation Result of Long-Period Wave Propagation (Insident Direction W)

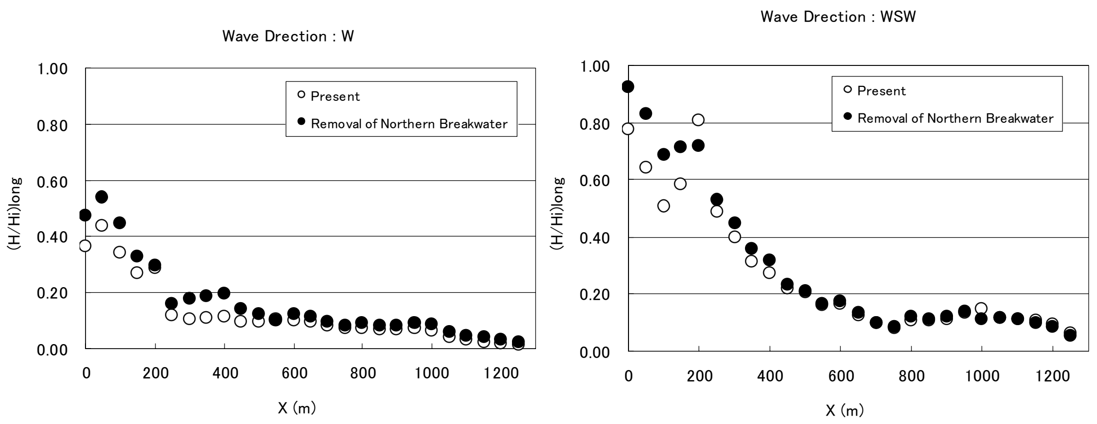


Figure III.2.2-6 Distribution of Relative Wave Height for Long-Period Waves in Existing Channel (Comparison of with and without Existing Northern Breakwater)

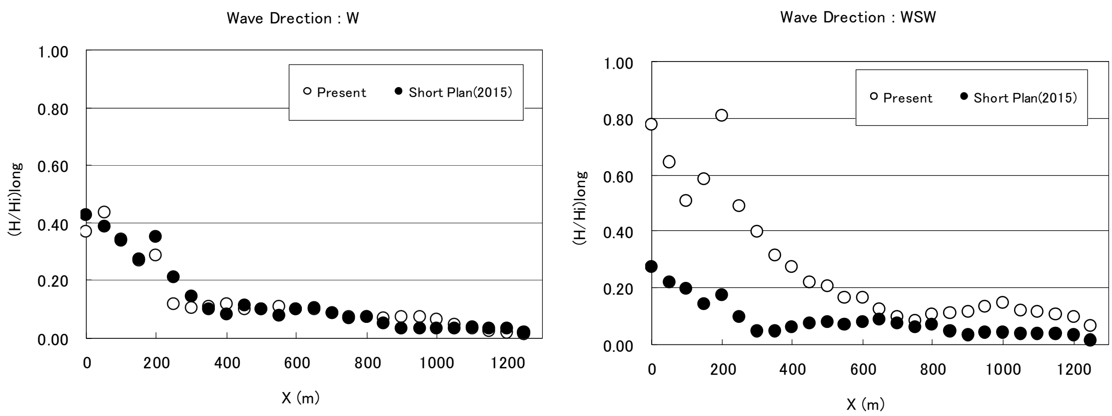


Figure III.2-2-7 Distribution of Relative Wave Height for Long-Period Waves in Existing Channel (Comparison of Present and Outer Port Plan)

Table III.2.2-4 Relative Wave Height for Long-Period Waves at New Berth Position for Outer Port Plan

Insident Long-Period Wave Direction	Estimation Point		
	I	II	III
W	0.31	0.19	0.17
WSW	0.36	0.25	0.27

2.3 Sedimentation Analysis for Inner Channel

2.3.1 General

In the Planning of Port Development (Part III), the possibility of developing the inner port expansion has been studied. As a part of the study, the change in the sedimentation rate in the expanded and deepened inner channel has become one of the technical study points, so the extent and magnitude of likely sedimentation with the inner port expansion has been examined. The numerical computation with Mud Transportation Model has been used for examination, where the experiment parameters have been assumed as follows:

Before construction

Depth: -14.0m in the channel up to International Ferry Terminal

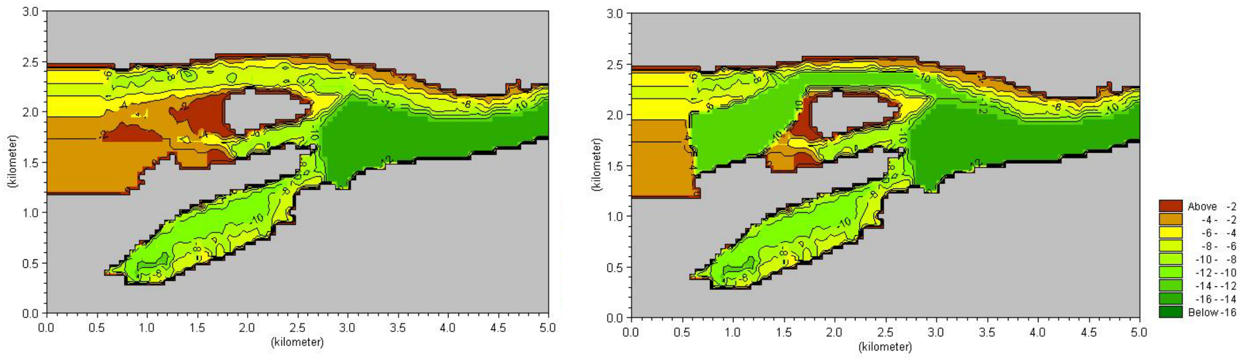
Flow Volume: Q1 = 600m3/s (in summer) and Q2 = 1200m3/s (in spring)

After construction

Depth: further extended to the inner port area (-12.0m) as shown in Figure III.2.3-1

Flow Volume: the same as Present Condition

The sedimentation volumes have been compared for each area, and the future volumes have been estimated based on the present volume.



(1) Before Construction **(2) After Construction**

Figure III.2.3-1 Bathymetric Condition

2.3.2 Estimation of Sedimentation Distribution

The present flows of currents and sedimentation in the inner channel have been reproduced as shown in Figure III.2.3-3. The sedimentation rate is small around the Kiaules Nugara Island and near the port entrance, because the current velocity is accelerated there. The sedimentation rate is high around the International Ferry Terminal, Malku Bay, and Inner Port where the fresh water is stagnated. The tendency mentioned above is similar to the result of the spot measurements performed by the JICA Study Team using portable equipment. Moreover, the sedimentation distribution produced from the computation, though slightly different zone by zone, is almost similar to that measured on site (see Figure III.2.3-2).

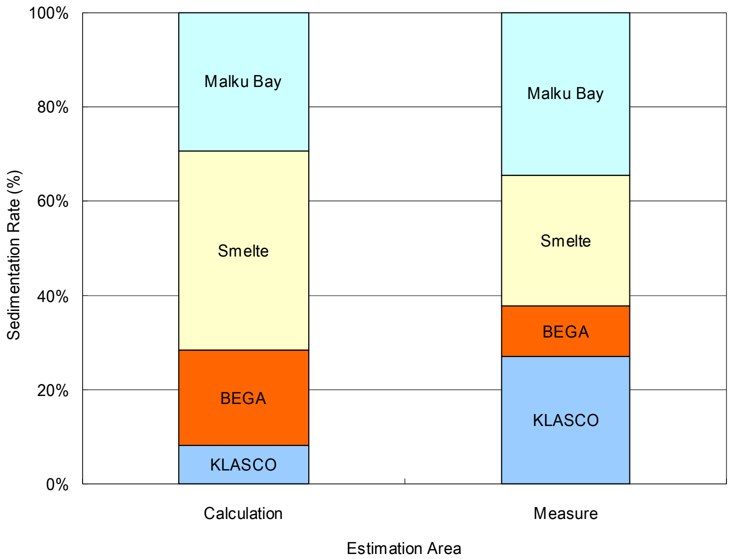


Figure III.2.3-2 Sedimentation Distribution by Zone based on Measurement and Computation Result

The computation result after the construction of inner port is shown in Figures III.2.3-4 and III.2.3-5 (refer to Appendix F 2 for all computation). The current velocity decreases suddenly at the deeper part of the inner port due to increase in its water depth, resulting in the existence of sedimentation.

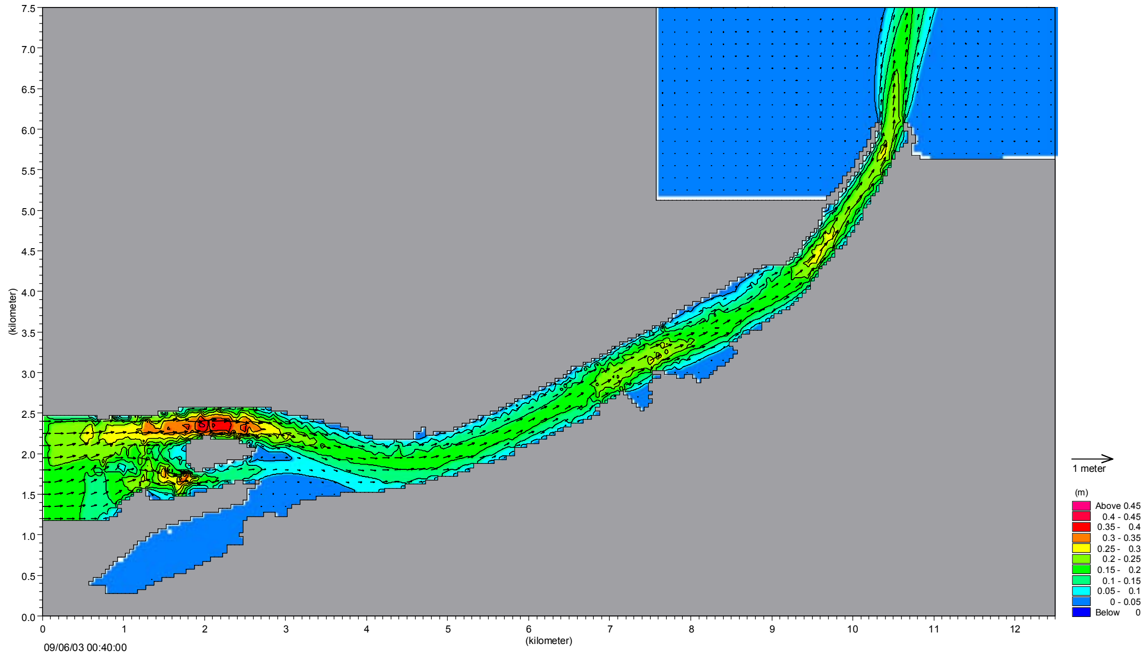
The sedimentation rate in each zone has been compared as shown in Figure III.2.3-7. The change in sedimentation rates after the inner channel dredging is shown in Figure III.2.3-8 in the conditions of different flow volumes, where the sedimentation rate is estimated as a share of sedimentation volume in each area against the total sedimentation volume. The volume of suspended load that flows out to the Baltic Sea is relatively high in the case of flow volume of 1,200m^3/s, but the sedimentation volume in the port area decreases. On the contrary, the sedimentation volume in the port area increases in the case of 600 m^3/s. Sedimentation is high around the Smelte berths and the Malku bay.

The total volume of sedimentation in the port area after construction of the inner port has been estimated at almost the same as that of before construction. The sedimentation volume in the planned inner port area after construction is almost the same as that at Smelte berths and the Malku bay, though it decreases slightly in the existing channel.

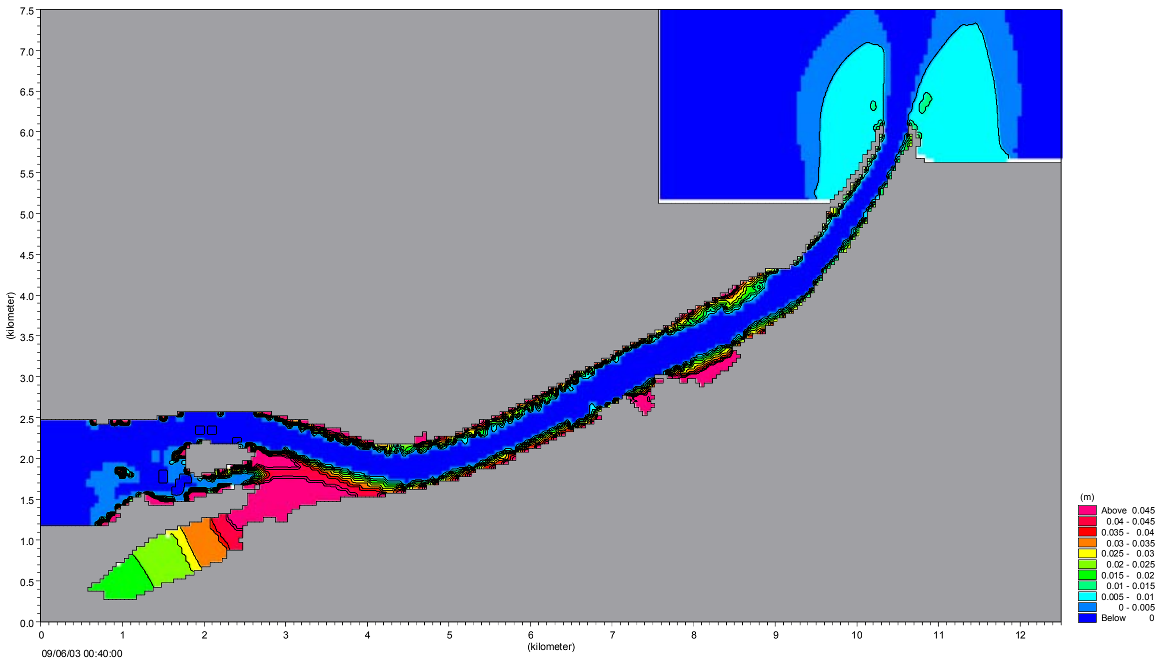
Based on the computation results and previous maintenance dredging records, the tendency of sedimentation in the planned inner port area is summarized as follows.

- After increasing the water depth in planned inner port area, remarkable sedimentation will be brought into the inner port area due to the sudden decrease of current velocity induced by increase in water depth.
- Sedimentation volume in the planned inner port area is almost the same as that at the Smelte berths and the Malku bay. As the maintenance dredging volume

is 50-70 thousand m^3/year at the Smelte berths and the Malku bay, there is a possibility that the similar sedimentation will occur in the planned inner port area after construction.



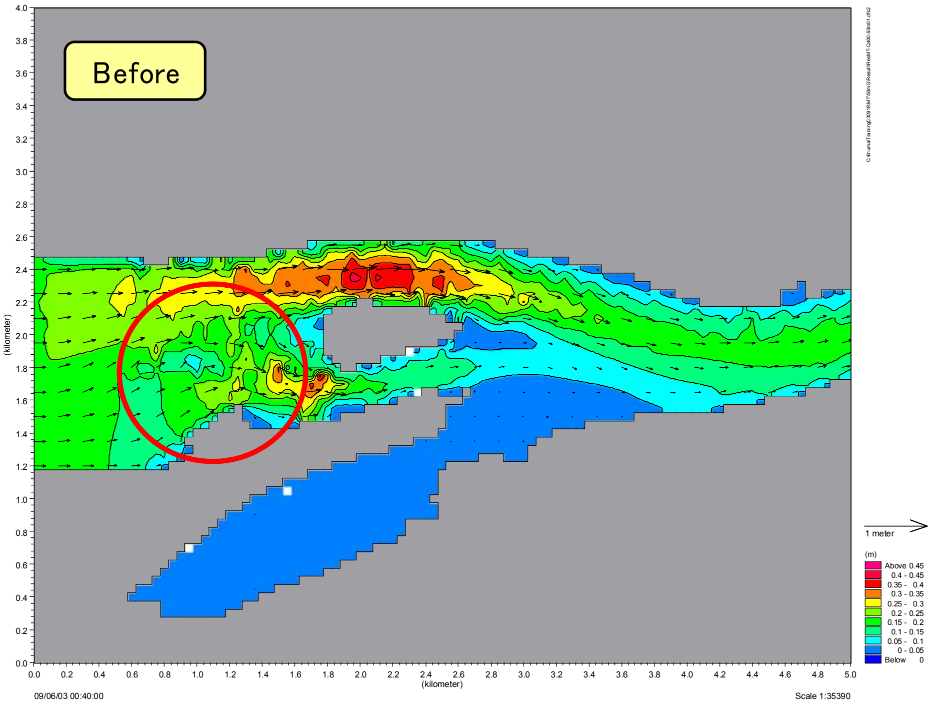
(1) Current Distribution



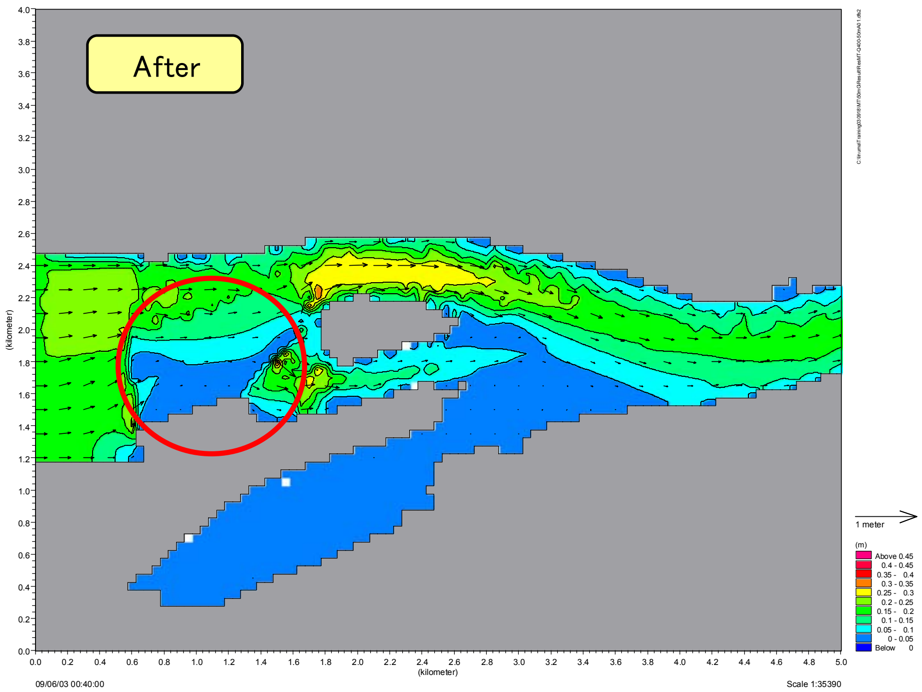
(2) Sediment Distribution

Source: The JICA Study Team

Figure III.2.3-3 Reproduction Result in Present Condition



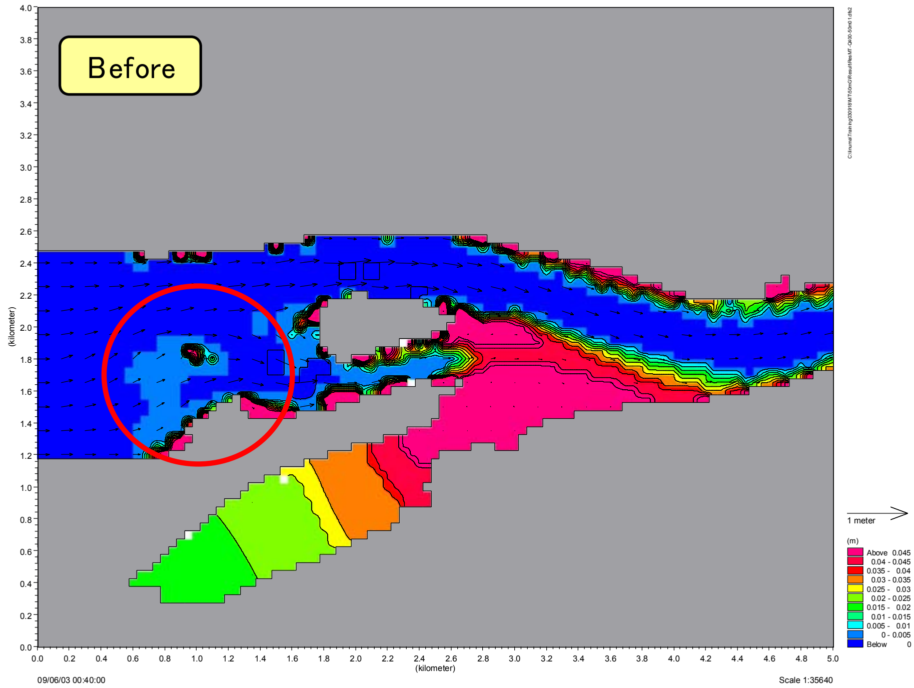
(1) Before Construction



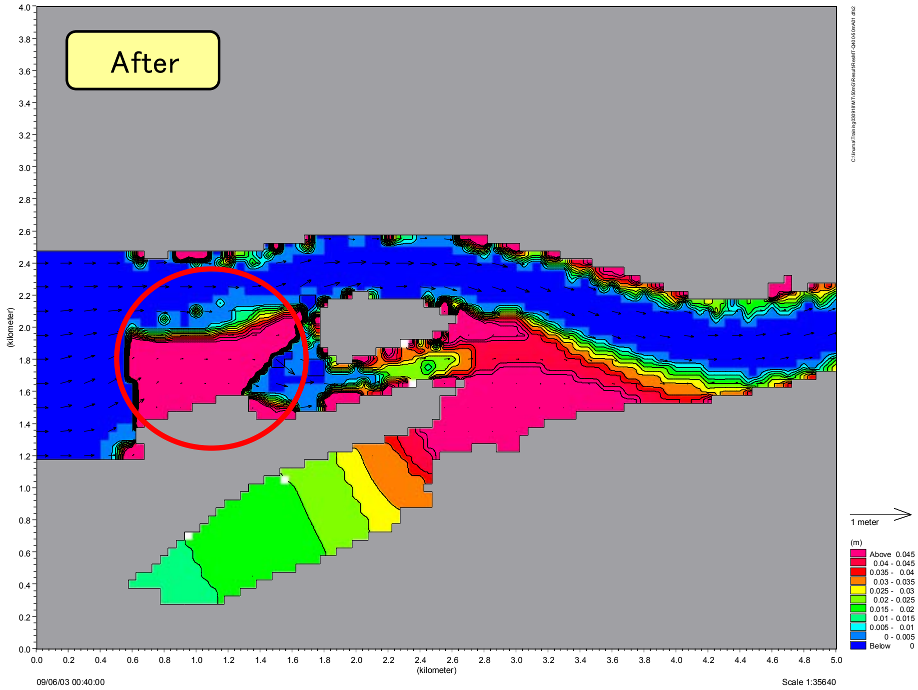
(2) After Construction

Source: The JICA Study Team

Figure III.2.3-4 Distribution of Current（in case of Q=600m^3/s）



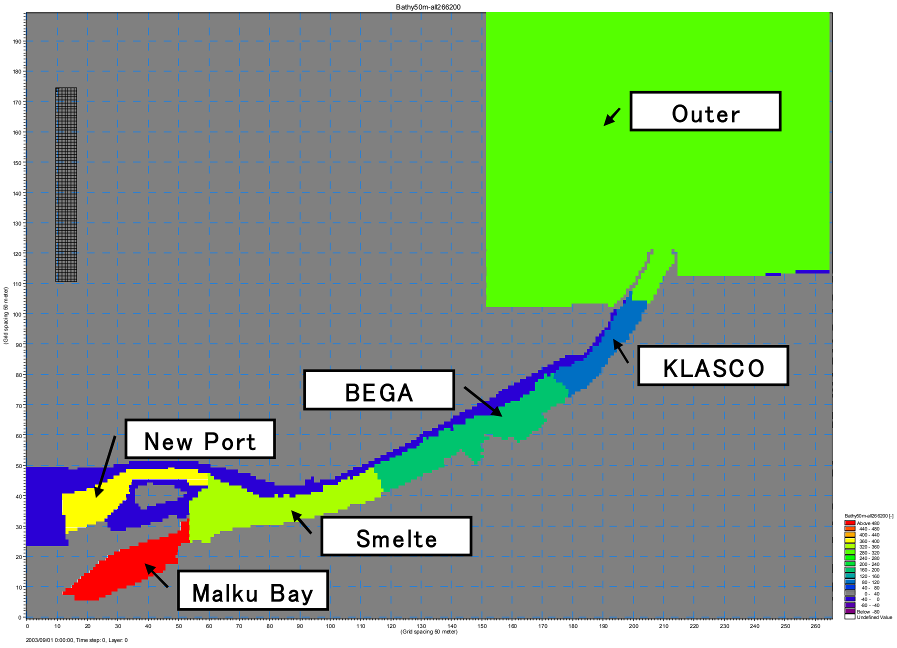
(1) Before Construction



(2) After Construction

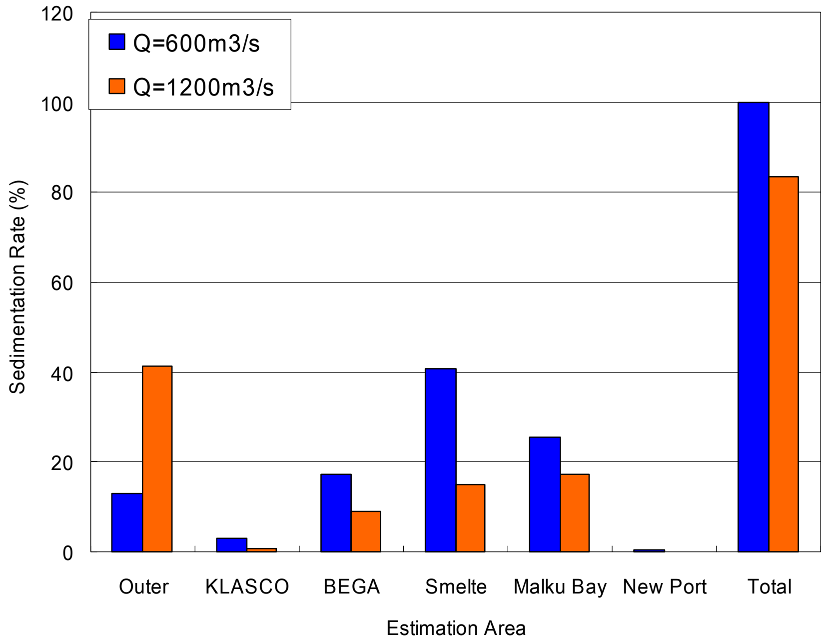
Source: The JICA Study Team

Figure III.2.3-5 Distribution of Sedimentation (in case of Q=600m^3/s)



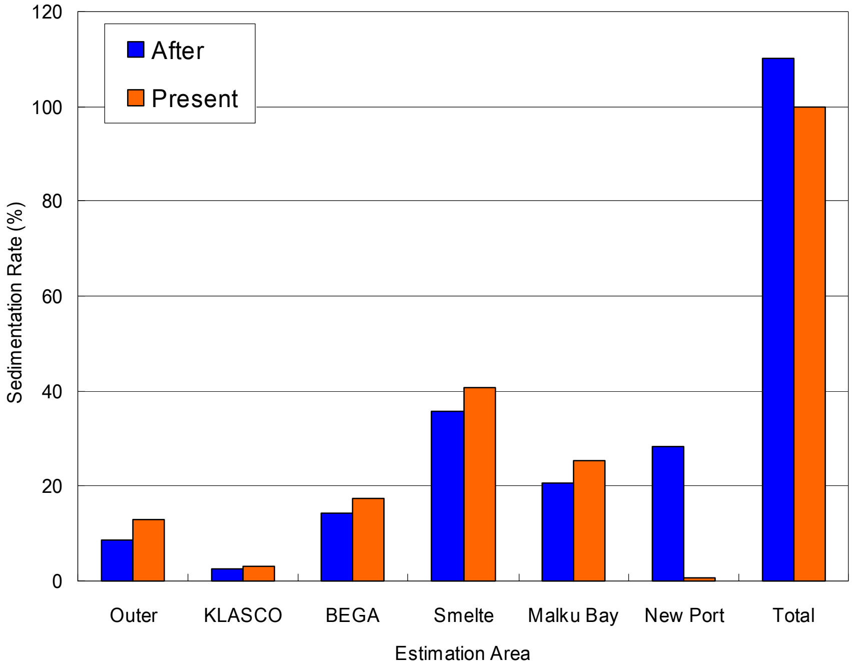
Source: The JICA Study Team

Figure III.2.3-6 Estimation Area

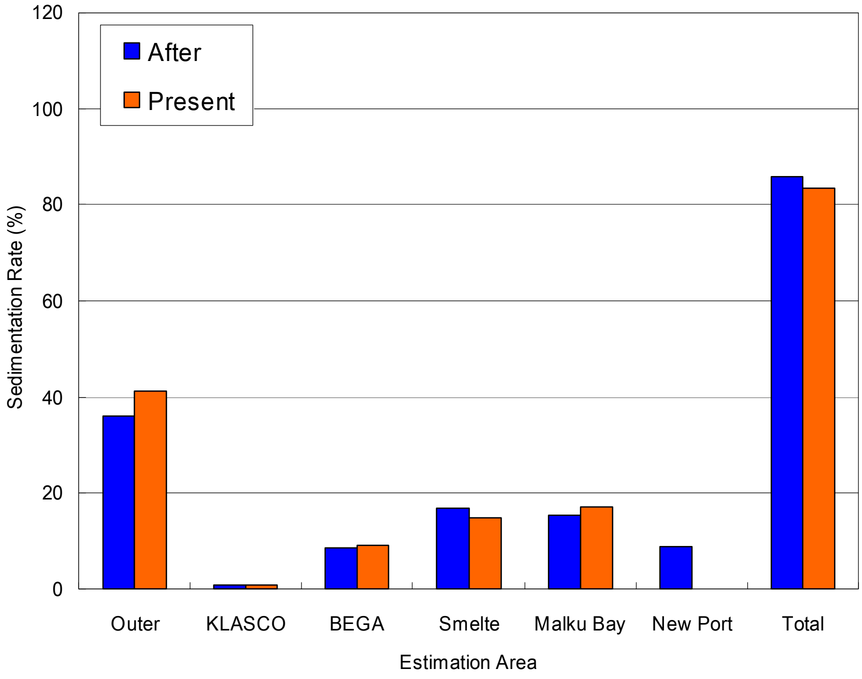


Source: Estimate by the JICA Study Team

Figure III.2.3-7 Comparison of Sedimentation Rate in Different Flow Condition



(1) Q=600m^3/s



(2) Q=1200m^3/s

Source: Estimate by the JICA Study Team

Figure III.2.3-8 Comparison of Sedimentation Rate between Before and After Construction

2.4 Analysis on Salinity Intrusion

2.4.1 General

The Port is located close to the Baltic Sea and belongs to a salinity intrusion zone in the Lagoon, while the area inshore of the existing port zone is hardly affected by salinity intusion. Salinity is a very sensitive factor to ecological system in the Lagoon. The water-intake canals exist near the prposed inner port zone to tap city water, so that any change there in salinity due to the inner port expansion would be crucial. This section deals with salinity intrusion in the port area.

2.4.2 Salinity Characteristics in the Port

It is presented in the technical study reports (for example, Geochemistry of Sediments of the Curonian Lagoon, Institute of Geography, 1998) that the salinity in the existing port area is controlled by the winds and flow from the Lagoon. The field observation executed by the JICA Study Team endorsed that the vertical distribution of salinity in the port area was greatly influenced by wind and wave conditions. The saline water stagnate only near the bottom of the dredged navigation channel, while the remaining seabed is covered by fresh water when it is calm. On the other hand, a vertical mixture is generated when the strong winds blow, which cause disturbance, and the salinity concentration appeares from the bottom to the surface. The salinity concentration appears less in the surface part, compared with that in the bottom part.

2.4.3 Influence of Salinity Intrusion on Inner Port Plan

The influence of the salinity intrusion on the inner port development has been examined by using a three-dimensional numerical computation model of salinity diffusion. Table III.2.4-1 shows the conditions of numerical computation for salinity intrusion. Figure III.2.4-2 shows the calculation area and the represented output line.

Table III.2.4-1 Conditions of Numerical Computation for Salinity Intrusion

Bottom Topography	2 Cases (before construction of inner port (After -14m channel dredging) construction) and after construction)
Flow Rate	2 Cases (Q=450m^3/s , 680m^3/s)
Wind	From no wind to 20 m/s (see Figure III.2.3-1) (Wind direction is constantly WSW direction)
Initial Condition of Salinity Concentration	Outside the port area: 7 ‰ inside the port area: 0 ‰
Computation Period	8 days (after 6 days wind acts)
Water Temperature	Set vertical distribution referring to the field investigation

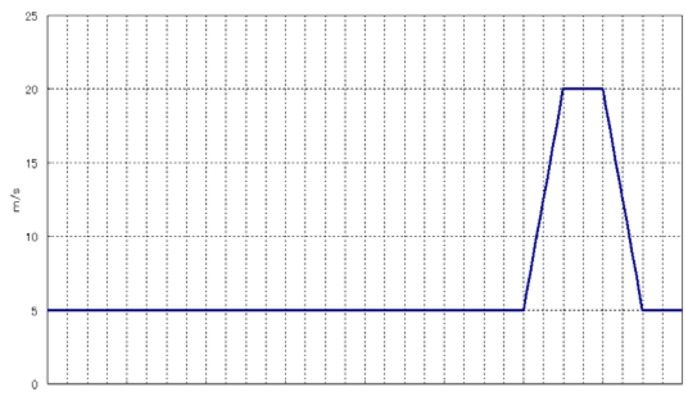
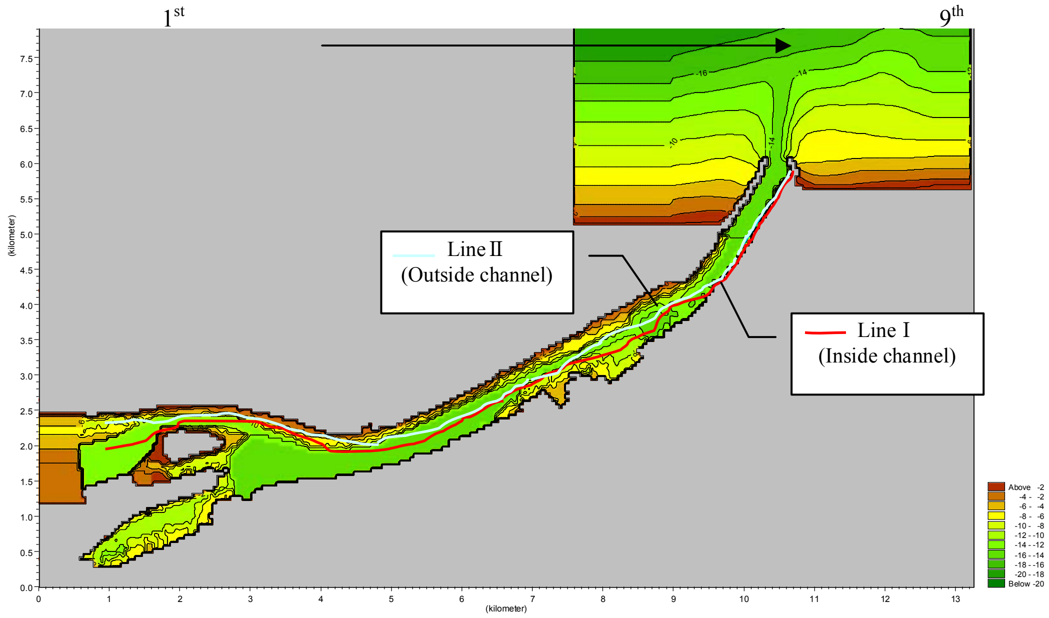


Figure III.2.4-1 Condition of Wind Strength (Wind Direction: WSW Constant)



Source: The JICA Study Team

Figure III.2.4-2 Calculation Area and Output Line

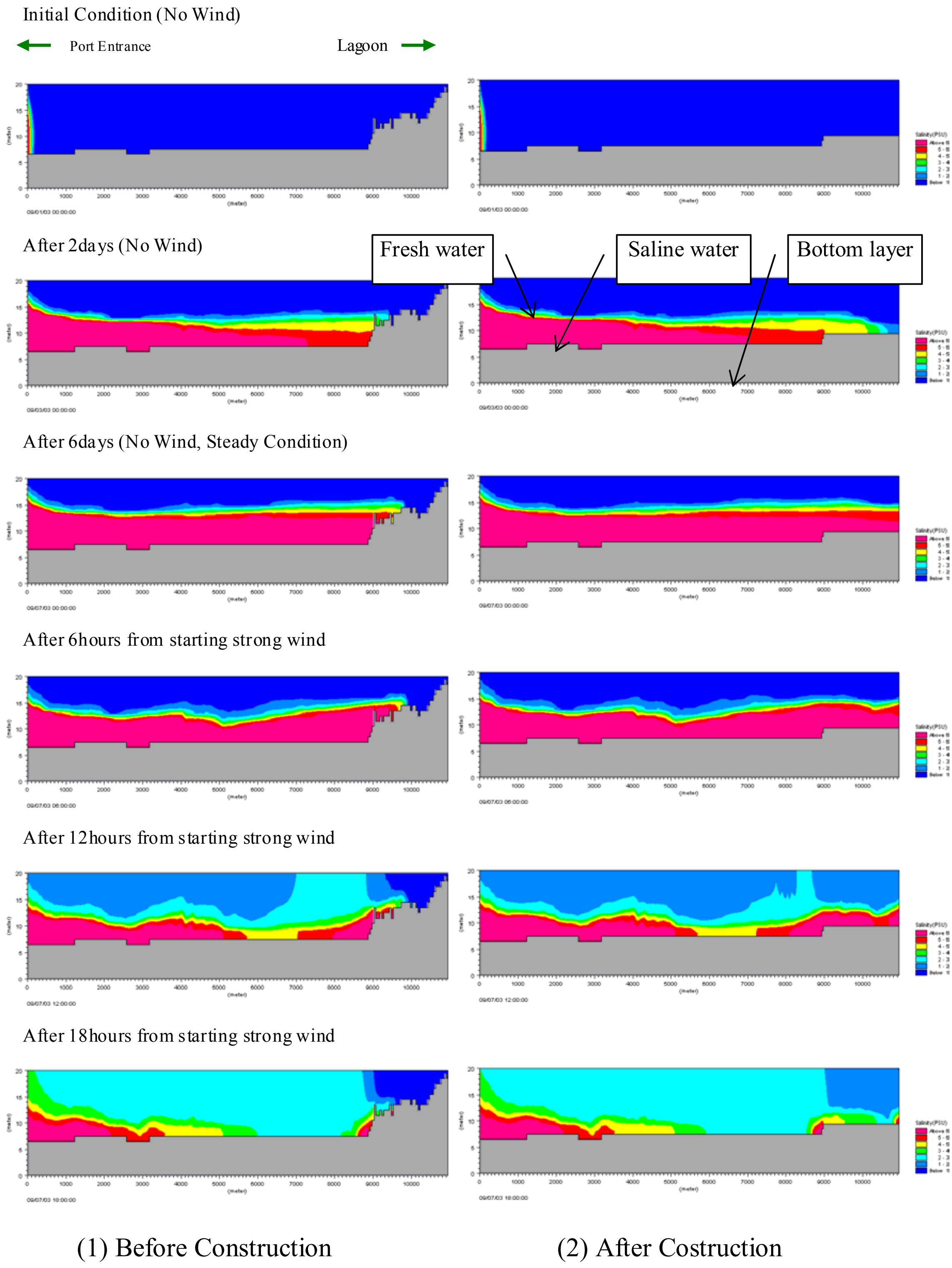
An example of numerical computation for salinity intrusion is shown in Figures III.2.4-1 and III.2.4-2. Figure III.2.4-3 shows the vertical distribution of salinity concentration for the case of Q= 450 m^3 inside the channel (Line I) and Figure III.2.4-4 shows that of outside the channel (Line II) (refer to Appendix F 3 for all computation results). General mechanism of salinity distribution can be explained below.

- In the initial state, there existes only the fresh water in the port area. Then, the saline water outside the port gradually invades the port area due to the difference in the density between saline and fresh water.
- Under a calm weather condition, the saline water exists only in the vicinity of bottom layer in the existing ship channel (Line I), and it does not exist in the other natural bottom area (Line II). Such phenomena are similar to the result of the field investigation for salinity.

- Before the inner port expansion, the saline water does not invade the inner port area. On the other hand, the saline water will invade to a new developed deep water area after construction.
- A vertical mixture of saline water is formed after the strong winds occur, and the stable stratification of salinity is destroyed. As the result, the concentration of salinity appears in the vicinity of bottom layer and in the surface both inside (Line I) and outside (Line II) the channel. The concentration of salinity decreases in the surface part when reaching closer to the inner port area.

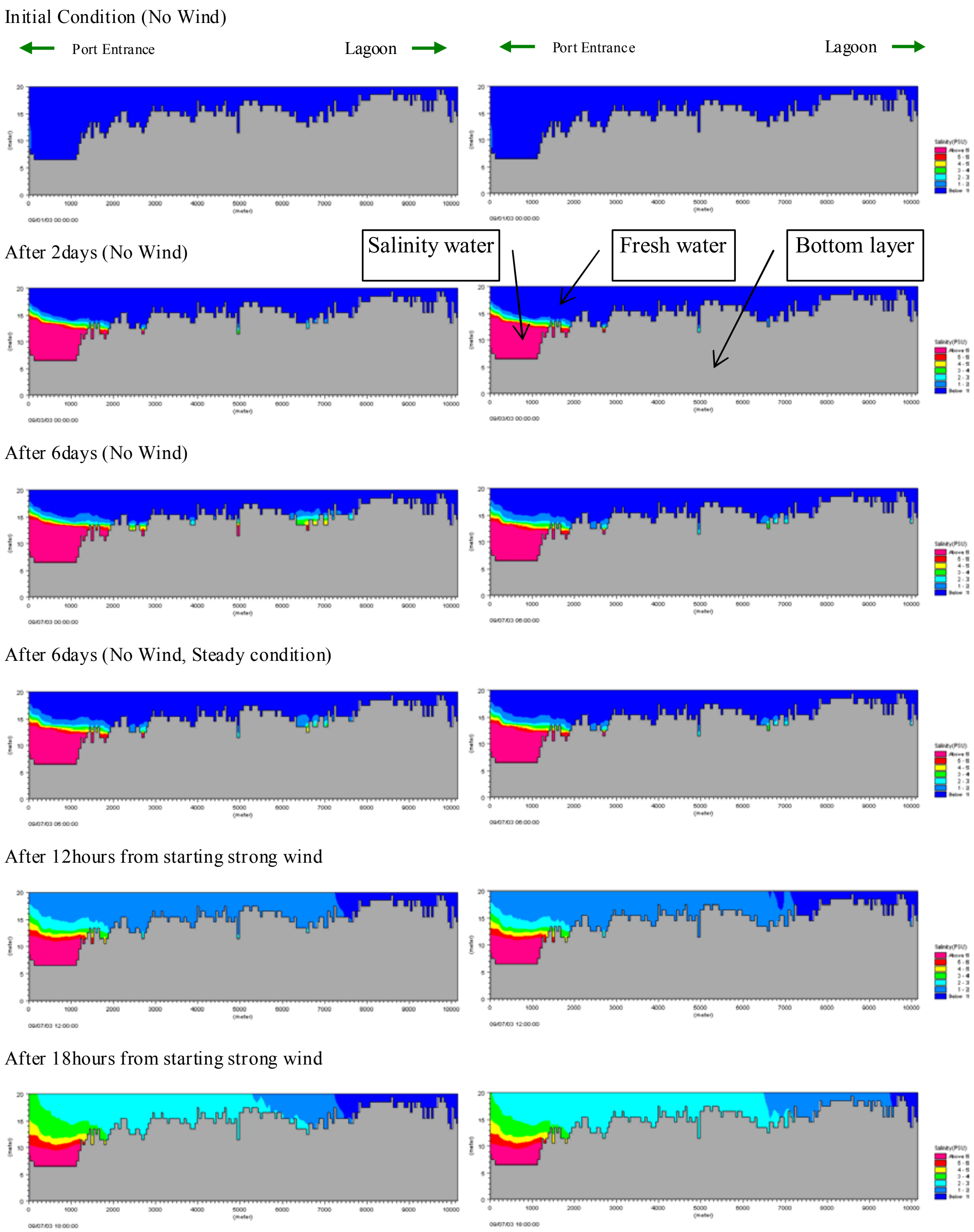
The influence of salinity water after construction of the inner port can be expected as follows.

- When it is calm and the stratification (saline water / fresh water) is formed, saline water doesn't appear at the inner port area (shallow water region). On the other hand, after construction of inner port, saline water comes always to stagnate in the vicinity of bottom layer due to increase in water depth with the channel dredging.
- When strong winds blow, water area is disturbed and the stratification is destroyed. As the result, saline water can be detected in all the port area by vertical mixing of the saline water at the bottom of the existing channel and by accelerating the sea water intrusion from the port entrance. Salinity, after construction of the inner port, will increase, compared with the present condition.



Source: The JICA Study Team

Figure III.2.4-3 Vertical Distribution of Salinity in Existing Channel (Line I) ($Q=450m^3/s$)



(1) Before Construction (2) After Construction

Source: The JICA Study Team

Figure III.2.4-4 Vertical Distribution of Salinity in Existing Channel（Line II）(Q=450m^3/s)

2.5 Sedimentation Analysis for Outer Channel

2.5.1 General

The mechanism of sedimentation at the outer channel is different from that inside the port area. The port entrance of the proposed outer port will be relocated offshore of the present port entrance. As the outer channel is deepened and expanded offshore, it is necessary to examine the influence of suspended loads from the Lagoon as well as the incoming drift sand caused by waves.

The influence of sand drift on the outer port including its channel portion has been examined by using the following two methods.

- Estimation based on the critical water depth for sediment transport
- Estimation based on the actual sediment situation in the existing outer channel

In the next step, the influence of the suspended load from the Lagoon has been examined by applying the numerical computation model of mud transportation.

2.5.2 Estimation of Critical Water Depth for Sediment Transport

The breakwaters and the port entrance of the outer port should be located to minimize the amount of drift sand entering the port and to ensure safe navigation of ships along the channel. The port entrance should be planned in the water depth greater than the critical water depth that causes sediment transport.

Several methods can be applied as below for assessing the critical water depth for sediment transportation.

- To trace back historical changes in contour lines around the existing seabed especially in the outer channel
- To feed the data of offshore wave climate and soil characteristics such as mean grain size into experimental formulas
- To analyse the distribution of grain size of seabed materials in on-offshore direction

In order to assess the critical water depth of sediment transport with high reliability, all these methods have been taken into account. I

(1) Analysis from the shape of contour line around the existing outer channel

Figure III.2.5-1 shows the water depth contour lines around the existing outer channel. The curvatures of contour lines in the vicinity of the outer channel on the south side have been examined. The contour lines deeper than -13m bend with acute angles, almost right angle. On the other hand, the contour lines shallower than -12m curb with round shapes. It can be understood that this change in shape from acute to obtuse angles is attributed to the existence of remarkable sediment movements around the shoulder portion of the channel slope. Based on this fact, it can be assumed that the critical water depth for sediment transport is about 12m - 13m.

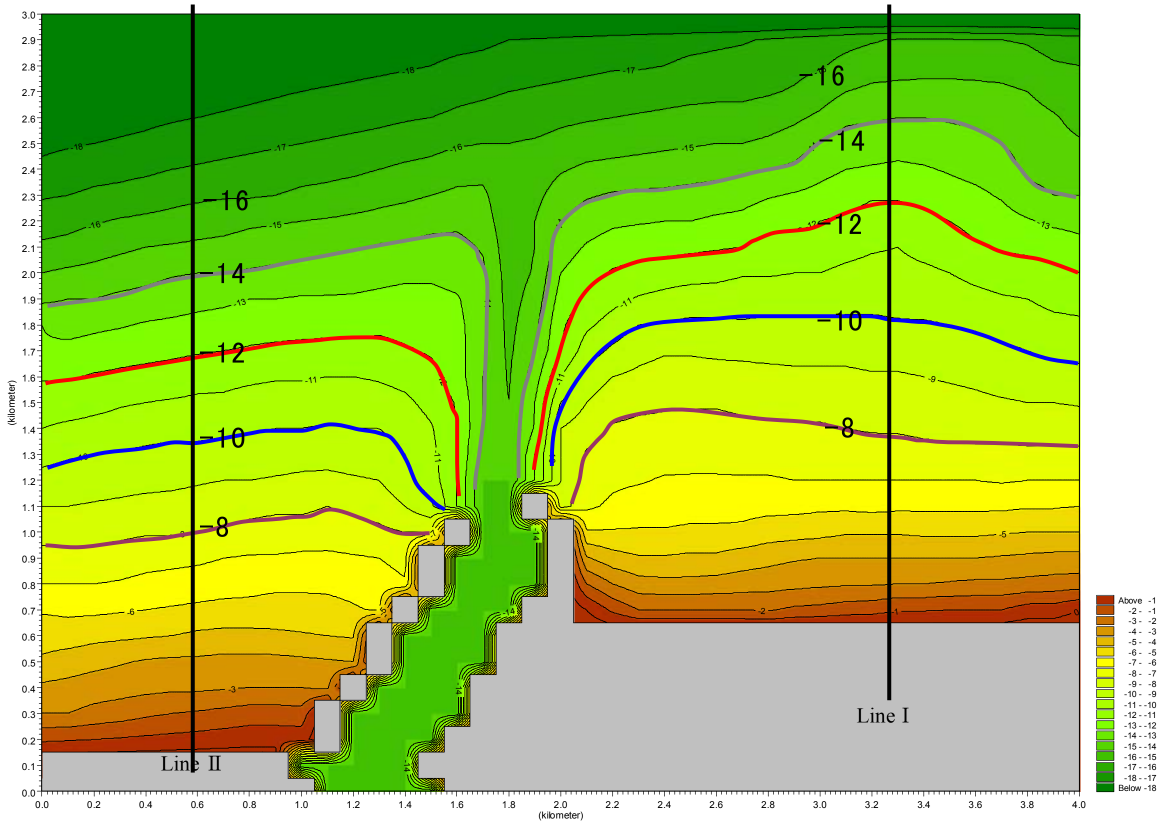


Figure III.2.5-1 Contour Line around Existing Outer Channel

(2) Analysis from the calculation formula with offshore wave condition and mean grain size

A calculation formula with the offshore wave condition and mean grain size has been used in estimating critical depth of sedimentation as below.

$$\frac{H_0}{L_0} = \alpha \left(\frac{d}{L_0} \right)^n \left(\sinh \frac{2\pi\, hi}{L} \right) \frac{H_0}{H}$$

Here,

- H_0 : Offshore Wave Height
- L_0 : Offshore Wave Length
- d : Mean Grain Size for the Sediment
- hi : Critical Depth for Sedimentation
- H : Wave Height
- L : Wave Length

The various coefficients should be assumed to respond movement forms, including initial movement, general movement, net transport of surface sediment and net transport of whole sediment. Among these, the critical water depth for net transport of both surface sediment and entire sediment is important. The coefficients for these two types of movement form were proposed by Dr.Sato and

Dr.Tanaka, as α = 1.35, n = 1/3 for net transport of surface sediment and α = 2.4, n = 1/3 for net transport of entire sediment respectively.

Table III.2.5-1 shows the calculation results of the critical water depth for net transport of both surface and entire sediment by using each offshore wave conditions. The mean grain size (D50) in this area is distributed between 0.12 to 0.2mm according to the field observation result of bottom sampling and grain size analysis. The calculated critical water depths change in corresponding to the wave conditions as shown in Table III.2.5-1.

It is known that a substantial part of the sediment transport is generate by a stormy climate with high-wave energy, but low appearance. So, the peak wave heights during the storm that appear about once a month have been extracted from the wave observation results, which shows the existence of 2.7-2.8m high waves. As such, the average wave of H1/3 = 2.75 m has been selected as the representative wave height. Applying this condition, the critical water depth for inducing sedimentation can be calculated at 7.5 m (surface sediment) ~ 13.4 m (Entire sediment) for the case of D50 = 0.2 mm, and 9.7 m (surface sediment) ~ 16.2 m (whole sediment) for the case of D50=0.1 mm respectively. On this basis, it has been presumed at about 12 m as the critical water depth for the grain size of D50=0.16 mm.

Table III.2.5-1 Critical Depth of Sedimentation

Wave Height H1/3 (m)	Wave Period T1/3 (s)	Critical Depth for Net Transport of Surface Sediment (m)		Critical Depth for Net Transport of Whole Sediment (m)		Remark
		D50=0.1 mm	D50=0.2 mm	D50=0.1 mm	D50=0.2 mm	
1.0	4.4	4.8	3.8	2.5	1.9	
1.5	5.1	7.8	6.3	4.3	3.3	
2.0	5.7	11.1	9.0	6.3	4.9	
2.5	6.3	14.5	11.9	8.5	6.6	
2.75	6.6	16.2	13.4	9.7	7.5	Peak Wave Height during Storm
3.0	6.8	17.9	14.9	10.8	8.4	
3.5	7.2	21.3	17.9	13.1	10.3	
4.0	7.6	24.5	20.8	15.4	12.2	
4.5	7.8	27.6	23.5	17.7	14.1	
5.0	8.1	30.4	26.1	19.9	16.0	
5.5	8.2	32.8	28.4	21.9	17.8	
6.0	8.3	34.8	30.4	23.7	19.4	

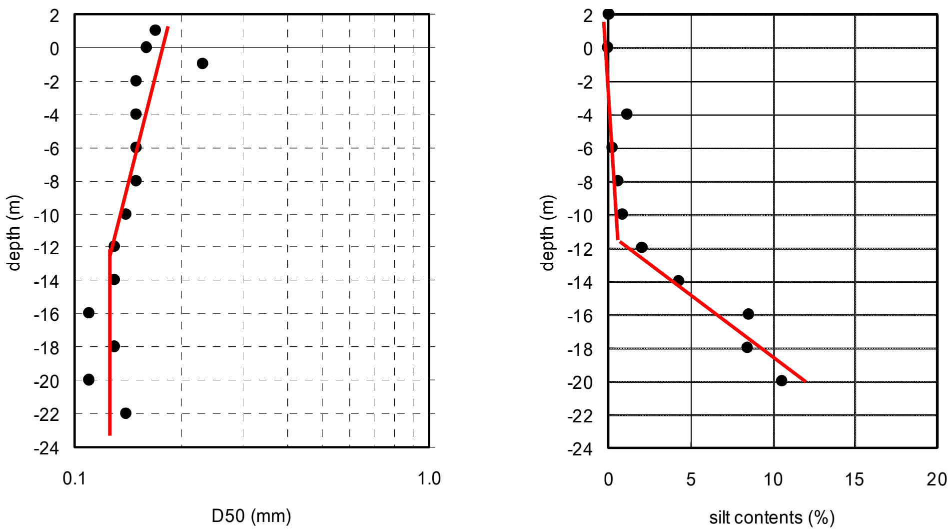
Source: Estimate by the JICA Study Team

(3) Analysis from grain size distribution in on-offshore direction

According to the graphs showing the correlation between grain sizes or silt contents and critical water depths, as prepared by Dr. Uda in Japan based on the field investigation result and the bottom sampling data, it can be said that the mean grain sizes become constant regardless of increase in the water depth and the silt contents become increasing toward the deep water area from the critical water depth.

Figure III.2.5-2 shows the distribution of the mean grain size and silt content obtained by the on-offshore line on south sides of the port entrance as shown in Figure III.2.5-1. A plotted line of mean grain size distribution and silt contents bend at the point of about -12m as shown in Figure III.2.5-2. Here, it has been presumed that the water depth of about -12m is the critical depth of sedimentation.

Same analysis has been conducted in on-offshore line on north side, however this expected tendency cannot be seen due to local sediment regime governed by dominant northerly flows from the Lagoon.



(1) Mean Grain Size(D50) (2) Silt Content

Figure III.2.5-2 Distribution of Mean Grain Size (D50) and Silt Contents for Each Depth in Line II

(4) Summary

The critical water depth for sedimentation transport has been analyzed by using several methods. As the result, it can be concluded that the critical water depth would be about 12m. Sediment transport is active in the area shallower than this critical water depth. If the port entrance is located shallower than this critical water depth, there is the possibility of causing sedimentation in the port basin, entrance, and access channel.

2.5.3 Sedimentation Analysis for Outer Channel based on Existing Dredging Record

The sedimentation in the existing channel is presented in III Section 1.7 (2). Using these results, the thickness of sedimentation in the outer channel can be calculated. Figure III.2.5-3 shows the average sedimentation thickness during eight months in 1999-2000 including winter season. Distance "0" indicates the position of port entrance. The right side of "0" indicates the offshore direction and the left side for port area side.

The proposed position of the entrance in the outer port is shown with an arrow mark. The sedimentation peaks in the vicinity of port entrance and decreases toward seaward. The deposition diminish to 0.1m thick in the water depth greater than -13m. The new entrance of the outer port will be located in this depth. Therefore, it is anticipated that the sediment transport now noticed in the vicinity of the port entrance will significantly decrease.

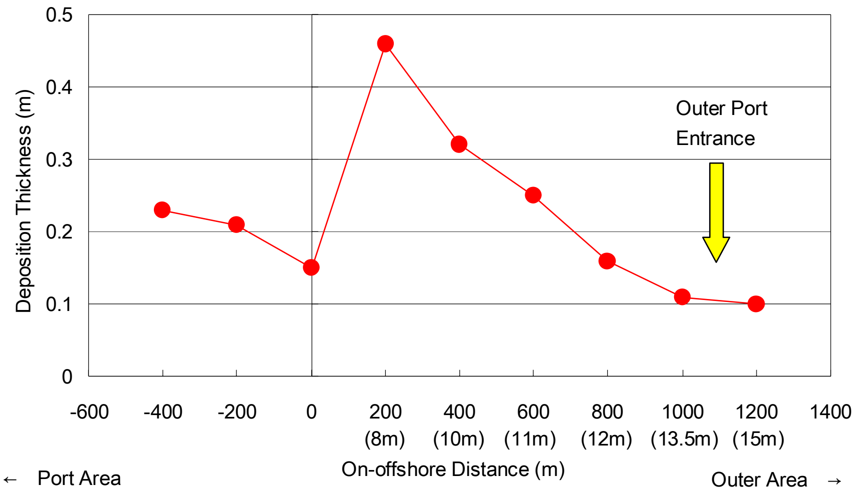


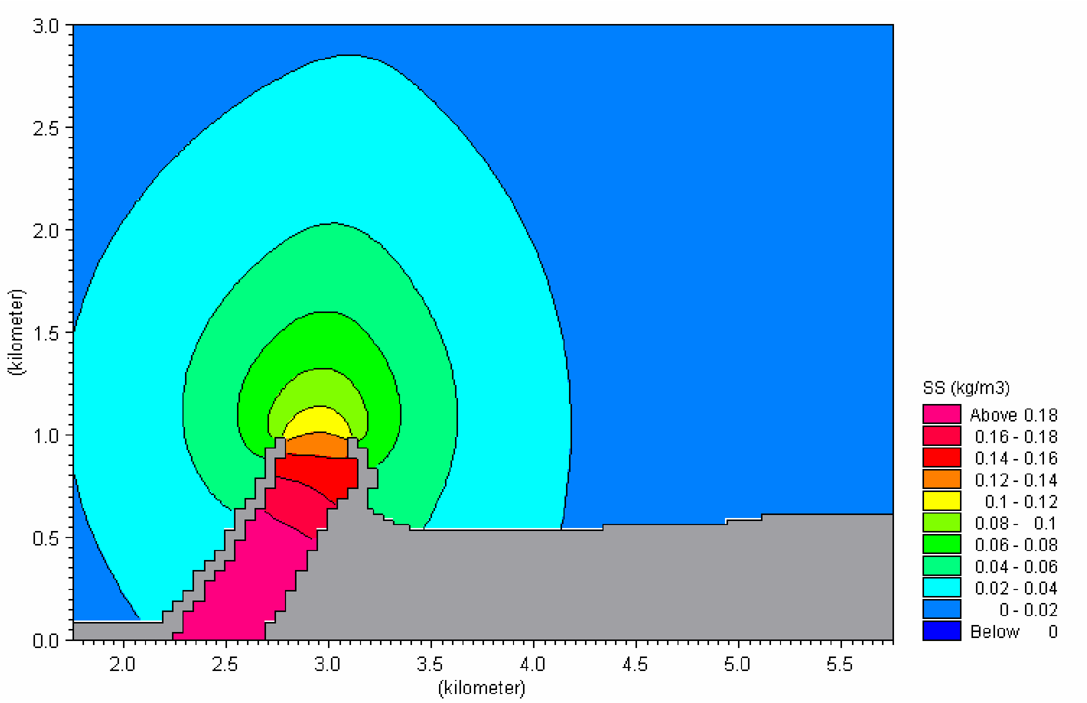
Figure III.2.5-3 Sedimentation Thickness for Outer Channel

2.5.4 Sedimentation in Outer Port by Suspended Load from Lagoon

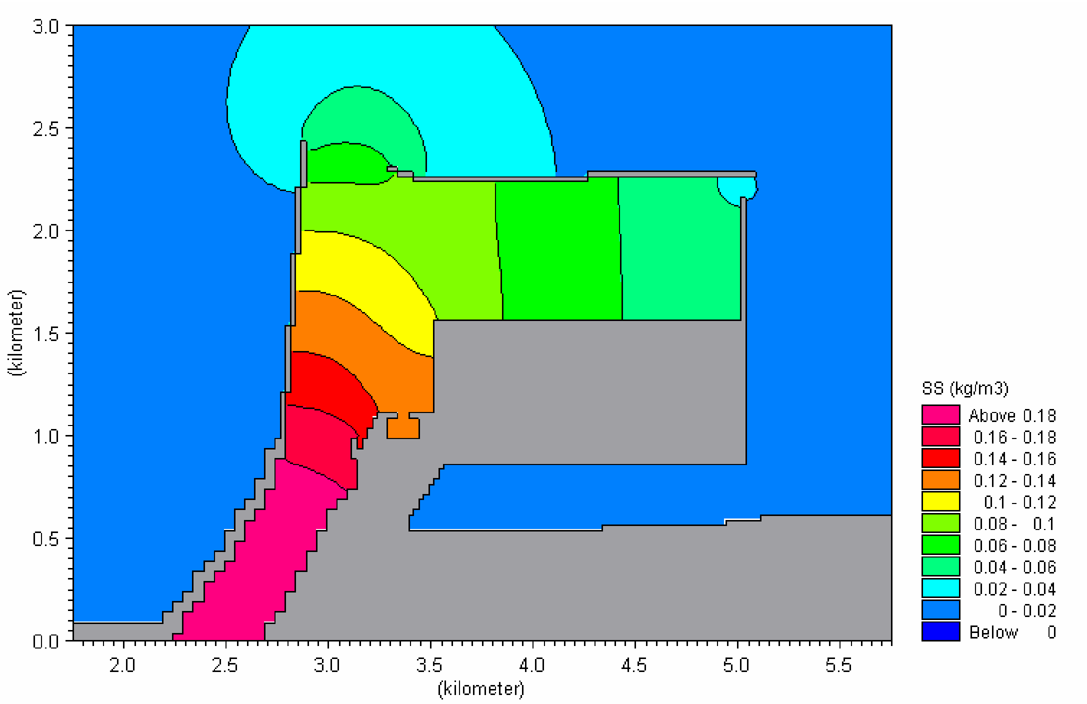
The sedimentation in the outer port basin that is caused by the suspended load from the Lagoon has been examined by using the numerical computation model for mud transportation, including such factors as convection, diffusion, settling, and sedimentation process. The numerical computation has been conducted for two cases - before and after construction of the outer port. The layout of the outer port plan has been taken from the master plan in 2025. In the computation, two kinds of flow rates ,Q=600m^3/s (average) and 1,200m^3/s(peak) have been used referring to the data from the Institute of Geography, Lithuania.

Figures III.2.5-4 and III.2.5-5 show the concentration of suspended load for the case of Q=1,200m^3/s and the distribution of sedimentation due to the suspended load, respectively. Figure III.2.5-6 shows the sedimentation rate, which has been calculated from the distribution of sedimentation inside and outside the port basin. The vertical axis indicates the sedimentation ratio to the total sedimentation volume outside of the Lagoon before and after construction with a flow volume of Q=1,200m^3/s (refer Appendix F 4 for all computation results). The following are the present understanding on the sediment regime expected.

- The suspended load from the Lagoon, at presemt, diffuses outside the port area and widely expands to the open sea.
- After the construction of the outer port, most of suspended load will stay in the new port basin that is a calm region, where the suspended load will settle and accumulates due to the stagnation of flow, and it causes remarkable sedimentation in the port basin.
- As the result of the numerical computation, 70 - 80 % of the total sediment volume will accumulate in the new outer port basin



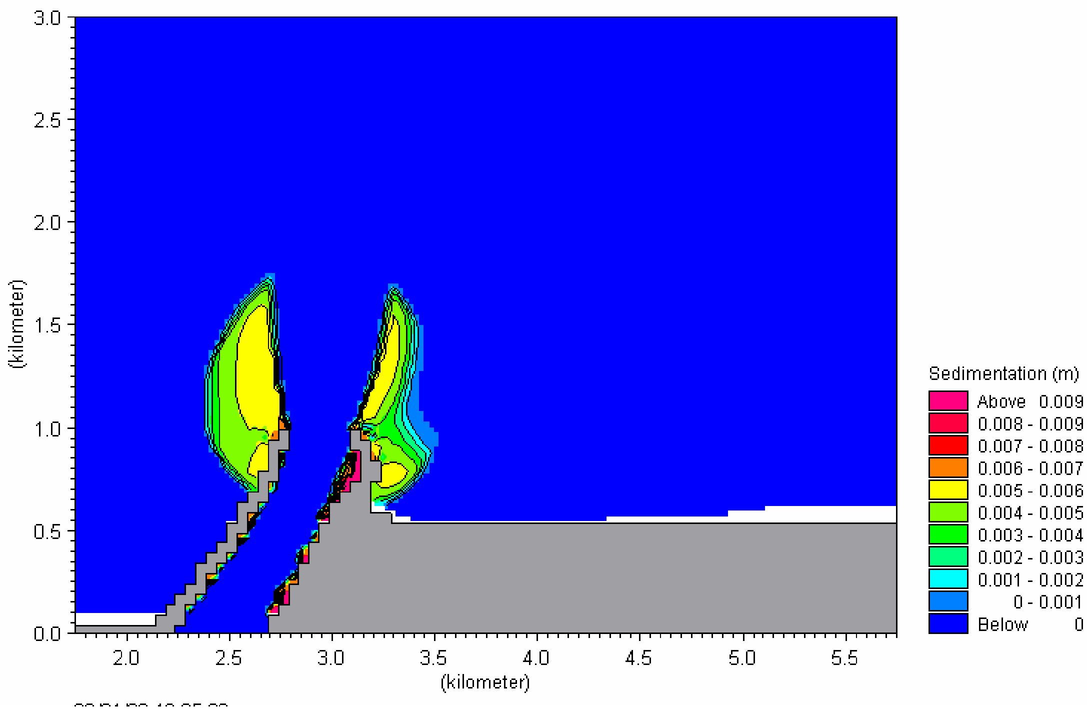
(1) Before Construction of Outer Port (Present Condition)



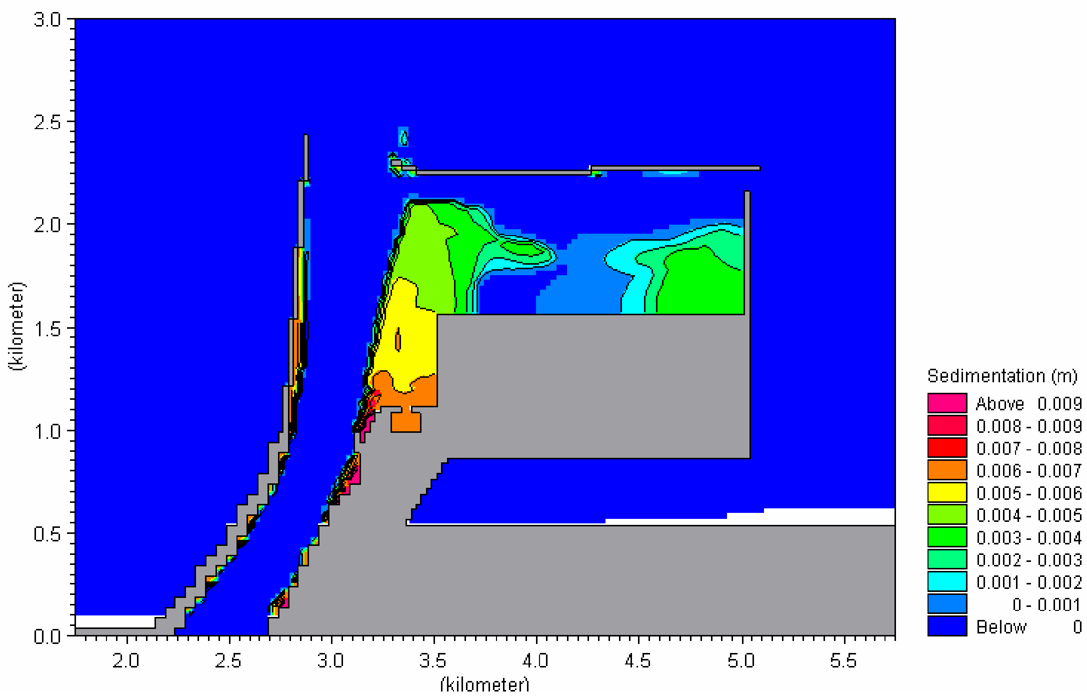
(2) After Construction of Outer Port

Source: The JICA Study Team

Figure III.2.5-4 Distribution of Concentration of Suspended Load ($Q=1200m^3/s$)



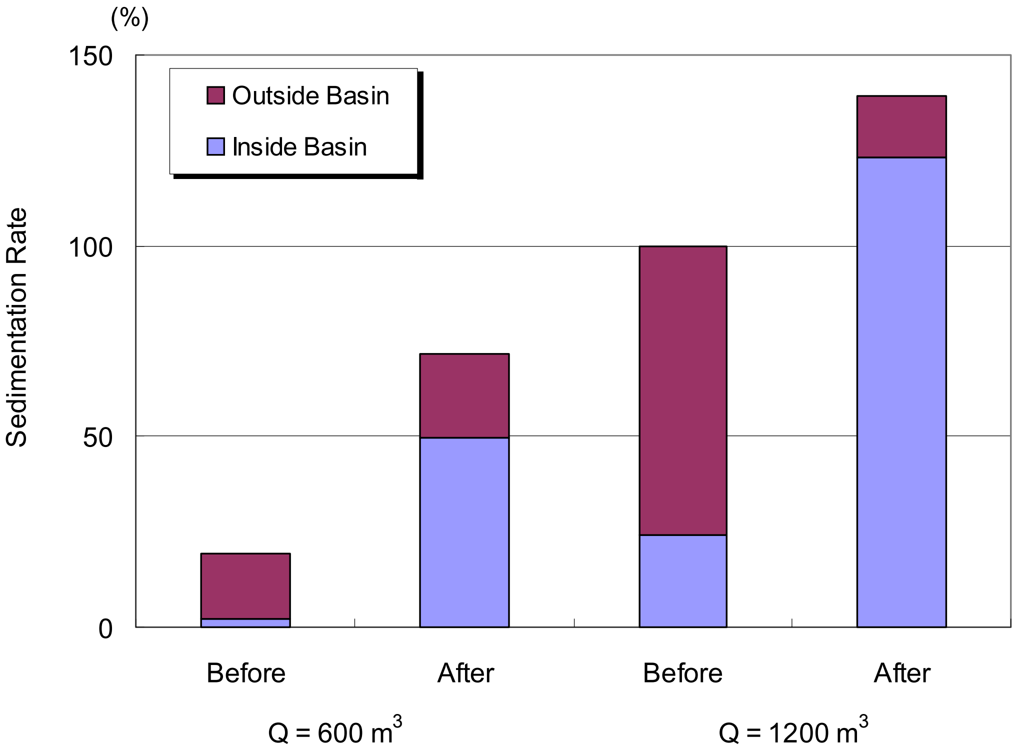
(1) Before Construction of Outer Port (Present Condition)



(2) After Construction of Outer Port

Source: The JICA Study Team

Figure III.2.5-5 Distribution of Sedimentation due to Suspended Load and Current ($Q=1{,}200m^3/s$)



Source: The JICA Study Team

Figure III.2.5-6 Comparison of Sedimentation Rates in Outer Area Before and After Construction of Outer Port

2.6 Influence of Outer Port Development on Surrounding Coastal Areas

With the development of the outer port, it is likely that the coastal areas surrounding the Port will experience geographycal changes, particularly so in the northern part. One of the concerns is the beach erosion with the interception of littoral drift due to the construction of the new breakwaters. Another concern would be the shoreline change induced by the formulation of a shadow region sheltered from nearshore waves after the construction of new offshore breakwaters. The former becomes a factor to cause the beach erosion in wider area about range of more than 10 km, and the latter becomes that in limited area of several km in general according to the scale of the harbors layout. The influence on these two points is described as follows.

2.6.1 Influence on the coast line due to interception of littoral drift

The characteristics of the northward littoral drift in Lithuanian coast is reported in III Section 1.7. This littoral drift is likely to cause beach erosion on the northern side of the port area and accumulation on the southern side.

There is a possibility of change in shoreline formation due to the dredging work of -14.5 m and the recent extension of breakwaters at the port entrance. The monitoring survey of the shoreline around Klaipeda Port started few years ago, but the data has not been well accumulated to date. Under these circumstances, high resolution aerial photographs (AGI in 1997) and the aerial photograph taken by the JICA Study Team from a helicopter in 2003 have been compared and the historical changes in shorelines have been analysed.

Figure III.2.6-1 shows these shoreline changes between 1997 and 2003. No remarkable change could be found on the southern coast line. On the northern coast, some retreats can be noticed in the vicinity of the northern breakwater about 1 to 2 km long, while there is almost no change from the point 0 up to the point of 6-7 km from the port entrance (Melnrage to Giruliai). Photo III.2.6-1 shows the satellite photo indicating the geomorphology of shoreline around port entrance. The orientation of the beach line from Melnrage to Karkle is different from that on the northern side of Karkle, almost perpendicul to the incident wave of WSW direction, which means that less magnitude of sedimentation occur. A sea cliff of glacial till extends about 1km long along the shore line of Karkle, where some boulders are observed in the glacial till layer (Photo III.2.6-2). These facts may lead to saying that the sea cliff at Karkle has functioned as the boundary for the movement of northward littoral drift, and leads to the local difference of beach line and maintain the balance of beach at the north part of port entrance. From these facts, it is concluded that there is no remarkable influence on the northern coast line due to intercept of northward littoral drift by the construction of outer port. It seems that the sea cliff at Karkle is under process of erosion from Photo III.2.6-2. However, it is unidentified whether this erosion becomes to be accelerated in recent years or not because there are not sufficient data of shoreline monitoring since past. To make these phenomena clear, it is needed the long term monitoring survey.

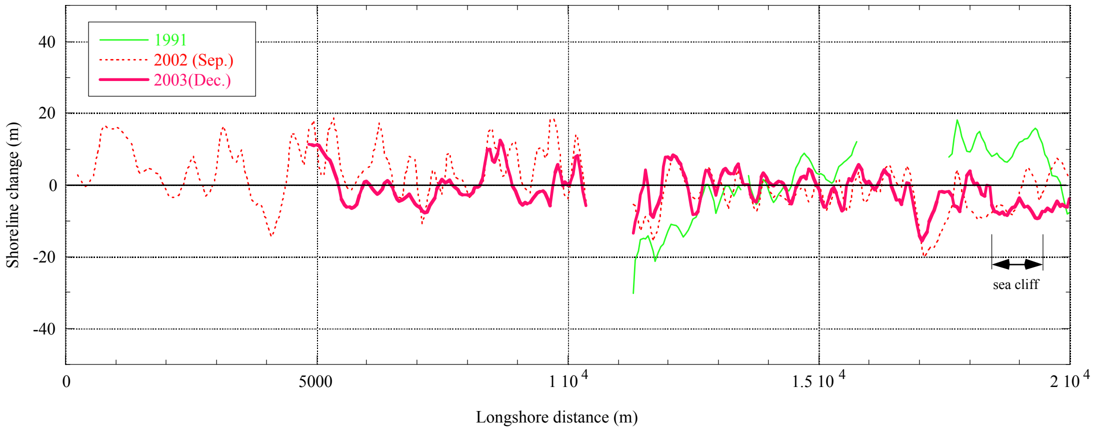


Figure III.2.6-1 Shoreline Change (based on 1997)

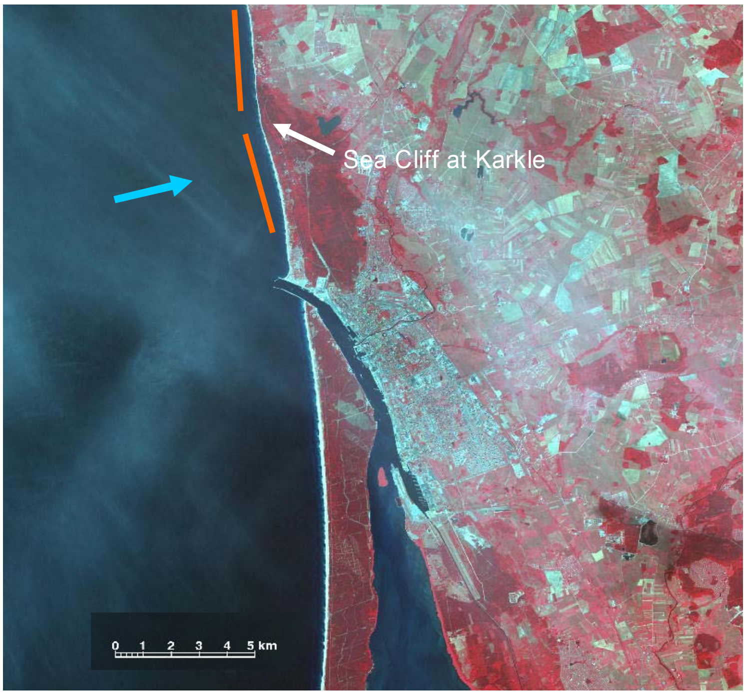
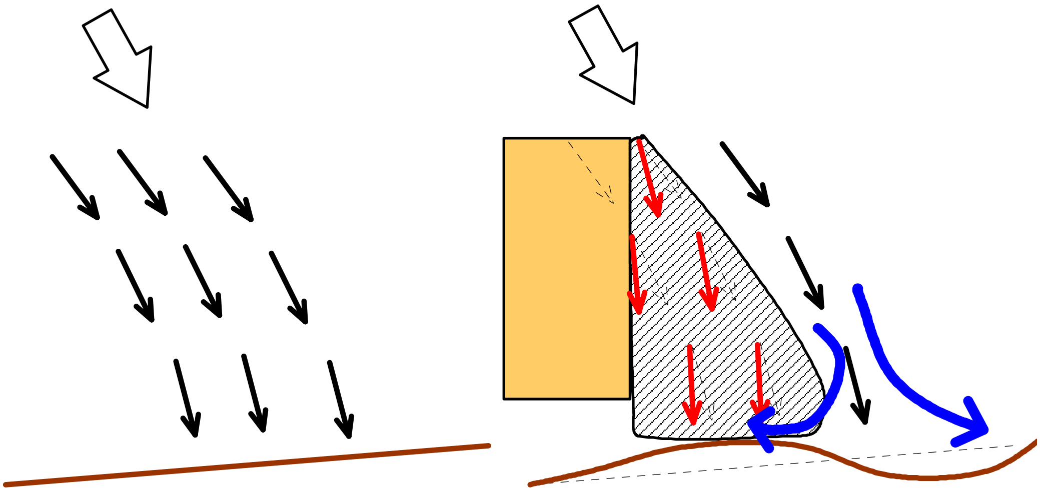


Photo III.2.6-1 Geomorphology Shape of Shoreline

Photo III.2.6-2 Sea Cliff at Karkle

2.6.2 Change in Shoreline by Forming Shadow Region of Waves

When a breakwater is constructed, a shadow region is created by the breakwater. This local sediment regime induces longshore currents, which allows the adjacent sand to move into this shadow region from outside regime. As the result, topographic change in shore line take place, most likely erosion occurs outside the shadow region and accresion inside the shadow region. The incident waves from W –WSW is predominant in the port area, so that the shadow region of waves will be created on the northern side of the new north breakwater as shown in Figure III.2.6-2. The wave direction will also change due to wave diffraction and circulation in this area. The changes in the fields of waves and currents cause the shoreline change, whose range is rather limited and it is known from the experience that the shoreline influenced on the downdrift side is about three times as large as the distance of the breakwater extending from the shore to the offshore end.



(1)Present Condition (2)Predictive Condition

Figure III.2.6-2 Shoreline Change Caused by Forming Shadow Area of Waves

It is difficult to predict such shoreline change quantitatively. Here, two kinds of study have been performed. One is to predict the outline of stable beach line by using mathematical formula which is based on many cases of experimental results of beach line for the location of behind the cape and offshore breakwater proposed by Dr. Hsu. The other is case study to know the example of actual beach line change for the island type port which has similar scale and layout with some clearance between shoreline and port area such as proposed outer port plan. The results for both studies are presented in Appendix F 6. The obtained results are brought together as follows (Figure III.2.6-3).

- The accumulation area in the shadow region is caused within the range of about 1.2km toward the north side from the northern breakwater, and the advance of beach width is expected about 200m from the results of two kind of study.
- There is a possibility that the retreat of shoreline is caused about order of 50m when assuming that the retreat, which volume is balanced to the accumulation, is caused until the vicinity of Karkle where sea cliff exists.

- Several countermeasures can be proposed as that for decreasing the predictive change of shoreline and for maintenance of sandy beach as follows.

1) To be decreasing the shadow region for waves (for example, change of shape at the north part of offshore breakwater from straight shape to make curvature, the adjustment of clearance; distance between shoreline and port area, etc).

2) Beach nourishment together with construction of additional coastal protection facility such as groins or headlands to decrease the retreat of shoreline.

3) To be decreasing the sand movement from northern side to southern accumulation area by filling up the sand in advance.

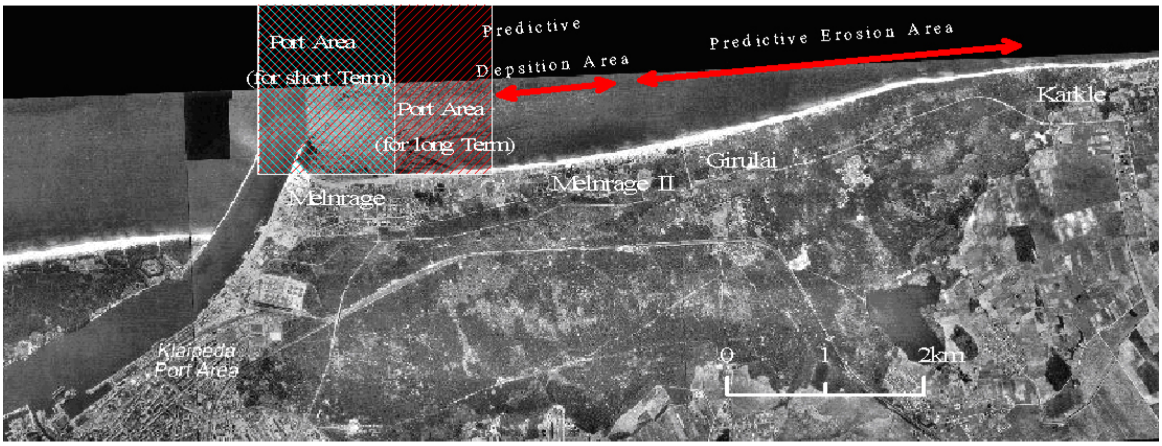


Figure III.2.6-3 Location of Predicted Shoreline Change in Northern Side

CHAPTER 3 PORT ENGINEERING STUDY

CHAPTER 3 PORT ENGINEERING STUDY

3.1 Design Manual, Standards and Codes

In Lithuania, the design manual named "Recommendations of the Committee for Waterfront Structures, (EAU 1996 - Harbours and Waterways)" is mainly used for the engineering design for marine structure. So, this manual has been basically applied and for the particular design items where the above manual was not applicable, other internationally accepted manuals such as British Standard, Shore Protection Manual, and Japanese Design Manual have been used among others.

The existing design norms, codes, and standards of Lithuania are mostly based on the Russian standards. The major norms, standards, and codes referred to in this study for the design of marine facilities are listed below:

i)	SNip 2.06.04-82	Loads and impacts of hydraulic works (waves, ice, and vessels)
ii)	SNiP 3.07.02-87	Hydraulic works of sea and inland waterway transport
iii)	SNiP 2.06.08-87	Concrete and reinforced concrete elements of hydraulic products
iv)	SNiP 2.03.01-84	Concrete and reinforced concrete elements
v)	LST 1341:1995	Concrete and R/C Components and Products
vi)	LST 1341:1995	Cement Composition, Technical Requirements, Signs of Compliance
v)	Nr. 297:1996	Technical Provisions of Railway Usage
vi)	GOST 9238-83	Construction and Rolling Stock Clearance Diagrams for the USSR Railways of 1520 (1524) mm Gauge

3.2 Design Criteria

The design criteria of natural conditions have been derived from the design manual, regulations and codes of Lithuania. The results from the field investigations carried out by the JICA Study Team and data/information collected through the Study have also been fully utilized in establishing the design criteria.

3.2.1 Water Levels

The astronomical tidal motion of the North Sea does not affec the coast of Lithuania, so the seasonal changes in the water levels in the Port occur due to the fluctuation of water volume from the Nemnas river. The daily change of water level in the Port is minimal, and the daily movements are quite small. Consequently, the difference in water levels between the sea water and the residual water at quays is negligible small. This local condition has been considered in structural design.

The datum of elevations used for the engineering design is referred to the Baltic Sea Level (BSL), which is equal to the mean sea water level at Baltic Sea.

The water levels in the Baltic Sea and Klaipeda Strait have been analysed by Lithuanian Energy Institute. These data are tabulated below together with return periods. It shows that the range of water levels in Klaipeda Strait is larger than those in Baltic Sea.

Table III.3.2-1 Maximum and Minimum Water Levels in Baltic Sea and Klaipeda Port

(unit : cm BSL)

Return Period		1 year	2 years	5 years	10 years	50 years
Baltic Sea	Maximum	+48	+80	+100	+115	+126
	Minimum	-18	-20	-26	-41	-58
Klaipeda Strait	Maximum	+45	+85	+110	+124	+162
	Minimum	-50	-68	-77	-83	-97

Source : Lithuanian Energy Institute

3.2.2 Design Waves

The characteristics of design waves have been developed from the data obtained through the wave observation by the JICA Study Team and other data available in Baltic Sea. And, the offshore incident design waves have been set up on a wave direction basais and with a return period of 50 years as tabulated below.

Wave Direction	Wave Height	Wave Period
SW	5.6 m	8.5 sec
WSW	5.9 m	8.5 sec
W	7.3 m	9.5 sec
WNW	5.6 m	8.5 sec
NW	4.9 m	7.5 sec

The above offshore waves have been transformed into nearshore waves as described in the Section 2.1. The design waves at the location of the proposed marine structures in the outer port area are summarized in Table III.3.2-2.

Table III.3.2-2 Design Waves (H1/3) at Each Location

Offshore Wave Direction/ Location	WSW		W		WNW	
	Wave Height	Incidental Angle	Wave Height	Incidental Angle	Wave Height	Incidental Angle
DW-1	5.6 m	247°	6.7 m	272°	5.6 m	289°
DW-2	5.4 m	250°	6.4 m	270°	5.1 m	270°
DW-3	5.5 m	249°	6.5 m	270°	5.1 m	270°
DW-4	5.6 m	252°	5.9 m	270°	5.2 m	270°
DW-5	3.8 m	270°	4.9 m	270°	4.1 m	284°
DW-6	2.0 m	270°	3.5 m	270°	3.7 m	283°
DW-7	1.8 m	270°	3.1 m	270°	3.2 m	283°
DW-8	5.2 m	270°	6.5 m	270°	5.5 m	285°
DW-9	4.3 m	270°	4.5 m	271°	4.3 m	283°
DW-10	2.4 m	250°	2.8 m	250°	1.9 m	250°
DW-11	2.1 m	235°	2.6 m	235°	1.4 m	235°
DW-12	1.2 m	225°	1.4 m	225°	0.7 m	225°

Source : Estimate by the JICA Study Team

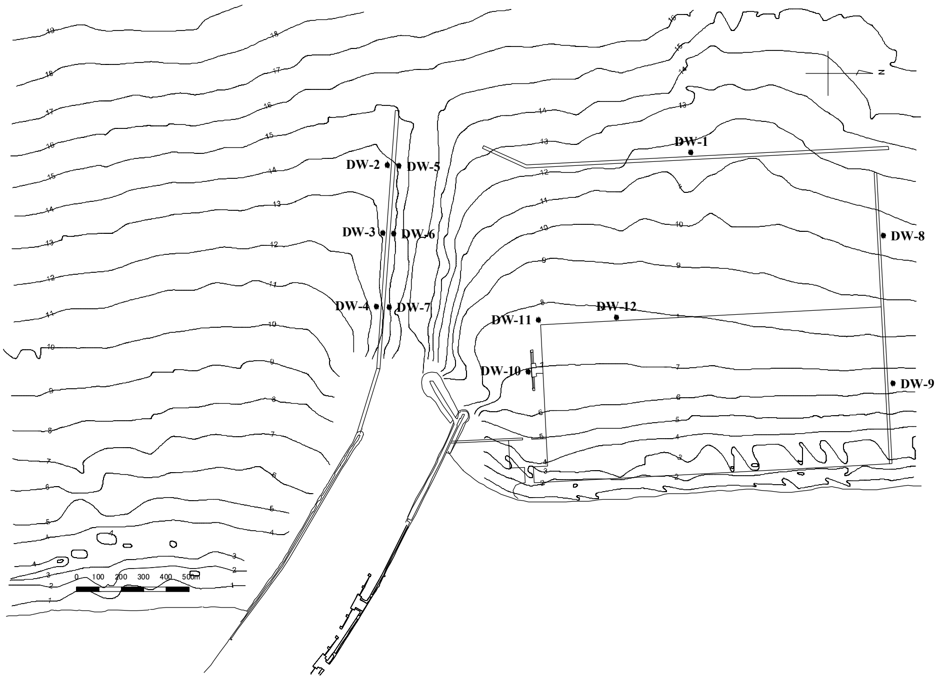


Figure III.3.2-1 Locations for Design Wave Analysis

3.2.3 Seismic Load

The seismic disturbance is reportedly so small around Klaipeda. To confirm this information, the past records of earthquake have been collected, including the date of occurrence, location (Latitude and Longitude), depth of epicenter and magnitude. The earthquake that occurred within the distance of 500 km from Klaipeda City for the last 50 years are listed in Table III.3.2-3.

The horizontal acceleration by the earthquake has been estimated based on the equation established in the “Standards for roads and bridges in Japan”. The maximum horizontal acceleration is estimated at 10.5gal for the earthquake occurred in 1995 which had magnitude of 4.3at about 132 km far from Klaipeda with a direction of WSW. The intencity is quite small, equivalent to 1% of gravity.

Considering the above analysis and safety of structural stability, the seismic coefficient has been determined at 0.05 (kh) for structural design.

Table III.3.2-3 Earthquake Research Results and Acceleration

Date (y/m/d)	Latitude	Longitude	Depth	Distance	Magnitude	Horizontal Acceleration
1976/10/25	59.16N	23.73E	33 km	413 km	4.50	3.561 gal
1984/10/17	52.02N	17.12E	10 km	488 km	4.10	2.206 gal
1995/02/23	54.61N	19.69E	10 km	153 km	4.00	7.291 gal
1995/06/20	55.23N	19.19E	10 km	132 km	4.30	10.499 gal
1996/11/06	51.41N	19.26E	10 km	496 km	4.20	2.327 gal
2001/04/17	51.44N	19.26E	5 km	492 km	4.80	3.620 gal
2002/12/18	55.89N	18.21E	10 km	181 km	4.20	7.080 gal

Source; Estimate by the JICA Study Team based on data by National Earthquake Information Center

3.2.4 Ice and Snow Loads

As Klaipeda is located in the 2nd snow region, the design snow load is 0.75KN/m^2. The minimum ice load is 250KN/m.

3.2.5 Subsoil Condition

The soil investigation carried out by the JICA Study Team are presented in Chapter 1, and more details are included in Appendix E. The soil borings show that the subsoil at the outer port area is composed of three stratum, namely a Holocene stratum in the upper part, a Pleistocene Limnic stratum in the middle, and a Pleistocene Glacial stratum in the lower part. The profiles of these layers are shown in Figure III.3.2-2.

a) Holocene Stratum – Upper Stratum

This stratum has a thickness of 4 to 7 meters, and is composed of layers Nos. 8, 9, 12, 14, and 15. It consists of loose-dence silty sand with shell, organic matter and gravel. The standard penetration test produced N-value of 20-40.

b) Pleistocene Limnic Stratum – Middle Stratum

This stratum has a thickness of 0.5 to 6.5 meters, and is composed of the layer Nos. 22 and 23. It consists of silty clay with gravel occasionally. The penetration N-value is about 30-40. About 75-90 % of soil particle of this layer is clay and silt, thus it is not suitable for reclamation fill.

c) Pleistocene Glacial Stratum – Lower Stratum

This stratum is represented by the layer No. 20. It consists of sandy clay with a low plasticity mixed with gravel and cobble. This layer is very hard with N-value of about 80, and is mainly composed of silt and clay having 50-60 % of soil particle.

For structural designing, the soil characteristics for each layer have been determined as below.

Table III.3.2-4 Soil Parameters for Preliminary Design

Stratum	N-Value	Dry Bulk Density	Int. Friction Angle	Cohesion
Upper Stratum	20 to 40	1.8 t/m^3	30°	—
Middle Stratum	30 to 40	1.8 t/m^3	22°	100 KN/m2
Lower Stratum	Over 50	2.0 t/m^3	32°	200 KN/m2

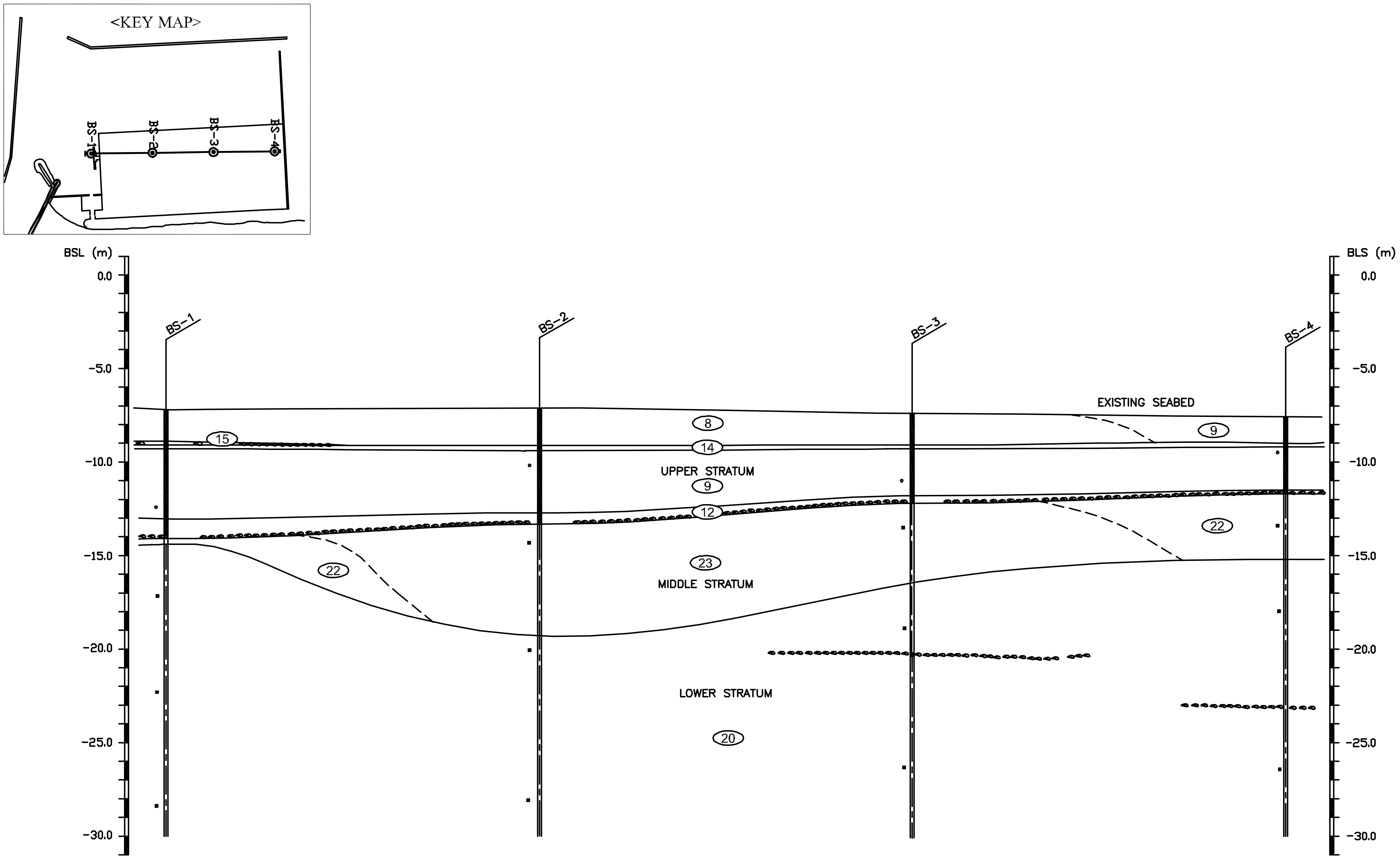


Figure III.3.2-2 Subsoil Profile at Outer Port Area (Scale V: 1/300, H: 1/7500)

3.3 Design of Breakwaters

3.3.1 Selection of Structural Type of Breakwaters

The outer port will be protected by three breakwaters namely:

- West Breakwater,
- South Breakwater, and
- North Breakwater.

The West Breakwater will be placed at the water depth of 15m to 12.5 m, and the head portion of the South and North Breakwaters will be located at -16.5 m and -14 m respectively.

The clay stratum lying at the middle and lower parts of subsoil, have high cohesion and internal friction angle. The upper silty sand stratum is chiefly composed of sand containing organic matter and gravel, but the penetration resistance is as high as N-Value of 20-40.

Between the sandy (upper) and silty clay (middle) strata appears a thin sandy layer of less than 1m, which contains gravel and cobble of 10cm-30cm in diameter. Moreover, it is believed that the maximum size of cobble would be much larger, because boulders of 1-2m size are found on the beach near Karkle (about 10km north of the port entrance), where the same sandy layer extends from the port.

As discussed in the foregoing section, the design wave heights for the West Breakwater has been set at 6.7 m. Judging from this size of design wave and the subsoil condition mentioned above, a gravity type of breakwater would be the optimum solution in terms of economy and stability. So, following three alternative gravity structures have been studied:

- Rock-Mound Type with Tetrapods,
- Rock-Mound Type with Accropods, and
- Caisson Type

The breakwaters recently constructed under the Port Entrance Rehabilitation Project is of a rock-mound structure armoured with concrete blocks (see details at Section G.1 of Appendix G). Since natural stones, both large and small, are not available in Lithuania, core materials of rock-mound structure were imported from Scandinavian countries and concrete blocks for armour stones were manufactured in Lithuania.

In the past, various kinds of concrete blocks were used for the construction of breakwater in the world, including Tetrapod and Accropod. To find the optimum block type, Tetrapod and Accropod have been added for comparison. Accropod is particularly of advantage in construction costs, because only single layer of accropod can be applied as armour stone instead of 2-3 layers generally applied for other alternatives.

Quarry-run ranging from 0.1 to 300 kg will be used for the core material of rock-mound A secondary armour layer will be laid on the trimed surface of the core materials, which functions as a filter between primary armour layer and core. After trimming the surface of secondary layer, artificial concrete blocks will be placed to form a primary armor layer.

Caisson structure is commonly applied for breakwaters where a firm foundation layer such as stiff clay, compacted sand or rocks could be found in a shallow subsoil elevation. Caisson structure has also advantage, when executed in deep water areas, because it can minimize time requirements for offshore marine works. Concrete-box can be manufactured onshore or on floating docks without any obstruction from marine climate. After manufactured, they are towed out to the sea and placed on the designated positions, making use of several calm days. However, a rock-mound structure consumes considerable times to complete the whole section of the structure in the sea.

3.3.2 Crest Elevation of Breakwaters

Crest elevations of breakwaters have been set at 0.6 times of significant wave height above high water levels, when overtopping waves are allowed. When a basin is narrow or some wharf facility is located behind breakwaters, crest elevations should be raised to reduce amount of overtopping waves. Since the water basin behind the West Breakwater is considerablly wide and the wharves will be constructed in the distance, overtopping free condition (coefficient of 0.6) has been taken for designing the crest elevations of all the breakwaters.

3.3.3 Size of Armour Blocks

Armour blocks should be sized in weight to withstand the design wave forces. In proportion to design wave heights, sizes of rock or artificial concrete blocks increase to dissipate corresponding wave energy.

The size of armour rock has been determined with the formula of Hudson as below:

$$W = (\rho s \times Hdes^3)/Krr \times (Sr - 1)^3 \times \cot \alpha$$

Where,

W : Weight of an individual armour unit (kN)

ρs : Mass density of the armour material (kN/m^3)

Hdes : Significant wave height ($H_{1/3}$ m)

Krr : Shape and stability coefficient

Sr : Specific gravity of armour unit relative to the water ($Sr = \rho s/\rho w$)

α : Slope angle of covered layer (degree)

Stability coefficient (Krr) for armour blocks in wave breaking condition had been assumed at:

- Krr = 8.0 for TETRAPOD
- Krr = 15.0 for ACCROPOD

A slope angle of 1 : 4/3 has been applied for rock mound consirdering armour block characteristics of TETRAPOD and ACCROPOD.

The required sizes of primary and secondary armours in the trunk portion for Tetrapod and Accropod have been computed as shown in Table III.3.3-1. The sizes of secondary layers have been determined following the recommendation of the Shore Protection Manual, where the secondary layer is defined in size of 1/10 to 1/15 of the primary block. At the head portion of the breakwaters, armour blocks should be increased 1.5 times in weight to secure higher stability of structure.

Table III.3.3-1 Required Sizes of Armour Concrete Block and Rock for Trunk Portion

	Depth	Design Wave Height	Primary Cover Layer		Secondary Cover Layer
			TETRAPOD	ACCROPOD	
West Breakwater	-15 to -12 m	6.7 m	40 t	9.0 m3 (22 t)	4 to 7 t Rock
South Breakwater	-15 to -13 m	4.7 m	16 t	3.0 m3 (7 t)	1 to 3 t Rock
North Breakwater	-14 to -12 m	4.7 m	16 t	3.0 m3 (7 t)	1 to 3 t Rock
	-12 to -9 m	4.3 m	12.5 t	2.5 m3 (6 t)	1 to 3 t Rock

The stability analysis has been conducted for each caisson-type breakwater by applying the design waves corresponding to their location. In determining the size of caisson box, it has been assumed that six boxes would be manufactured on a 6,000 to 8,000 floating dock, which is an effective size for the planned caisson structure. Table III.3.3-2 shows the sizes of the proposed caisson boxes.

Table III.3.3-2 Required Caisson Boxe Sizes for Trunk Portion

	Depth	Design Wave Height	Caisson Size (Width x Length x Height)
West Breakwater	-15 to -12 m	6.7 m	18 m x 18 m x 12 m
South Breakwater	-15 to -13 m	4.7 m	12 m x 12 m x 12 m
North Breakwater	-14 to -10	4.7 m	12 m x 12 m x 9 m

The sizes of toe concrete blocks and toe protection concrete blocks, both of which are required to protect rock mound from scouring action by waves, have been determined as shown in Table III.3.3-3.

Table III.3.3-3 Required Toe Concrete Block and Toe Protection Block

	Depth	Toe Concrete Block	Toe Protection Armor Block
West Breakwater	-15 to -14m	42.3	16 t
	-13 m	37 t	16 t
South Breakwater	-15 m	37 t	6 t
	-14 to -13 m	24.8 t	6 t
North Breakwater	-14 to -13 m	37 t	12 t
	-12 to -10 m	24.8 t	12 t

3.3.4 Standard Section of Breakwaters

The required cross sections of rock-mound breakwaters with armour concrete blocks of TETRAPOD and ACCROPOD, and concrete caisson box have been determined based on the design criteria established in the previous sections. The typical cross sections of the West Breakwater at the trunk portion are shown in Figures III.3.3-1, III.3.3-2 and III.3.3-3.

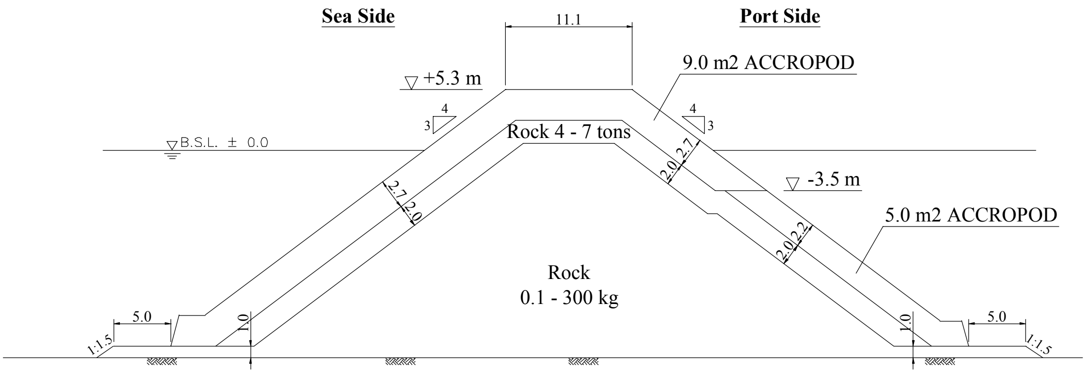


Figure III.3.3-1 Rock Mound Type West Breakwater with TETRAPOD (Trunk Portion)

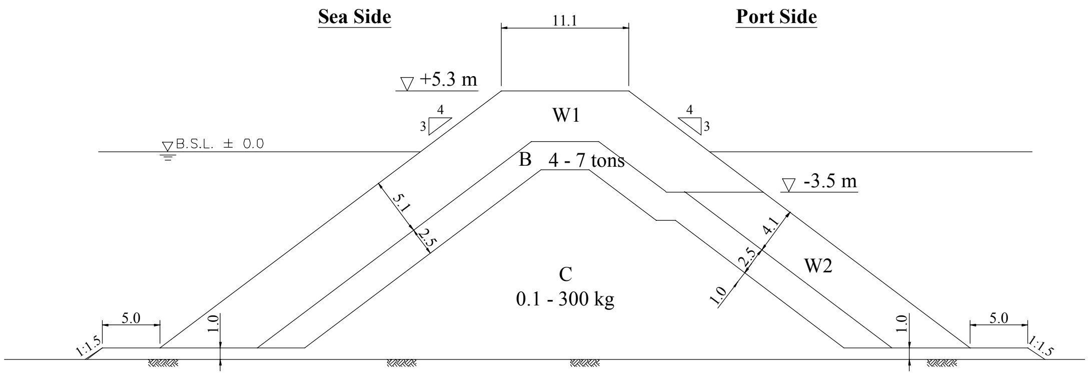


Figure III.3.3-2 Rock Mound Type West Breakwater with ACCROPOD (Trunk Portion)

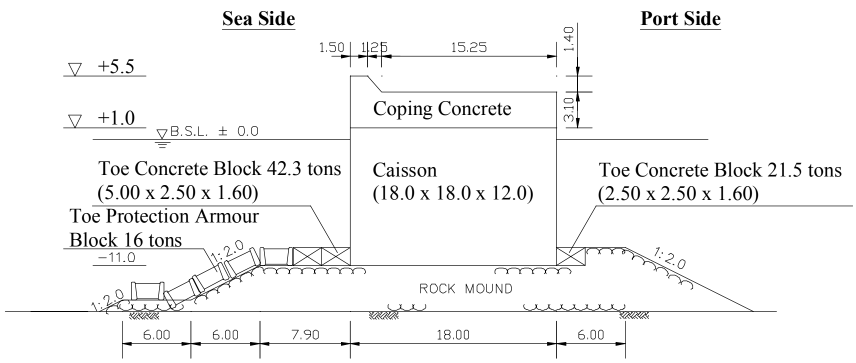


Figure III.3.3-3 Caisson Type West Breakwater (Trunk Portion)

3.3.5 Selection of Structural Type of Breakwaters

When evaluating the three alternatives structures, several factors should be taken into account, including environmental constraints, cnstruction cost and time requirements, among which no quantifiable difference could be found other than a cost factor. The comparison of cost between three alternatives for the West Breakwater is presented in Figure III.3.3-4. It shows that the rock mound type with ACCROPOD would be the most economical structure for the water depths between -16 m to -12 m. Thus, this structural type has been selected for the West Breakwater, where the natural water depths are in the range of 13m to 14m.

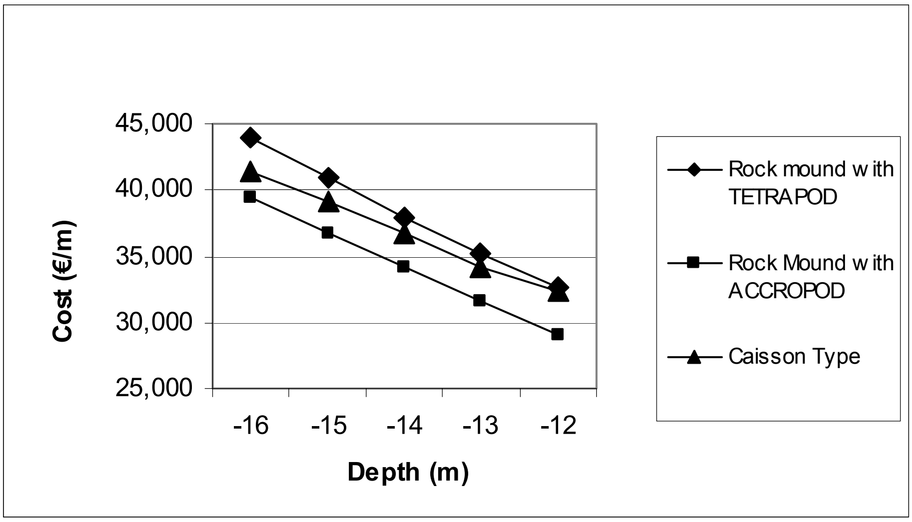


Figure III.3.3-4 Comparison of Cost for Alternatives of West Breakwater

Similarly, Figure III.3.3-5 shows the cost comparison for the South Breakwater. When comparing two rock mound types, ACCROPOD gives less construction cost for all water depths. The caisson type structure is most economical for the water depths greater than -15.0 m, and more economical than ACCROPOD. Therefore, it has been determined that the caisson type would be used for the section deeper than -15m, while the rock mound type for shallower than -15m.

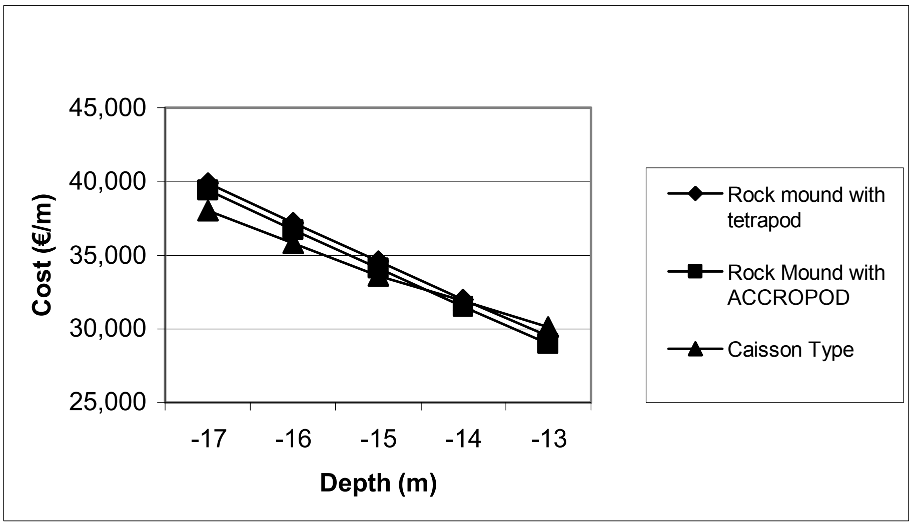


Figure III.3.3-5 Comparison of Cost for Alternatives of South Breakwater

For the North Breakwater, similar comparison has been made and the results are shown in Figure III.3.3-6, which reveals that the rock mound type with ACCROPOD is the most economical for all water depths.

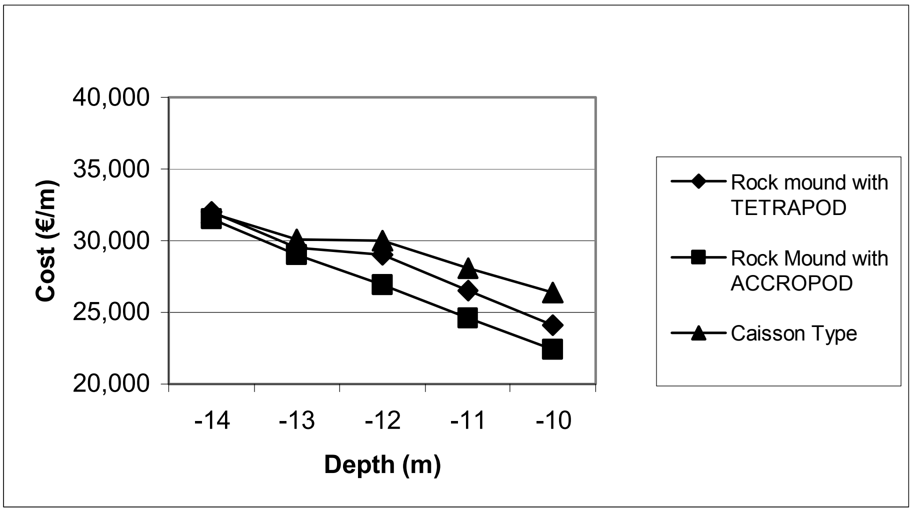


Figure III.3.3-6 Comparison of Cost for Alternatives of North Breakwater

3.4 Design of Quaywall Structures

3.4.1 Design Parametres

(1) Ship Particulars

The berth structures have been designed to receive the maximum size of the vessels expected in the outer port area. The vessels calling in the port are described in the Chapter 2 of Part II and design vessel sizes (see Table III.3.4-1) .

Table III.3.4-1 Design Vessel Size for Berths of Outer Port

Berth	Vessel Size	LOA	Breadth	Draft
Berth No. 1 - Petroleum Jetty	109,000DWT	244 m	42.3 m	14.9 m
Berth No. 2 - Grain Bulk	123,000DWT	266 m	40.6 m	15.4 m
Berth No. 3 – Fertilizer	123,000DWT	266 m	40.6 m	15.4 m
Berth No. 4 – Fertilizer	74,000DWT	225 m	32.3 m	13.5 m
Berth No. 5 – General Cargo	74,000DWT	225 m	32.3 m	13.5 m
Berth No. 6 - Container	4,800TEU	294 m	32.2 m	13.5 m

(2) Surcharge and Live Load

The surcharge and live loads on each berth have been determined following EAU and taking operational condition of each berth into account. The live loads from cranes and cargo handling equipment have been presumed based on the commodities handled and operational method.

Table III.3.4-2 Surcharge and Live Loads

Berth	Crane Load	Uniform Load (Normal)
Berth No. 1	Oil Loading/Unloading Arm	10 KN/m2
Berth No. 2	Grain Loader 1,500 t/hr, Unloading Pipe for Liquid Bulk	Apron 10KN/m2, Yard 50 KN/m2
Berth No. 3	Loader 2,500 t/hr, Level-ruffing Crane 40 t	50 KN/m2
Berth No. 4	Loader 2,500 t/hr, Unloader 1,000 t/hr Level-ruffing Crane 40 t	50 KN/m2
Berth No. 5	Level-ruffing Crane 40 t	50 KN/m2
Berth No. 6	Gantry Crane	Apron 20 KN/m2, Yard 50KN/m2

(3) Elevation of Quaywall

The elevation of quaywall shall be determined in consideration of cargo handling patterns, vessels sizes expected to call at the port and the local variation of water levels. When fixing the working level, following principal factors have been taken into account:

For the port with a tidal range less than 3.0 m, top elevations of quaywalls, for larger vessels, say more than 10,000DWT, are generally set at 1.0 to 2.0 m above high water levels in Japan. The fluctuations of water levels in the Port is not significant. The possible higher water level with 10-year return period is 1.15 m and the lower water level is -0.41 m,

In a flood-free harbour, the elevation of wharf structures shall be 2.0 to 2.5 m above mean water level (EAU). The top elevations of quaywall structures at Klaipeda Port are in a range between 2.0 and 2.5 m BSL,

Vessels expected to call at the outer port will be larger than those presently calling in the existing port, and the maximum vessels expected at each berth of the outer port are as shown in Table III.3.4-1. In order to reduce reclamation volume, it would be economical to lower the top elevation as much as possible.

Taking into account the above engineering points, the top elevation of berth structures has been set at +3.0 m except for Berth No. 1, where the extreme design wave height would be as high as 2.8 m. This is attributed to its closeness to the port entrance where incidental waves will reach the berth without significant decay. Therefore, the top elevation of the platform of Berth No. 1 has been set at +5.0 m, which can clear the crest of the extreme waves.

(4) Mooring Forces

The bollards should be installed along the berth front to withstand the following mooring forces corresponding to the displacement tonnage of design vessels listed in Table III.3.4-3.

Table III.3.4-3 Line Pull Force of Bollard

Ship Displacement	Line Pull Force
Up to 100,000 ton	1,000 KN
Up to 200,000 ton	1,500 KN
Over 200,000 ton	2,000 KN

3.4.2 Structural Type of Quaywall

In selecting optimum structure for the quaywall facilities, following points have been considered.

(1) Maritime Conditions

The water levels vary quite slow in the port area, thus it is not necessary to consider residual water, resulting in elimination of the water pressure acting behind the quaywall structure.

With a large opening of the port entrance, particular care should be taken for wave actions at the southeastern part of the berths in outer port. Taking this slightly higher design waves into account, Berths No. 1 and No. 2 have been designed with a wave dissipating type.

(2) Geophysical and Geological Conditions

It is unlikely for the Port to experience large earthquakes. The horizontal seismic coeeficient has been determined as low as 5 % of gravity. This factor gives advantage to a gravity type structure.

One of outstanding subsoil conditions at the outer port is the existence of hard sandy clay layer situated at the elevation of -16 to -18 m BSL. This layer is covered by well consolidated sandy layer, thus it is suitable as a foundation layer without settlement. This layer has a considerablly high cohesion and intenal friction angle.

A thin sandy layer with gravel and cobble exists at the top of silty clay layer (middle stratum : see Figure III.3.2-2). If this layer is located above the planned dredging level, pile structure as well as gravity structure would be available, since the bearing capacity of the silty clay layer is high enough to support vertical load from the structure. Meanwhile, it would be a problem for piling operation, if this layer could be found below the dredging level.

(3) Operational Conditions

The quaywalls of the outer port shall be designed with the water depth of 17.0 m and 15.0 m. The commodities to be handled at each quaywall vary in its cargo format, such as liquid bulk, bulk of minerals, break bulk, container, etc.

Liquid cargoes are planned to be handled at Berth No. 1. The loading and unloading arms will be installed at the platform and the pipelines will be connected to storage tanks at the NAFTA tank farm.

The major commodities to be handled at Berth No. 2 would be grain, all of them handled through loader and belt conveyor system. A liquid cargo of UAN Solution will also be handled through the outlets of pipelines. Furthermore, apatite is planned to be unloaded at the berth and unloaded into the railway wagons through movable hoppers.

Berth No. 3 will function as a general cargo berth in the Short-term Plan, where raw sugar, ferro alloy, and steel products will be handled. In the Master Plan, bulk cargoes of fertilizer will be solely handled. In order to allow cargo handling equipment to move efficiently on the wharf apron in both plans, fixed superstructure like belt conveyor should be avoided.

Berths No. 4, No. 5, and No. 6, as planned in Master Plan, cargoes of fertilizer, general cargo, and containers will be handled respectively.

(4) Construction Conditions

In Lithuania, large floating equipment such as a dredger, floating crane and pile driving barge are not commonly available. However, several numbers of large floating docks are available at the port, thus it gives slight edge to concrete caisson.

(5) Existing Berth Structures

The structural types used for the existing berths are presented in Section G.1 of Appendices. In recent years, many berths have been renovated and deepened with a structural type of steel sheet pile. For instance, the berths No.5 and 6 have been rehabilitated with a steel steel pile of AZ48 (section element of 4.8 x 10^3 cm^3/m) as a front wall. However, the subsoil condition in outer port area is not preferable for this type due to difficulty in piling operation to the sandy layer with gravel and cobble. Moreover, the berths in outer port require deeper water depths for which larger section element is required for sheet piles such as steel piped sheet pile.

3.4.3 Selection of Alternative Quaywall Structures

Considering the above site conditions and required functions of the quaywall facilities, optimum structural types have been selected as below.

(1) Structural Type for Berth No. 1

At Berth No. 1, the sandy layer with gravel and cobble is expected to appear below the planned dredging elevation of -17.0 m for the port basin. To receive various sizes of oil tankers, a dolphin type structure would be the most suitable and economical. Thus, the preliminary design for the dolphin type structure has been conducted.

(2) Structural Type for Berths No. 2 to No. 6

The quayside depth of the Berths No. 2 and 3 is -17.0 m, and Berths No. 4 to No. 6 is -15.0 m. The subsoil condition gives an advantage to gravity-type structure, but cause some difficulty in constructing multi-storied concrete block structures. As such, a caisson type structure has been selected as a representative structure of gravity type wharf.

In addition to gravity type structures, a relieving platform structure has been selected as an alternative for comparative analysis.

3.4.4 Design for Alternative Structures

(1) Structure for Berth No. 1

To provide a working space for oil loading and unloading operation, a platform will be provided. Its top elevation has been set at +5.0 m to clear the crest elevation of incidental waves propagating to the berth front.

The size distribution and detailed ship particulars of tankers calling at Berth No. 1 is not known, so that it has been assumed that they would be in the range from 30,000 DWT to 110,000 DWT.

To accommodate these tankers, three sets of breasting dolphines and three pairs of mooring dolphines have been planned. The mooring dolphines will be equipped with quick release hooks of 80 tonnes pull capacity. The design pull for one mooring dolphine has been assumed at 260 tf and 150 tf for larger two sets and smaller four sets of mooring dolphines respectively.

Figure III.3.4-1 shows plan and front view of Berth No. 1.

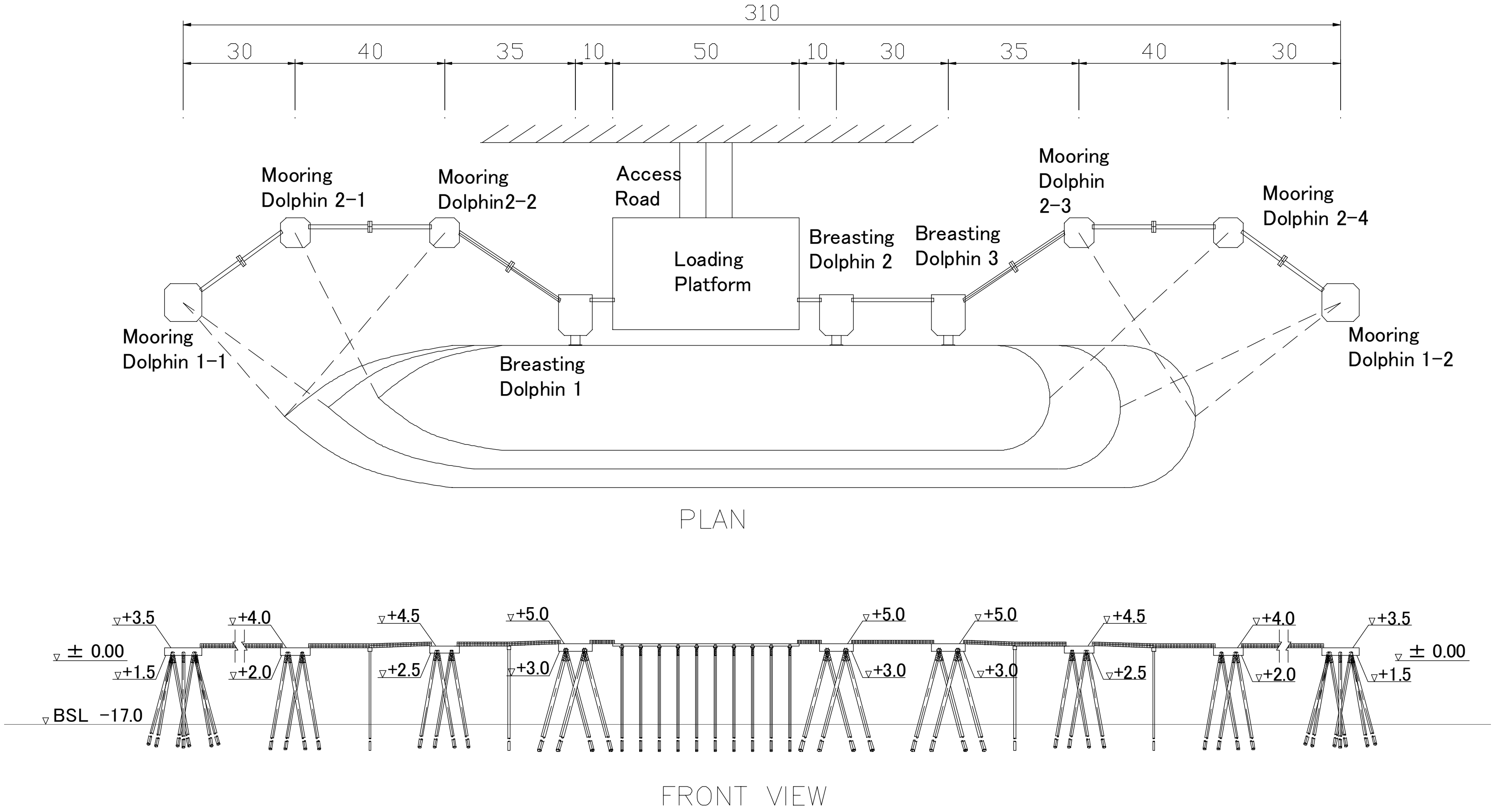


Figure III.3.4-1 Plan and Front View of Berth No. 1

(2) Structures for Berths No. 2 to No. 6

The Berths No. 2 and No. 3 will have a quayside depth of 17.0 m to receive the maximum vessel of 123,000 DWT. The live loads from grain / fertilizer unloaders as well as general cargo cranes have been considered for designing wharf structures.

A relieving platform type, as one of alternatives structures, has been designed to support the above live loads, berthing force of vessels, etc. The front row will require steel piles with a diametre of 1,000 mm, and the remaining rows, both of vertical and batter piles, will need a diameter of 800 mm Figure III.3.4-2 shows a standard cross section of relieving platform type for Berth No. 2.

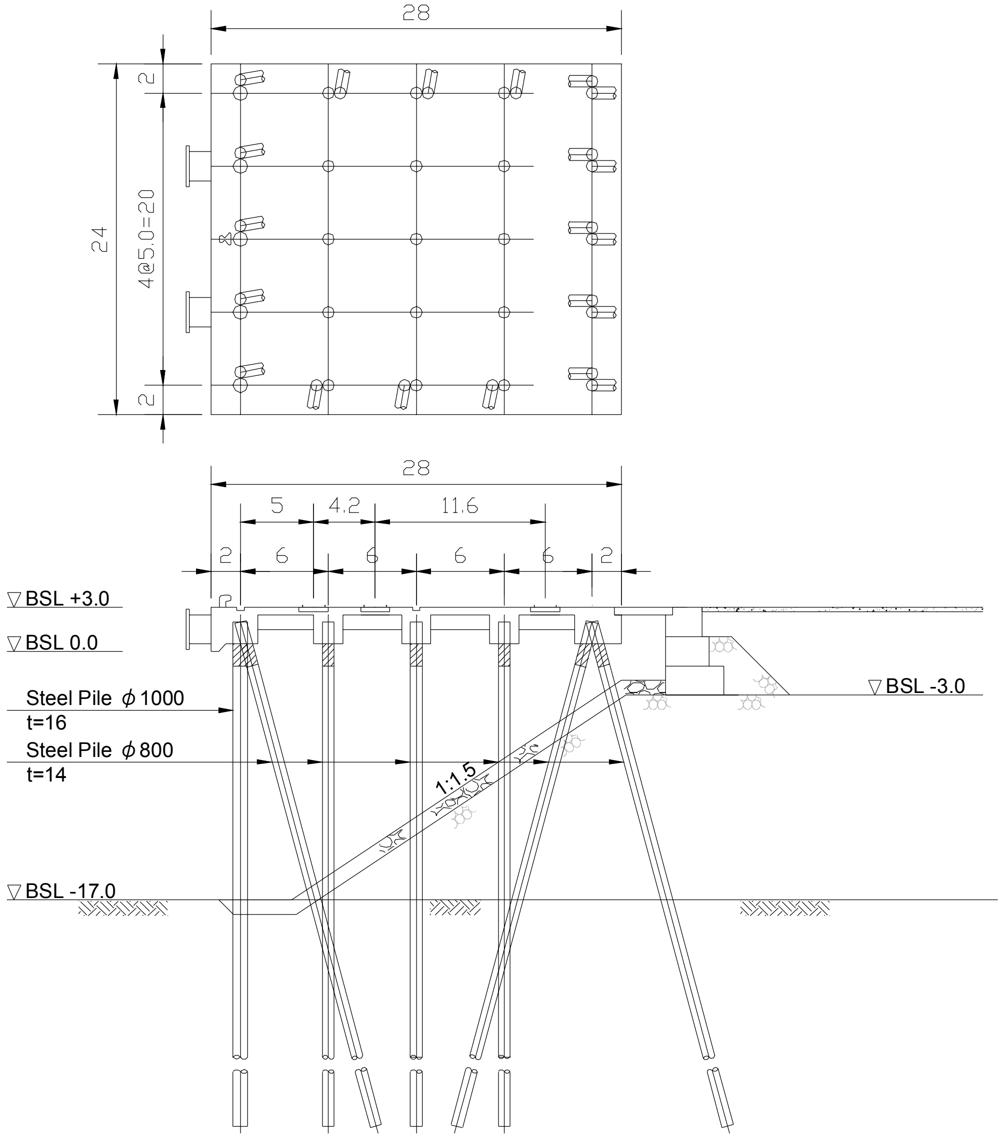


Figure III.3.4-2 Cross Section of Relieving Platform Type Structure (Berth No. 2)

A caisson type wharf structure is mainly composed of rock mound, concrete caisson box and crown concrete. Assuming that four caisson boxes are manufactured on a floating dock of 8,000 tonnes class, the design weight of a caisson boxe has been limitted to less than 2,000 tonnes. In this structure, all the live loads will be supported with caisson boxes. As the result of structural calculation, the size of the caisson boxes has been determined at 14 m (width) x 16 m (length) x 17.5 m (height) and 15.5 m x 16 m x 17.5 m for Berths No. 2 and No. 3 respectively.

Figure III.3.4-3 shows a cross section of Berth No. 2 with caisson type wharf.

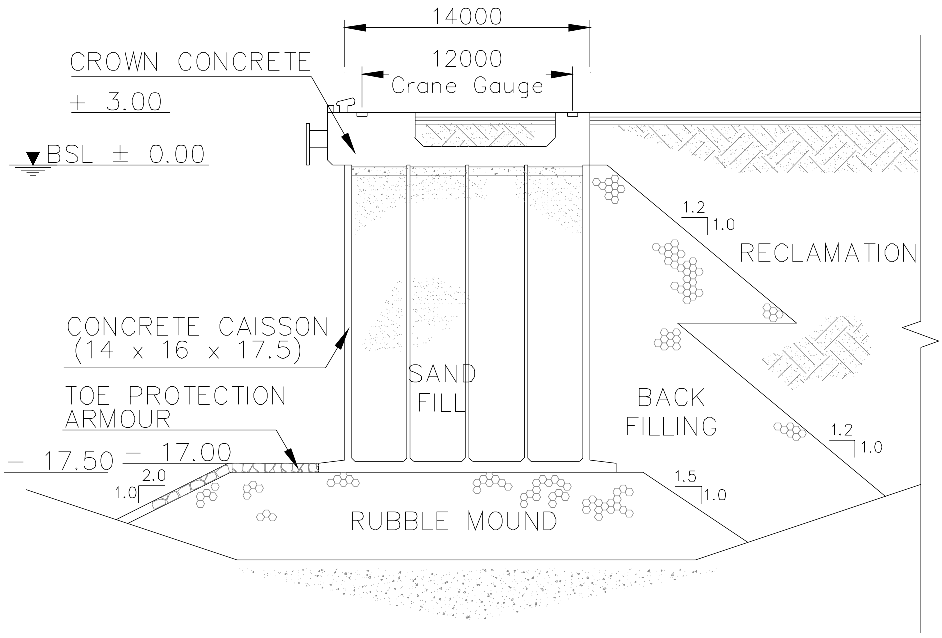


Figure III.3.4-3 Cross Section of Caisson Type Structure (Berth No. 2)

Berths No. 4 to No. 6 with a quayside depth of 15.0 m to accommodate a general cargo vessel of 74,000 DWT and a container vessels of 4,800 TEU. Loaders and unloaders for fertilizer as well as for bagged cargo will be provided at Berth No. 4. At Berth No. 5, general cargo cranes will be provided. hadve been taken into account. Two sets of gantry cranes will be installed for handling containers at Berth No. 6. The loads from these cargo handling equipment have been assumed for structural design

Berth No. 6 has been designed with a relieving platform structure, supported with steel pipe piles of 1,000 in diametre in front row and 800 mm in remaing rows of piles, including a rear crane rail foundation. Figure III.3.4-4 shows a standard cross section of relieving platform type for Berth No. 6.

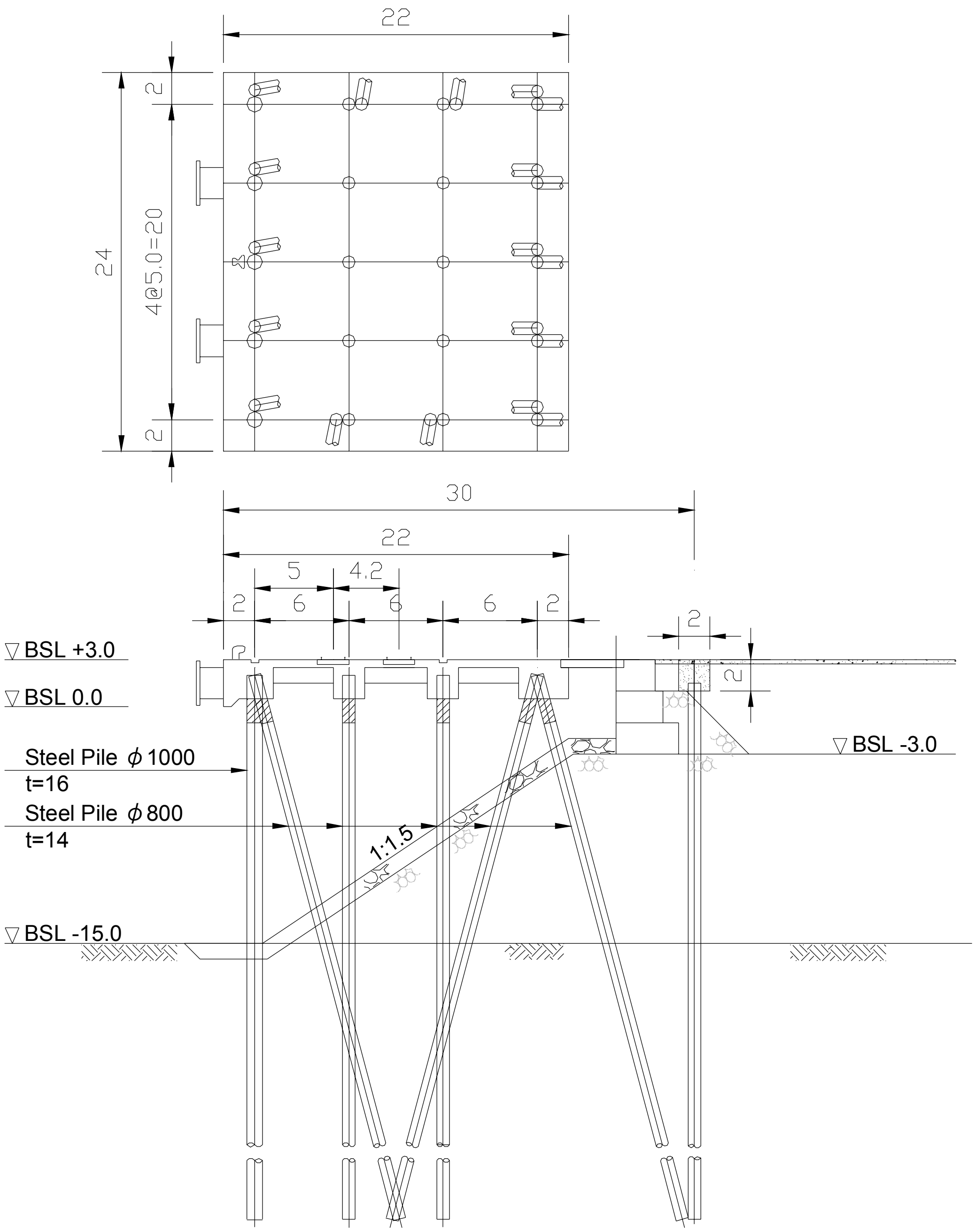


Figure III.3.4-4 Cross Section of Relieving Platform Type Structure (Berth No. 6)

The size of caisson boxes has been determined at 14 m (width) x 16 m (length) x 15.5 m (height) for Berths No. 4 & 5 and 12.5 m x 16 m x 15.5 m for Berths No. 6.

Figure III.3.4.-5 shows a cross section of Berth No. 6 with caisson type wharf.

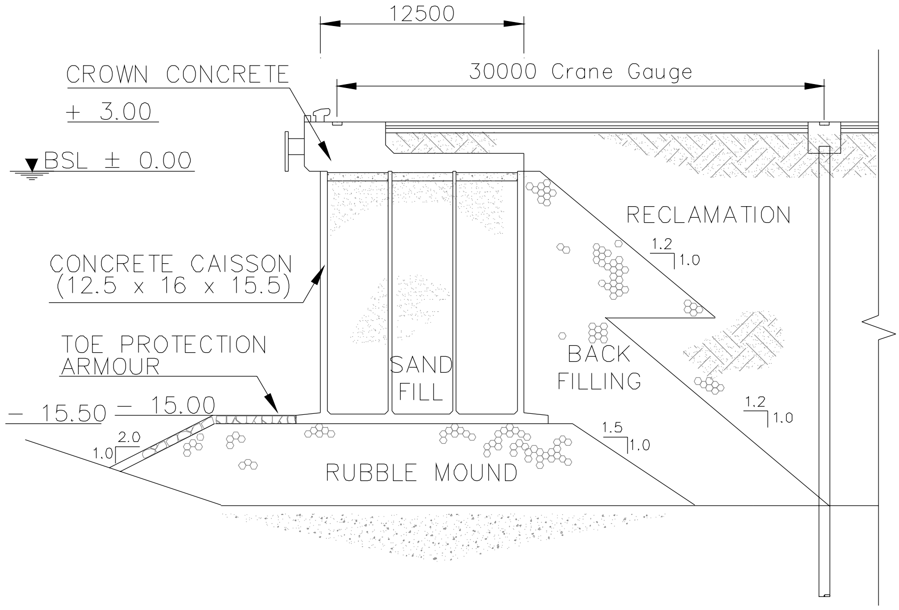


Figure III.3.4-5 Cross Section of Caisson Type Structure (Berth No. 6)

3.5 Design for Railway Facilities

The designing of railway alignments and structures, in principle, has followed the Lithuanian Railway Standard and Regulations. The Technical Provisions of Railway Usage, Techninio Gelezinkeliu Naudojimo NUOSTATAI and Construction and rolling stock clearance diagrams for the USSR railways of 1520 (1524) mm gauge GOST 9238-83 have also been applied for the engineering design of railway structures. The major track geometry is summarized in Table III.3.5-1.

Table III.3.5-1 Track Structure

Item	Description
Gauge	1,520 mm
Minimum Curve Radius	Main Track: 2,000 m (800 m: complicated condition) Siding, Access Track: 200 m Station, Yard: Straight (1,500 m: complicated condition)
Maximum Grade	Main Track: 15/1000 Siding, Access Track: 20/1000 Station, Yard: 1.5/1000
Type of Rail	R65 or UIC60 – 25m
Sleeper	Concrete / Wood Sleeper
Sleeper Space	500mm, 2,000 unit/km on straight track 543mm, 1,840 unit/km on curved section (less than R=350 m)
Ballast	Depth 350 mm (under the sleeper) Depth 200 mm (sand under the ballast)
Superelevation	Maximum: 150 mm ($C=12.5QV^2/R$)
Type of Switch	Main Track: 1/11 Marshalling Yard: 1/9 (1/6 symmetrical turnout: complicated condition)
Distance between Track Centres	Main Track: 4.1 m (more than three tracks: 5.0 m) Station, Yard: 4.8 m

Source : Tchnical Provision of Railway Usage

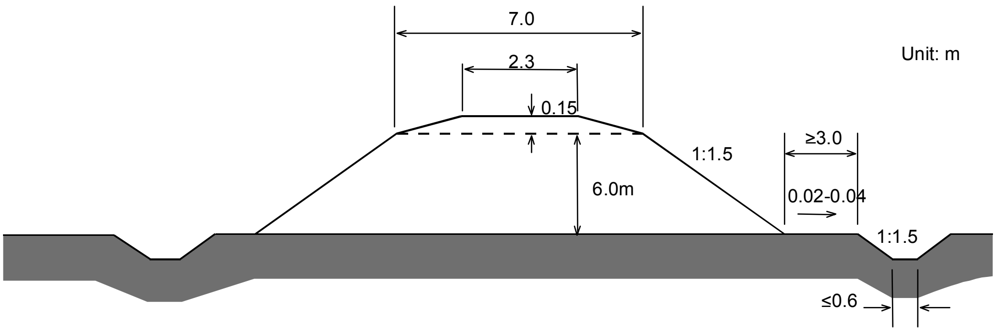
Major performance and specifications of locomotive are shown in Table III.3.5-2.

Table III.3.5-2 Major Performance and Specifications of Locomotive

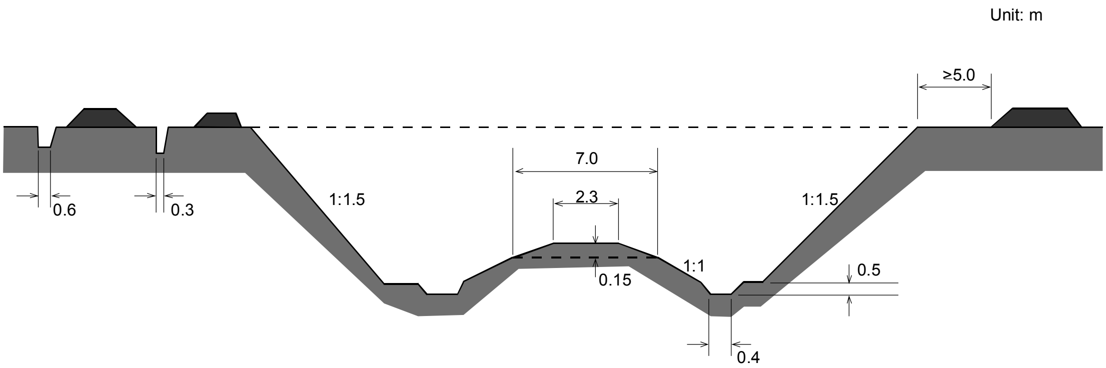
Item \ Type of Locomotive	2M62	M62	CME3 (Shunting)	TEM2 (Shunting)
Axle Arrangement	2 x Co-Co	Co-Co	Co-Co	Co-Co
Engine Power (kW)	2 x 1,470	1,470	994	883
Maximum Speed (km/h)	100	100	95	100
Weight (ton)	240	116.5	123	120
Axle Load (ton)	20	19.4	20.5	20
Electric Transmission	DC/DC	DC/DC	DC/DC	DC/DC
Continuous Tractive Effort (kN)	2 x 19.5	20.0	23.0	21.0
Maximum Height (mm)	4,615	4,615	-	5,115
Maximum Width (mm)	2,950	2,950	2,950	2,950
Length (mm)	17,550 x 2	17,550	-	16,970
Wheel Base (mm)	4,200	4,200	4,200	4,200
Wheel Diameter (mm)	1,050	1,050	1,050	1,050

Source : Lithuanian Railways Figures and Facts

A typical cross section of embankment and cutting portions have been designed as shown in Figure III.3.5-1 and typical cross section of subgrade is shown in Figure III.3.5-2. Construction and rolling stock gauges are shown in Figures III.3.5-3 and III.3.5-4.



Embankment Section



Cutting Section

Figure III.3.5-1 Typical Cross Section of Embankment and Cutting

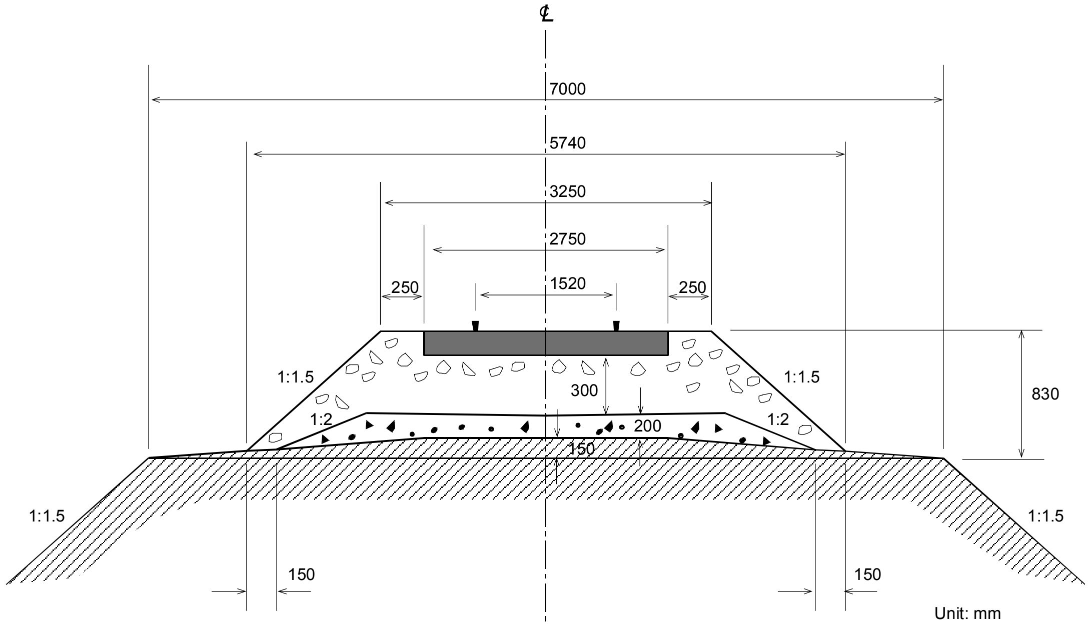
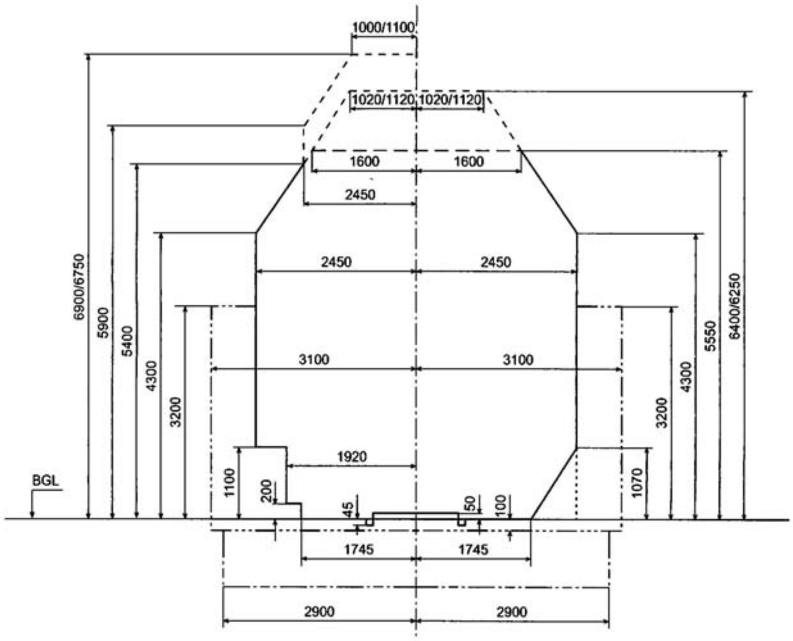


Figure III.3.5-2 Typical Cross Section of Subgrade



Line of the distance to the bridges, tunnels, galleries, platforms, floorings of the crossings, signalling facilities located in their vicinity.

Line of the distance to the facilities and equipment, which is not electrified.

Line of the distance to the buildings, facilities and equipment (except the supports of the bridges, structural elements of the tunnels, galleries, platforms), located at the external side of the outer ways of stages and stations as well as at the tracks located separately at the stations.

Line which should not be exceeded by any kind of equipment within the stages and useful length of the tracks within stations except engineering facilities, floorings of the crossings, signalling facilities and centralization and blocking equipment located in their vicinity.

Line of the distance to the basements of the building and supports, underground wires, cables, pipelines and other facilities.

Line of the distance to the structural elements of the tunnels, railings on the bridges, viaducts, and other engineering facilities.

Figure III.3.5-3 Construction Gauge

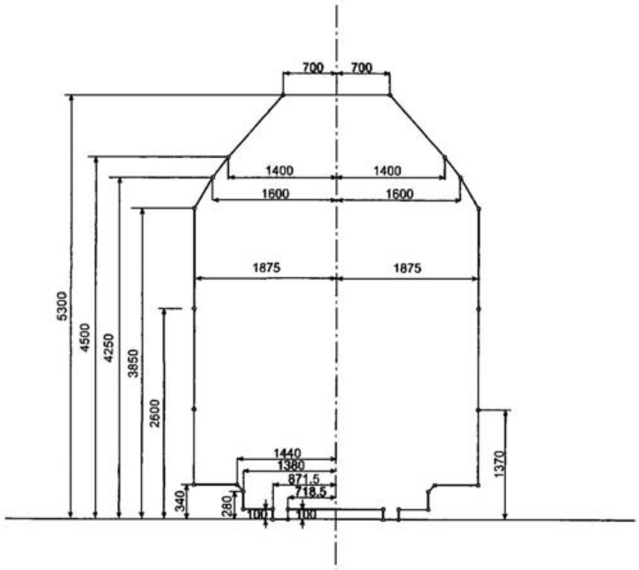


Figure III.3.5-4 Rolling Stock Gauge

3.6 Road Structure

An access road from the public road to the outer port area will be required. The port service roads will need a flyover bridge at the crossing point with the railway yard that will be located on the shoreside of the outer port area. The roads have been designed with four-lanes to accommodate future demand of port related traffic. Typical cross sections at grade and flyover are shown in Figures III.3.6-1 and III.3.6-2.

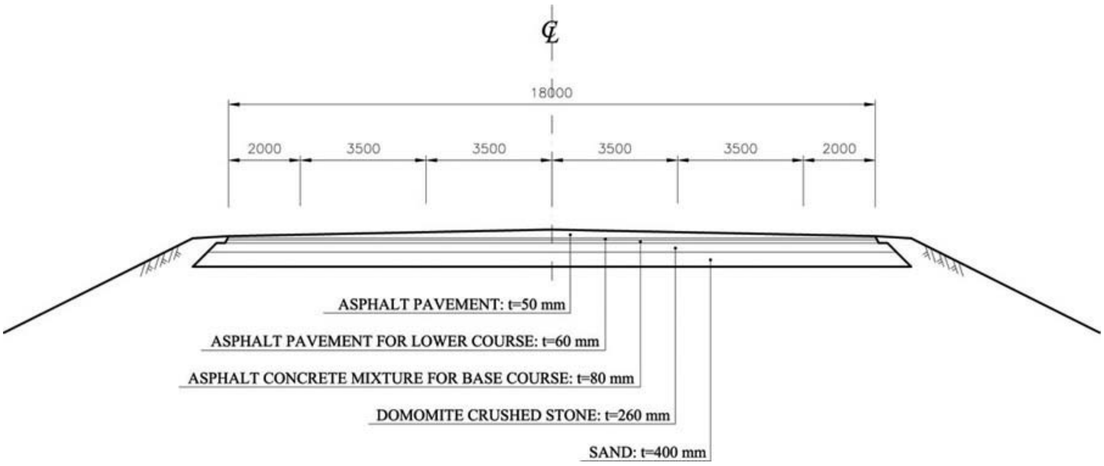


Figure III.3.6-1 Typical Cross Section of Access Road

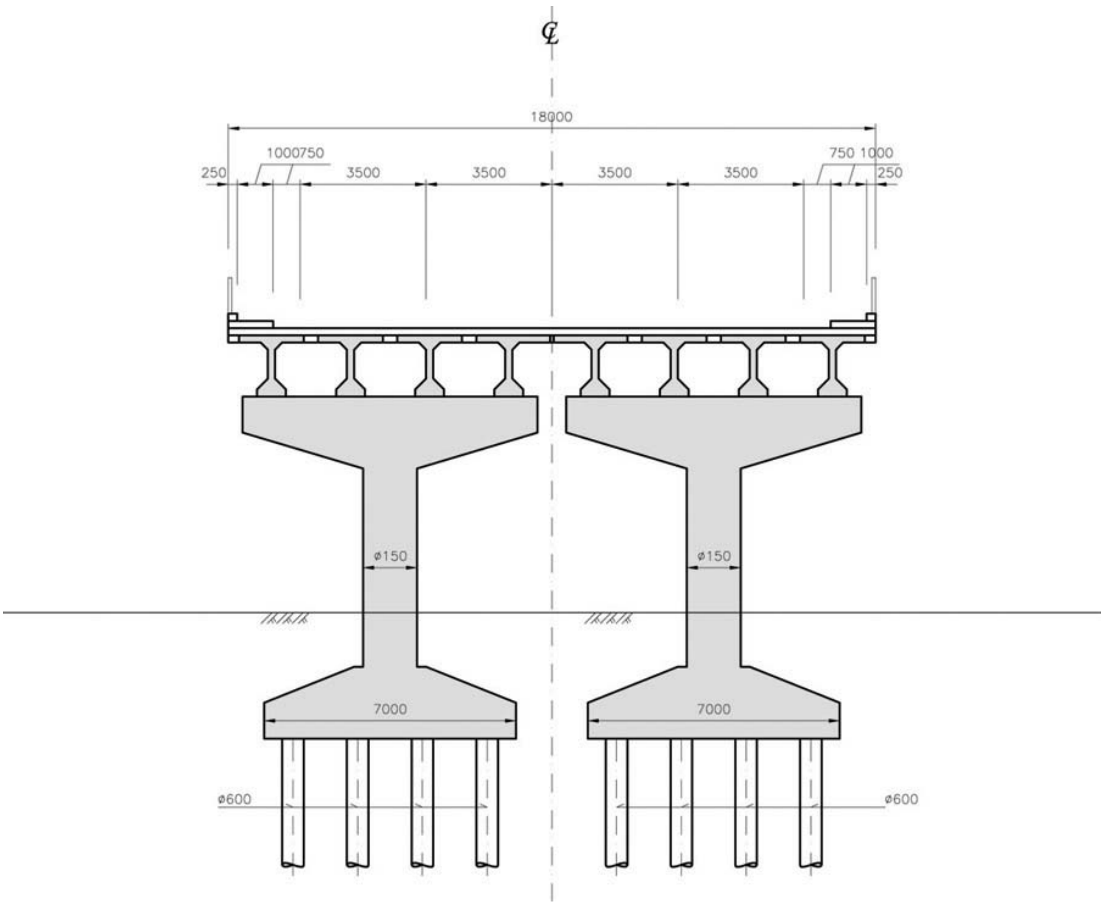


Figure III.3.6-2 Typical Cross Section of Flyover

CHAPTER 4 PROJECT IMPLEMENTATION PROGRAM

CHAPTER 4 PROJECT IMPLEMENTATION PROGRAM

4.1 Law and Regulations for Construction

4.1.1 Laws and Guidelines on Procurement

The basic law applied for the project implementations under the state budget in Lithuania is Law No. I-1491, which is called the "Law on Public Procurement. This law describes the procurement procedures and the basis of contract. In case of the project to be implemented with financial assistance from international organisations and banks, it allows to apply these guidelines.

4.1.2 Construction Permit

Construction Law No. I-1240 stipulates that a construction permits is required for execution of construction, reconstruction and repair works. In order to secure the construction permit, an implementing authority or agency concerned is obliged to submit necessary documents to municipal mayor even if the permit is obtained from the county head administration.

The basic requirements of documents to be submitted include:

- Application of Standard Form;
- Engineering data of facility by fulfilling the approved forms and supplemental information, if required;
- Document confirming property right;
- Decision of institution in charge concerning possibility of foreseen economic activities and environmental impact assessment, if it is obligatory by the Law;
- Document concerning appointment of construction management and control, if it is obligatory;
- Certificate of facility cadastre measurement and legal registration; and
- Others as required.

Upon receipt of application, municipal mayor shall pass the documents to the standing committee for construction to assess if the construction meets the requirements of land use management plan and design conditions. The committee formalise the results of assessment and give recommendation to mayor.

In case the permit is to be issued by county, municipal mayor is obliged to submit report of standing committee. The construction permit is valid for 10 years and permit for demolition is valid for 3 years.

4.1.3 Labour Law

The labour code No. IX-926 regulates protection of labour rights and obligations of both employers and employees. The working time shall not exceed 40 hours per week. The minimum annual leave defined in this code is 28 calendar days. The minimum wage was set at 450 Litas per month as of 1st September 2003.

4.1.4 Taxes

The following tax regulations are closely related to the construction activities:

- VAT is collected from the added value created in the process of production of goods and rendering of services and of the goods imported. The standard rate of VAT for construction sector is 18%. Deduction of VAT is allowed for the amount paid for goods and services.
- Under the Law on Profit Tax, profit and/or income is subjected to the profit tax. The normal profit tax rate is 15%.
- All employees are subject to income tax on income earned. Income tax rate is flat rate of 33% after deducting non-taxable minimum income as established by the Government.

4.1.5 Social Security

The social security system in Lithuania comprised the social insurance system, medical insurance system, and social support system. The most significant components of the state social security system are the social insurance system and medical insurance system. Employers must pay to the State Social Security Fund a mandatory social security contribution for every employee.

The employer's contribution equals to 31% of the gross wage of employees which is composed of 1% for labour accident insurance, 3% for medical insurance, 22.5% for pension insurance, 3% for sickness and maternity or paternity insurance, and 1.5% for unemployment insurance.

Employers must withhold 3% of gross wage as a social insurance as a contribution by employees (2.5% for pension insurance and 0.5% for sickness and maternity or paternity insurance) payable to the State Social Security Fund.

4.2 Workable Days

Workable days have been estimated for offshore and onshore works respectively. Non-workable days due to the following reasons have also been counted:

- Public holidays and rest days,

- Rough wave conditions, and

- Heavy rain.

(1) National Public Holidays and Working Calendar Days

In Lithuania, there are 12 public holidays, of which 2 days are fixed on Sundays. Holidays fall on Saturdays and Sundays are not compensated in week days. Therefore, annual number of weekdays is computed aa follows:

365day x 5days/7days x (365-10)/365 = 254 days

Introduction of labour work formation and work shift varies in construction industry depending on the type of work. Night work is restricted for those works that cause large noise and vibration exceeding permissible levels.

(2) Non-workable Days due to Rough Wave Conditions

Marine construction works to be performed in an area exposed to offshore waves shall be suspended in high wave condition. The limits in wave heights have been assumed for each type of marine works as shown in Table III.3.4-1. The table shows also the frequency of wave occurrence within the assumed limits of wave heights estimated based on the offshore wave data (see Table III.1.2-1).

Table III.4.2-1 Limit of Workable Wave Height and Occurrence

Type of Work	Limit of Workable Wave Height	Occurrence of Respective Wave Height
Rock Work	1.2 m	72 %
Trimming Work of Rock Surface	1.0 m	64 %
Placing of Concrete Blocks	0.8 m	67 %

(3) Non-workable Days due to Heavy Rain

Also some type of construction works such as surface finishing of pavement, concrete casting and earth work can not be executed during or after heavy rain. Based on the precipitation records in Klaipeda for 2001 to 2003, the number of average days with daily precipitation more than 20 mm, 15-20 mm, and 10-15 mm are computed as tabulated below.

Table III.4.2-2 Number of Days for Precipitation Range

Range of Precipitation	Average Number of Days	Occurrence Percentage
Precipitation 10 – 14.9 mm/day	9.7 days	2.7%
Precipitation 15 – 19.9 mm/day	4.7 days	1.3%
Precipitation more than 20 mm/day	3.0 days	0.8%

Source : Precipitation Data from Meteorology Service Klaipedas

It has been assumed that on-shore works such as earth work and pavement work would be suspended in excess of 10 mm/day precipitation. This criteria give occurrence of 4.8 %.

(4) Workable Days by Type of Work Item

Taking into account the above work restrictions, workable days for various kinds of offshore and onshore works by type of activities have been calculated below.

Table III.4.2-3 Working Calendar Days by Types of Work

Type of Work	Assumed Limitation of Work	Workable Days
Dredging Work	24 hours operation by 3 shifts without suspending on Saturday, Sunday, and holidays except for repair and special holidays.	350 days
Rock Work	Wave less than 1.2 m	(365 x 6/7) x 355/365 x 72 % = 219 days
Trimming of Rock Surface	Wave less than 1.0 m	(365 x 6/7) x 355/365 x 67 % = 204 days
Placing of Concrete Blocks	Wave less than 0.8 m	(365 x 6/7) x 355/365 x 64 % = 195 days
Onland Structural Work	Except for Saturdays, Sundays, and Holidays	(365 x 5/7) x 355/365 = 254 days
Pavement and Earth Work	Precipitation less than 10 mm	(365 x 5/7) x 355/365 x 95.2 % = 254 days

4.3 Construction Materials and Equipment

4.3.1 Construction Materials

Among the major construction materials, the following are produced in Lithuania:

- Ordinal portland cement;
- Timber, wood and plywood;
- Sand; and
- Precast concrete blocks.

Other construction materials including steel products, PVC, etc. are not manufactured or produced locally, but imported. Present market situation of common construction materials are described below.

1) Cement

UAB CEMEKA is a sole local company producing ordinary portland cement. The factory is located in Naujoji Akmene, Siauliai, which is about 180 km north-east of Klaipeda. The cement is delivered by train wagons or tank trucks for bulk and cargo track for bagged cement. For the concrete mixing companies, spur lines of train lead to inside the factory sites, allowing direct shipment of bulk cement.

The factory produces cement types of CEM I/42.5N, CEM I/42.5R, CEM I/52.5N, and CEM II/A-L 42.5N in accordance with Lithuanian Standard.

Special marine type cement is not produced in the company, so it is necessary to import from European countries.

2) Sand and Gravel

Geography of Lithuania is characterized by flat terrain. High mountains do not exist (the highest is 294 m at Juozapine). Accordingly, massive rock materials can not be found. Only small cobble, boulder, and gravel stone are available at certain borrow areas where sand and fill materials are excavated.

Figure III.4.3-1 shows the location of available sand/gravel borrow areas in the vicinity of Klaipeda City. The sources of sand and gravel are scattered mostly at the central part of the Klaipeda District. The deposit in each location is small in volume. Sand and gravel is excavated by open cut method using clamshells and draglines. The information on quarry sites in volume, including thickness of deposit and overburden, and percentage of gravel contents are summarized in Table III.4.3-1.

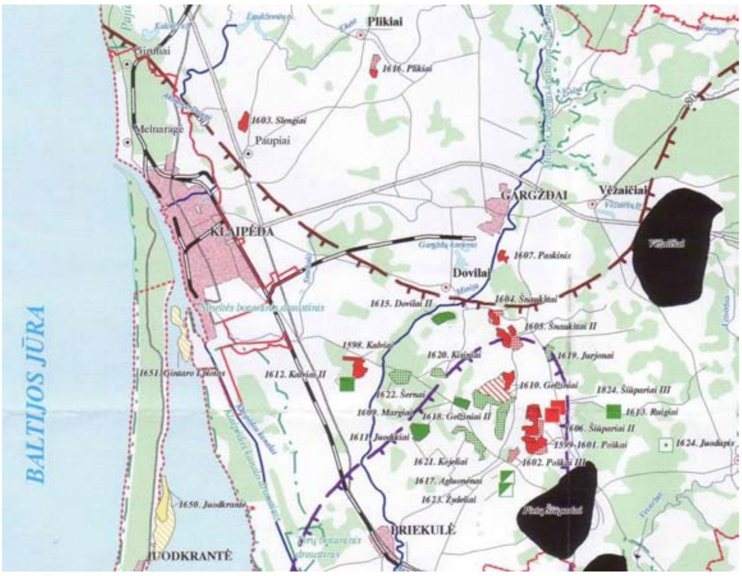


Figure III.4.3-1 Location Map of Borrow Area

Table III.4.3-1 Present and Potential Sources of Sand and Gravel

Location		Name of Operator	Total Deposit Volume (m^3)	Ave. Thick-ness of Deposit (m)	Ave. Thick-ness of Overburden (m)	Percentage of Gravel Contents (%)
No.	Name of Place					
(1) Gravelly Sand						
1598	Kalviai	AB Hidrostatyba	3,385	4.9	0.7	35.0
1599	Poskai IV		2,713	3.3/4.8	0.6/1.4	18.2
1601	Poskai I		2,877	2.9/3.1	1.0/1.2	38.2
1605	Snaukstai II	AB Hidrostatyba	4,057	4.6/4.0	0.9/1.4	44.0
1607	Paskinis		1,126	5.0	0.5	10.5
1610	Gelzinai	AB Hidrostatyba	6,908	2.5/3.2	0.8	23.3
(2) Sand						
1611	Juodikiai		3,373	4.9	0.8	0.3
1615	Dovilai	AB Silmega	1,046	2.3	0.4	-
1617	Agluonenai		1,270	4.7	0.5	4.3
1618	Gelziniai II		5,300	3.2	0.4	7.6
1621	Kojeliai		2,350	2.9	0.6	1.6
1622	Sernai		4,740	7.9	0.7	4.3

Source : Lithuanian Geology Institute
Note : Listed sources are more than 1,000m^3 of deposit

Several crushing plants are in operation among the quarries listed above, where boulders and cobble are screened out from excavated materials. Generally, they are small in scale due to the limited quantity of boulders contained in each deposit.

Large size stones such as armour stone, cobble for foundation, rubble larger than the diametre over 10cm, etc. are generally imported from the Scandinavian countries or Belarus.

3) Wooden Products

In Klaipeda, there are many wood-processing factories, where timber and plywood are produced. Since the resources of logs are also sufficient in the country, wooden construction materials such as wooden pile, timber, wooden building materials, plywood for formwork, etc. can be procured without difficulty.

4.3.2 Construction Equipment

Construction equipment for common onshore works such as bulldozers, track-cranes, pavement equipment, dump-tracks, etc. are generally owned by local construction companies.

There are several leasing companies for onshore construction equipment, though, in case of large-scale construction it is needed to procure equipment from outside the country or import major equipment, because local equipment is limited in number and sometimes short in capacity.

Floating construction equipment are not available on a lease basis except for working boats which are moored in the port area such as tug boats, floating dock, crane barge, flat barge, etc.

There are six major lease companies for construction equipment in Klaipeda as listed in Table III.4.3-2.

Table III.4.3-2 Major Leasing Company in Klaipeda

Name of Major Leasing Company
(1) ALTIMA NEW MARKETS
(2) HKL BAUMASCINEN UAB
(3) INKOMSTA
(4) Klaipedos Kranai
(5) Klaistvita UAB
(6) Klaistvita UAB

4.3.3 Concrete Mixing Plants

There are two concrete mixing plants in the cities one in PERDANGA and another in Musu Statyba. The production capacities of these concrete mixers are 120m^3/hour and 90m^3/hour respectively. An automated computer system is used for measuring concrete components.

Both plants also produce pre-cast concrete products of drainage facilities, road pavement, units of building structure, etc. They have covered working yards equipped with ceiling cranes, considerably large open storage yard to produce and keep high quality concrete products.

They have sufficient laboratory testing equipment and quality control system certified by the Certificate Centre attached to the Ministry of Environment. Since agitator tracks are rather limited in number, sealed dump tracks are commonly used for transportation of concrete.

4.4 Construction Plan

4.4.1 Major Work

Scope of construction work for Short-term Development and Master Plan are summarized below:

		Short-term Development Plan (Year 2015)	Master Plan (Year 2025)
1.	Offshore Facilities		
1.1	Breakwaters		
	West Breakwater	1,020 m	1,800 m
	South Breakwater	1,380 m	1,380 m
	North Breakwater	500 m	500 m
1.2	Dredging and Reclamation		
	Dredging Sand and Reclamation	6,730,000 m^3	8,740,000 m^3
	Dredging Hard Clay and Dumping	180,000 m^3	180,000 m^3
	Reclamation Fill	300,000 m^3	1,200,000 m^3
1.3	Quay Facilities		
	Berth No. 1 (Dolphin Type)	310 m (-17.0 m)	310 m (-17.0 m)
	Berth No. 2 (Caisson Type)	310 m (-17.0 m)	310 m (-17.0 m)
	Berth No. 3 (Caisson Type)	310 m (-17.0 m)	310 m (-17.0 m)
	Berth No. 4 (Caisson Type)	-	260 m (-15.0 m)
	Berth No. 5 (Caisson Type)	-	260 m (-15.0 m)
	Berth No. 6 (Caisson Type)	-	330 m (-15.0 m)
	Transition Part	50 m (-9 to -17 m)	50 m (-10 to -15 m)
1.4	Revetments		
	North Revetment	700 m	700 m
	South Revetment	460 m	460 m
	South-East Revetment	300 m	300 m
	East Revetment	1,000 m	1,850 m
1.5	Basin for Port Service Boats		
	Wharf	400 m (-6.0 m)	400 m (-6.0 m)
1.6	Removal of Existing North Breakwater	220 m	220 m
2.	Onshore Facilities		
2.1	Road and Pavement		
	Concrete Pavement for Apron	18,600 m^2	44,100 m^2
	Asphalt Pavement for Port Service Roads	47,000 m^2	57,000 m^2
	Container Yard Pavement	-	132,000 m^2
	Flyover Bridge with Approach	1 unit	1 unit

2.2	Railway		
	Port Area	8.2 km	13.3 km
	Access Railway (Pauoscio Yard-Port)	2.5 km	2.5 km
	Improvement of Pauoscio Yard	6.5 km	9.3 km
2.3	Drainage and Water Supply	1 lot	1 lot
2.4	Electrical Work	1 lot	1 lot
3.	Cargo Handling Facilities		
3.1	Shore Crane	3 units	10 units
3.2	Belt Conveyor System	400 m	1,300 m
3.3	Belt Conveyor System	400 m	1,300 m
3.4	Yard Equipment	1 lot	1 lot
3.5	Rail Transfer Crane	-	4 units
3.6	Miscellaneous Buildings	1 lot	1 lot

4.4.2 Environmental Constraints

(1) Fish Migration

In the construction works for the existing port facilities, suspension of certain works was considered during the period of fish migration. In the Klaipeda Port Entrance Rehabilitation Project, the following fish migration and intensive migration periods were set out.

1) Spring Season

Migration Period : 1st April to 31st May

Intensive Period : 15th April to 15th May

2) Summer Season

Migration Period : 15th August to 15th October

Intensive Period : 15th September to 15th October

In the above project, dredging work was suspended during intensive fish migration period and fish migration survey was performed during the fish migration period. Meanwhile, the condition given by MOE for the project of port deepening work to -14.0 m at the inner port area was eased. As the result, the contractor could perform the dredging work by opening one side of channel so as not to interrupt the course of fish migration.

Therefore, the sequence of dredging and rock work at the outer port area shall be well planned to keep a certain stretch of fish migration course.

(2) Fishing Permission

In the outer port area, total of six companies hold the permissions for commercial fishing from shoreline to the offshore area in the water depth of -20 m according to the Marine Environment Protection Agency. In Lithuania, there is no fixed value about compensation against fishing right. Thus, it is impossible to evaluate the value of fishing permission at this moment.

(3) Noise

Noise is one of the major environmental concerns in construction activities, since the proposed outer port area is close to the residential area of Melnrage. The construction work should be executed under the environmental control in accordance with requirements of EIA.

4.4.3 Construction Method

(1) Temporary Work

A large construction yard will be required to accommodate a contractor's site office, storage, concrete plants, concrete block manufacturing and storage yards, rock storage areas, temporary jetties, etc. One possible zone would be near the NAFTA Oil Terminal. The present unused and reserved port area is not large enough for manufacturing and storage concrete blocks. It is proposed to acquire the area between present port boundary and proposed access railway line so that the area can be used for the construction activity and future usage for various port related activities.

(2) Dredging and Reclamation

The dredging volume totals approximately 6.7 million cubic metres for the Short-term Development Plan and 8.7 million cubic metres for the Master Plan. Major part of dredging volume is sandy soil with N-Value of 20-40, contained with small percentage of hard silty clay. It was assumed that the contents of silty clay is 2 to 3 % based on the available boring data, but it is required to perform extensive number of boring in the basin and channel area to confirm the volume in well ahead of detailed design.

Considering the characteristics of soil to be dredged, cutter suction pump dredger would be suitable, while grab dredger with large bucket of 23 cubic meters or more is recommended for silty clay soil. Grab dredger is required not only for dredging at the basin, but also for dredging foundation area for quay structures.

The dredging work will commence after major parts of breakwaters are completed and calmness of dredging area is substantially secured.

When one cutter suction dredger of 8,000 ps is used, estimated production rate of dredging would be 14,000 m^3/day or 400,000 m^3/month. With this production rate, the period required for dredging work is estimated about 17 months for the Short-term Development Plan and 22 months for the Master Plan.

The sandy soil is suitable for reclamation, the dredged materials by cutter suction dredger is pumped to the reclamation area through discharge pipe line. Since maximum length of discharge pipe is about 1,500 m, dredged material can be dumped directly without additional booster pump.

Discharge water shall be controlled at the discharge weir to reduce as much as possible of turbidity water that flows to outside the reclamation area.

Between the sandy and silty clay layers, a thin sandy layer with gravel and cobble is likely to exist, for which cutter suction dredger can not be applied since large cobble may reach to the size of 2 metres. By a grab dredger with 23 m^3 class bucket, estimated production volume is 2,500 m^3/day or 60,000 m^3/month. It means that the period required for dredging silty clay layer is about 3 months.

Silty clay soil has contents of silt and clay portion more than 75 %, thus it is not suitable as reclamation fill. The dredged soil shall be transported by sand barges and dumped at about 20 km offshore of Klaipeda port where the water depth of more than 40 m.

Composition of major equipment of dredging fleet are:

1) Cutter Suction Dredging Fleet

Cutter Suction Pump Dredger 8,000 ps	: 1 no
Anchor Boat 35 tons	: 1 no

2) Grab Dredging Fleet

Grab Dredger 23 m^3	: 1 no
Sand Barge 650 m^3	: 2 nos.
Tug Boat 1,000 ps	: 1 no

(3) Breakwater Work

The West Breakwater shall be constructed from both ends to minimise the construction period, since dredging and quay-wall construction should start after calmness of water basin at the construction site has been secured.

Prior to the commencement of rock work of mound, considerable number of artificial concrete blocks shall be manufactured in stock yard so that the surface of rock mound can be covered without damaging the uncovered surface by wave attack.

Artificial concrete blocks will be manufactured and stored at the temporary construction yard near the NAFTA area. The yard shall have sufficient storage area.

Quarry run rock with a grade of 0.1 to 300 kg will be transported by bottom open barges and discharge rock directly at the bottom sea when the barges are set at the position.

The secondary cover layer of 4 to 7 tonnes rocks will be placed by crane barge on the core portion made of quarry run. The surface of the secondary cover layer will be trimmed to allow artificial armour concrete blocks in place.

For the section of the South Breakwater placed in a water depth more than -15.0 m, the caisson type structure has been recommended. A concrete caisson boxes of 12.0 m x 12.0 m x 12.0 m will be manufactured on a floating dock. Caisson boxes will be placed at the top of rock mound formed by quarry-run and trimmed. After placing at the designated location, empty chambers of caisson boxes will be filled with sand.

(4) Quay-wall

Except for Berth No. 1 of oil terminal, caisson type has been selected for quay-wall structure. Berth No. 1 has been designed with a dolphin type for which piling and concreting work are to be executed. Total of 148 numbers of piles are drilled to hard clay layer. At this location, sandy layer with gravel and cobble is expected to situate above -17 m in depth, most of this layer will be removed by dredging of basin. In case of hitting large cobble during drilling, reverse circulation drill shall be applied inside the pile to break and remove it.

Deck concrete is cast by bucket carried by crane barge into the forms with re-bar set at position.

Same as the south breakwater, four concrete caisson boxes are manufactured on a floating dock of 8,000 tonnes class moored along an available space of existing port area. One cycle of caisson manufacturing is estimated at 44 days. Total number of caisson boxes had been estimated at 42 boxes for the Short-term Development Plan while 95 boxes for the Master Plan. Therefore, manufacturing of caisson for quay-wall takes 18.5 months and 42 months by one floating dock.

4.5 Construction Cost

4.5.1 Basis of Cost Estimate

Unit rates of construction materials, labour, and equipment have been collected through market research and interview survey. Such unit rates are presented in Appendix G.

Based on the unit rates and collected contract price of recent construction works in the port, assessment of construction cost of the outer port for the Master Plan has been carried out. The construction cost has been estimated with following conditions:

- Construction costs are composed of direct and indirect costs, 6% of engineering cost, 18 % of value added tax, and 10% of contingencies on the top;
- Foreign exchange rates were assumed as of end of January 2004 at,

  1 Euro = 3.44 Litas = 130 Japanese Yen = 1.24 US$
- Estimated costs were expected expense of KSSA, concessionaires, and state Government.

4.5.2 Project Costs

The total project costs of the Short-term Development Plan and Master Plan have been estimated at 355 million Euros and 638 million Euros respectively.

Table III.4.5-1 Estimated Project Cost

	Short-term Plan	Master Plan
Outer Port	350 million Euros	633 million Euros
Southern Access railway Improvement	5 million Euros	5 million Euros
Total	355 million Euros	638 million Euros

The itemized project costs are tabulated in Tables III.4.5-2 and III.4.5-3.

Table III.4.5-2 Estimated Project Cost of Short-term Development Plan

(Unit : EURO)

Description	Unit	Quantity	Unit Rate	Amount
Mobilization Cost of Floating and Heavy Equipment	sum			**500,000**
West Breakwater - Rock Mound	sum			**38,117,000**
South Breakwater - Caisson or Rock Mound	sum			**44,994,000**
North Breakwater - Rock Mound	sum			**13,676,000**
Dredging and Reclamation	sum			**14,659,000**
Quay Facilities				
Berth No. 1 (-17m) - Petroleum	L.S	1	5,000,000	5,000,000
Berth No. 2 (-17m) - Grain Bulk	m	310	48,300	14,973,000
Berth No. 3 (-17m) - Fertilizer	m	310	50,500	15,655,000
Transition Part	m	50.0	48,400	2,420,000
			Sub Total	**38,048,000**
Navigation Aid	sum			**2,867,000**
Revetments	sum			**27,543,000**
Basin for Port Service Boats	sum			**2,916,000**
Removal of Existing North Breakwater	sum			**4,618,000**
Road and Pavement	sum			**11,975,000**
Drainage & Water Supply	sum			**3,000,000**
Electrical Work	sum			**3,000,000**
Railway				
Port Area Railway	L.S			5,830,000
Access Railway from Pauoscio Yard to Port	L.S			1,700,000
Pauoscio Yard Improvement	L.S			4,320,000
	L.S			**11,850,000**
Cargo Handling System and Storage				
Loader for Berth No. 2 (1,500t/hr)	unit	1	3,000,000	3,000,000
Jib Crane for Berth No. 3 (40 ton)	unit	2	1,100,000	2,200,000
Belt Conveyor	m	400	3,000	1,200,000
Silo for Grain (Berth No. 2)	L.S.	1	9,000,000	9,000,000
UAN Solution Tank (Berth No. 3)	tank	5	1,500,000	7,500,000
Warehouse for Fertilizer	M2	10,800	1,000	10,800,000
Miscellaneous Buildings for Concessionnaires	L.S.	1		2,000,000
Other Cargo Handling Equipment	L.S.	1		885,000
			Sub Total	**36,585,000**
Total for Construction Cost				**254,348,000**
Engineering Cost (6%)				**15,261,000**
Total excluding VAT				**269,609,000**
VAT (18%)				**48,529,620**
Total including VAT				**318,138,620**
Contingencies (10%)				**31,813,862**
Grand Total				**349,952,000**

Table III.4.5-3 Estimated Project Cost of Master Plan

(Unit : EURO)

Description	Unit	Quantity	Unit Rate	Amount
Mobilization Cost of Floating and Heavy Equipment	sum			**700,000**
West Breakwater - Rock Mound	sum			**63,265,000**
South Breakwater - Caisson or Rock Mound	sum			**44,994,000**
North Breakwater - Rock Mound	sum			**16,835,000**
Dredging and Reclamation	sum			**21,178,000**
Quay Facilities				
Berth No. 1 (-17m) - Petroleum	L.S	1		5,000,000
Berth No. 2 (-17m) - Grain Bulk	m	310	48,300	14,973,000
Berth No. 3 (-17m) - Fertilizer	m	310	50,500	15,655,000
Berth No. 4 (-15m) - Bulk	m	260	44,100	11,466,000
Berth No. 5 (-15m) - General Cargo	m	260	44,100	11,466,000
Berth No. 6 (-15m) - Container	m	330	43,600	14,388,000
Transition Part	m	50.0	38,600	1,930,000
			Sub Total	**74,878,000**
Navigation Aid	sum			**3,395,000**
Revetments	sum			**35,053,000**
Basin for Port Service Boats	sum			**2,916,000**
Removal of Existing North Breakwater	sum			**4,618,000**
Road and Pavement	sum			**15,976,000**
Drainage & Water Supply	sum			**5,000,000**
Electrical Work	sum			**5,000,000**
Railway				
Port Area Railway				12,130,000
Access Railway from Pauoscio Yard to Port				1,700,000
Pauoscio Yard Improvement				10,450,000
			Sub Total	**24,280,000**
Cargo Handling System and Storage				
Loader for Berth No. 2 (1,500t/hr)	unit	1	3,000,000	3,000,000
Jib Crane for Berth No. 3 (40 ton)	unit	3	1,100,000	3,300,000
Loader/Unloader for Berth No. 4 (2,500/1,000 t/hor)	unit	2	3,000,000	6,000,000
Jib Crane for Berth No. 5	unit	2	1,100,000	2,200,000
Gantry Crane for Berth No. 6	unit	2	5,700,000	11,400,000
Belt Conveyor	m	1,300	3,000	3,900,000
Silo for Brain (Berth No. 2)	L.S.	1		12,000,000
UAN Solution	tank	7	1,500,000	10,500,000
Warehouse for Fertilizer (Berth No. 3)	m2	21,600	1,300	28,080,000
Warehouse for Fertilizer (Berth Nos. 4 & 5)	m2	29,700	1,400	41,580,000
RTG for Container Yard	unit	5	1,200,000	6,000,000
Rail Transfer Crane	unit	4	2,000,000	8,000,000
Miscellaneous Buildings of Concessionnaires	L.S.	1		5,000,000
Other Cargo Handling Equipment	L.S.	1		885,000
			Sub Total	**141,845,000**
Total for Construction Cost				**459,933,000**
Engineering Cost (6%)				**27,595,980**
Total excluding VAT				**487,528,980**
VAT (18%)				**87,755,216**
Total including VAT				**575,284,196**
Contingencies (10%)				**57,528,420**
Grand Total				**632,813,000**

4.6 Implementation Program of Key Projects

4.6.1 Implementation Schedule of Short-term Development

As explained earlier in this study report, the Short-term Development Plan comprises the KSSA's already implemented or planned projects as well as the Key Projects that have been idenified through the JICA Study. The former ones iuclude the re-construction of Berths Nos. 82-89, the channel dredging, re-arrangement of storage areas, etc. The latter ones are the Outer Port Development and the Southern Access Railway Improvement. All the projects of the Short-term Development should be implemented in a timely manner in order to run the Port efficiently without causing port traffic congestion. The overall implementation program of the Short-term Development is shown in Figure 12.1, where various kinds of pre-constuction works are also proposed, inclusive of EIA, financial arrangements and selection of operators.

4.6.2 Implementation Schedule of the Key Projects

The Outer Port Development Project should be completed by 2014, and the Southern Access Railway Improvement Project by 2011. The durations of the both projects, including necessrary period for engineering design, selection of the contractors, and construction period have been estimated at 5.5 years and 2 years as shown in Figure 12.2 and 12.3.

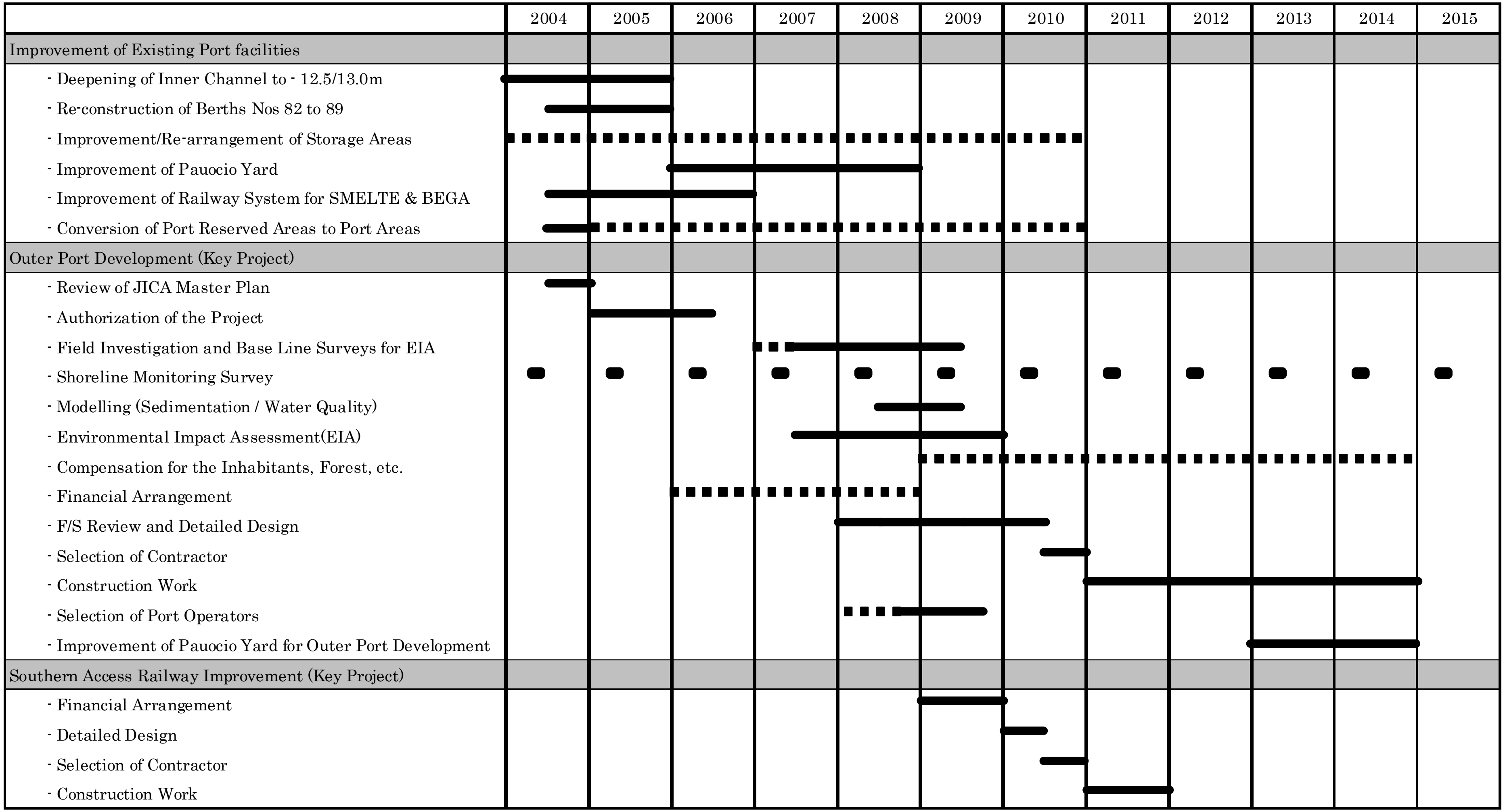


Figure III.4.6-1 Implementation Schedule for Short-term Development

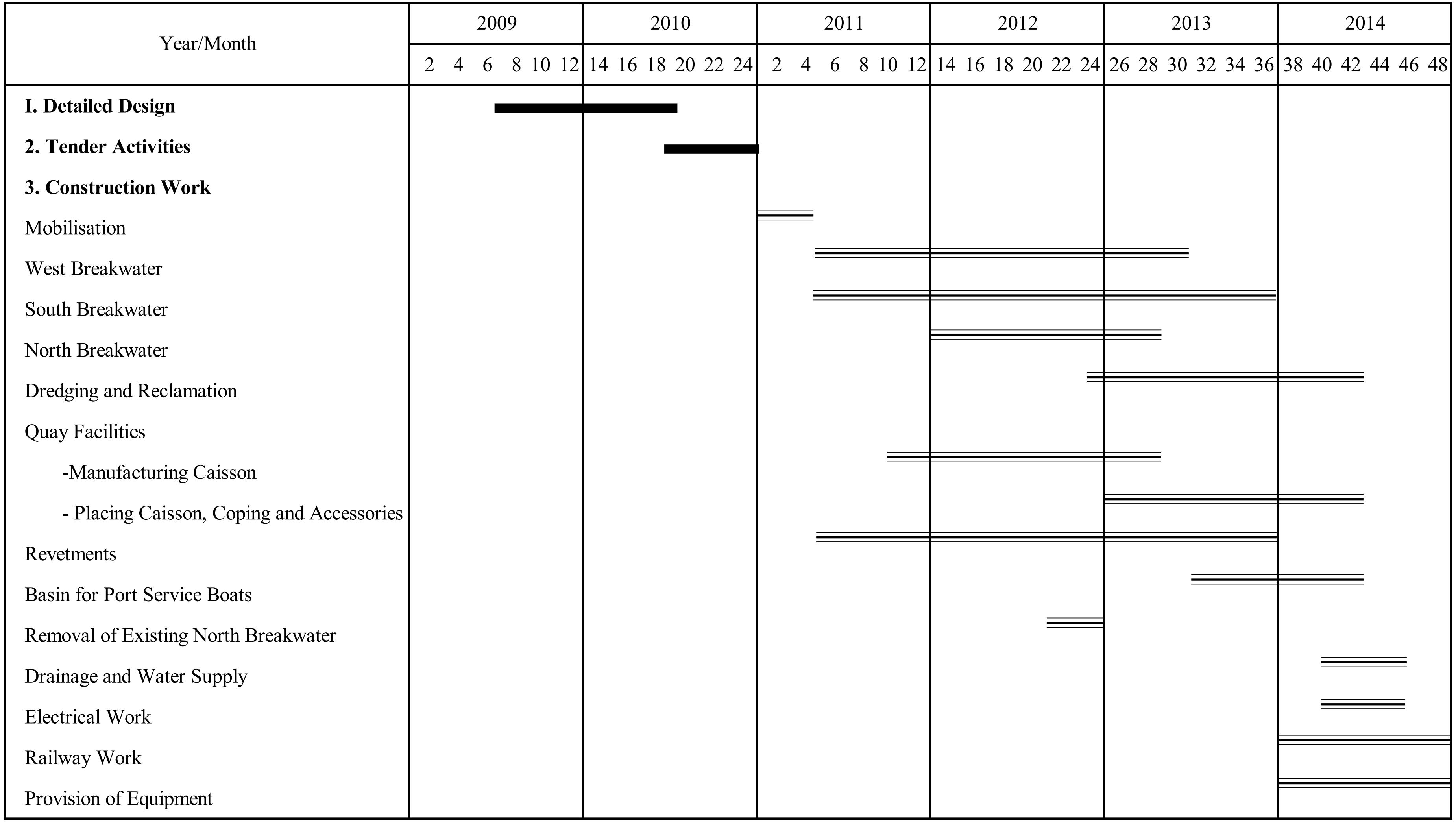


Figure III.4.6-2 Implementation Schedule for Outer Port Development Project

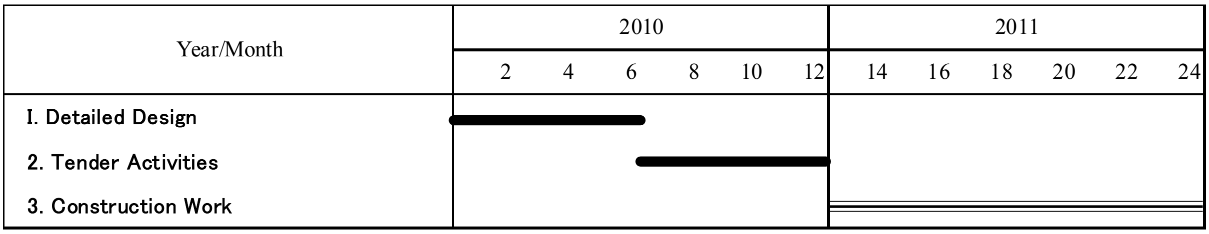


Figure III.4.6-3 Implementation Schedule for Southern Access Railway Improvement Project

4.6.3 Annual Fund Requirement

Based on the estimated costs and implementation schedule, annual fund requirements for key projects 1 and 2 have been estimated as shown in Tables III.4.6-1 and III.4.6-2. The details of the Key Projects 1 and 2 are shown in Appendix G.4.

Table III.4.6-1 Annual Fund Requirement for Outer Port Development Project

(Unit : million Euros)

	2009	2010	2011	2012	2013	2014
Key Project 1	4	4	49	100	87	106

Table III.4.6-2 Annual Fund Requirement for Southern Access Railway Improvement Project

(Unit : million Euros)

	2010	2011
Key Project 2	0.2	4.5