The Preliminary Study on Land Reclamation Alternatives at the Pacific Entrance to the Panama Canal

March 2003

Japan External Trade Organization (JETRO)

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CHAPTER 1 INTRODUCTION

1.1 Objectives of the Preliminary Study

Autoridad del Canal de Panama (ACP) is planning Panama Canal Expansion Project. The proposed Third Locks Project and related expansion of the Panama Canal will generate significant quantities of excavated materials amounting to some 50-70 million m^3. With a view to the beneficial usage of excavated materials coming from Panama Canal Expansion Plan activities, land reclamation at the Pacific entrance to the Panama Canal is under consideration.

The objectives of this JETRO Preliminary Study are to inquire into the technical and environmental aspects of offshore/onshore land reclamation alternatives for the beneficial usage of the excavated materials. These land reclamation alternatives will include Japanese technologies and experiences in constructing artificial islands.

1.2 Study Area

The Study area is at the Pacific entrance to the Panama Canal. Offshore land reclamation alternative (artificial island) and onshore land reclamation alternative (peninsula) are to be studied (see **Figure 1.1**).

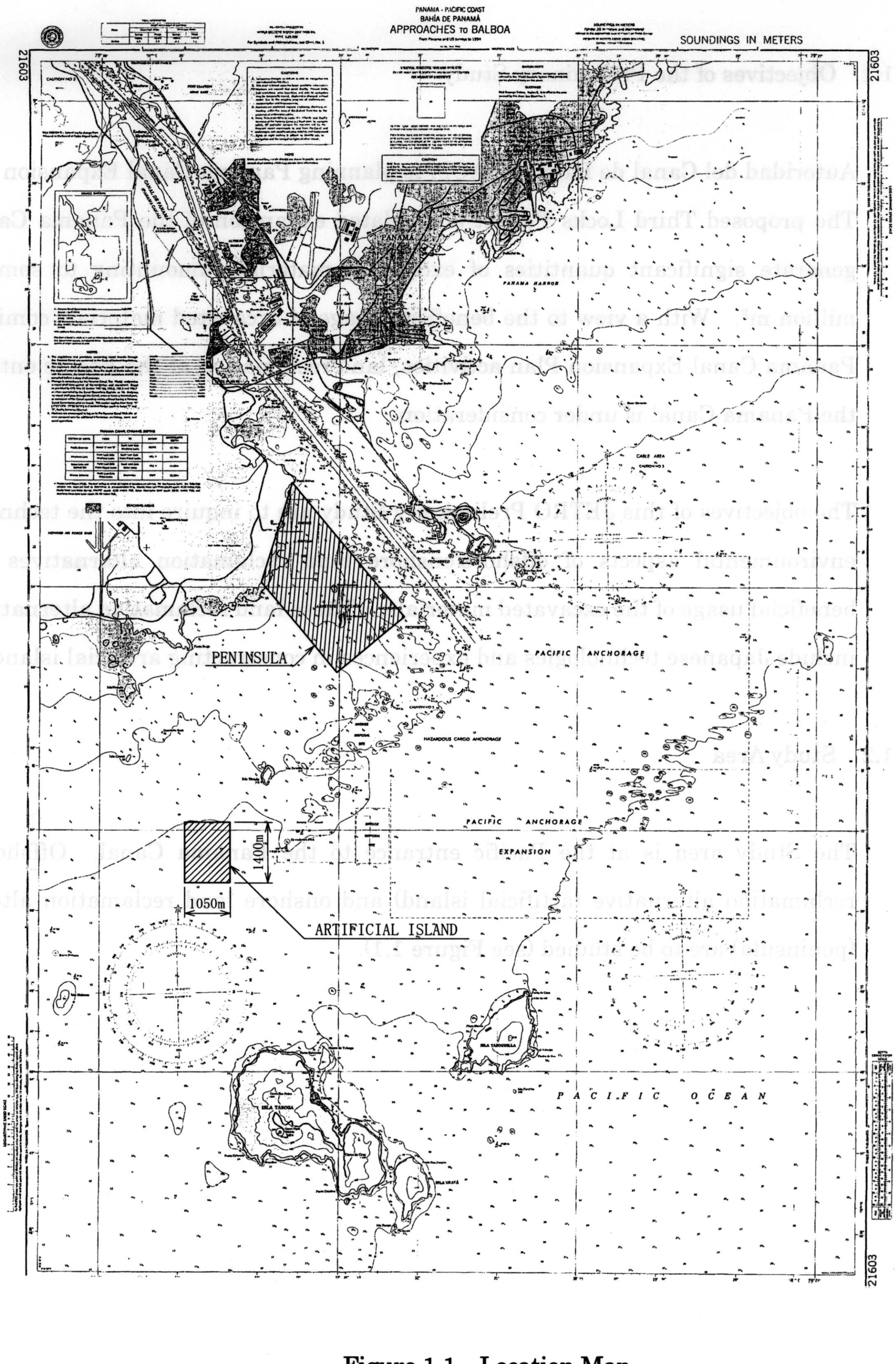


Figure 1.1 Location Map

1.3 Scope of the Study

To achieve the objectives mentioned above, the Preliminary Study will cover the following work items:

A. Preparatory Work:

a. Collection and review of the existing engineering studies, surveys and investigations relevant to the dry and wet excavation material disposal site alternatives from Panama Canal Expansion studies for the Pacific side.

b. Identification, collection and analysis of the engineering data necessary for the design of the offshore/onshore land reclamation alternatives, including meteorological, geo-technical, oceanographic, hydrographic, physical constraints (air space, navigation), etc.

c. Field visits to new lock sites and dry excavation disposal site alternatives.

B. Basic Proposal for Land Reclamation Alternatives:

a. Proposal for designing the construction of two land reclamation alternatives: artificial island and peninsula by using Pacific new locks dry excavation material. Due to time and budget constraints, this preliminary study will include basic technical, and cost comparisons between both alternatives.

b. Proposal for optimal artificial island size based on container port operation activities.

c. Proposal for implementing seawall technology in the construction of land reclamation alternatives depending on the conditions of soil and ocean geology. The proposal will include technical, and cost comparisons between seawall and embankment technologies.

d. Proposal for onshore and offshore land reclamation plans with regard to the methods of filling and sea transportation from a designated site. The study will assume the island and peninsula location as proposed by the ACP artificial island preliminary study, and preliminary screening of alternative disposal sites for materials from locks excavation.

e. Recommendations for potential options for island and peninsula developments based on Japanese experience.

f. For both land reclamation alternatives, JETRO will study the land filling and transportation, and will use ACP artificial island preliminary study data as follows:

	Transportation type	**Transportation to the site**	**Reclamation**	**Take-over point**
ARTIFICIAL ISLAND	Railway	ACP data	JETRO design	Delivered at reclamation site
	Barge	JETRO estimate	JETRO design	FOB Loading Point (ACP data) + Loading Cost+ Cost estimation of transportation between Loading Point and Reclamation Point
PENINSULA	Railway	ACP data	JETRO design	Delivered at reclamation site
	Barge	JETRO estimate	JETRO design	FOB Loading Point (ACP data) + Loading Cost + Cost estimation of transportation between Loading Point and Reclamation Point subject to navigation feasibility of Barges.

g. Proposal for environmental monitoring plans (EMP) during time periods before-construction, during-construction and after construction.

C. Cost Estimation:

Estimation of approximate costs of constructing seawalls, land reclamation, and dry excavation materials transportation by barges.

1.4 Schedule of the Preliminary Study

	Dec. 2002	Jan. 2003	Feb. 2003	Mar. 2003
(1) Collection of Engineering Data				
(2) Proposal for Land Reclamation Alternatives				
(3) Cost Estimation				
(4) Preparing Report				*

Note: Study in Panama, Study in Japan, * Submit final report

CHAPTER 2 PRELIMINARY STUDY ON CONSTRUCTING ARTIFICIAL ISLAND

2.1 Artificial Island Size

2.1.1 Introduction

Worldwide maritime container demand has been increasing strongly over the past three decades. The containerization of general cargo started first in the developing countries in Europe, North America and Japan, followed by the newly industrializing countries of East Asia, and is now expanding to the developing countries.

2.1.2 Worldwide Trend of Container Transportation

World container trade is estimated to have reached 70.1 million TEU in 2000. On the basis of the individual route assessment, the main East-West axis is estimated to have generated 39.6% of container traffic volumes, with 24.3% carried on North-South trades and an ever increasing 36.1% in intra-regional markets. Over time, the percentage shares of the three main market sectors have changed remarkably in favour of the intra-regional trades, which have almost caught up with the East-West sector.

Due to the growing global containerization and rising trade volumes, large vessels have been required more and more. These have put demands on ports to continually improve their capabilities in terms of quay length and capacity, gantry crane specification, berth depth and storage spaces. The most advanced ports are currently gearing themselves for 7,000 TEU vessels and are preparing for possible future needs for larger vessels.

Post-panamax container gantry cranes have become the norm and a number of ports are now investing in super-post panamax gantry cranes, capable of reaching across vessels with 22 container cells width.

The economics of operating large vessels require ship calls to be limited to large volume ports, and kept to the minimum length of stay. To maintain competitive, major ports have had to become extremely efficient, and new technology is being used increasingly at all stages of port operation to rationalize, automate, and accelerate processes.

Since 1980, global container port handling has grown at an average annual rate of 9.4%, a growth rate, which has continually exceeded underlying trade growth by several percentages.

The development of worldwide container port throughput by region is shown in Table 2.1.1 and Figure 2.1.1. It is clear that the world market for container cargo handling has expanded very rapidly, with annual average growth rate 9.7% over from 1985 to 2001 taking total throughput from 56.17 million TEUs to 245.83 million TEUs. Growth has been maintained through all major regions.

Table 2.1.1 World Container Throughput by Region

Year	1985	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000	2001
Europe	17.33	23.68	25.46	27.39	28.90	31.91	34.64	38.22	43.38	48.08	47.65	52.25	54.31
East Asia	12.80	22.93	25.95	28.72	31.93	36.49	40.62	44.56	48.52	51.42	60.95	71.42	75.12
South East Asia	4.37	11.44	13.44	15.60	18.54	22.01	25.37	28.25	31.26	32.30	35.97	41.33	43.02
North America	12.62	16.46	17.28	18.02	18.43	20.27	22.34	22.91	24.77	26.81	28.45	30.83	30.74
Caribbean & Central America	2.54	3.66	3.95	4.27	4.74	5.20	5.93	6.59	7.96	9.08	8.75	9.5	9.81
South America	0.96	1.47	1.70	2.12	2.49	2.83	3.71	4.23	4.92	5.20	7.08	7.81	8.19
Others	5.55	7.14	8.14	8.94	9.91	10.68	11.89	12.83	13.97	14.96	20.7	22.78	24.64
Total	56.17	86.78	95.92	105.06	114.94	129.39	144.5	157.59	174.78	187.85	209.55	235.92	245.83

Source: Drewry Shipping Consultants / Ocean Shipping Consultants

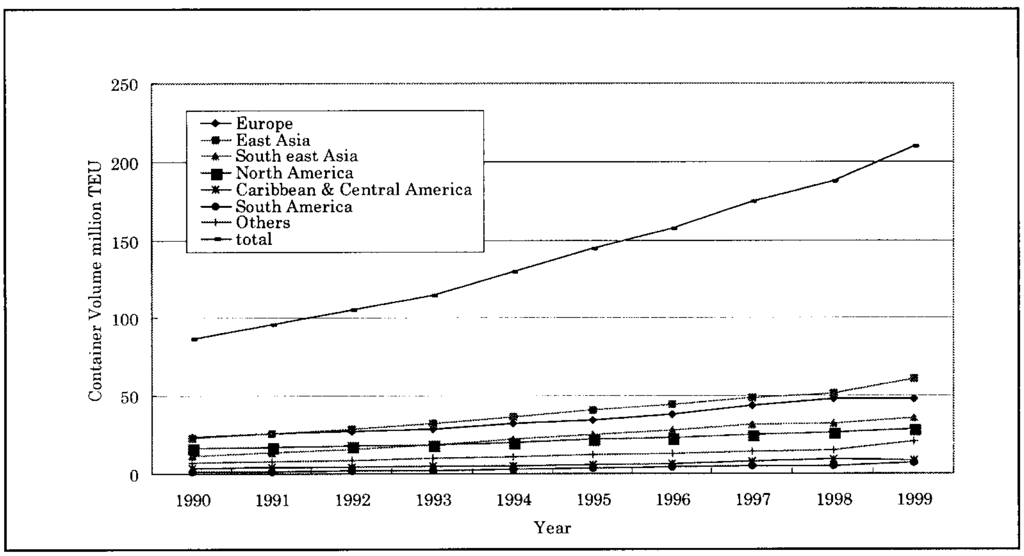


Figure 2.1.1. Graph of World Container Throughput by Region

2.1.3 Regional Container Cargo Volume Throughput (Pacific Coast of Latin America)

Latin America

Historical trend of container cargo throughput in Latin America is shown in Table 2.1.2 and Figure 2.1.2 Though in the past 16 years the average annual growth rate Worldwide is 9.7%, Latin America has recorded a 10.8% average growth rate.

Table 2.1.2 Latin America Container Throughput

Year	1985	1990	1991	1992	1993	1994	1995	1996	1997	1998
Caribbean	1,819.8	2,364.6	2,569.4	2,573.7	2,787.8	2,975.9	3,341.9	3,617.1	4,177.4	4,586.8
Central America East Coast	653.0	1,125.0	1,216.0	1,501.5	1,730.3	1,948.0	2,298.6	2,660.7	3,366.8	4,058.6
Central America West Coast	67.6	169.3	160.4	193.0	218.1	273.9	293.7	309.3	412.5	437.3
South America Atlantic Coast	719.8	1,006.8	1,086.4	1,327.4	1,589.5	1,820.0	2,505.7	2,630.8	3,060.1	3,171.0
South America Pacific Coast	243.4	458.5	615.0	788.6	895.7	1,007.8	1,204.0	1,600.4	1,855.2	2,026.5
Total	3,503.6	5,124.2	5,647.2	6,384.2	7,221.4	8,025.6	9,643.9	10,818.3	12,872.0	14,280.2

Source: Ocean Shipping Consultants

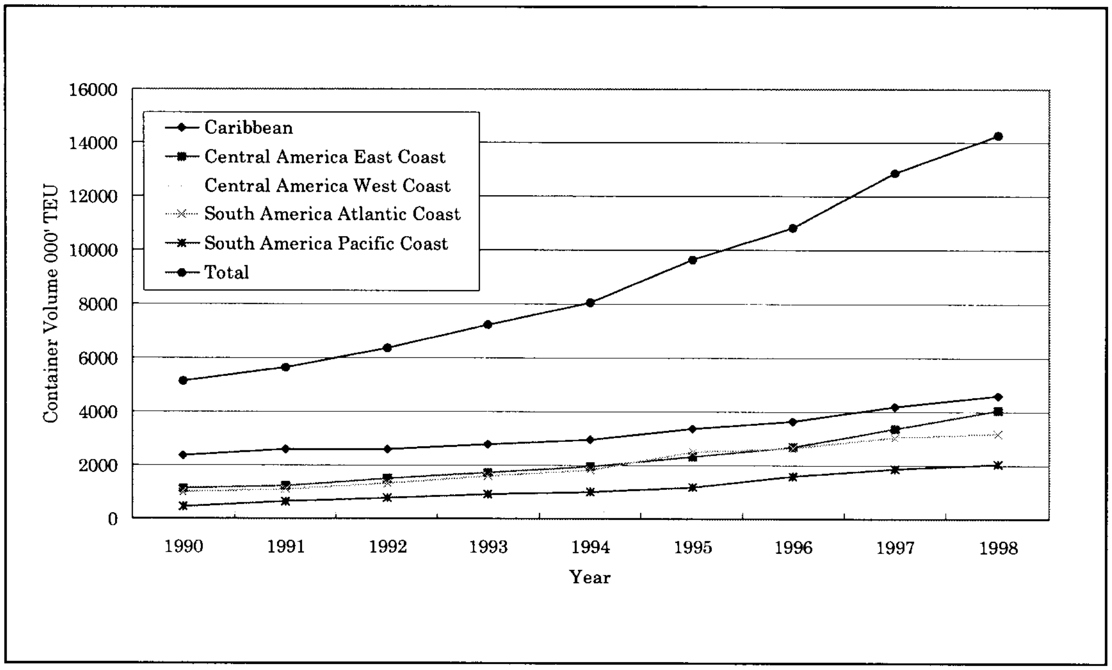


Figure 2.1.2 Graph of Latin America Container Throughput

Central America and Caribbean Container Throughput

Container Traffic in the Central America and Caribbean region has witnessed significant growth over several years. This expansion has included substantial investment in port facilities and infrastructure throughput the region, thus preparing the region for further increases in the future. In 1998, this region as a whole 9.1 million compared with only 3.7 million TEU in 1990. This equates to an average annual increase in container traffic of around 10.7 per cent. It is important to note that there has been a sharp upturn in growth rates since 1996 – primary as a result of increased transshipment volumes. Pacific Coast Latin America Container throughput is shown in Table 2.1.3 and Figure 2.1.3.

Table 2.1.3 Pacific Coast Latin America Container Throughput

Year	1985	1990	1991	1992	1993	1994	1995	1996	1997	1998
Central America West Coast	67.6	169.3	160.4	193.0	218.1	273.9	293.7	309.3	412.5	437.3
South America Pacific Coast	243.4	458.5	615.0	788.6	895.7	1,007.8	1,204.0	1,600.4	1,855.2	2,026.5
Total	311.0	627.8	775.4	981.6	1,113.8	1,281.7	1,497.7	1,909.7	2,267.7	2,463.8

Source: Ocean Shipping Consultants

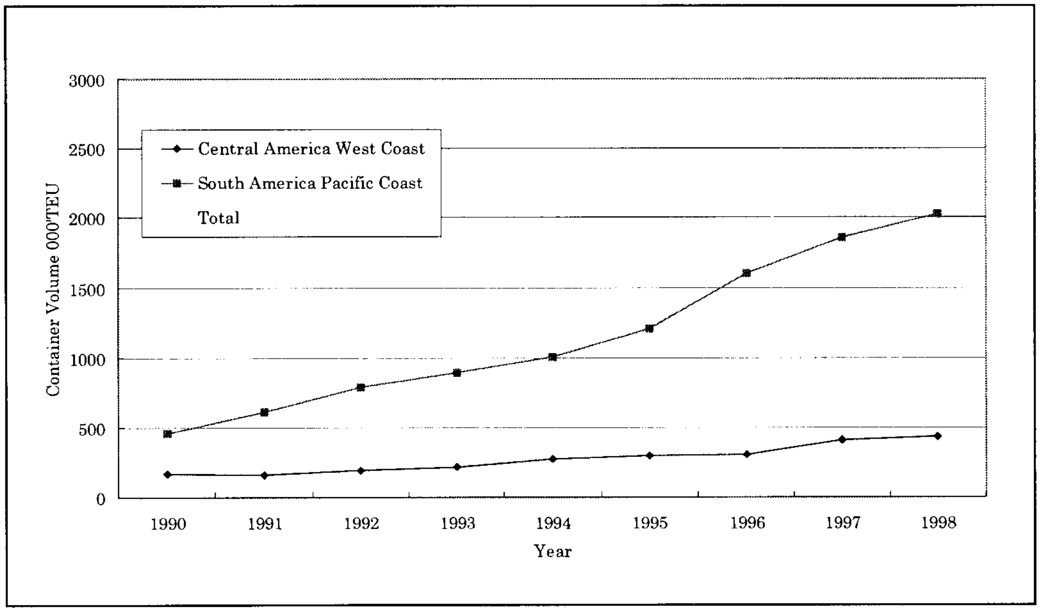


Figure 2.1.3 Graph of Pacific Coast Latin America Container Throughput

2.1.4 Demand Forecast of Latin America Pacific Region for the year 2010～2020

The target year of demand forecast is year 2010～2020, where Panama Canal expansion project is expecting to be completed.

Although the pace of economic expansion in the developing world is increasingly dependent upon factors intrinsic to the countries and regions concerned, the patterns of growth in the developed economies will set the tone for the world economy as a whole. For Latin America and the Caribbean, the size and proximity of the US economy implies that long-term development will depend on US growth, as well as continuing efforts within the region to open up economies and introduce market reforms.

Table 2.1.4 shows the regional Container throughput and size of regional GDP volume from 1990 to 1998.

There is a quite high correlation between the size of the regional GDP volume (GDP volume 1.0, base year 1990) and regional container cargo volume.

$Y = 6{,}353 * X - 5824 \quad (R2 = 0.979)$

Where: Y : Container Volume
X : Regional GDP volume (GDP volume of base year 1990 is 1.0)

Table 2.1.4 Container Throughput and Historical Trend of Regional Size, 1990-1998

Year	1990	1991	1992	1993	1994	1995	1996	1997	1998
Container Throughput	627.8	775.4	981.6	1,113.8	1,281.7	1,497.7	1,909.7	2,267.7	2,463.8
Size of Regional GDP Volume	1.000	1.033	1.067	1.102	1.139	1.176	1.215	1.255	1.297

Container Throughput: '000 TEU
Source: Ocean Shipping Consultants

Regional GDP growth rate forecasted by the World Bank is adopted for future projection. Table 2.1.5 and Figure 2.1.4 show the forecasted GDP growth rates by the World Bank. Study team adopted same growth rate of 3.9 % for the year until 2020.

Table 2.1.5 Historical Trend and Expected GDP Growth Rate in Region

Year	1991 ~ 1998	1999	2000	2001	2002	2003	2004 ~ 2020
GDP Growth rate (%)	3.3	0.1	3.8	0.9	2.5	4.5	3.9

Source: World Bank, Regional Economic Prospects, and Study Team

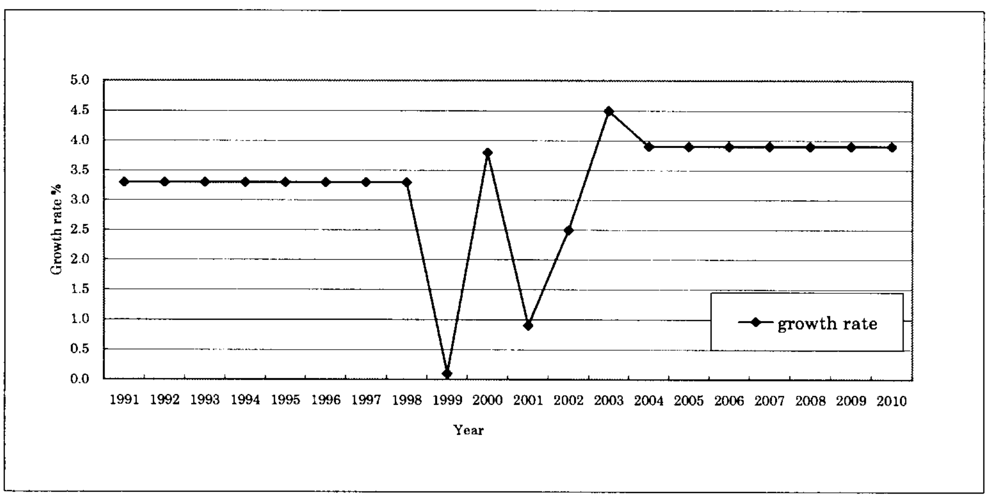


Figure 2.1.4 Historical Trend and Expected GDP Growth Rate in Region

Table 2.1.6 shows the projected regional GDP volume ratio compared with the base year 1990's GDP volume.

Table 2.1.6 Regional GDP Volume Projection for Year 2020

Year	1990	2000	2001	2002	2003	2004	2005	2010	2015	2020
GDP Growth Rate (%)	-	3.8	0.9	2.5	4.5	3.9	3.9	3.9	3.9	3.9
Size of FDP Volume	1.000	1.347	1.359	1.393	1.456	1.513	1.572	1.903	2.304	2.79

Source: World Bank, Study Team

Therefrom, the projected regional GDP volume for the years 2010, 2015 and 2020 are 1.902, 2.304 and 2.790 times of the base year 1990's respectively.
Using the same correlation equation, the container cargo projection volume of the target years 2010, 2015 and 2020 is obtained.

Volume Projection

Year 2010 GDP = 1.903 (GDP =1.0 in 1990)
Y = 6,353 * 1.903 – 5824 = 6,266
Regional container handling volume : 6,266,000 TEUs

Year 2015 GDP = 2.304 (GDP =1.0 in 1990)
Y = 6,353 * 2.304 – 5824 = 8,813
Regional container handling volume : 8,813,000 TEUs

Year 2020 GDP = 2.790 (GDP =1.0 in 1990)
Y = 6,353 * 2.790 – 5824 = 11,900,000
Regional container handling volume : 11,900,000 TEUs

2.1.5 Container Demand Forecast Volume of New Port

Expansion of Panama Canal and construction of new port for post-panamax vessel will accelerate the post-panamax vessel initially on the run in the Latin America region.
The new port has a great potential to be the regional hub port taking advantage of geographical location and modern equipment for the highly automated and advance facilities as well as the gateway port of Panama.

Table 2.1.7 shows the estimated transshipment container volume and Incidence from 1990 to 2000.

Table 2.1.7 Container Throughput and Estimated Transshipment Container Volume from 1990 to 2000

Year	1990	1991	1992	1993	1994	1995	1996	1997	1998	1999	2000
Container Throughput	87.6	96.1	105.8	116.3	130.3	144.2	156.6	174.1	188.4	207.6	233.3
Estimated Transshipment Container Volume	16.2	18.9	21.6	24.9	28.6	32.7	36.4	41.2	45.2	50.5	57.1
Ratio (%)	18.5	19.7	20.4	21.4	21.9	22.7	23.2	23.7	24.0	24.3	24.5

Source: Drewey Shipping Consultants

The large ships on the axial route take in cargo from an ever-wider area, move it along the east-west pipeline to a hub port and then feed it into either a regional transshipment system or an international relay network. Indeed, transshipment has become such an important container throughput generator that the natural hinterland or the landside transport infrastructure- formerly important aspects – are of very little consequence to specialized hubs such as Marsaxlokk, Gioia Tauro or Tanjung Pelepas where the majority of throughput never passes through the terminal gates.

Transshipment has been a major factor not just in the growth of world container port activity, but also in the demand for specific sizes/type of vessel, and in the present organization and structure of the container shipping industry.

Worldwide average ratio of Transshipments container volume has increased from 18.5% in the year 1990 to 24.5% in the year 2000 along with the improvement of port facilities and enlargement of container vessel. Transhipment container volume rose from an 16.2 million TEUs in 1990 to 57.1 million TEUs in 2000. In the future, transshipment as a generator of port handling activity, is expected to increase in importance, as carriers rely on filling their ever larger vessels with cargo which originates from an ever geographical radius.

If it is profitable for shipping lines to use new port as a transshipment hub instead of direct calls to the Pacific Latin American ports, the transshipment container volume at new port will increase significantly. Such potential transshipment is closely related to the facilities and services offered at the new port.

From this scenario, study team assumed that the new port would handle 20% to 30% of total regional container volume. Projected transshipment container cargo volume at new port is shown in Table 2.1.8.

Table 2.1.8 New Port Container Cargo Demand Forecast for Year 2010-2020

Unit: TEU

Year	Total Volume	Transshipment Ratio			
		10%	20%	30%	40%
2010	6,266,000	627,000	**1,253,000**	**1,880,000**	2,506,000
2015	8,813,000	881,000	**1,763,000**	**2,644,000**	3,525,000
2020	11,900,000	1,190,000	**2,380,000**	**3,570,000**	4,760,000

Study team assumed that the new port will handle from 1,253,000 TEUs to 3,570,000 TEUs container volume in the year 2010-2020.

2.1.6 Port Development Activity in Panama

Throughout Central America, Panama has witnessed by far the most dramatic port development programme. The Panama, being closed to the proximity of the canal, has placed them in a favorable position to expand their transshipment operations. Consequently, container throughput volumes have increased rapidly leading to large-scale investment in port infrastructure. In 1998 total port volumes on the eastern side of Panama reached a level of 1 million TEUs, with this being funded primarily by surging transshipment demand.

Major terminal development included the July 1996 award to Hatchinson Whampoa of Hong Kong of the 25 year concession to manage the port of Cristobal. Having won concession, Hutchinson Whampoa created a local subsidiary, Panama Ports Company (PPC), to take over operations at the port. Over the next five years, PPC has pledged to invest a total of US$50 million in upgrading the port facilities at Cristbal.

The most recent attempt to become the main Central American transshipment hub has been launched by Evergreen with its new Colon Container Terminal (CCT) which opened in March 1998. The CCT is located at Coco Solo North, adjacent to the Manzanillo International terminal and just North of Cristobal. At present, Evergreen's US$100 million, 25ha container facility has 612m of quays on two berths, both dredged to 14m. This, in conjunction with four post Panamax cranes and six RTGs, has enabled the port to accommodate two Panamax vessels simultaneously and has pushed capacity to approximately 0.4 million TEU per annum.

Also designed as a transshipment hub, the Manzanillo International Terminal (MIT), adjacent to Coco Solo, began container operations in 1995. The US$ 210 million facility, a joint venture between Motores Internacionales (Moinsa) and the Stevedoring Services of America (SSA), also continuously benefits from its close proximity to the Colon Free Trade Area. At present, the facility has 950m of berths capable of handling vessels up to 13m draught. The facility also has four post Panamax and two Panamax cranes enabling it to accommodate the largest container ships. Container operations at the 25ha yard are also enhanced by 10 RTGs and the recent inclusion of the terminal in the queuing zone of the Canal.

In comparison to the Atlantic Coast and Caribbean Sea, port development in the Pacific Coast has been limited, with new investment only attracted in more recent time following the rapid expansion of the Far Eastern markets

The port of Balboa situated at the mouth of the Panama Canal, is the only major container port in the Panamanian Pacific Coast. In spite of the growing trade links between the Far East and the West Coast of the Americas, there has been, until recent times, only limited interest in container ports of this Coast of Panama. In July 1996, the 25 years concession to operate and manage the port, along with Cristobal on the Atlantic Coast was awarded to Hutchinson Whampoa. The Panama Ports

Company (PPC), the local subsidiary formed by Hutchinson Whampoa, is responsible for the day-to-day operations at both ports and has US$50 million investment at Balboa over the next five years.

2.1.7 Design Development of the New Port

Required Number of Berths

The development of port facilities has historically progressed hand-in-hand in line with the development in the vessel size. Larger vessels could not be introduced until there were enough suitably equipped ports abling to handle them.

The required number of berths is determined based on cargo volume and handling capacity. The required number of berths is calculated by the following formula.

$$N = Nc / Ch / (Dy * Hd) / Ro$$

N : Required number of berth
Nc : Cargo throughput per year
Ch : Cargo volume per berthing time
Dy : Annual operation days
Hd : Working hours per day
Ro : Planning berth occupancy ratio

*Ch = 24 cycle/hour *0.8(efficiency) = 19.2 box/hour/crane*

In 2010, six (6) numbers of container berth, each equipped with two (2) gantry cranes, will be constructed to accommodate the prospected numbers of container cargo volumes.

To increase the handling capacity of container cargo, it is necessary to increase the number of gantry crane installation and improve the cargo handling efficiency.

Year 2010 : Two number of Gantry Crane / Berth
1,880,000 /2 / (19.2 * 2) / (350 * 24 * 0.8) / 0.6 = 6.0 Berth (O.K)

Year 2015 : Three number of Gantry Crane / Berth
2,644,000 /2 / (19.2 * 3) / (350 * 24 * 0.8) / 0.6 = 5.7 Berth (O.K)

Year 2020 : Three number of Gantry Crane / Berth 0.75
3,570,000 /2 / (19.2 * 3) / (350 * 24 * 0.8) / 0.75 = 6.0 Berth (O.K)

The maximum draft of the vessel which can transit the Canal is limited to less than 39.5 ft (12.4m), and waterways and basins are generally designed as 42 ft (12.73 m) deep in consideration of the extent of oscillatory motion of the ship due to the natural condition such as waves, winds and tidal currents, and trim.

The necessary berth length is usually estimated as 350m for the berths with a depth of 15-16 meters.

Terminal Area

The terminal area needed for one berth is estimated at least 12-15 ha.

This means the width of terminal yard space will be at least 350 meters under the condition that the length of each berth is 350 meters. In planning a full-scale container terminal, the maximum size shall be secured in the area. See attached layout plan.

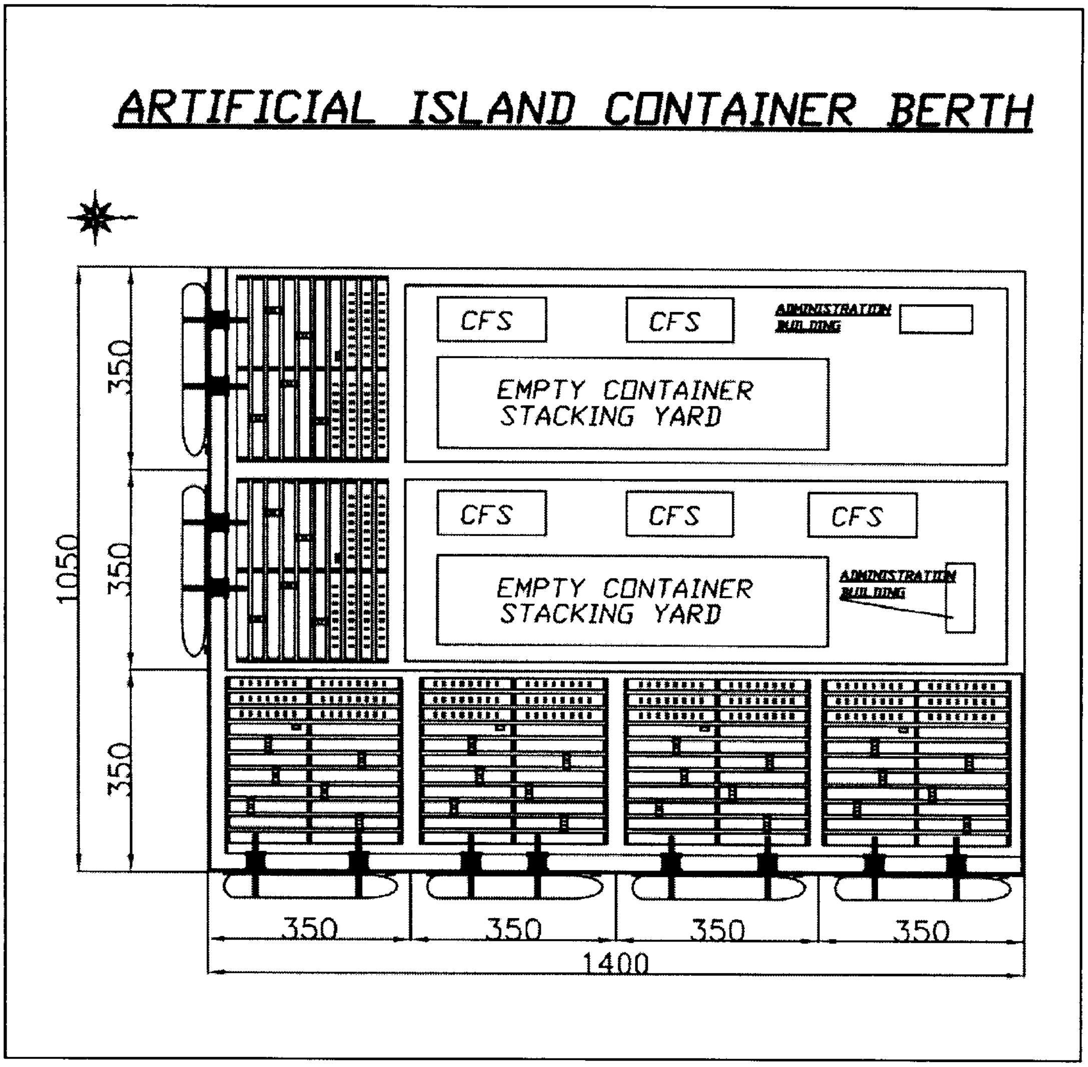


Figure 2.1.5 Artificial Island Size

2.2 Plan and Design of Quaywall and Revetment for Artificial Island

2.2.1 Design Objectives and Design Policy

2.2.1.1 Design Objectives

This Section introduces structural design of quaywall and revetment for "Artificial Island", including the proposal of high-advanced technology.

Design Objectives: 1) Quaywall and Revetment for Artificial Island

Figure 2.2.1 shows general plan of Artificial Island. In this study, the location of Artificial Island is planned at the same site as proposal in ACP Report (2001), "Preliminary Study of island Development at the Pacific Entrance of the Panama Canal - Final Report". Access causeway will connect Artificial Island with continental land. Structural design of access causeway is out of scope in this study.

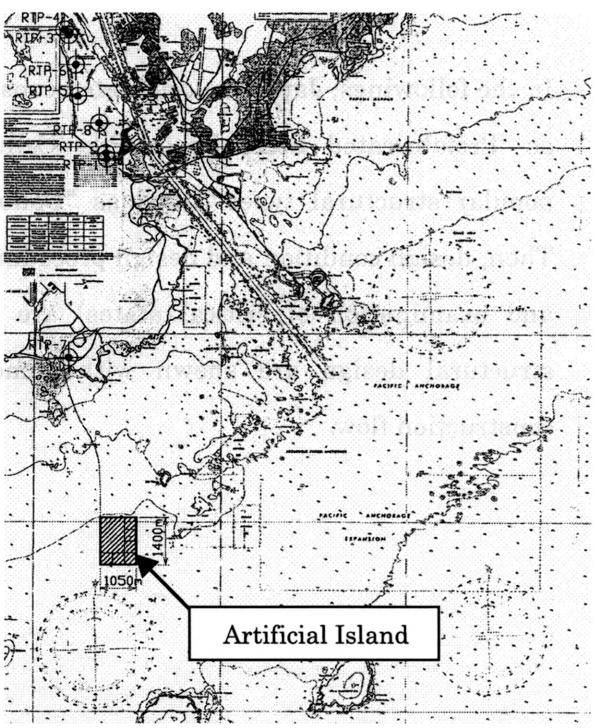


Figure 2.2.1
Location of Artificial Island in Planning

2.2.1.2 Design Policy

The design policy should be displayed before the proceeding to design of these infrastructures. The design policy for artificial island is shown in order of priority as:

Design Policy:

1) Minimum impact on environment
2) Shortening construction period
3) Minimizing construction cost
4) High structural durability

The most principal observance is to achieve the minimum impact on environment during and after construction. Particularly, the seawater pollution must be avoided carefully. Next, the construction period should be short to associate well with the related projects around Panama Canal. Construction cost is also important aspect for assessment of the feasibility of this construction project. Finally, the revetment and quaywall should possess sufficient durability in the whole lifetime.

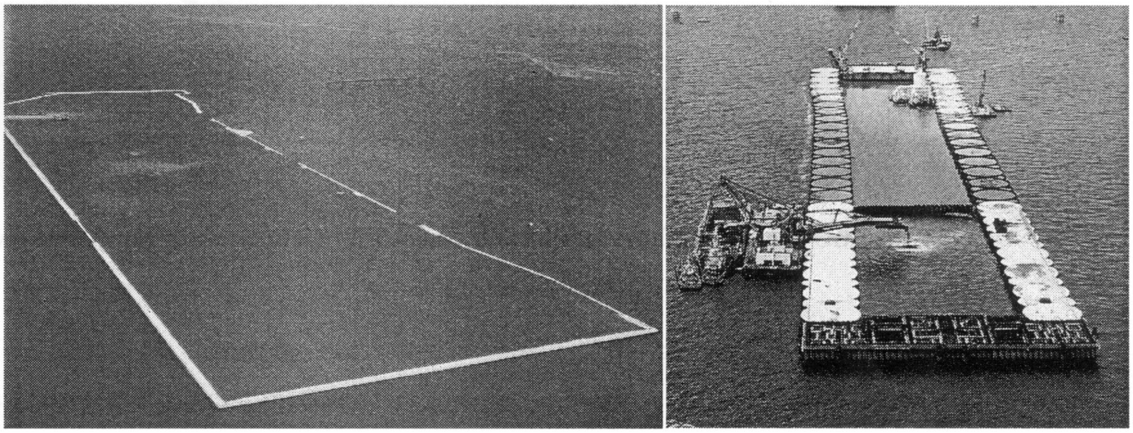
In the followings, Japanese-originated construction for reclaimed island is introduced as 'Prefabricated Steel Sheet Pile Cellular-bulkhead Quaywall'. Next, another popular structural types called as "Double Steel Sheet Pile Wall" is introduced. Then, design condition and design procedures (design standards, structural modeling and examination of critical states) are explained briefly. Lastly, the results of structural design are shown with some drawings with visual explanation of construction flow.

2.2.2 Construction of Reclaimed Island in Japan

2.2.2.1 Construction Method in Japan

Nowadays, when constructing an artificial island in Japan, the reclamation is implemented after all construction of surrounding revetment. Considering anti-contamination of seawater, reclamation of inside area should be isolated from external seawater.

Bird's view of construction of reclaimed island is shown in Figure 2.2.2 as two examples in Japan. These photographs are constructions of Kansai International Airport and Tokyo-wan Aqua Line (Umihotaru Island). Kansai International Airport was the first artificial airport island in Japan. The area of the artificial island is 800ha surrounded by revetments of total 12km. Seawater is deeper than 25m. Then, Tokyo-wan Aqua Line is consists of undersea tunnel of 10km and continuous girder bridge of 5km approximately. Umihotaru Island is constructed at the connection point of undersea tunnel and girder bridge. This artificial island is 600m long and 100m wide and the water depth is 17m approximately. Reclaimer vessel can be seen just working in this photograph.



(a) Kansai International Airport

(b) Tokyo-wan Aqua Line (Umihotaru Island)

Figure 2.2.2 Construction Examples of Reclaimed Islands in Japan

As shown in Figure 2.2.2, reclamation is always executed after construction of surrounding revetment in Japan. Therefore, environmental impact can be minimized because the external seawater is isolated perfectly from reclamation area.

2.2.2.2 Steel Sheet Pile Cellular-bulkhead Quaywall

Among popular structural types in Japan are concrete caisson, double steel sheet pile wall and steel sheet pile cellular-bulkhead quaywall. Concrete caisson is very popular in many countries. However, double steel sheet pile wall and prefabricated steel sheet pile cellular-bulkhead quaywall (called as 'sheet pile cellular-bulkhead quaywall') are also popular in Japan but not so in some countries. In waterfront infrastructures of Japan, more than 20 sheet pile cellular-bulkhead quaywalls have been constructed since 1970s.

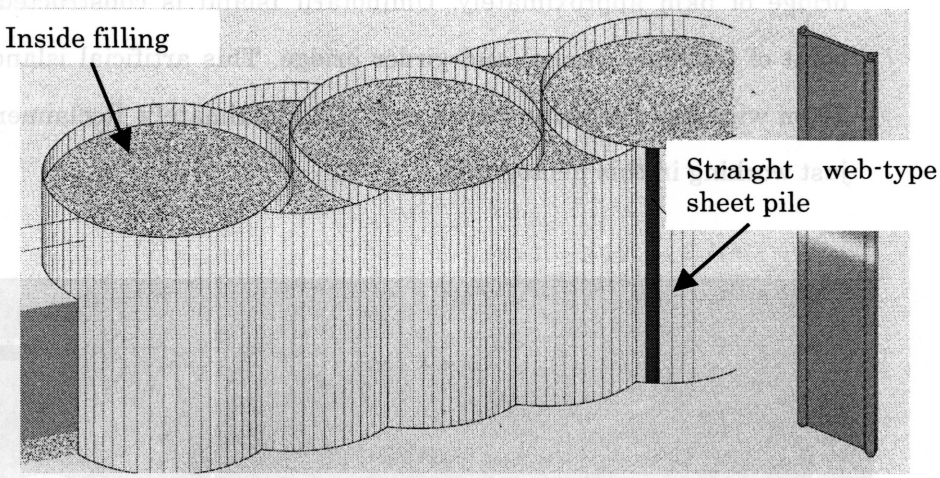
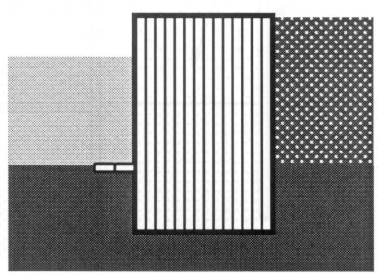
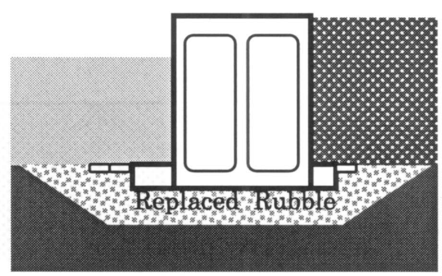


Figure 2.2.3 Prefabricated Steel Sheet Pile Cellular-bulkhead Quaywall

Figure 2.2.3 explains a structural configuration of sheet pile cellular-bulkhead quaywall. The stability is kept highly with sheet pile cellular and inside filling. Firstly, straight web-type sheet piles are driven by vibrohammer. Secondly, inside filling is executed. Straight web-type sheet piles can resist the hoop tension in circumferential direction produced by earth pressure from inside filling. Straight web-type sheet piles have stronger tension strength than U-type and Z-type sheet piles. Inside filling needs 1 to 2 days for one cellular-bulkhead quaywall. Calm climate is desirable for inside filling because empty condition of cellular-bulkhead quaywall is unstable against storm wave.

Sheet pile cellular-bulkhead quaywall is compared roughly with concrete caisson in Table 2.2.1. This Table compares both structures as for environment in installation and reclamation, construction period and cost, and durability. There are two differences between them, as in environment in installation and construction period. In construction (installation) of sheet pile cellular-bulkhead quaywall, driving sheet piles and then inside filling can keep seawater clean from contamination. On the other hand, in construction of concrete caisson, rubble replacing may make seawater dirty.

Table 2.2.1 Comparison between Sheet Pile Cellular-bulkhead Quaywall and Concrete Caisson

	Prefabricated Steel Sheet Pile Cellular-bulkhead Quaywall	Concrete Caisson
Side view		
Environment in installation	Good	No good (due to replaced rubble)
Environment in reclamation	Good	Good
Construction period	Short	Mediate
Construction cost	Mediate	Mediate
Durability	Good	Good
Evaluation	Recommended	Mediate

Furthermore, casting, haulage and emplacement of concrete caisson need generally longer time than driving sheet piles. Both structural types are even in construction cost and durability. Additionally, casting work and concrete plant need large sized area near the construction site in case of concrete caisson. Consequently, sheet pile cellular-bulkhead quaywall is recommended in this Study. For final selection of the best structural type, so more detailed and elaborate evaluations should be carried out in full scope feasibility study of the future.

2.2.2.3 Double Steel Sheet Pile Wall

Double steel sheet pile wall is also competitive in Japan. Figure 2.2.4 shows vertical section of double sheet pile wall.

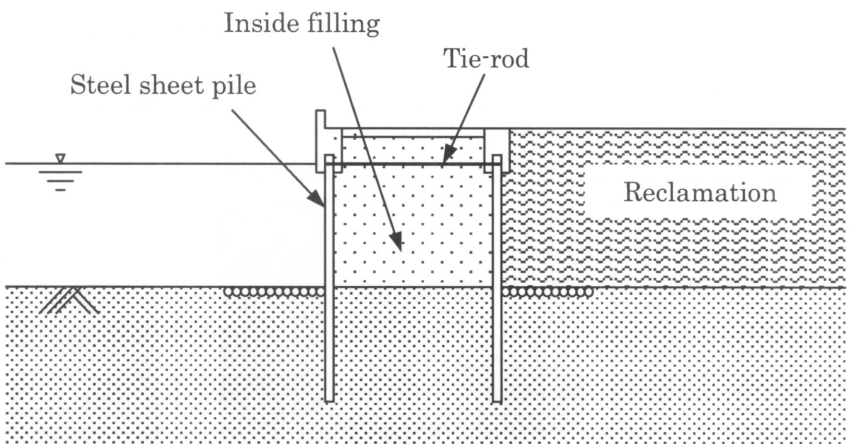


Figure 2.2.4 Double Steel Sheet Pile Wall

This structure is constructed with inside filling between two facing sheet piles connected one another by tie-rods. Structural stability and flexibility can be possessed at high level due to using ductile sheet piles penetrated into stable underground layer. Inside filling is carried out after driving sheet piles and connecting tie-rods tightly.

As well as sheet pile cellular-bulkhead quaywall, double steel sheet pile wall can keep seawater clean during construction (installation) and in reclamation. In construction period and durability, double steel sheet pile has almost the same performance as sheet pile cellular-bulkhead quaywall. Construction cost can be changeable more or less, based on various condition as water depth and soil layer configuration, compared with sheet pile cellular-bulkhead quaywall. The comparison between two structural types will be discussed after the design executions in this Chapter.

2.2.3 Design Condition

2.2.3.1 Design Standards

To design sheet pile cellular-bulkhead quaywall and double steel sheet pile wall, the following Japanese standard is used in this study. This is the most authoritative design code as for water front infrastructure in Japan. Sheet pile cellular-bulkhead quaywall Chapter 7 and Section 17.5 in this standard deal with sheet pile cellular-bulkhead quaywall and double steel sheet pile wall, respectively.

> Technical Standards and Commentaries for Port and Harbour Facilities in Japan", *The Overseas Coastal Area Development Institute of Japan*, 1999.

In this study, the above standard is called as "Technical Standards in Japan".

2.2.3.2 Oceanographic Condition

(1) Tides

Tidal level is assumed as shown in Table 2.2.2 based on JICA Report (1997), "The Study on the Development of the Port of Balboa in the Republic of Panama".

Table 2.2.2 Tidal Level [JICA Report 1997]

Tidal Data	P.L.D.=0.000	M.L.W.S=0.000
Highest Water (H.W.)	3.596	5.918
Mean Monthly Highest High Water (M.H.H.W.)	3.023	5.345
Mean High Water (M.H.W.)	2.140	4.462
Lowest High Water (L.H.W.)	0.676	2.998
Mean Sea Level (M.S.L.)	0.307	2.629
0.00 Precise Level Datum (P.L.D.)	**0.000**	2.322
Highest Low Water (H.L.W.)	-0.327	1.995
Mean Low Water (M.L.W.)	-1.696	0.626
Mean Low Water Spring (M.L.W.S.)	-2.322	**0.000**
Mean Monthly Lowest Low Water (M.L.L.W.)	-2.788	-0.466
Lowest Water (L.W.)	-3.445	-1.123

Means are from 1973 to 1991. Extremes are from 1909 to 1991.

These data are almost the same as Drawing, 'Underkeel Clearance and Over the Sill Clearance for 39.5 Feet Vessels' given by ACP. The left column and right column mean each level above P.L.D and M.L.W.S., respectively.

In this study, indication above M.L.W.S. is used because bathymetric survey and boring exploration was carried out previously based on M.L.W.S..

(2) Design Water Level

Design water levels are set at mean monthly highest high water (M.H.H.W.) and mean monthly lowest low water (M.L.L.W.) as shown in Table 2.2.3. In structural design, M.H.H.W. is water level for design case of wave attack. M.L.L.W. is assumed for ordinary case and earthquake case in design.

Table 2.2.3 Design Water Level

Design Water Level	Design Case
Mean Monthly Highest High Water (M.H.H.W.)	Wave Attack
Mean Monthly Lowest Low Water (M.L.L.W.)	Ordinary, Earthquake

Furthermore, residual water level in sheet pile cellular-bulkhead quaywall and double sheet pile wall is calculated as:

$$\text{Residual Water Level} = 2/3\,(\text{H.W.L.} - \text{L.W.L.}) + \text{L.W.L} \qquad (2.2\text{-}1)$$

$$= \text{M.L.W.S.} + 3.41\text{m} \qquad (2.2\text{-}2)$$

(3) Existing Water Depth at Construction Site

Existing water depth at construction site is set based on the results of bathymetric survey submitted by ACP. For Artificial Island, the planning site is located on bathymetric map as shown in Figure 2.2.5. Water depth around the construction site is from -11m to -13m. In this study, the existing water depth in design is assumed as -12.0m for Artificial Island. Thus the existing water depth in design are summarized as shown in Table 2.2.4.

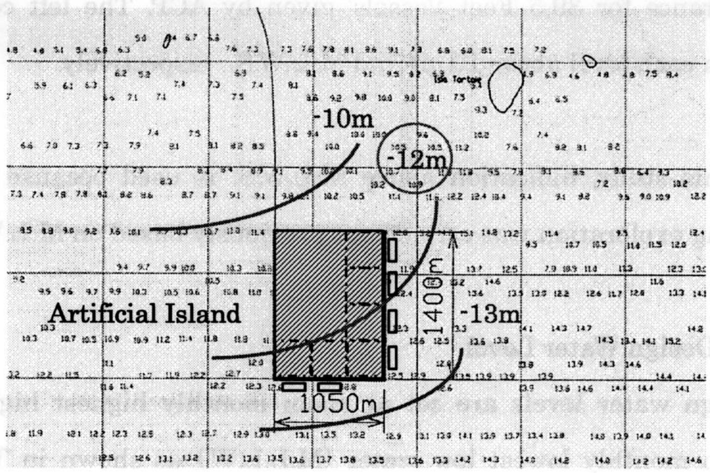


Figure 2.2.5 Existing Water Depth at Artificial Island

Table 2.2.4 Assumed Existing Water Depth at Artificial Island

	Artificial Island
Existing Water Depth	-12.0m

(4) Design Water Depth

According to ACP Report (2001), "Preliminary Study of island Development at the Pacific Entrance of the Panama Canal - Final Report", the new Third Locks will be designed to accommodate the following vessel dimensions in Table 2.2.5 as:

Table 2.2.5 Vessel Dimension [ACP Report 2001]

	Vessel Dimension
Length	385.7m
Beam	54.9m
Draft	15.2m
DWT	105,000

Berth facility is a necessary function in construction of Artificial Island. Berth depth is decided under consideration of adding safety factor and dredging tolerance to loaded draft. Consequently in this study, the berth depth is set at -16.75m for Artificial Island. This depth is the same as the value in the above ACP Report (2001). Therefore, dredging is needed for quaywall side (berth side) construction of Artificial Island because the existing water depth is 12.0m as described previously. Revetment side can be constructed on seabed of -12m depth without any dredging works.

On the other hand, berth facility is not needed in case of Reclamation Peninsula. Dredging is not necessary for construction of any revetment of Reclamation Peninsula. Consequently, (design) water depths are summarized as shown in Table 2.2.6.

Table 2.2.6 Water Depth for Each Side at Artificial Island

	Artificial Island
Planning Water Depth	-16.75m for Quaywall (Berth) Side : dredged from -12.0m
	-12.0m for Revetment Side : constructed as existing depth

(5) Significant Wave Height

According to ACP Report (2002), "Artificial Island Feasibility Study - Wave Transformation Study: Existing Conditions", significant wave heights were calculated by Gumble distribution approach based on hindcast record for 31 years. Results of the analysis are indicated in Table 2.2.7. For sheet pile cellular-bulkhead quaywall and double sheet pile wall, examination of global stability is executed against significant wave height.

Table 2.2.7 Return Periods and Wave Heights [ACP Report 2002]

Return Period (years)	Significant Wave Height (m)
2	2.6
5	2.8
10	3.0
20	3.2
50	3.4
100	3.6

2.2.3.3 Geological and Geotechnical Condition

(1) Design Ground Level

Design ground level is important for use of berth function and volume capacity of reclamation. Technical Standards in Japan recommends typical crown heights of mooring facility above high water level as indicated in Table 2.2.8. Tidal range in this study accounts approximately for 6 m. Based on this Table, the tidal range is larger than 3m. Water depth is so deep as 16m approximately. This depth belongs to the category for large vessel. Therefore, the crown height should be set to +0.5 to +1.5m from M.H.H.W..

Table 2.2.8 Typical Crown Heights of Mooring Facility above High Water Level

	When the tidal range is 3.0m or more	When the tidal range is less than 3.0m
Mooring Facilities for Large Vessel (with a water depth of 4.5m or more)	+0.5 ～ 1.5m	+1.0 ～ 2.0m
Mooring Facilities for Small Vessel (with a water depth of less than 4.5m)	+0.3 ～ 1.0m	+0.5 ～ 1.5m

Moreover, considering rain season and squall in Panama, the crown height should be a little higher as +1.655m. Consequently, the Ground level of container terminal is set at MLWS +7.0m (= +5.345m+1.655m) here. Finally, the design water level and the design ground level are summarized in Figure 2.2.6.

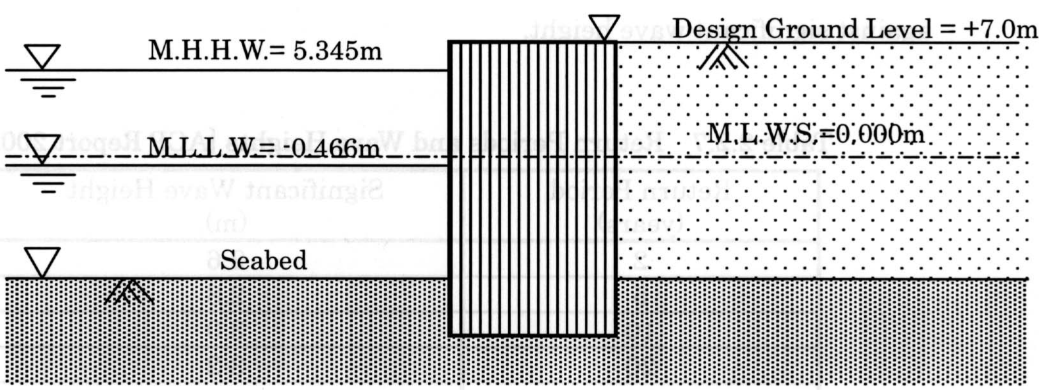


Figure 2.2.6 Design Water Level and Design Ground Level

(2) Soil Condition

Figure 2.2.7 shows existing boring logs around construction site of Artificial Island. This figure also indicates another plan as "Reclamation Peninsula". Although there are totally 7 boring logs, the locations of all logs are far away from construction site of Artificial Island. The nearest boring log is RTP-7.

Horizontal profile of these boring logs is arranged in Figure 2.2.8. In most of the boring logs except RTP-1 and 2, bearing soil layer with 50 of N-value exists from -6m to-10m level. Soil layers above the stiff layer are categorized into silty sand and N-values are comparatively large as 10 to 30. Bearing soil layers are deep in only RTP-1 and 2. This area is something like sedimentary basin, so fat clay is deposited in this low-lying land. However, all sites except RTP-1 and 2 are regarded as having similar configuration of soil layers.

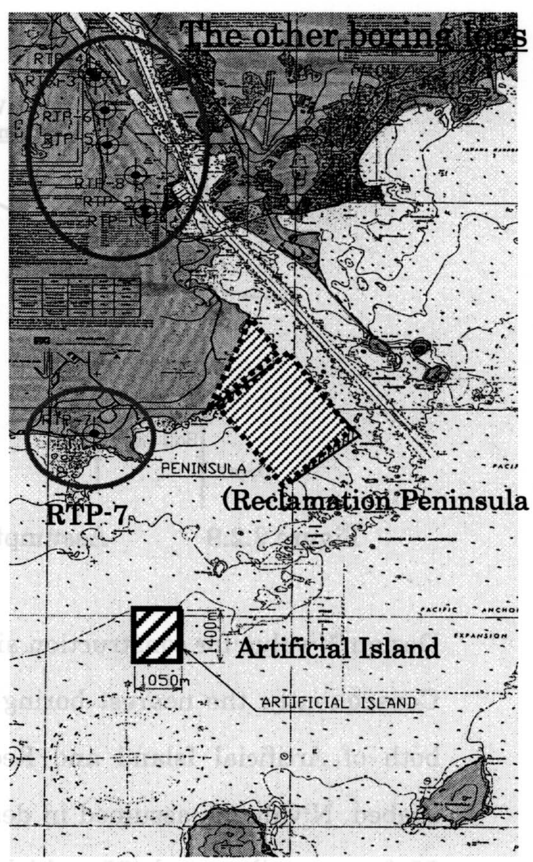


Figure 2.2.7
Location of Existing Boring Logs

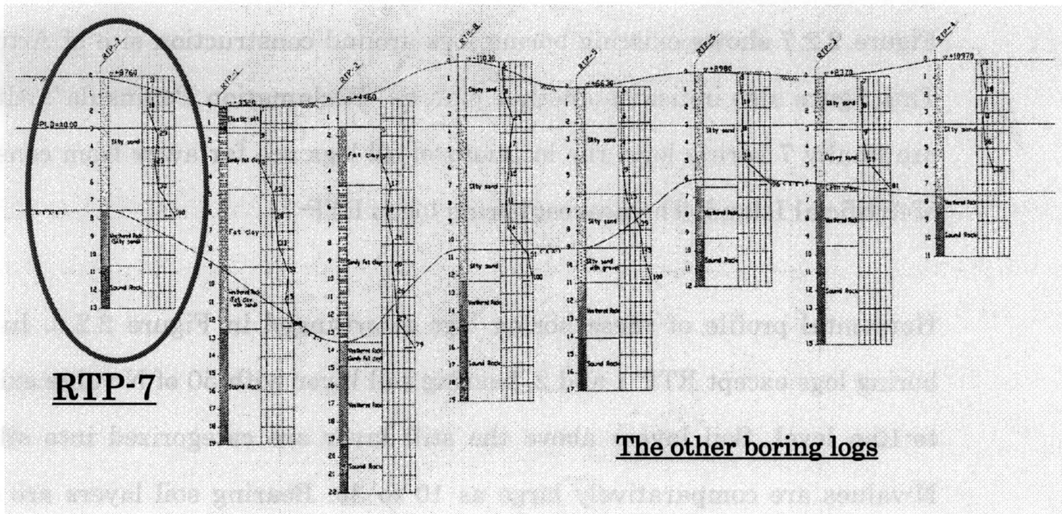


Figure 2.2.8 Profile of Existing Boring Logs

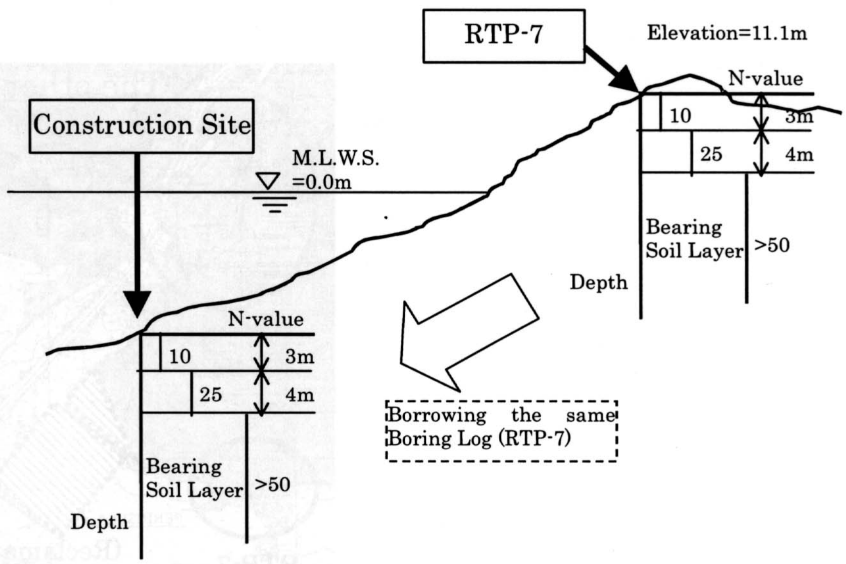


Figure 2.2.9 Assumption of Soil Condition at Construction Site

Design N-value for construction site of Artificial Island is illustrated in Figure 2.2.9. Consequently, the nearest boring log "RTP-7" is borrowed as in-situ boring log for both of Artificial Island and Reclamation Peninsula in consideration. Below the seabed, N-value is assumed in design modeling as 10 for first silty sand (3m thick), 25 for next silty sand (25m thick) and then 50 for weathered rock (as bearing soil layer). Since this assumption is extremely rough, boring exploration must be carried out during full scope feasibility study in the future.

(3) Earthquake

Technical standards for buildings in Panama includes the map of " Coefficient of Maximum Acceleration relative to Velocity for the Republic of Panama, REP-94, January 1994" as shown in Figure 2.2.10. According to this Figure, maximum acceleration relative to velocity (Av) accounts for 0.10 at the construction site of Artificial Island.

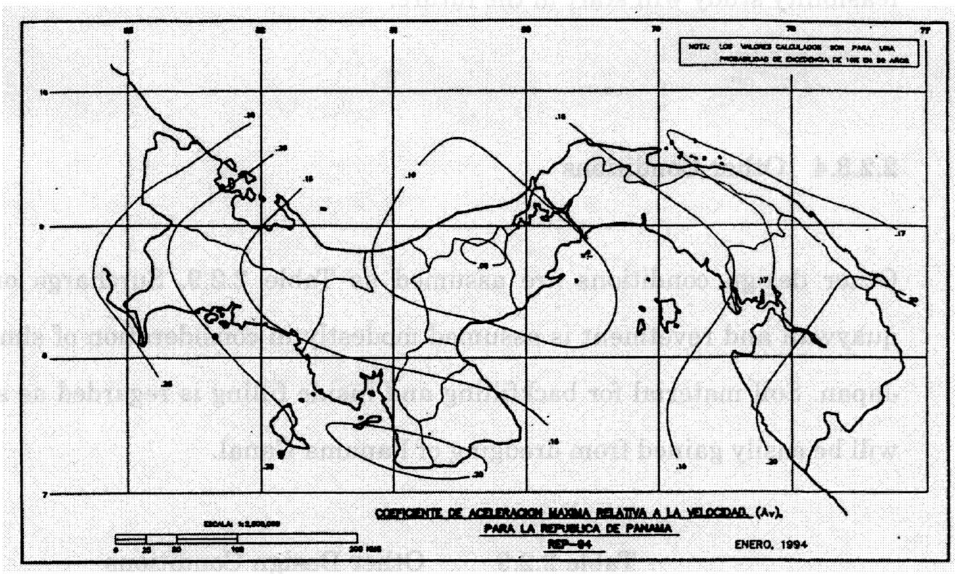


Figure 2.2.10 Maximum Acceleration relative to Velocity for the Republic of Panama

Coefficient of base shear (C_s) , which means the ration of horizontal force to dead load of structure, is calculated by the following equation:

$$C_s = 2.5\, A_a / R \le 2.5\, A_v / R \qquad (2.2\text{-}3)$$

where A_a : Effective peak acceleration

A_v : Maximum acceleration relative to velocity

R : Response modification factor

Since there is no information in terms of response modification factor R for waterfront infrastructures, at this moment, R is assumed as 2.5. The value of 2.5 is not so large in buildings whose range of R distributes from 1.25 to 8. By calculating the equation (2.2-3), the coefficient of base shear (seismic coefficient) is set at 0.1 consequently in this study.

However, the earthquake intensity around construction site should be evaluated in detail and the seismic coefficient should be decided more carefully when full scope feasibility study will start in the future.

2.2.3.4 Other Conditions

Other design conditions are assumed as Table 2.2.9. Surcharge on the crown of quaywall and revetment is assumed modestly in consideration of similar facilities in Japan. Soil material for backfilling and inside filling is regarded as stiff rock, which will be easily gained from dredging of Panama Canal.

Table 2.2.9 Other Design Conditions

<table>
<tr><th colspan="3">Items or Parts</th><th>Artificial Island</th></tr>
<tr><td colspan="2" rowspan="3">Surcharge on Quaywall and Revetment</td><td>During Construction (Wave Attack)</td><td>$0kN/m^2$</td></tr>
<tr><td>After Construction (Ordinary)</td><td>$20kN/m^2$</td></tr>
<tr><td>After Construction (Earthquake)</td><td>$10kN/m^2$</td></tr>
<tr><td rowspan="3">Soil Materials (Backfilling and Inside Filling)</td><td colspan="2">Angle of internal friction</td><td>40°</td></tr>
<tr><td colspan="2">Specific weight above residual water level</td><td>$18kN/m^3$</td></tr>
<tr><td colspan="2">Specific weight below residual water level</td><td>$10kN/m^3$</td></tr>
</table>

2.2.3.5 Loading Cases

The design cases of loading condition are shown in Table 2.2.10. Design case is divided into two categories of during construction and after construction. The design case during construction is that wave attacks sheet pile cellular-bulkhead quaywall when inside filling is just completed. The structures have to withstand individually against wave attack (design wave height of 2.6m).

On the other hand, the most critical design case after construction assumes when dredging and backfilling are finished. Ordinary case, in which sheet pile cellular-bulkhead quaywall is subjected to dead load, surcharge and earth pressure, is considered in structural design. Furthermore, seismic force is also considered in the design as earthquake case.

Table 2.2.10 Design Cases of Loading Condition

Design Case (Period in consideration)	Loading Condition	Design Execution
During Construction	Wave Attack	Considered
After Construction	Ordinary	Considered
	Earthquake	Considered

For example, Figure 2.2.11 illustrates structural modeling during construction and after construction in case of the existing water depth of 12.0m. In the case during construction, the design situation is that wave attacks sheet pile cellular-bulkhead quaywall or double sheet pile wall when water rises to M.H.H.W.. As total construction period of the structures will be estimated roughly as 2 years or less, the design wave height is assumed as 2.6m based on consideration of relationship between return period and significant wave height in Table 2.2.7.

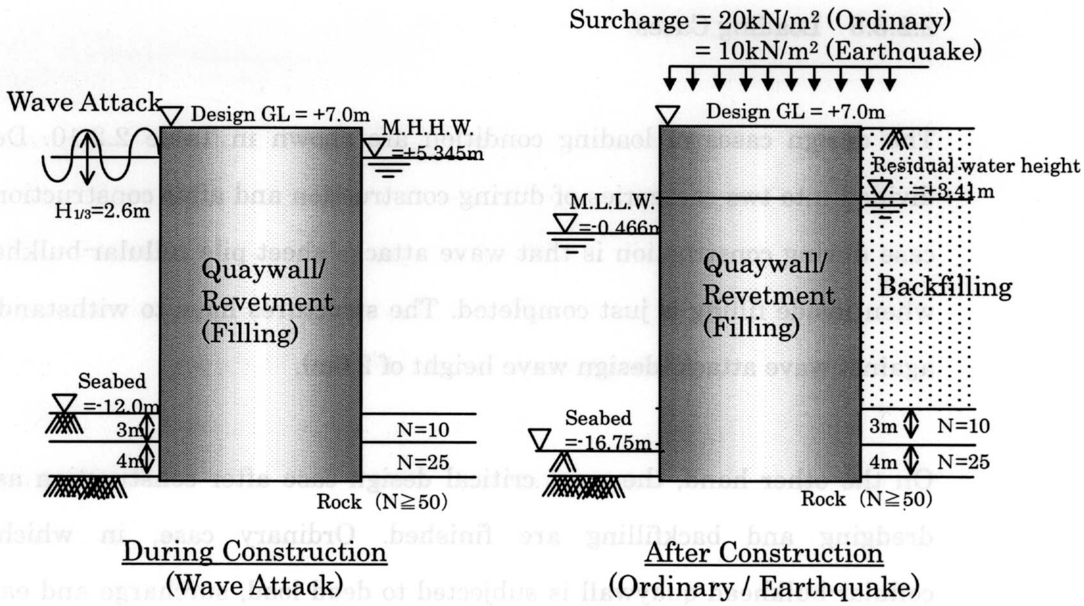


Figure 2.2.11 Structural Modeling for Loading Cases

Empty condition of cellular-bulkhead quaywall and double sheet pile wall is not considered in the design. Inside filling should be carried out fastly in calm weather because empty condition of the structures is quite unstable against storm wave. Inside filling needs only 1 to 2 days for one sheet pile cellular-bulkhead quaywall.

In the case of after construction, sheet pile cellular-bulkhead quaywall or double sheet pile wall is subjected to earth pressure from backfilling and surcharge. Surcharge is set at $20kN/m^2$ in ordinary case and $10kN/m^2$ in earthquake case. Water level is set at M.L.L.W. in front of the structure and residual water level in the structure and backfiling. It is noted that the seabed in front of guaywall is dredged from existing water level to berth (design) water level of M.L.W.S. -16.75m.

2.2.4 Design Method of Steel Sheet Pile Cellular-bulkhead Quaywall

Design flowchart for sheet pile cellular-bulkhead quaywall is introduced in Figure 2.2.12. Each design step is explained briefly in this section.

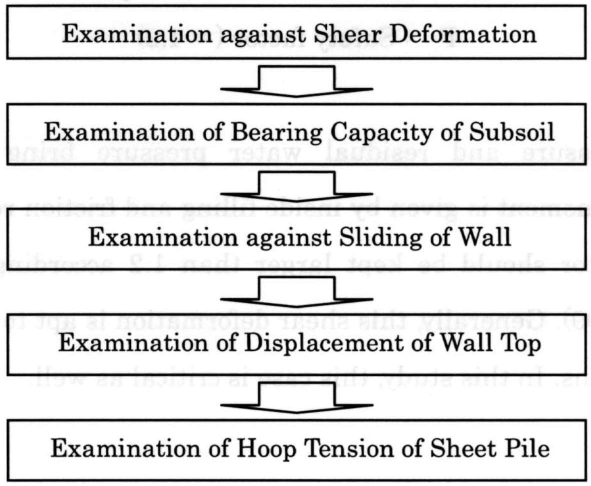


Figure 2.2.12 Design Flowchart for Sheet Pile Cellular-bulkhead Quaywall

(1) Examination against Shear Deformation

Firstly, steel sheet pile cellular-bulkhead quaywall should keep the initial shape from illegal deformation as shear deformation or bending. Figure 2.2.13 illustrates the design model for shear deformation.

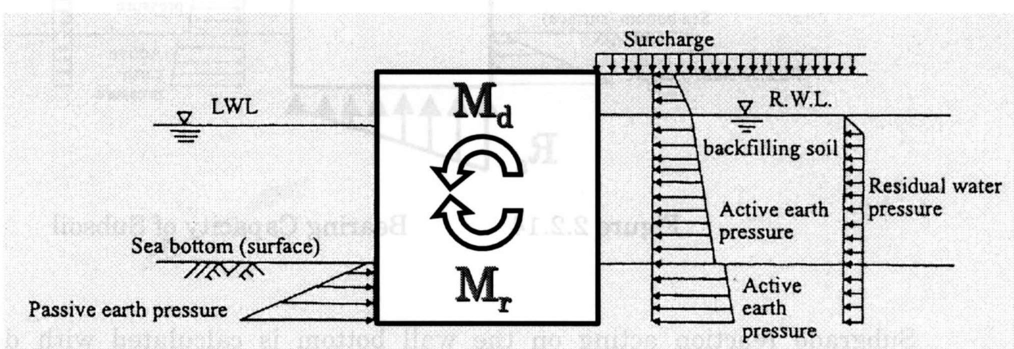


Figure 2.2.13 Shear Deformation

Resisting moment should be larger than deformation moment as the following equation:

$$M_r > M_d\, F \qquad (2.2\text{-}4)$$

where M_r : Resisting moment (Filling , Friction force of sheet pile joint)

M_d : Deformation moment (Earth pressure , Residual water pressure)

F : Safety factor (= 1.2)

Earth pressure and residual water pressure bring out deformation moment. Resisting moment is given by inside filling and friction resistance in sheet pile joints. Safety factor should be kept larger than 1.2 according to Technical Standards in Japan (1999). Generally, this shear deformation is apt to be the most critical in shape deformations. In this study, this case is critical as well.

(2) Examination of Bearing Capacity of Subsoil

Secondly, bearing capacity should be examined as illustrated in Figure 2.2.14. This is one of the evaluations for global stability.

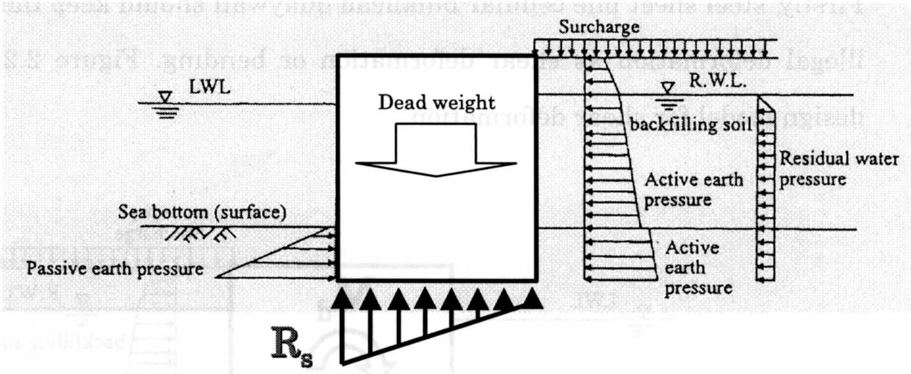


Figure 2.2.14 Bearing Capacity of Subsoil

Subgrade reaction acting on the wall bottom is calculated with dead weight of cellular-bulkhead quaywall combined by the deformation moment explained in previous page. Safety factor is set at 3 as shown in the following:

$$B_c > R_s\, F \qquad (2.2\text{-}5)$$

where B_c : Bearing capacity

R_s : Subgrade reaction acting on the wall bottom

(Dead weight , Earth pressure , Residual water pressure)

F : Safety factor (= 3)

In this study, this examination cannot be any serious problems because bearing soil layer has a sufficient bearing capacity.

(3) Examination against Sliding of Wall

Another examination in terms of global stability is against sliding of wall as shown in Figure 2.2.15.

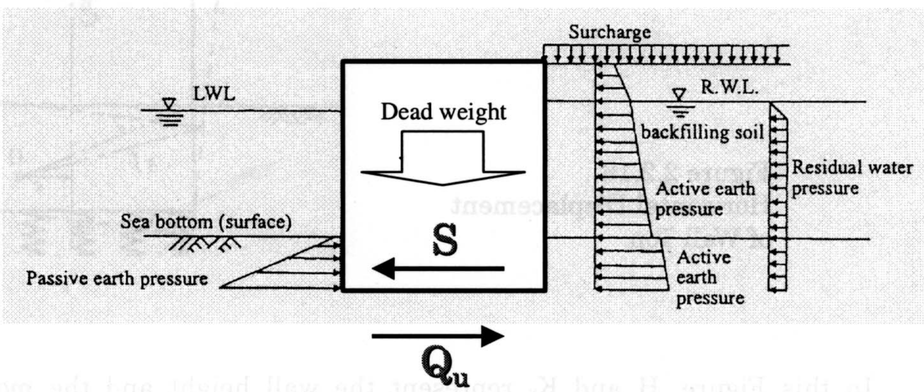


Figure 2.2.15 Sliding of Wall

Shear resistance force given from friction between the wall and the ground should be larger than acting shear force. Safety factor should be kept larger than 1.2 as the following:

$$Q_u > S\, F \qquad (2.2\text{-}5)$$

where Q_u : Shear resistance force acting between the wall and the ground

S : Shear force acting between the wall and the ground

(Earth pressure , Residual water pressure)

F : Safety factor (= 1.2)

(4) Examination of Displacement of Wall Top

Next, horizontal displacement of wall top is examined. Horizontal displacement in quaywall's rotation is brought out by earth pressure, waves and earthquakes. Rotational displacement is sustained by the subgrade. Horizontal displacement can be calculated in spring model with coefficient of subgrade reaction as shown in Figure 2.2.16.

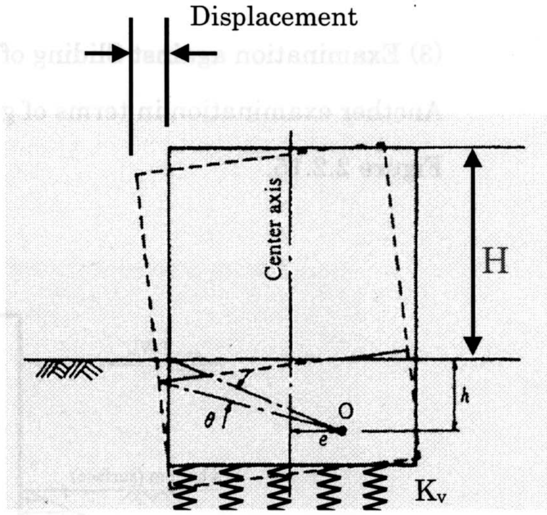


Figure 2.2.16 Horizontal Displacement of Wall Top

In this Figure, H and K_v represent the wall height and the modulus of vertical subgrade reaction, respectively. Allowable horizontal displacement is often set to 1.5% of wall height and 20cm empirically in Japan as:

Displacement < 0.015 H (Structural stability : 1.5%) (2.2-7)

< 20 cm (Safety of approaching) (2.2-8)

As a result of calculation, the horizontal displacement accounts for 1 or 2 cm only in this study because of no earthquakes in Panama and bearing soil layer.

(5) Examination of Hoop Tension of Sheet Pile

Last examination is hoop tension of sheet pile as illustrated in Figure 2.2.17. As shown in the left figure, large earth pressure acts on the bottom of quaywall. As shown in the right figure, the hoop tension occurs in circumferential direction.

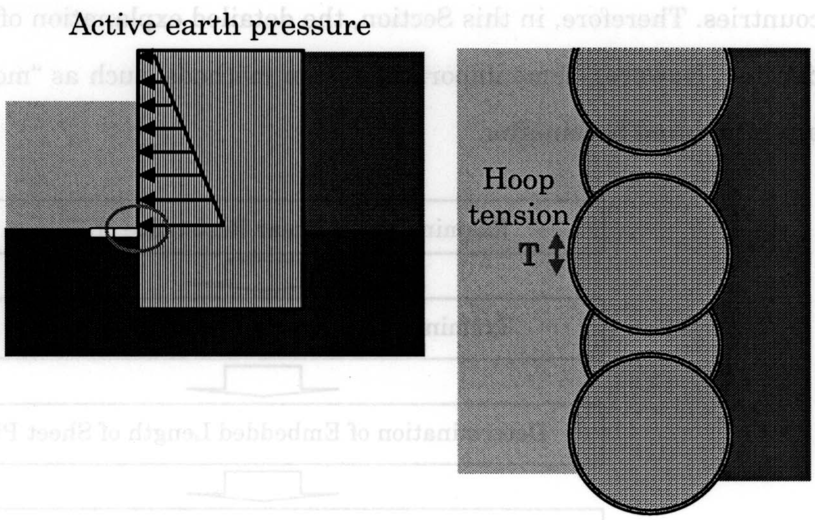


Figure 2.2.17 Hoop Tension of Sheet Pile

Steel sheet pile should resist against this hoop tension as follows:

$$T_a > T \tag{2.2-9}$$

where T_a : Allowable sheet pile tension value

T : Hoop tension of sheet pile

Tensional strength of joints of steel sheet pile produced in Japan is designed stronger than full-sectional strength of steel web.

2.2.5 Design Method of Double Steel Sheet Pile Wall

Design flowchart for double sheet pile wall is introduced in Figure 2.2.18. Each design step is explained briefly in this section. Comparing with sheet pile cellular-bulkhead quaywall, double sheet pile wall may be more popular in many countries. Therefore, in this Section, the detailed explanation of the design method is omitted. However, some important design methods, such as "modified Row's method" are introduced hereinafter.

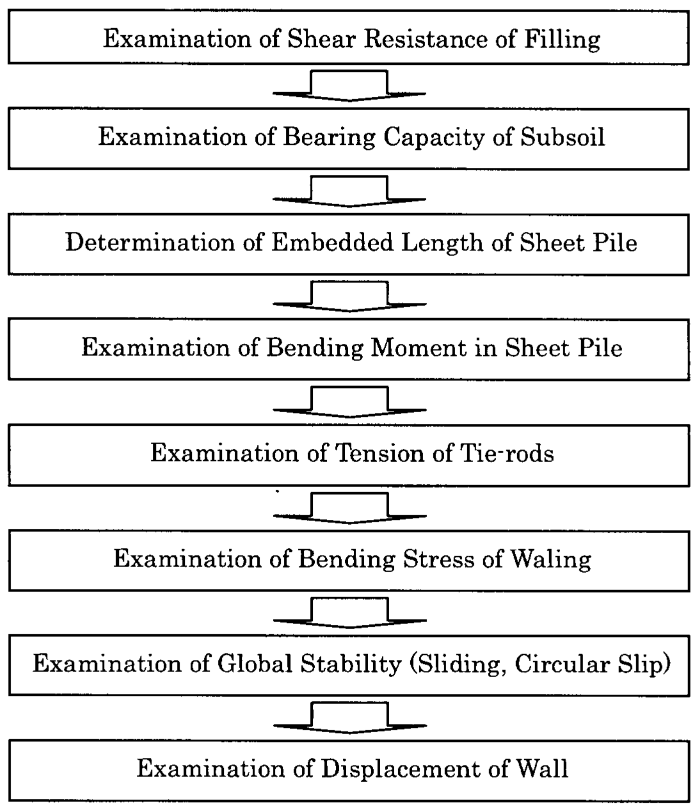


Figure 2.2.18 Design Flowchart for Double Sheet Pile Wall

(1) Free Earth Support Method to calculate Embedded Length of Sheet Pile

The mechanical behavior of sheet pile wall changes according to the embedded length. Among several design methods, the "free earth support method" based on a classic earth pressure has been traditionally used to determine the embedded length. The following equation should be satisfied for between the moment caused by passive earth pressure and the moment caused by active earth pressure.

$$M_P = M_A F \tag{2.2-10}$$

where M_P : Moment at the tie-rod setting point by the passive earth pressure

M_A : Moment at the tie-rod setting point by the active earth pressure and residual water presure

F : Safety factor

(2) Equivalent Beam Method to calculate Bending Moment of Sheet Pile

In the past, as design water depths for quaywall were relatively shallow, the bending moment reflection to calculate the inflection point of bending moment was located around the seabed when used in sandy soil ground with a medium or high degree of compaction. Therefore, simplified classical method called as 'the equivalent beam method' had been used. This method is so simple to calculate the beam assumed as suspended by tie-rod setting point and seabed level. However, if designed with the equivalent beam method, the bending moment inflection point may be located below seabed level and the section forces may be underestimated

(3) Modified Row's method

The free earth support method and the equivalent beam method have been popularly used because of its ease of use. However, this is not a design method that takes into consideration of the cross-sectional rigidity of the sheet pile.

Therefore, Row's method is well known to regard the embedded part of sheet pile as a beam set on an elastic media. The basic equation for the embedded part is as follows:

$$EI\ d^4y/dx^4 = P(x) = P_{A0} - (l_h/D)\ xy \tag{2.2-11}$$

where E : Young's modulus of sheet pile

I : Geometric moment of inertia of sheet pile wall per unit width

P_{A0} : load intensity at seabed generated by the active earth pressure and residual water pressure

l_h : Coefficient of subgrade reaction to the sheet pile wall

D : Penetration depth of sheet pile

Technical Standards in Japan (1999) recommends to compare between free earth support method and modified Row's method and chose the safer result. Modified Row's method is proposed by Takanashi *et al*, in which earth pressure distribution for analysis of sheet pile wall is illustrated in Figure 2.2.19. Based on Row's method, they amended this method to better reflect the behavioral characteristics of actual sheet pile walls as the following equation:

$$EI\, d^4y/dx^4 = P(x) = P_{A0} + K_{AD}\,\gamma\, x - K_0\,\gamma\, x - [(l_h / D_F\, r_f)]\, xy \qquad (2.2\text{-}12)$$

where K_{AD}: Coefficient of active earth pressure in the embedded part of sheet pile

γ : Unit weight of soil

K_0 : Coefficient of earth pressure at rest

D_F : Converged embedded length of sheet pile wall

r_f : Ratio of the exerting depth of the primary positive reaction earth pressure acting on the front surface of the embedded part of sheet pile to D_F

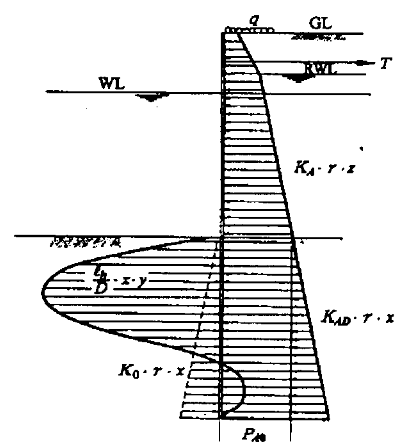


Figure 2.2.19
Earth Pressure Distribution for Analysis of Sheet Pile Wall

In practical design by using modified Row's method, some modifying coefficients, such as similarity number and flexibility number, are used for evaluate the embedded length and bending moment of sheet pile.

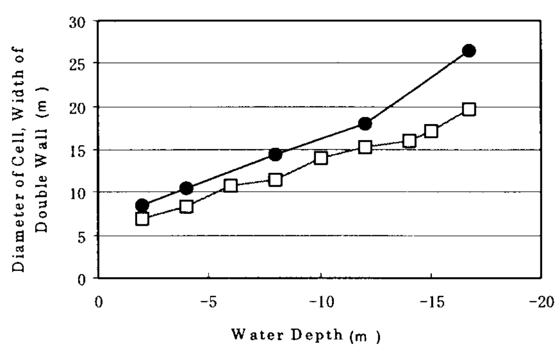
2.2.6 Best Selection of Structural Types

Structural basic design was executed for sheet pile cellular-bulkhead quaywall and double sheet pile wall. To select the better structural type from these structural types, comparative evaluation is effective particular in terms of construction cost (quantity).

In the design of both structures, the design water depth was changed from –2m to –17m to evaluate the effect of water depth on the construction quantity. The results of these parametric designs are shown in Figure 2.2.20.

The left Figure shows calculated relationship between design water depth and diameter of cellular-bulkhead quaywall or width of double sheet pile wall. For all depth range from –2m to –17m, diameter of cellular-bulkhead quaywall is shorter than width of double sheet pile wall at each water depth.

Relationship between design water depth and weight of steel sheet pile per unit length (m) of quaywall is shown in the right Figure. In shallower depth range than –5m, the quantity of steel sheet pile is larger in cellular-bulkhead quaywall than in double sheet pile wall. However, in the depth range deeper than –5m, cellular-bulkhead quaywall needs smaller quantity of steel materials than double sheet pile wall.



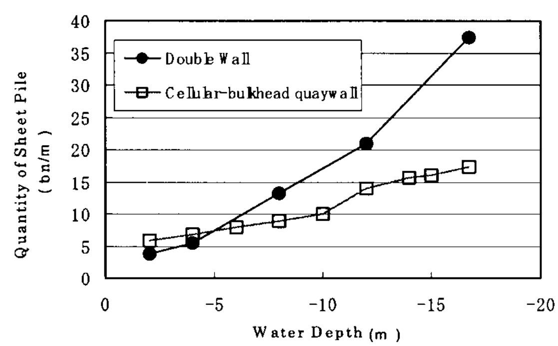


Figure 2.2.20 Comparison of Construction Quantity between Sheet Pile Cellular-Bulkhead Quaywall and Double Sheet Pile Wall

In case of Artificial Island in consideration, the design water depths are –16.75m for quaywall side and –12.0m for revetment side. Evaluating Figure 2.2.20, it can be concluded that the sheet pile cellular-bulkhead quaywall is cheaper than the double sheet pile wall in construction of Artificial Island. Therefore, sheet pile cellular-bulkhead quaywall is strongly recommended as the most suitable structural type for quaywall and revetment in Artificial Island.

2.2.7 Design Results of Quaywall and Revetment in Artificial Island

2.2.7.1 Design Results of Sheet Pile Cellular-bulkhead Quaywall

The designed results for steel sheet pile cellular-bulkhead quaywall are shown in Table 2.2.11 for quaywall side and Table 2.2.12 for revetment side, respectively.

As shown in these two Tables, the design water depth is different from quaywall side and revetment side. Water depth at quaywall side is dredged from -12m to -16.75m. As a result of design, the diameter of cell lead to 21.0m for quaywall side and 15.3m for revetment side, respectively. Steel material YSP-FXL SY390 for quaywall is stronger in tension than YSP-FXL SY295 for revetment side. Total weight of sheet pile of both quaywall side and revetment side accounts for 71,293 ton. For both sides, the most critical design values were derived from shear deformation and hoop tension in sheet pile. Extraordinary cases are governed by earthquake case, but not by wave attack, for both quaywall side and revetment side.

Table 2.2.11 Design Results of Sheet Pile Cellular-bulkhead Quaywall for Quaywall Side

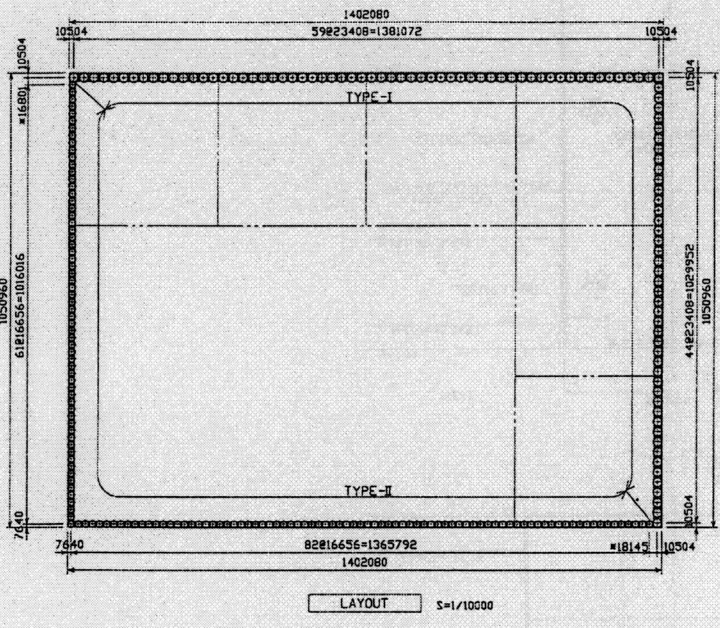
Wall Side		**Quaywall Side**	
Condition	Existing Water Level	M.L.L.W.-12m	
	Design Water Depth	M.L.L.W.-**16.75m**	
Specification	Steel Sheet Pile	YSP-FXL **SY390**	
	Diameter of Cell	**21.0m**	
	Installment Pitch of Cell	**23.4m**	
Total Size	Total Wall Length	**2,411m**	
	Total Weight of Sheet Pile	**35,760 ton**	
Loading Case (*: earthquake or wave attack)		Ordinary	Extraordinary*
Design Values	Shear Deformation	**1.33** > 1.20 OK	-
	Bearing Capacity	541 kN/m² < 4,406 OK	920 kN/m² < 7,327 OK
	Sliding	2.70 > 1.20 OK	1.56 > 1.0 OK
	Displacement of Wall Top	10mm < 356 OK	28mm < 356 OK
	Hoop Tension in Sheet Pile	**2,620 kN/m** < 2,750 OK	**2,541 kN/m** < 2,750 OK

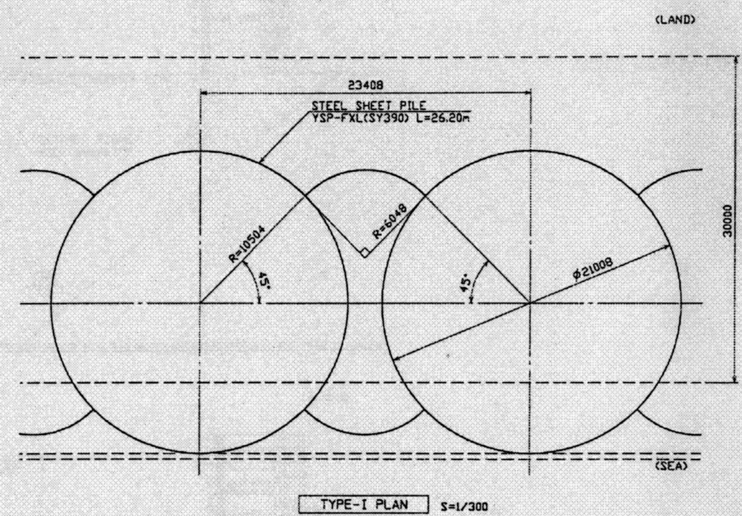
Table 2.2.12 Design Results of Sheet Pile Cellular-bulkhead Quaywall for Revetment Side

Wall Side		Revetment Side	
Condition	Existing Water Level	M.L.L.W.-12m	
	Design Water Depth	M.L.L.W.-**12.0m**	
Specification	Steel Sheet Pile	YSP-FXL **SY295**	
	Diameter of Cell	**15.3m**	
	Installment Pitch of Cell	**16.7m**	
Total Size	Total Wall Length	**2,382m**	
	Total Weight of Sheet Pile	**35,533 ton**	
Loading Case (*: earthquake or wave attack)		Ordinary	Extraordinary*
Design Values	Shear Deformation	**1.36** > 1.20 OK	-
	Bearing Capacity	679 kN/m^2 < 4,953 OK	2,108kN/m^2 < 8,208 OK
	Sliding	3.07 > 1.20 OK	2.28 > 1.0 OK
	Displacement of Wall Top	19mm < 285 OK	168mm < 285 OK
	Hoop Tension in Sheet Pile	**1,798 kN/m** < 2,106 OK	**1,736 kN/m** < 2,106 OK

Finally, the design drawing for plan of quawall side and revetment side is shown in Figure 2.2.21. The vertical sections of sheet pile cellular-bulkhead quaywall are drawn in Figure 2.2.22. In these drawings, TYPE-Ⅰ and TYPE-Ⅱ mean quaywall side and revetment side, respectively. Inside filling is made of crushed stone which can be obtained from dredging Panama Canal. Steel sheet piles should be driven deeply into the weathered rock layer.

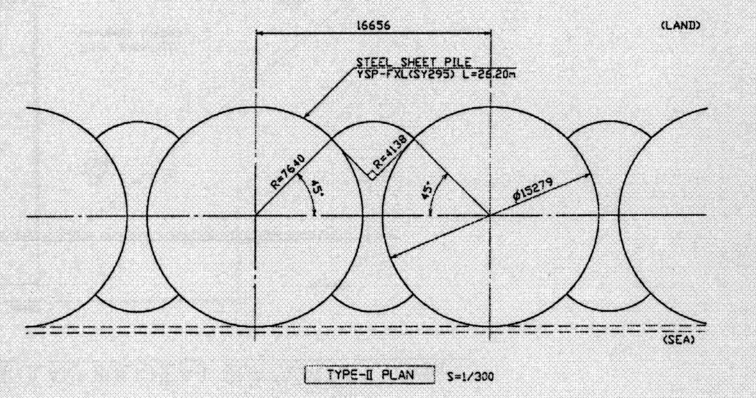
PREFABRICATED STEEL SHEET PILE CELLULAR-BULKHEAD QUAYWALLS





Quantity Calculation of Steel Sheet Pile

	Item	Type	Length (m)	No. of pieces	No. of cells	Unit Weight (kg/m)	Total Weight (ton)	Steel Grade
Sheet Pile								
TYPE-I	Main Body	YSP-FXL	26.200	128	104	77.2	26925.4	SY390
	Arc	YSP-FXL	26.200	36	104	77.2	7572.8	SY390
TYPE-II	Main Body	YSP-FXL	26.200	92	144	77.2	26795.9	SY295
	Arc	YSP-FXL	26.200	24	144	77.2	6990.2	SY295
							68284.3 t	
Joint-Sheet Pile								
TYPE-I	Main Body	YSP-FXL	26.200	4	104	115.8	1262.1	SY390
TYPE-II	Main Body	YSP-FXL	26.200	4	144	115.8	1747.8	SY295
							3009.7 t	
						Sheet Pile	68284.3 t	
						Joint-Sheet Pile	3009.7 t	
						Total Weight	71294.0 t	



PZ-C1PLA.DWG

Figure 2.2.21 Plan of Steel Sheet Pile Cellular-bulkhead Quywall in Artificial Island

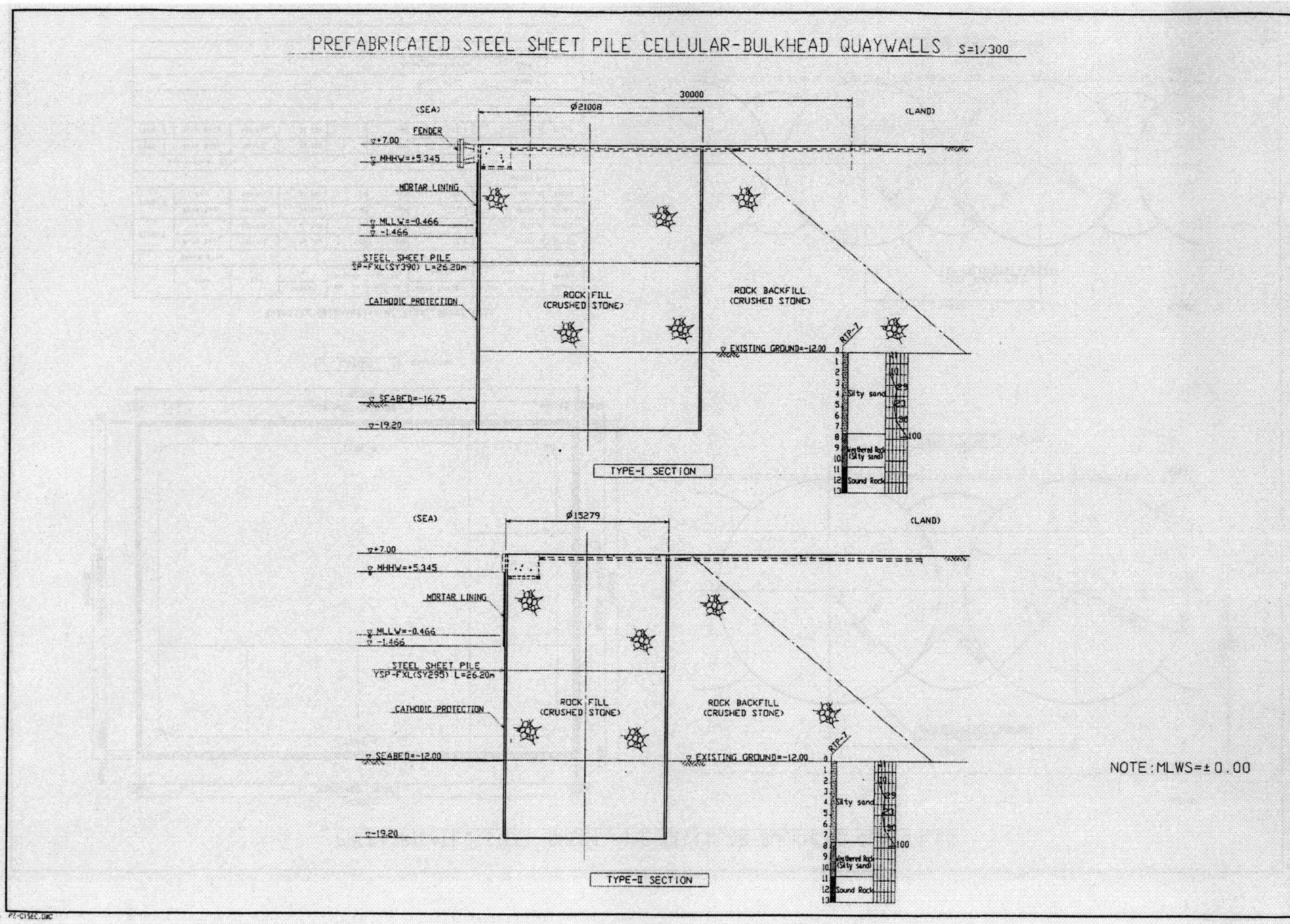


Figure 2.2.22 Section of Steel Sheet Pile Cellular-bulkhead Quywall in Artificial Island

2.2.7.2 Anti-corrosion for Steel Sheet Pile

Steel material is very durable in strength but not in corrosion. Countermeasure as anti-corrosion should be considered for steel sheet pile. Figure 2.2.23 shows the anti-corrosion plan for steel sheet pile in this study. Two major countermeasures are introduced as concrete lining and cathodic protection.

Concrete lining covers and protects the surface of steel sheet pile above level of –1m, including splash zone. This is because the splash zone is apt to be not only corroded but also injured by flowing objects as wooden piece and disposal wastes.

In the seawater under the level of -1m, cathodic protection with aluminum alloy anodes is suitable to keep the steel material from corrosion. It is noted that aluminum alloy anodes must be renewed regularly, according to erasing speed of alloy.

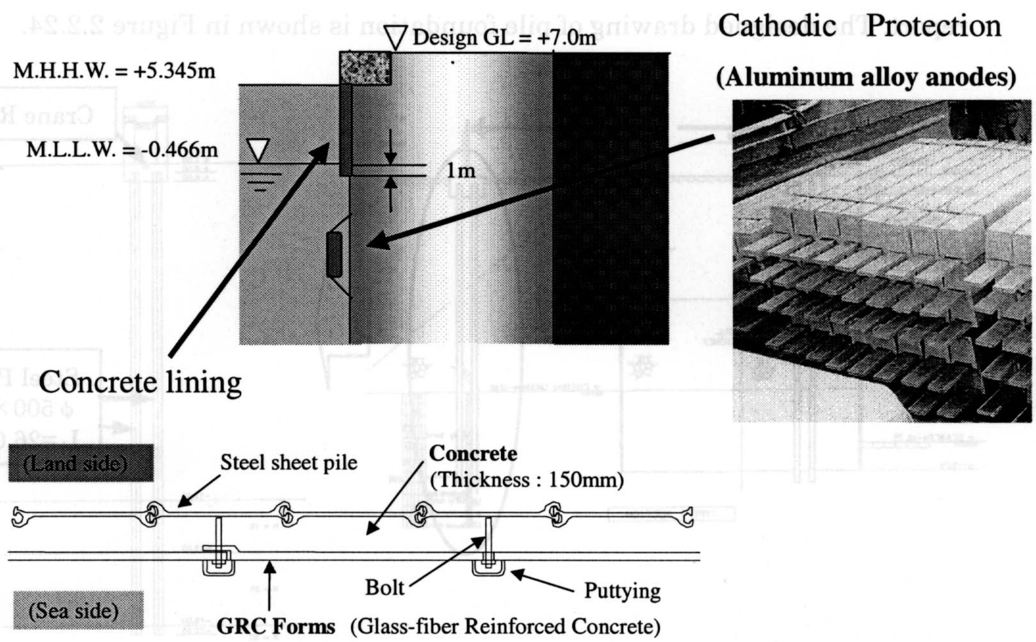


Figure 2.2.23 Anti-corrosion for Steel Sheet Pile

2.2.8 Pile Foundation of Gantry Crane

This Section explains briefly the design of gantry crane for quaywall side as berth facility. The loading condition of gantry crane is borrowed from existing crane (carrying capacity of 30ton) in Japan and assumed as Table 2.2.13. Span of crane legs is planned as 30m.

Table 2.2.13 Loading Condition for Gantry Crane

	Sea Side		Land Side	
No. of Legs	2		2	
Travel Weels	6 wheels per leg		4 wheels per leg	
Loading Condition	Vertical Load	Horizontal Load	Vertical Load	Horizontal Load
In Working	323kN	5.9kN	284kN	8.8kN
Storm	265kN	39.2kN	353kN	59.8kN
Earthquake	353kN	38.2kN	480kN	56.8kN

Gantry crane is sustained by pile foundation. The steel pile (ϕ 500×9, L =26.0m) is used as the pile material. The detail of design method and design values are omitted in this report. The designed drawing of pile foundation is shown in Figure 2.2.24.

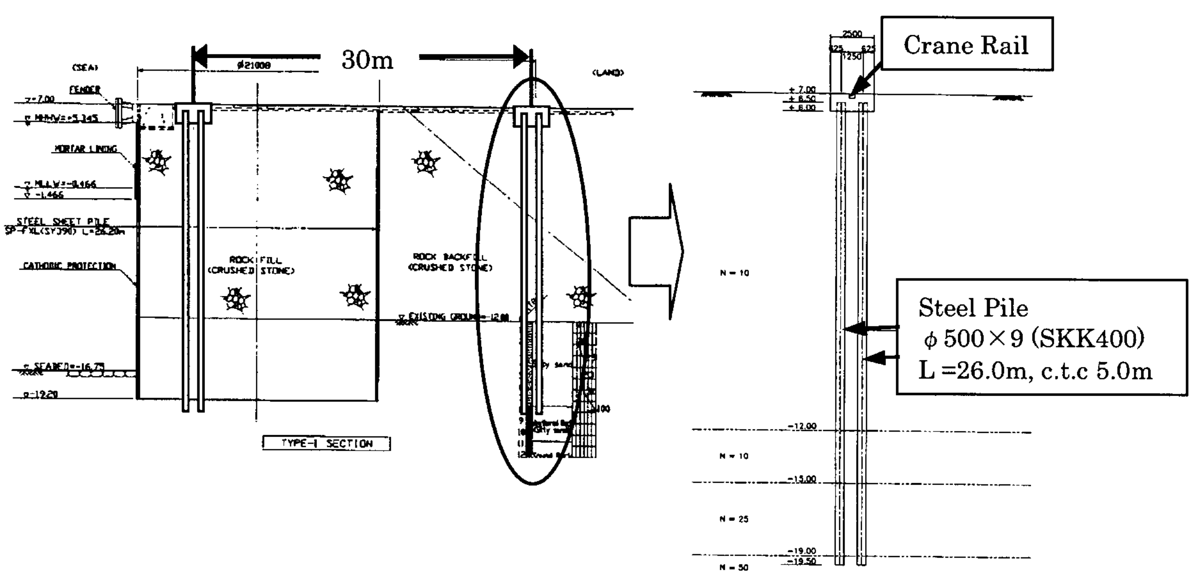


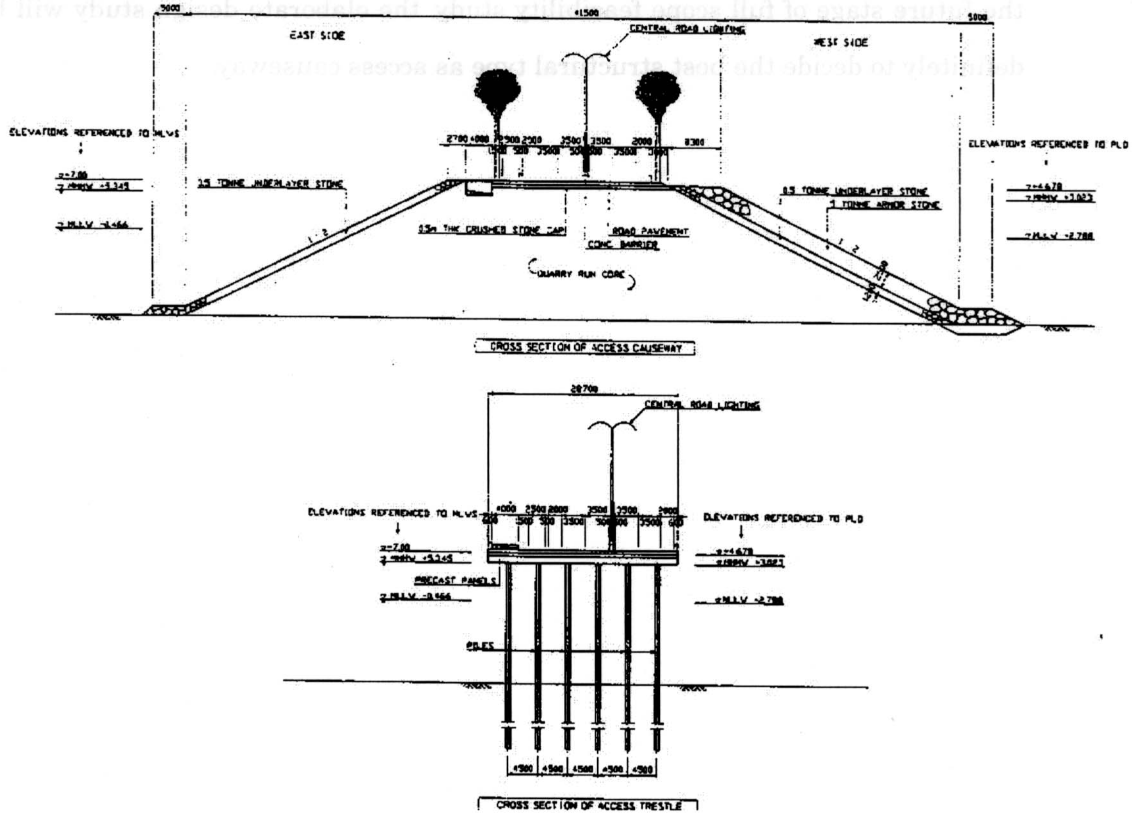
Figure 2.2.24 Pile Foundation of Gantry Crane in Quaywall Side

Additional Notes:

As mentioned previously, the structural design of access causeway is out of scope of this study. Whereas, the typical cross sections of access causeway and trestle are illustrated in Additional Figure A2.2.1. This specification is fundamentally borrowed from ACP Report (2001), "Preliminary Study of island Development at the Pacific Entrance of the Panama Canal - Final Report". However, this slope embankment may give mal function to environment because of shutting-out of the existing tidal flow. Another structure, such as continuous bridges, may be more desirable to minimize the environmental impact. In the future stage of full scope feasibility study, the elaborate design study will be needed definitely to decide the best structural type as access causeway.

Addition 2.2.1 Cross Sections of Access Causeway and Trestle

The typical cross sections of Access Causeway and Trestle are shown in Additional Figure A2.2.1. These sections were not decided from the study in this report due to out of this study's scope. Fundamental specification is just borrowed from ACP Report (2001), "Preliminary Study of island Development at the Pacific Entrance of the Panama Canal - Final Report".



Additional Figure A2.2.1 Typical Cross Sections of Access Causeway and Trestle

2.3 Construction Plan of an Artificial Island

The methodology of this plan has considered the basic requirements for suitable facility providing conveniences for fabricating cell and the ease of related activities for the success of this plan. Some of these factors include the water depth to accommodate the vessel draft, design length of the waler guide frame and transportation of fabricated cell.

2.3.1 Flowchart of the Steel Sheet Pile Cellular-bulkhead Wall Construction

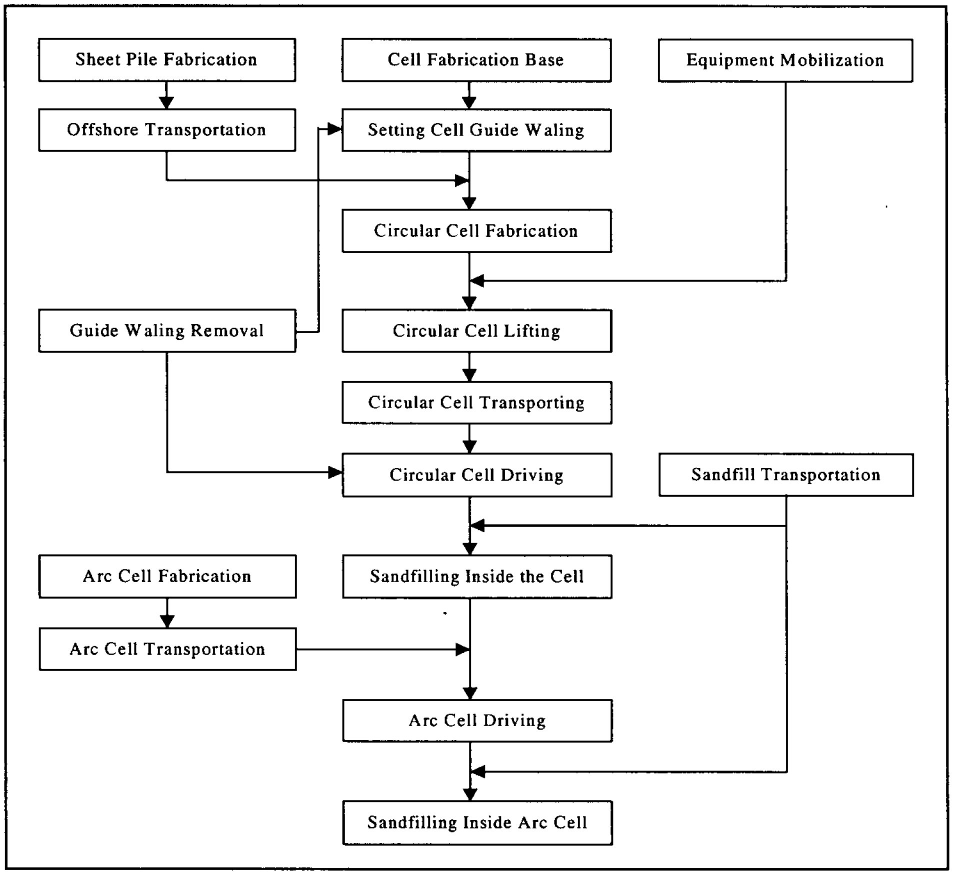


Figure 2.3.1 Flowchart of the Steel Sheet Pile Cellular Wall Construction

2.3.2 General Methodology of Constructing Steel Sheet Pile Cellular-bulkhead Wall

A. Construction of Temporary Loading Jetty

The construction temporary loading jetty as shown in Figure 2.3.2 will be located near the shore of the New Locks sites. Its size will depend on the water depth to suitably accommodate the floating barge for the collection and transportation of sand to the artificial island site. The condition of the prevailing navigational traffic at the channel is also a factor in choosing the appropriate site of this temporary jetty.

Figure 2.3.2 Picture of Temporary Loading Jetty

B. **Construction of Offshore Cell Fabrication Base**

Steel pipe piles with suitable size will be driven as foundation for the tower crane installation. The fabricated crane post and the crane unit will be installed on the driven pile head that treated with concrete base coping.

Another steel pipe piles will be driven within the fabrication area to serve as the working platform for the waler guide frame. Base plates of ring shape that conforms to the circular size of the waler guide frame will be installed on the driven pile head. The circular waler guide frame will be pre-fabricated ready for use at the fabrication base.

C. **Special Equipment for Circular Cell**

This equipment is a hanging type steel frame carrying the total weight of the fabricated cell, multiple vibro-hammer with chuck and the waler guide frame.

Each vibro-hammer with chuck installed in the steel frame has a capacity of driving 3 to 6 steel sheet piles. They are set in the same elevation of hanging wire of the hanging steel frame and each has the standard hammer capacity of 60 kw, and 90 kw for the T-type. Each steel sheet pile is fixed with safety pin to prevent from dropping down or disconnection to the waler guide frame. See Figure 2.3.3 showing the feature of vibro-hammer.

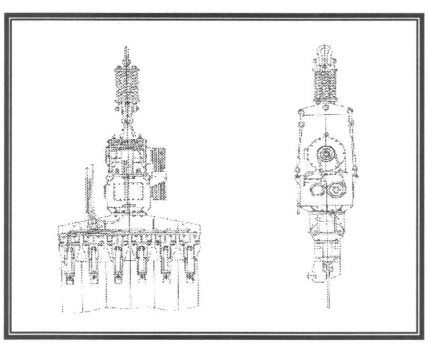


Figure 2.3.3 Feature of Vibro-Hammer

D. Fabrication of the Circular Cell

1) Steel sheet piles installation on the circular waler guide frame

The upper ring of circular waling guide frame is marked with uniform pitch for installation and adjustment of steel sheet piles. Each pile will be picked up from its tip using the hydraulic chuck and will be locked by safety pin on the waler guide frame. This work will be carried out with a rope to prevent the piles from swaying and rolling.

The steel sheet piles are installed in a single direction as aligned referring to the marking of the upper and lower ring of the waler guide frame. Sheet piles will be installed symmetrically against the center of the ring forming a single cell.

2) Towing of fabricated circular cell

After fabricating a single cell, it will be lifted up and towed by floating crane up to its designated location. The timing and method of towing the fabricated cell will be determined by the prevailing site condition, such as sea wave condition, etc. Moreover, proper communication and close coordination with the safety station authority and tower control personnel will be done.

3) Positioning / driving of circular cell

Nearing to reach its designed position, the cell will be lowered down up to 50 cm above the seabed. The multiple vibro-hammers will be pre-operated and checked the inclination of the waling guide frame.

After confirmation of the good condition of vibro-hammer, waling guide, steel sheet piles (i.e. verticality, twist and rolling, driving condition, etc.) and checking the water condition, the vibro-hammers operate to start the driving of the cell.

These vibro-hammers will be stopped at one time upon reaching the depth of about 4 to 5 m of the seabed, in order to allow the checking of the joint condition of the sheet piles. Thereafter, driving will be continued up to the designed depth.

Finally, the sheet pile head will be adjusted to the pre-indicated markings of designed elevation by operating the vibro-hammers one by one.

4) Removal of circular waler guide frame

All the safety locks and pins will be detached from driven sheet piles of the cell to release the hanging circular waler guide frame from the cell, then it will be lifted up to remove out from the driven cell.

A ring anchor will be installed on the top surface of the cell using the floating crane for sand filling work after the frame removal.

The pontoon will set back the frame at the fabrication base, to start the same cycle of work for fabricating, towing and driving the circular cell until all the required number of cell have been fabricated.

5) Sandfilling of the installed circular cell

Immediately after installation of the circular cell, sandfilling will be executed in order to secure the stability of the structure. Sandfilling operation will be in a uniform elevation and dropping at a minimum level starting from the center of the cell. Care shall be taken to avoid any deformation of the cell while performing the sand filling. See Figure 2.3.4 showing this operation

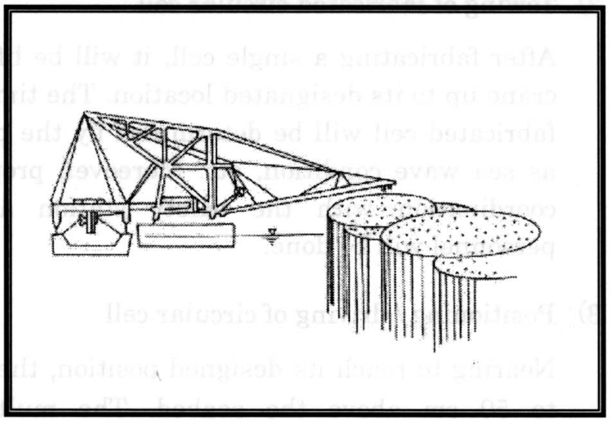


Figure 2.3.4 Sandfilling of the Cell by Reclaimer Barge

D. Fabrication of the Arc Cell

1) Fabrication of the arc cell

The basic method of fabricating arc cell is similar to the circular cell except for the shape. It will also re-use the circular waler guide frame for production of arc cell by modifying some parts to suit the arc form of the cell. In addition, installation would be quite different since the arc cell has to employ the fixing guide equipment in order to maintain its curvature during driving.

2) Installation of the arc cell

Fabricated arc cell will be set and installed in between the installed cellular cell. Prior to this work, a T-type steel pile will be driven in the two contact points of the arc cell and circular cell as basis for the arc cell connection. In addition, fixing guide equipment for arc cell will be provided to maintain its curvature during the driving operation. See Figure 2.3.5 showing the feature of fixing guide equipment for arc cell

In case of some difficulty in driving the arc cell up to its design depth by the installed multiple vibro-hammer, the sheet pile will be driven by hydraulic hammer or diesel hammer.

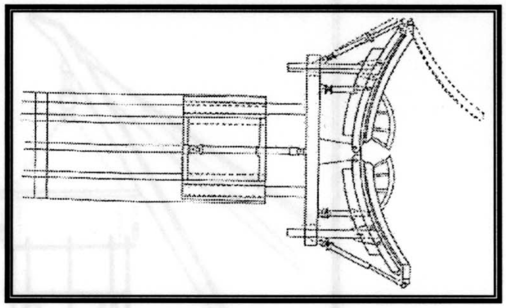
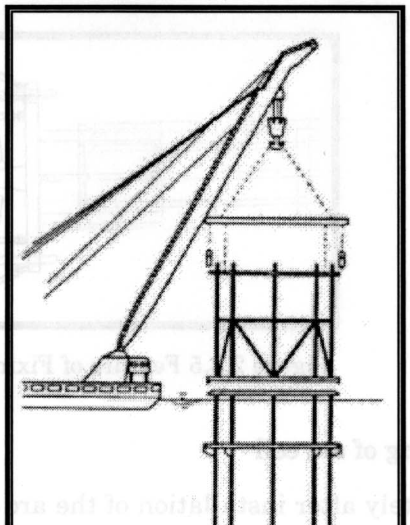


Figure 2.3.5 Feature of Fixing Equipment

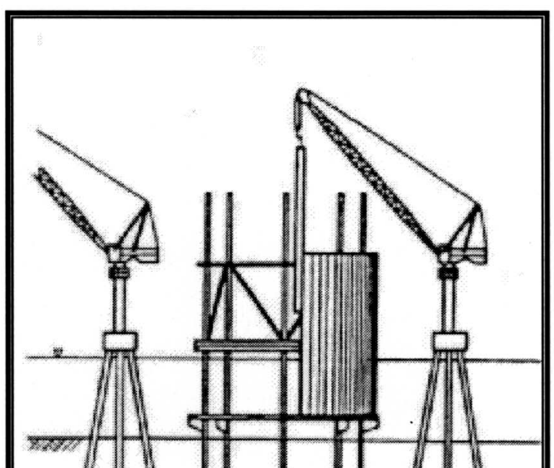
3) Sandfilling of arc cell

Immediately after installation of the arc cell, sandfilling will be executed in order to secure the stability of the structure. Sandfilling operation will be in a uniform elevation and dropping at a minimum level starting from the center of the cell. Care shall be taken to avoid any deformation of the cell while performing the sand filling.

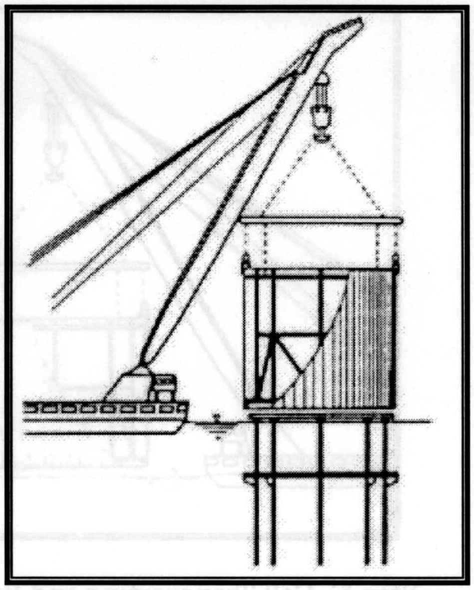
2.3.3 Picture presentation of Steel Sheet Pile Cellular Wall Construction



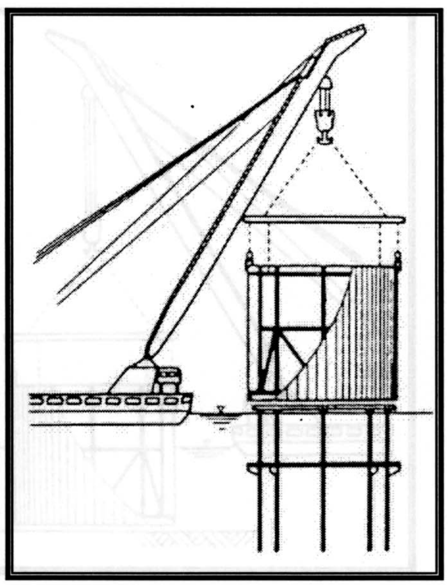
Step 1: Waling Guide Frame Setting



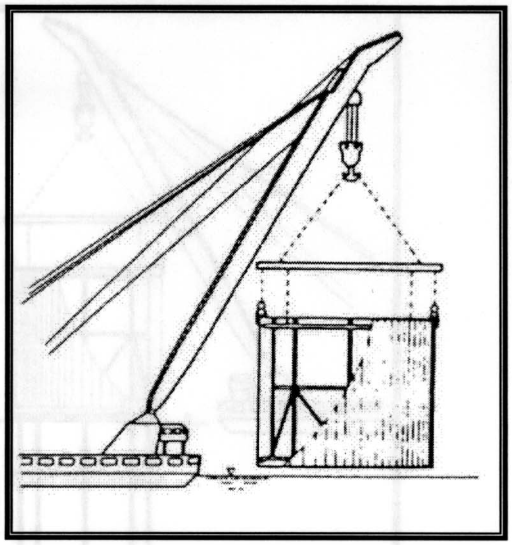
Step 2: Steel Sheet Piles Installation



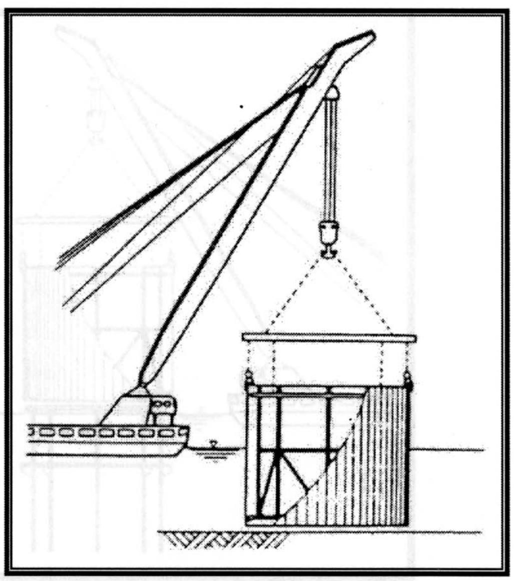
Step 3: Chucking



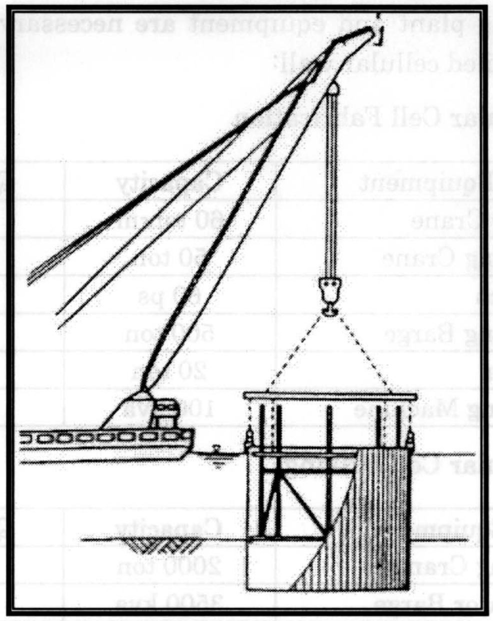
Step 4: Cell Lifting



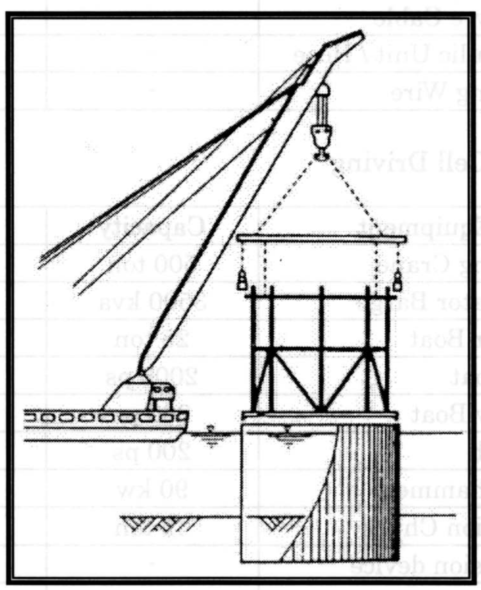
Step 5: Cell Transporting and Positioning



Step 6: Cell Driving Operation



Step 7: Cell Driving Completion



Step 8: Cell Waling Guide Frame Lifting

2.3.4 List of Plant and Equipment Required

The following plant and equipment are necessary for the construction of the steel sheet-piled cellular wall:

A. For Circular Cell Fabrication

Equipment	Capacity	Quantity
Tower Crane	60 ton-m	4 nos.
Floating Crane	50 ton	2 nos.
Launch	60 ps	2 nos.
Floating Barge	500 ton	2 nos.
Trailer	20 ton	2 nos.
Welding Machine	100 kva	2 nos.

B. For Circular Cell Driving

Equipment	Capacity	Quantity
Floating Crane	2000 ton	2 nos.
Generator Barge	3500 kva	2 nos.
Anchor Boat	50 ton	2 nos.
Tug Boat	2500 ps	2 nos.
Survey Boat	200 ps	2 nos.
Launch	200 ps	2 nos.
Generator	100 Kva	2 nos.
Vibro-Hammer	90 kw	52 nos.
Vibration Chuck	42 ton	128 nos.
Admission Device	-	1 Ls
Operation Board	-	1 Ls
Cab Tyre Cable	-	1 Ls
Hydraulic Unit / Hose	-	1 Ls
Hanging Wire	-	1 Ls

C. For Arc Cell Driving

Equipment	Capacity	Quantity
Floating Crane	500 ton	2 nos.
Generator Barge	3500 kva	1 nos.
Anchor Boat	25 ton	2 nos.
Tug Boat	2000 ps	2 nos.
Survey Boat	200 ps	2 nos.
Launch	200 ps	2 nos.
Vibro-hammer	90 kw	14 nos.
Vibration Chuck	42 ton	72 nos.
Admission device	-	1 Ls
Operation board	-	1 Ls
Cab Tyre cable	-	1 Ls
Hydraulic unit / hose	-	1 Ls
Hanging wire	-	1 Ls

2.3.5 Reclamation Work

Prior to commence the reclamation operation, the installed steel sheet pile cellular wall will be stabilized for cofferdam type quay wall of the artificial island. Slip circle analysis will be conducted for the stability of the ground soil in order to determine future occurrence of the soil slipping.

Based the result of the aforesaid analysis, the reclamation work shall be executed in layers progressively. Reclamation production is estimated at approximately 2,000,000 m^3/month.

A. Flowchart of the Reclamation Work

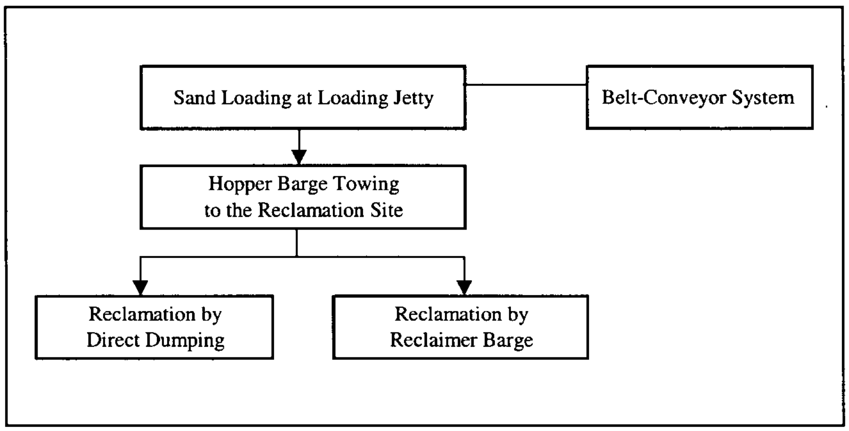


Figure 2.3.6 Flowchart of Reclamation Work

B. General Methodology of Reclamation Work

1) Sand loading and transporting to the reclamation site

Sand will be loaded onto hopper barge at the temporary jetty by conveyor system. As the hopper barge fully loaded, it will then be towed to the reclamation site offshore.

In this plan, the estimated frequency of transporting sand to the reclamation site is at 10~12 barges per day. Moreover, the draft of barge when fully loaded is at 5~6 meter.

2) Reclamation by Direct Dumping Using Hopper Barge

The hopper barge will directly dump its load with the use of DGPS positioning system, which enable to make accurate dumping at the designated location. See Figure 2.3.7 showing the operation of hopper barge.

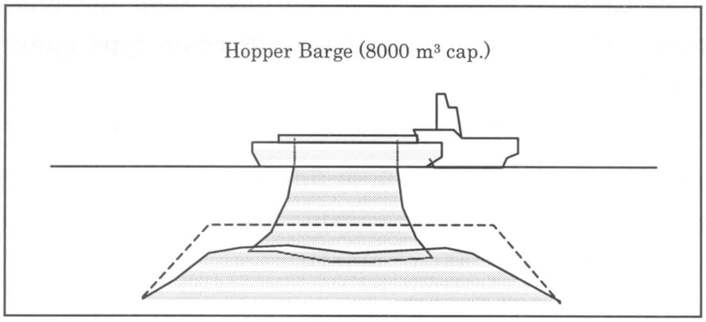


Figure 2.3.7 Direct Dumping by Hopper Barge

3) Reclamation by Reclaimer Barge

The hopper barge will be anchored on the side of reclaimer barge where it will unload sand from the hopper barge using its mounted belt conveyor and will conveyed the sand inside the steel sheet pile cellular wall type cofferdam. Reclamation work will proceed layer by layer in order to avoid the circle slip of the soil. This method requires two (2) hopper barges to continuously feed the conveyor of the reclaimer barge. See Figure 2.3.8 showing the operational layout of this method.

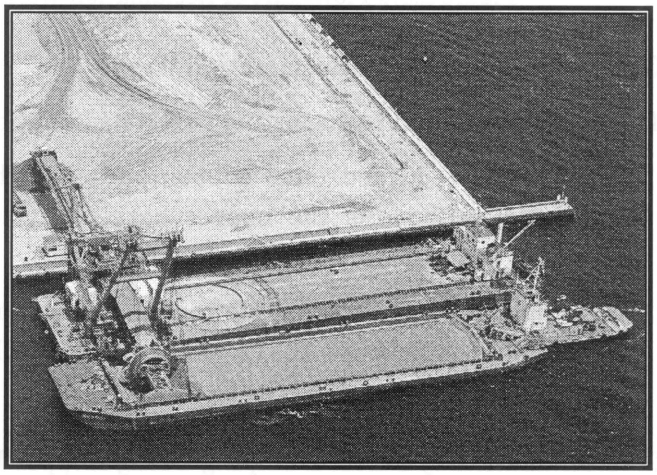


Figure 2.3.8 Reclamation by Reclaimer Barge

C. List Major Equipment Required

Description	Capacity	Quantity
Hopper Barge	8000 m^3	4 nos.
Pusher Boat	4000 ps	4 nos.
Reclaimer Barge	3500 m^3/h	1 nos.
Reclaimer Barge	3000 m^3/h	1 nos.
Survey Boat	200 ps	1 nos.
Anchor Boat	15 ton	2 nos.
Bulldozer	21 ton	-
Grader	4 m	-
Compaction Roller	5 ton	-
Vibration Roller	10 ton	-
Dump Truck	10 ton	-
Excavator	2.0 m^3	-
Wheel Loader	2.0 m^3	-
Water Pump	4 inch	-

CHAPTER 3 PRELIMINARY STUDY ON CONSTRUCTING PENINSULA

3.1 Peninsula Size

There are considerable factors in determining the size of peninsula. In this study, the depth of the water is one of the major factors in deciding the planned size of the peninsula, where it ease the navigation of the required equipment during construction providing economic cost. See Figure 3.1.1 showing the layout plan of the peninsula.

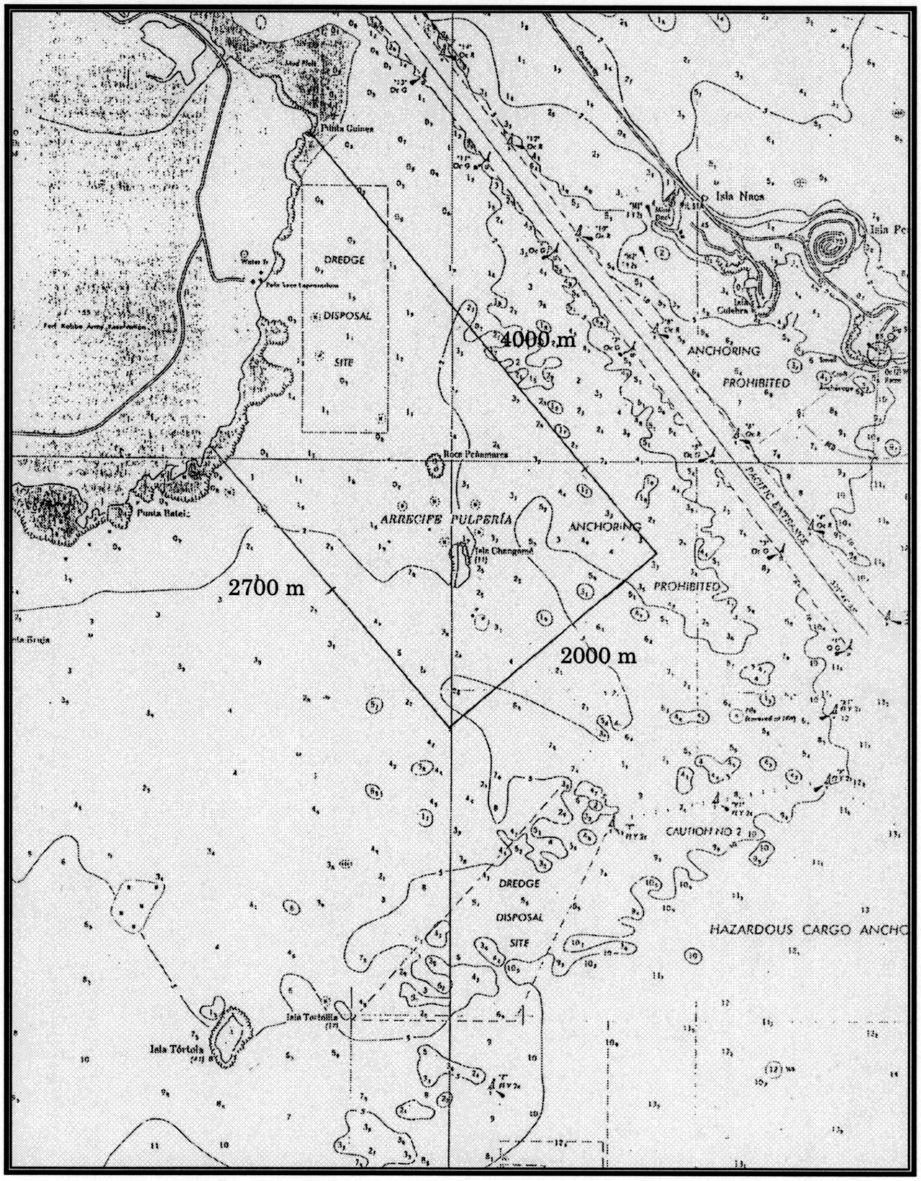


Figure 3.1.1 Layout Plan of Peninsula

3.2 Plan and Design of Revetment for Peninsula

3.2.1 Design Objectives and Design Policy

3.2.1.1 Design Objectives

This Section introduces structural design of revetment for "Peninsula", which is alternative plan as dredge disposal site only. Alternative plan for Peninsula does not include a function as berth facilities.

Design Objectives: 1) Revetment for Peninsula

This most of design policy, design condition and design method for revetment of Peninsula are the same as the quaywall and revetment of Artificial Island which is discussed in Section 2.2 before. Therefore, the same description is omitted in this Section. Only differences are mentioned hereinafter.

Figure 3.2.1 shows general plan of Peninsula. In this study, the planning location of Reclamation Peninsula is set at the site indicated tentatively by ACP.

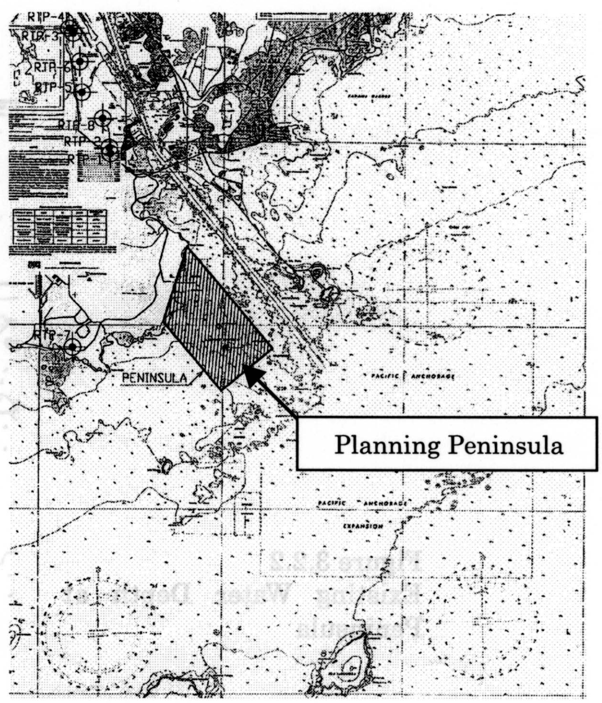


**Figure 3.2.1
Location of Peninsula in Planning**

3.2.1.2 Design Policy

see Section 2.2.1.2.

3.2.2 Design Condition

see Section 2.2.3 except the followings:

(1) Existing Water Depth at Construction Site

Existing water depth at construction site is set based on the results of bathymetric survey submitted by ACP. For Peninsula, planning site is located on bathymetric map as shown in Figure 3.2.2. Water depth around the construction site is from –0.3m to –5.6m. In this study, the existing water depth in design is assumed as –2m and –4m for the structural design of representative revetment in Peninsula. Thus the existing water depth in design are summarized as shown in Table 3.2.1.

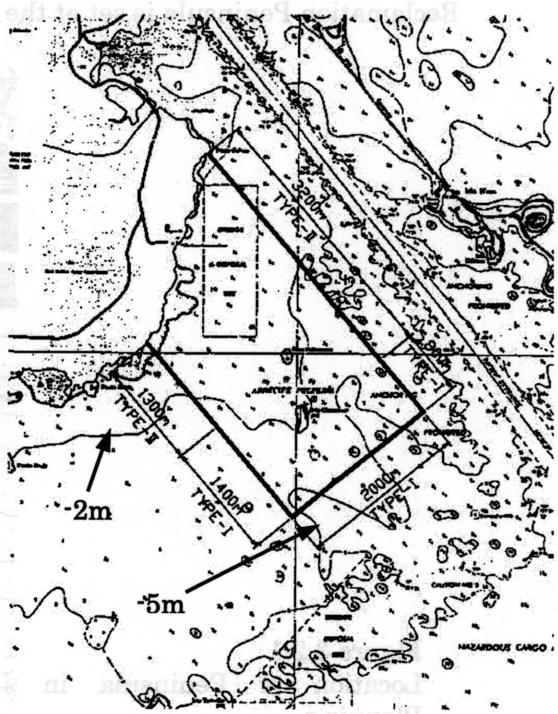


Figure 3.2.2
Existing Water Depth at Peninsula

Table 3.2.1 Assumed Existing Water Depths at Peninsula

	Artificial Island
Existing Water Depth	–2.0m and –4.0m

(2) Design Water Depth

There are no needs to construct the berth facility in Peninsula. Therefore, the design water depth in Peninsula leads to the same as the existing water depthas shown in Table 3.2.1 before.

(3) Other Conditions

Other design conditions are assumed as Table 3.2.2. It is noted that only surcharge is different from Artificial Island. Surcharge on the crown of revetment is assumed modestly in consideration of similar structures in Japan. Soil material for backfilling and inside filling is regarded as stiff rock, which will be easily gained from dredging of Panama Canal.

Table 3.2.2 Other Design Conditions

<table>
<tr><th colspan="2">Items or Parts</th><th>Artificial Island</th></tr>
<tr><td rowspan="3">Surcharge on Quaywall and Revetment</td><td>During Construction (Wave Attack)</td><td>0kN/m^2</td></tr>
<tr><td>After Construction (Ordinary)</td><td>10kN/m^2</td></tr>
<tr><td>After Construction (Earthquake)</td><td>5kN/m^2</td></tr>
<tr><td rowspan="3">Soil Materials (Backfilling and Inside Filling)</td><td>Angle of internal friction</td><td>40°</td></tr>
<tr><td>Specific weight above residual water level</td><td>18kN/m^3</td></tr>
<tr><td>Specific weight below residual water level</td><td>10kN/m^3</td></tr>
</table>

3.2.3 Design Method of Revetment for Peninsula

Design method for

Design flowchart for sheet pile cellular-bulkhead quaywall and double sheet pile wall are the same as described in Section 2.2.4 and Section 2.2.5, respectively. See the related Section.

3.2.4 Best Selection of Structural Types

The design water depth at Peninsula is very shallow as –2m or –4m. As mentioned previously in Figure 2.2.20 of Section 2.2.6, the double sheet pile wall can be more economical in such shallow water depth in comparison with sheet pile cellular-bulkhead quaywall. Therefore, it can be concluded that double sheet pile wall is strongly recommended as the most suitable structural type for revetment in Peninsula.

Comparing with a sloping embankment, double sheet pile wall can keep the seawater clean during construction as mentioned previously in Section 2.2.2.3. Moreover, it is noted that double sheet pile wall also can function as a quaywall for shipping if necessary. In case of Peninsula, double sheet pile wall can be utilized as recreation area, such as yacht harbor or angler's port.

3.2.5 Design Results of Revetment in Peninsula

The designed results for double steel sheet pile wall are shown in Table 3.2.3 for deep (–4m) side and Table 3.2.4 for shallow (–2m) side, respectively.

Steel sheet pile of NSP-IVw SY390 and NSP-IIIw SY390 are selected for deep side and shallow side, respectively. Total weight of sheet pile of both sides accounts for 39,771 ton. For both sides, ordinary case is the most critical for the steel members as sheet pile, tie-rod and waling. On the other hand, extraordinary case gives the most critical design values for global stability as shear resistance and bearing capacity. Extraordinary cases are governed by earthquake case, but not by wave attack, for both deep side and shallow side.

Table 3.2.3 Design Results of Double Sheet Pile Wall for Deep (–4m) Side

<table>
<tr><td colspan="2">Revetment Side</td><td colspan="2">Deep (-4m) Side</td></tr>
<tr><td rowspan="2">Condition</td><td>Existing Water Level</td><td colspan="2">M.L.L.W.-4m</td></tr>
<tr><td>Design Water Depth</td><td colspan="2">M.L.L.W.-4m</td></tr>
<tr><td rowspan="4">Specification</td><td>Steel Sheet Pile</td><td colspan="2">NSP-IVw SY390</td></tr>
<tr><td>Tie-rod</td><td colspan="2">ϕ55 ×11.22m@ 2.4m, HT690</td></tr>
<tr><td>Waling</td><td colspan="2">2-Channel 250×90×9×13, SS400</td></tr>
<tr><td>Width of Double Wall
(c. to c. of sheet piles)</td><td colspan="2">10.0m</td></tr>
<tr><td rowspan="2">Total Size</td><td>Total Revetment Length</td><td colspan="2">4,200m</td></tr>
<tr><td>Total Weight of Sheet Pile</td><td colspan="2">23,002 ton</td></tr>
<tr><td colspan="2">Loading Case (*: earthquake or wave attack)</td><td>Ordinary</td><td>Extraordinary*</td></tr>
<tr><td rowspan="7">Design Values</td><td>Stress in Sheet Pile</td><td>234 kN/m² < 235 OK</td><td>280 kN/m² < 353 OK</td></tr>
<tr><td>Embedded Length of Sheet Pile</td><td>4.5m</td><td>4.5m</td></tr>
<tr><td>Stress in Tie-rod</td><td>170 kN/m² < 176 OK</td><td>198 kN/m² < 264 OK</td></tr>
<tr><td>Stress in Waling</td><td>129 kN/m² < 140 OK</td><td>146 kN/m² < 210 OK</td></tr>
<tr><td>Shear Resistance</td><td>1.36 > 1.20 OK</td><td>1.25 > 1.20 OK</td></tr>
<tr><td>Sliding</td><td>2.55 > 1.20 OK</td><td>1.47 > 1.00 OK</td></tr>
<tr><td>Bearing Capacity</td><td>2.72 > 1.20 OK</td><td>1.10 > 1.00 OK</td></tr>
</table>

Table 3.2.4 **Design Results of Double Sheet Pile Wall for Shallow (-2m) Side**

Revetment Side		**Shallow (-2m) Side**	
Condition	Existing Water Level	M.L.L.W.-2m	
	Design Water Depth	M.L.L.W.-2m	
Specification	Steel Sheet Pile	**NSP-Ⅲw SY390**	
	Tie-rod	**ϕ44 ×9.06m@ 2.4m, HT690**	
	Waling	**2-Channel 200×80×7.5×11, SS400**	
	Width of Double Wall (c. to c. of sheet piles)	**8.0m**	
Total Size	Total Revetment Length	**4,500m**	
	Total Weight of Sheet Pile	**16,769 ton**	
Loading Case (*: earthquake or wave attack)		Ordinary	Extraordinary*
Design Values	Stress in Sheet Pile	179 kN/m^2 < 235 OK	214 kN/m^2 < 353 OK
	Embedded Length of Sheet Pile	4.7m	4.7m
	Stress in Tie-rod	**169 kN/m^2** < 176 OK	198 kN/m^2 < 264 OK
	Stress in Waling	**138 kN/m^2** < 140 OK	150 kN/m^2 < 210 OK
	Shear Resistance	1.45 > 1.20 OK	**1.32** > 1.20 OK
	Sliding	2.62 > 1.20 OK	1.54 > 1.00 OK
	Bearing Capacity	2.67 > 1.20 OK	**1.13** > 1.00 OK

Finally, the design drawings for layout plan of revetment and vertical section of double sheet pile wall are shown in Figure 3.2.3. In these drawings, TYPE- I and TYPE- II mean deep (-4m) side and shallow (-2m) side, respectively. Total length of deep side and shallow side is 4,200m and 4,500m, respectively. Inside filling is made of crushed stone obtained easily from dredging Panama Canal. Steel sheet piles should be driven deeply into the silty sand layer. Dredged stiff rock is assumed as a material for backfilling.

In addition, countermeasure as anti-corrosion of double sheet pile wall is fundamentally the same as that of sheet pile cellular bulk-head quaywall as mentioned in Section 2.2.7.2 previously.

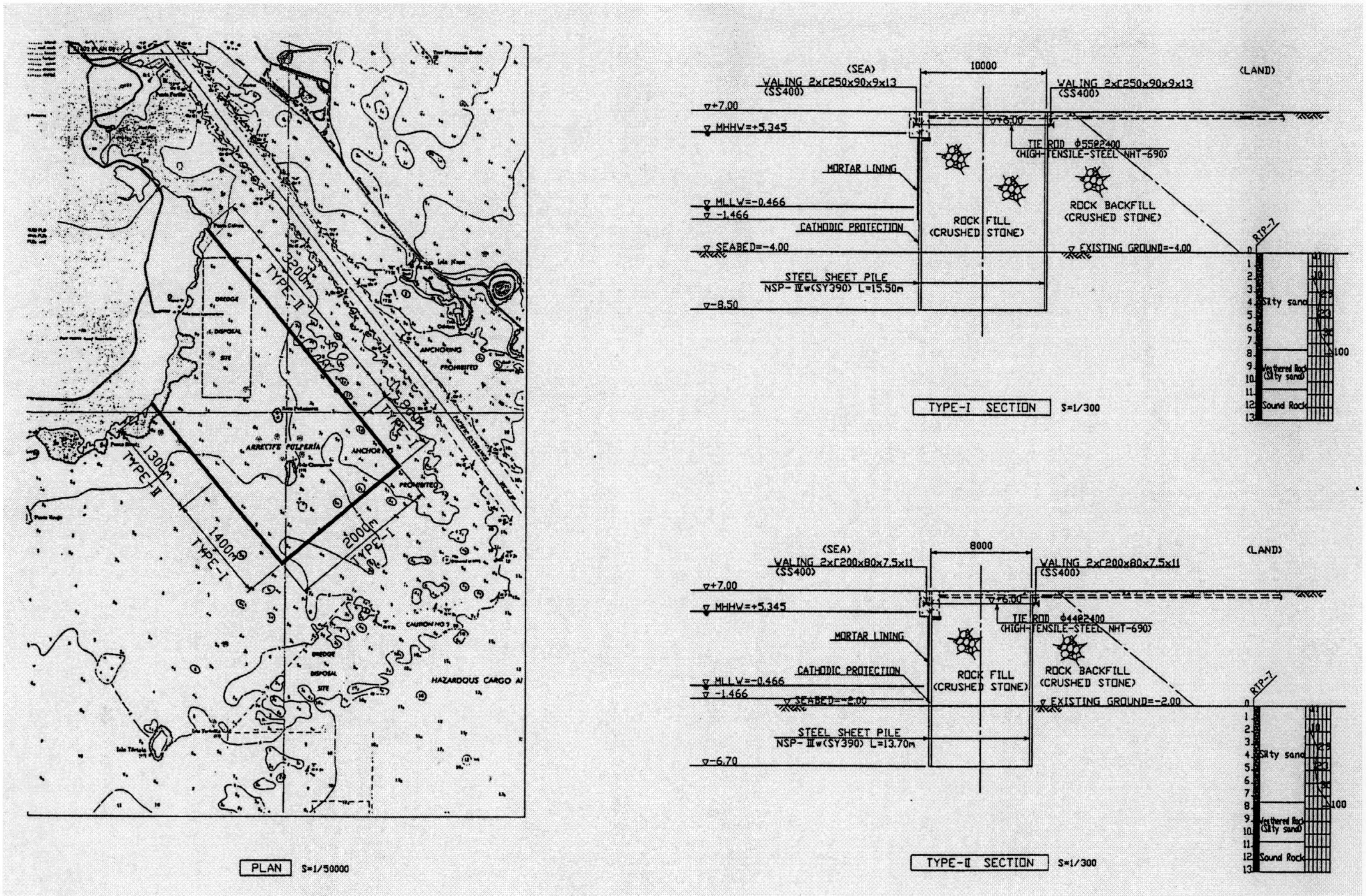


Figure 3.2.3 Layout Plan and Section of Double Sheet Pile Wall in Peninsula

3.3 Construction Plan of Peninsula

The methodology of this plan includes the technical aspects of the design and various requirements as set forth herein and this also includes the consideration of the water depth, stone handling and transportation, equipment capabilities. The targeted reclamation production of this plan is at approximately 2,000,000 m^3/month for which the construction period is based on. So it should be noted that the success of this plan might also depend on the productivity of high capability soil in the New Lock Sites.

Figure 3.3.1 shows the flowchart of the work to be undertaken to complete the construction of peninsula.

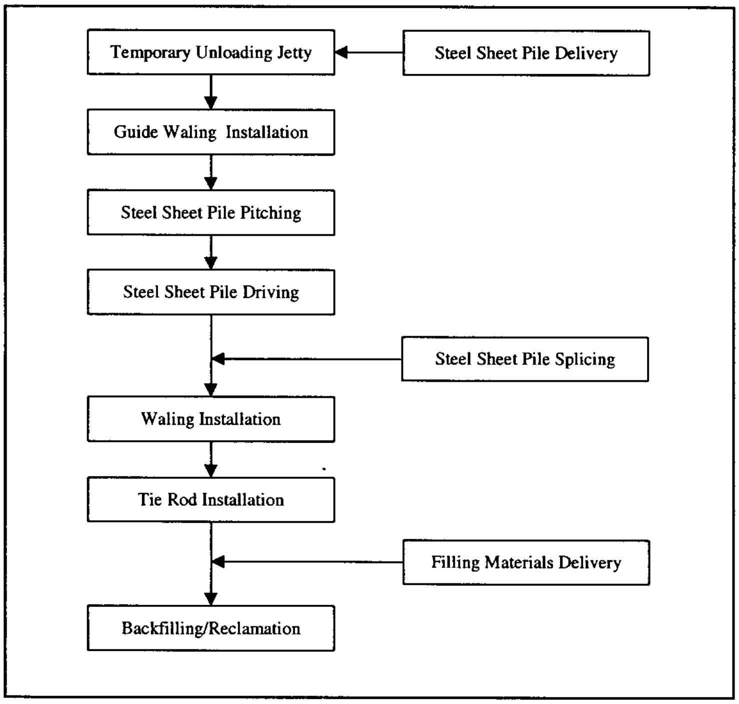


Figure 3.3.1 Flowchart of Construction of Peninsula

3.3.1 General Methodology

A. Construction of Temporary Loading Jetty

The construction temporary loading jetty as shown in Figure 3.3.2 and Figure 3.3.3 will be located near the shore of the New Locks sites. Its size will depend on the water depth to suitably accommodate the floating barge for the collection and transportation of stone to the peninsula site. The condition of the prevailing navigational traffic at the channel is also a factor in choosing the appropriate site of this temporary jetty.

Figure 3.3.2 Picture of Temporary Loading Jetty

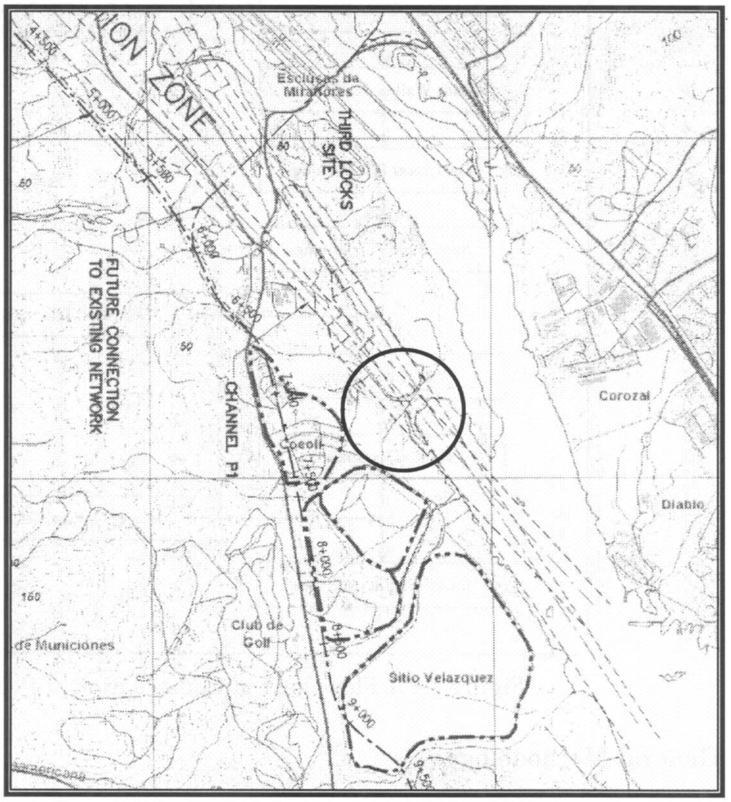


Figure 3.3.3 Location of the Loading Jetty

B. Construction of Guide Waling Beam for Sheet Piling

Prior to commence the steel sheet piles driving operation, two (2) lines of guide waling beams will be set in parallel using H-beam bearing piles to be driven by floating crane as shown in Figure 3.3.4.

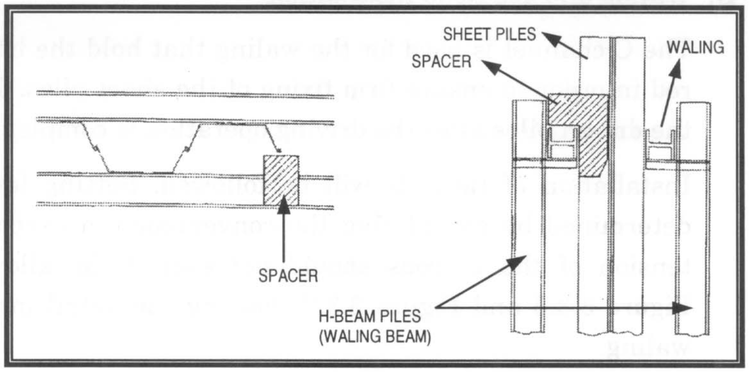


Figure 3.3.4 Picture of Guide Waling Beam

C. **Steel Sheet Pile Pitching and Driving**

The steel sheet piles will be loaded unto the floating barge by truck crane at jetty. Then, the pontoon will be towed to the piling site by tugboat.

The piling pontoon as shown in Figure 3.3.5 will pitch and set the sheet piles on its driving position and interlocked with spacer between the guide waling beam to prevent the swaying and rolling of the sheet piles.

Each sheet pile will be driven by vibro hammer in stages with a depth of 1-2 meter per stage and during this operation the position, alignment and verticality of the sheet piles will be observed.

In case of the piles is short of length to meet the designed level, it will be extended by full strength butt welding after the lower part of the pile has been driven to a certain level. This welding works must be carried out by an experienced and qualified welder.

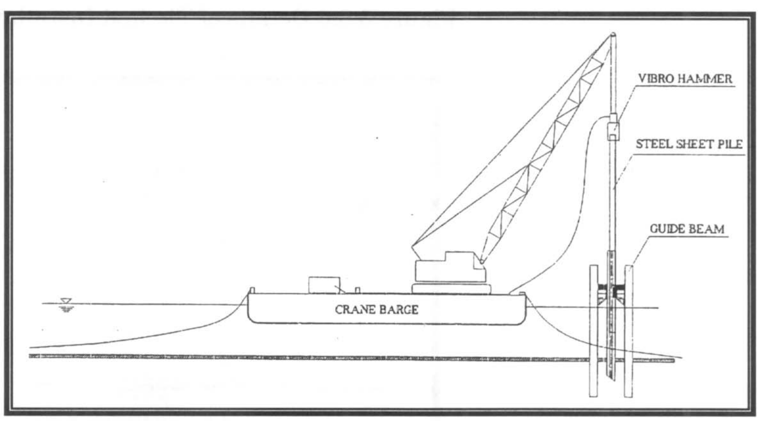


Figure 3.3.5 Operation of the Piling Pontoon

D. Waling and Tie Rods Installation

The C-channel is used for the waling that hold the bolted high tension type tie rod in order to ensure firm fixing of the sheet piles. This channel is welded on the driven piles after the driving operation is completed.

Installation of tie rods will be followed. Setting level of tie rods should be determined by considering the convenience in execution of the tie rods. The tension of the tie-rods should not exceed the allowable tensile stress. See Figure 3.3.6 and Figure 3.3.7 showing the detail installation for tie rods and waling.

Tie rods will be installed perpendicular to the line of steel sheet piles and the rods adjustment should not be done by cutting and welding, instead turnbuckle should be used. The tension of the tie rod after installation shall be similar to each other in order to maintain the stability of the sheet piles.

Ring joints as shown in Figure 3.3.6 shall be set in two (2) positions, such that one at the offshore facing sheet pile and the other at the anhor sheet pile. It shall be installed in a direction that can allow the rolling closely to the sheet piles.

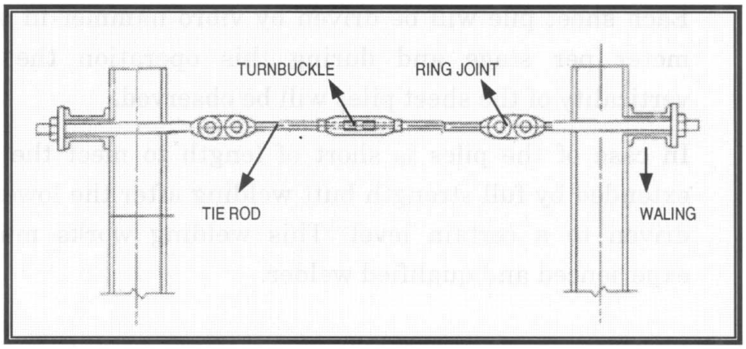


Figure 3.3.6 Details of Tie Rod Installation

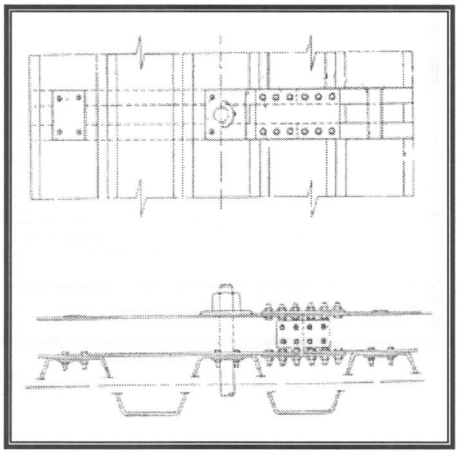


Figure 3.3.7 Details of Waling and Tie Rod

E. Backfilling / Reclamation Work

The method of filling operation is to be carried out by the reclaimer barge and auxiliary floating barges for the transportation of fill materials from the loading jetty. See Figure 3.3.8 showing those equipment.

The planned delivery of fill materials to the reclamation area is at 10~12 barge per day considering its distance to the temporary loading jetty as well as the prevailing traffic at the channel. In addition, for the maximum size of stone for filling is about the size of human head to be navigated by the barge, which has a draft of 5 - 6 meter when fully loaded.

Backfilling will be performed by the reclaimer barge. This operation will be controlled layer by layer to avoid any displacement or deformation of the sheet piles and tie rods that may caused by concentrated soil pressure.

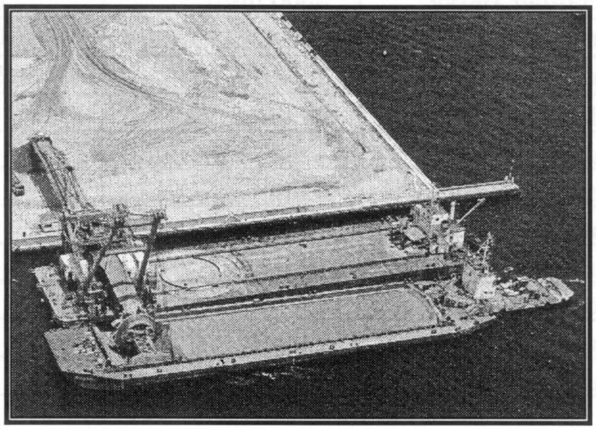


Figure 3.3.8 Reclaimer Barge Operation

3.3.2 Major List of Equipment Required

Equipment	Capacity	Quantity
Floating Crane	50 ton	4 nos.
Floating Barge	500 ton	4 nos.
Truck Crane	20 - 22 ton	1 nos
Tug Boat	500 ps	4 nos.
Anchor Boat	10 ton	4 nos.
Vibro-Hammer	45 kw	4 nos.
Generator	100 kva	4 nos.
Welder	D 300A	4 nos.
Hopper Barge	8000 m^3	4 nos.
Pusher Boat	4000 ps	4 nos.
Reclaimer Barge	3000 m^3/h	1 nos.
Reclaimer Barge	3500 m^3/h	1 nos.
Survey Boat	200 ps	1 nos.
Anchor Boat	15 ton	2 nos.
Bulldozer	21 ton	-
Grader	4 m	-
Compaction Roller	5 ton	-
Vibration Roller	10 ton	-
Dump Truck	10 ton	-
Excavator	2.0 m^3	-
Wheel Loader	2.0 m^3	-
Water Pump	4 inch	-

3.4 Construction Period

ITEM No.	DESCRIPTION	MONTH																																			
		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24	25	26	27	28	29	30	31	32	33	34	35	36
1	Double Steel Sheet Pile Wall Cofferdam																																				
	a. Sheet Pile Driving (TYPE 1 : 2000m)																																				
	b. Sheet Pile Driving (TYPE 1 : 2200m)																																				
	c. Sheet Pile Driving (TYPE 2 : 3200m)																																				
	d. Sheet Pile Driving (TYPE 2 : 1300m)																																				
	e. Wale・ Tie-Rod Installation (TYPE 1 : 4200m)																																				
	f. Wale・ Tie-Rod Installation (TYPE 2 : 4500m)																																				
2	Filling Sand for Double Sheet Pile Wall Cofferdam																																				
3	Reclamation																																				

3.5 Cost Estimation

3.5.1 Revetment Construction and Land Reclamation Costs

Table 3.5.1 shows revetment construction and land reclamation costs for the peninsula. Total cost is about 205 million dollars.

Table 3.5.1 Revetment Construction and Land Reclamation Costs

Item	Units	Quantity	Unit Price ($)	Total ($)
Revetment Construction and Land Reclamation Costs				
General				
Mobilization and Demobilization	LS	1	2,000,000.00	*2,000,000*
Embankment Construction				*67,836,560*
Double Wall Piling	m	8,700	7,412.00	64,484,400
Inside Filling	m3	1,022,000	3.28	3,352,160
Land Reclamation				
Land Reclamation	m3	54,000,000	2.50	*135,000,000*
Total				***204,836,560***

CHAPTER 4 ENVIRONMENTAL ASPECT

The Artificial Island Project could be regarded as one of the mitigation measures to reduce environmental impacts caused by disposing excavated material in the process of widening the Panama Canal from a viewpoint of the global environment.

Therefore, in the JETRO Study, prefabricated steel sheet pile cellular-bulkhead quaywall structure with loading jetties has been proposed taking into account the design concept “Minimum Impact on Environment”. This proposal has the following advantages from an environmental point of view.

- Reduction of construction period
- Reduction of reclamation area
- Reduction of dispersal of disposed material

Obtained has been little information on application and procedures for Panama’s environmental impact analysis including applicable regulations and laws and environmental baseline data on natural conditions. Therefore, this Chapter chiefly introduces JBIC’s environmental guidelines and an artificial island project in Japan. Items introduced in this Chapter are follows:

- present situation of the artificial island;
- legal framework;
- JBIC environmental guideline;
- review of existing environmental conditions;
- tidal current analysis;
- introduction of environmental measures in Japan;

4.1 Present Situation of the Artificial Island Project

The Panama Canal is located in a strategic position, serving international trade including that of Japan and the Canal provided passage for more than 850,000 vessels about 88 years ago. These vessel transit projections indicate that the capacity of the Canal will be exceeded by traffic demand between 2010 and 2020. Therefore, the Panama Canal Authority (ACP) begun to carry out a preliminary feasibility study to expand the existing Canal in 1998.

The proposed expansion project and related third lock project of the Panama Canal will generate significant quantities of excavated and dredged material. The volume of dredged material is estimated at 55.0 to 85.0 million m^3.

The Artificial Island Development Project has been planned by ACP. The use of a large amount of excavated and dredged materials from the related expansion projects of the Panama Canal would definitely create an artificial island by reclamation and complement the canal operations and minimize the environmental impacts caused by land disposal operations.

In other words, both expansion and improvement of the Panama Canal together with constructing an artificial island are important and urgently required due to the ever-increasing demands of Panama as well as other related countries.

4.2. Legal Framework

The proposed Artificial Island Development Project is subject to all the relevant legislative requirements of Panama as well as internationally binding agreements signed by the Government of Panama.

4.2.1 International Protocols

Over the years, the Government of Panama has signed a number of international agreements and conventions that stipulate a worldwide binding status as far as environmental protection is concerned.

Among them, the most pertinent protocols to the proposed Artificial Island Project are "The United Nations Convention on the Law of the Sea." and "The International Convention for the Prevention of Pollution from Ships, (MARPOL 73/78, IMO)". These important protocols are described respectively below.

(1) The United Nations Convention on the Law of the Sea[1].

"The United Nations Convention on the Law of the Sea (Montego Bay, 1982)", the Government of Panama signed this protocol in 1982 with many countries.

[1]) *Source: The United Nations Convention on the Law of the Sea (Montego Bay,1982)*

"Part XII: Protection and Preservation of the Marine Environment" in the United Nations Convention on the Law of the Sea, it was stipulated that "Part XII: Protection and Preservation of the Marine Environment"

- Article 192: General obligation (States have the obligation to protect and preserve the marine environment
- Article 194: Measure to prevent, reduce and control pollution of the maritime environment
- Article 204: Monitoring of the risks or effects of pollution
- Article 206: Assessment of potential effects of activities
- Article 207: Pollution from land-based sources
- Article 208: Pollution from sea-bed activities subject to national jurisdiction
- Article 209: Pollution from activities in the Area
- Article 210: Pollution by dumping
- Article 211: Pollution from vessels
- Article 212: Pollution from or through the atmosphere

(2) MARPOL 73/78[2].

The International Convention for the Prevention of Pollution from Ships, it was adopted by the International Conference on Marine Pollution convened by International Maritime Organization (IMO) in 1973. This Convention was subsequently modified by the Protocol of 1978 relating thereto adopted by the International Conference on Tanker Safety and Pollution Prevention (TSPP Conference) convened by IMO in 1978. The Convention, as modified by the Protocol, is known as the International Convention for the Prevention of Pollution from Ships, 1973, as modified by the Protocol of 1978 relating thereto or, in a short from, MARPOL 73/78 Regulations covering the various sources of ship generated pollution are contained in below six annexes of the Convention.

- Annex I: Regulations for the Prevention of Pollution by Oil
- Annex II: Regulations for the Control of Pollution by Noxious Liquid Substances
- Annex III: Regulations for the Control of Pollution by Harmful Substances in Packaged Forms
- Annex IV: Regulations for the Prevention of Pollution by Sewage
- Annex V: Regulations for the Prevention of Pollution by Garbage

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[2]) *Source: MARPOL 73/78 Consolidated Edition, 1991, International Maritime Organization.*

- Annex VI: Regulations for the Prevention of Air Pollution (Un-issued)

4.2.2 National Legislation [3]

In July 1998, the National Environment Authority (ANAM) was organized to control environmental problems in Panama. However, the environmental management and operation have been controlled by not only ANAM but also many organizations such as related ministries, autonomies, semi-autonomies, institutes, local governing bodies, systematized groups, NGOs and etc.

Thus, due to inefficient arrangement between the above entities, there have been difficulties in observing existing laws and modifying them. Since the JETRO Study Team did not make discussions with ACP about regulations and laws on EIA in the case of the Artificial Island Development Project, adopted regulations and laws will be clarified and determined later.

The following environmental laws have been retrieved from the relevant materials.

- National Action Plan of Environmental Health in Sustainable Development (1998-2002), Ministry of Health (July 1997)
- Law on National Environment No.41 (1998)
- Law on EIA for Tourism No.8 (1994)
- Law on Enterprises Affecting Detrimental Effects on Environment No.30 (1994)
- Law on Environmental Education to Prevent Natural Resources Deterioration No.10 (1992)
- Law on Prohibition of Importing Harmful Products No.10 (1992)
- Law on Prevention of Material Dispersal No.51 (1975)
- Law on the Mitigation Measure for Water Pollution No.16 (1973)
- Law on Prevention for Air Pollution No.66 (1947)
- Decree on Noise Control Regulation No.150 (1971)
- Law on Wild Life Conservation No.24 (1995)
- Law on Forest Conservation No.1, No.24 (1994)

Especially, MARPOL has adopted operational regulations[4] to manage and/or control in the Panama Canal. These regulations are as follows;

[3] *Source: The Report on Environmental Information for Panama Oct, 1999, Japan International Cooperation Agency (JICA).*
[4] *Source: Panama Canal Authority.*

- Law on Land Use around Canal on Local Land Development Plan, Preservation and Development Plan No.21 (1997)
- Law No.19, The Panama Canal Authority is Organized, Panama Legislative Assembly (June 11, 1997), Chapter I: Character, Definitions, and General Rules, Article 6
- Agreement No.13, The Regulation on Navigation in Panama Canal Waters is approved, The Board of Directors of the panama canal authority (June 3, 1999)
- Agreement No.20, The Regulation on the Panama Canal Authority Board of Inspectors are approved, The Board of Directors of the panama canal authority (July 15, 1999), Chapter II: Investigation of Marine Accidents
- Agreement No.23, The Regulation on Sanitation and Communicable Disease Prevention are approved, The Board of Directors of the panama canal authority (September 23, 1999), Section Nine: Discharge Vessel Wastes, Ballast, and Sewage

4.3 JBIC's Environmental Guidelines

The Japan Bank for International Cooperation (JBIC) is one of the financial bodies. The objective of the JBIC environmental guidelines is to guarantee that all aspects of the development project have been assessed from an environmental point of view.

Moreover, it is one of the mandatory items to receive approval by JBIC for a financial loan. Therefore, it is necessary that the Panama side (if required) should conduct an EIA to implement the Artificial Island Project.

With this intention, JBIC requires that all mitigation measures with regard to the environment are taken by the recipient country for the proposed project. In this regard, compliance with all the legal requirements of the recipient country (the Government of Panama) as well as the international conventions is essential.

According to the JBIC guidelines, harbor and port development projects such as the Artificial Island Development Project is classified as "Class A" and the Project requires a full environmental impact assessment (EIA) study.

Based on their past experience with similar projects supported by other international financiers (e.g., the Asian Development Bank and the World Bank), it is necessary that the ACP and/or the Government of Panama anticipates that an EIA study fulfilling the requirements of the National

Environment Authority (ANAM) and other related organizations will satisfy the JBIC criteria.

4.3.1 JBIC's EIA Procedures

General contents and general procedures of an EIA are the following.

1) Screening: Screening is an initial assessment to identify potential magnitude of impacts. This is used to determine the depth of analysis required.

2) Scoping: Scoping is a process to identify issues that are required for a detailed study and eliminate items that are not required. Scoping defines appropriate boundaries of an EIA study, impacts to be considered and time frame of the study.

(The results of the preliminary scoping of the Artificial Island Project were provided by the JETRO Study Team in Additional Table A4.3.1)

3) Assessing: Assessing evaluates and predicts the likely environmental impacts based on existing natural, social and cultural conditions at the harbor project site.

4) Mitigation: Based on the environmental impacts predicted in 3) (Assessing), mitigation measures and alternatives that are available to reduce to environmental and human impacts are considered. Additional Table A4.3.2 presents an example of Environmental Impact Factors and Environmental Preservation Measures related to Port and Harbor Projects in accordance with JBIC's guidelines.

5) Reporting: An EIA report contained the study and analysis results of 1) through 4) (above) is prepared and open to affected people and relevant organizations.

6) Reviewing: The aim of reviewing is to check contents of the EIA report and to ensure the EIA report is adequate for alleviation of environmental impacts

7) Monitoring and Management: The aim of monitoring and management is to confirm that mitigation and compensation measures described in the EIA report are implemented. It is also used to obtain data and information on the performance of mitigation or compensation measures.

8) Public Involvement Relating to the Above All Stages: The involvement of affected people and communities and relevant organizations is an important element of the EIA process.

The EIA procedure is shown in Figure 4.3.1. [5]

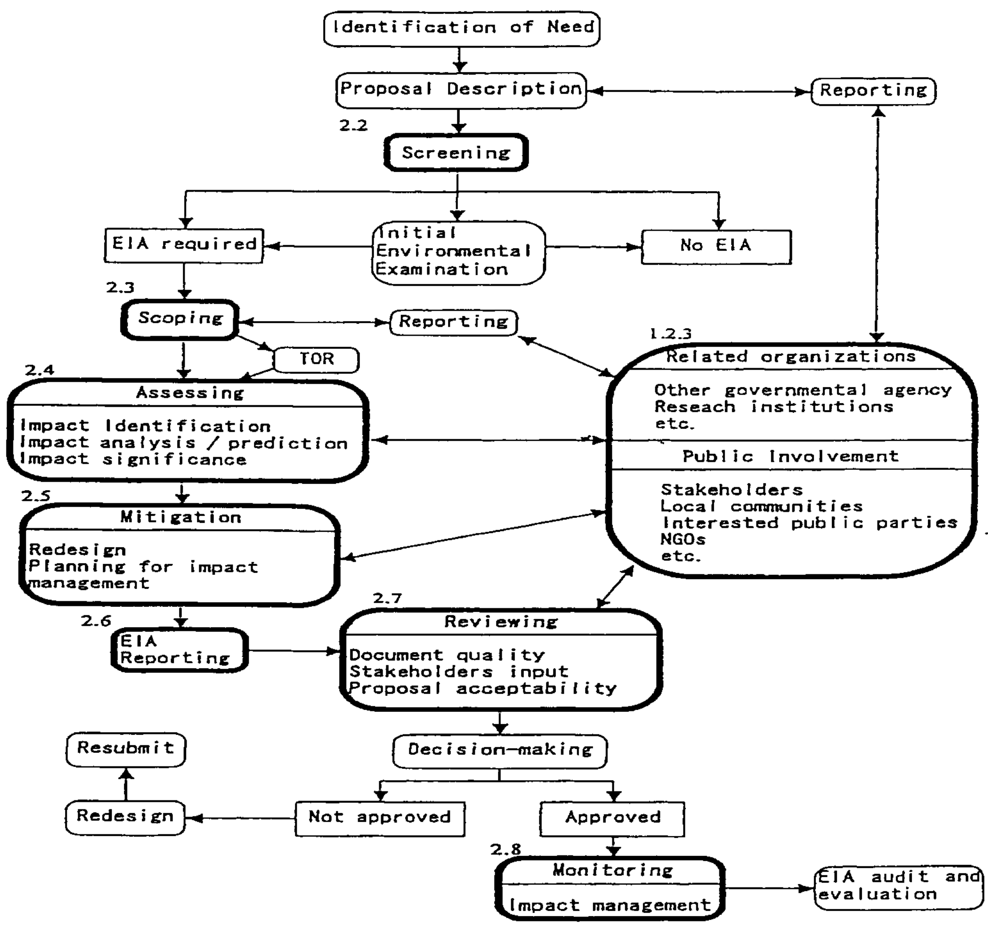

Figure 4.3.1 EIA Procedure

4.3.2 Necessity of the Environmental Management and Monitoring

After the Project is started, monitoring should be performed to validate the environmental impacts predicted in the EIA and the effectiveness of mitigation. If monitoring reveals that mitigation is not effective, follow-up mitigation should be taken. It is important to compare the contents

[5] *Source: "A Guide to Preparing an Environmental Impact Assessment(Harbour Sector) March 1998", JBIC(in past called The Overseas Economic Cooperation Fund, Japan)*

described in the EIA with actual environment conditions during construction and operational phases.

The objective of monitoring is to assess environmental conditions periodically and systematically. Monitoring is also performed for environmental management.

The objective of monitoring for environmental management is to implement various mitigation measures to achieve preservation targets which are set up to maintain desirable environmental conditions for human health and ecological protection.

The primary objective of EIA is to reduce environmental impacts through the EIA process. In order to realize this objective, feedback of EIA results to actual project implementation by monitoring and follow-up are very important.

The objective of monitoring and the environmental management plan in EIA is to confirm whether mitigation measures (environmental preservation plan) are implemented during construction and operation, and whether expected effectiveness is achieved when mitigation measures are implemented.

Environmental management and monitoring procedures following the completion of the port and harbor construction should be formulated, covering details as technical, operational, institutional, and financial aspects.

In this Section, environmental monitoring and environmental management plans in EIA are described respectively.

1) Environmental Monitoring

Environmental monitoring is essential to check whether actual environmental impacts are consistent with predicted ones, whether any unpredictable environmental phenomena appeared, and if additional environmental measures will be required. Periodical monitoring is mandatory during construction as well as during the operation of the port and harbor.

The planning of Environmental Monitoring Programs generally include the following procedures and items:

- Items to be monitored: Based on the results of environmental impact predicted, items necessary to assess environmental impacts should be selected.
- Time and frequency of monitoring: Appropriate time and frequency should be determined.
- Location of monitoring points: Monitoring locations for the field investigation should be identified and plotted on an appropriate map and/or topographical map.
- Monitoring methods: Specific monitoring methods should be set out for each item. (In principle, the same methods applied in the baseline data collection should be applied.)
- Monitoring criteria: Monitoring criteria should follow environmental preservation targets decided upon in impact assessment stages.
- Monitoring system: Monitoring system and organizational chart, and their supporting systems should also be established.

2) Environmental Management Plan (EMP)

The Environmental Management Plan (EMP) is one of the most important outputs in the EIA process. EMP is variously called the environmental action plan, the environmental protection plan or the environmental construction plan.

EMP is the synthesis of all proposed mitigation and monitoring actions, set to a timeline with specific responsibility assigned, and follow-up actions defined.

A well-designed EMP addresses both construction and operational components of the Project and includes the following:

- A listing of all project-related activities and impacts, organized by development stage, namely planning, construction and operation;
- A listing of regulatory agencies such as the Ministry of Public Works, the Autoridad Nacional de Ambiente (ANAM), the Autoridad de la Region Interceanica (ARI), the Ministry of Health and the Autoridad Maritima de Panama (AMP) involved and their responsibilities;
- Specific remedial and monitoring measures presented for construction period activities and impacts, operational period activities and impacts;
- A clear reporting schedule, including discussion of what to submit, to whom and when;
- Cost estimates and sources of funding for both one-time costs and recurring expenses for EMP implementation.

Additional Notes:

Additional Table A4.3.1 Preliminary Scoping[6]

Environmental Impact Factors / Environmentally Affected Items			During Construction		During Operation		
			Change of topography	Operation of construction machinery and vehicles	Presence of facilities (occupation of the space	Utilization of facilities	Vessel sailing
Environmental Pollution	1	Air pollution					o
	2	Water contamination	*	o	*	*	o
	3	Noise		o			o
	4	Vibration		o			
	5	Waste		o		o	o
Natural Environment	6	Topography, Geology	*				
	7	Change in hydrology	o		*		
	8	Coastal erosion	o		*		
	9	Fauna and flora	*	o	*		
	10	Landscape		o	o		
Social Environment	11	Involuntary resettlement					
	12	Minority races					
	13	Socioeconomy (1) Traffic and living facilities		o	o		
		(2) Regional division					
		(3) Water right, the right of common			o		
	14	Fishery	*	o	*	*	o
	15	Ruins, cultural assets					
	16	Health and sanitation		o			
	17	Disaster (risk)		o		o	*

Note) *: Special attention must be paid to the presence of the project itself depending upon the scope of the environment impact and the adequacy of the measures to be taken.

o: The impact may become enlarged, depending upon the scale of the project and the situation of the project location.

No mark: As the impact is minimal, no detailed investigation and study are normally required.

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[6] *This table shown as a preliminary study only, so modification is necessary in future JETRO F/S.*

Additional Table A4.3.2 Example of Environmental Impact Factors and Environmental Preservation Measures Related to Port and Harbor Projects[7]

1.1 Operation of working ships and construction equipment	1.1.1 Air pollution	1.1.1 Modification on construction schedule; restriction on working time zone; smoke prevention fence
	1.1.2 Generation of noise and vibration	1.1.2 Choice of construction method and equipment; restriction on working time zone; revised layouts of noise and vibration sources
	1.1.3 Changes in terrestrial ecosystem	1.1.3 Choice of construction method and equipment
1.2 Dredging/subsoil stirring/dumping into the sea	1.2.1 Water/sediment pollution (SS, toxic substances)	1.2.1 Application of silt basin; and coagulant agent; choice of construction method
	1.2.2 Offensive odor	1.2.2 Choice of construction method and equipment; adoption of offensive odor treatment method
	1.2.3 Decrease in aquatic fauna	1.2.3 Application of silt basin; coagulant agent; choice of construction method and equipment; turbidity diffusion protector; choice of construction period; contamination of catches
	1.2.4 Contamination of aquatic products	1.2.4 Application of silt basin and coagulant agent; choice of construction method and equipment; turbidity diffusion protector
	1.2.5 Drop in value of tourist/resort resources	1.2.5 Application of silt basin; coagulant agent; choice of construction method and equipment; turbidity diffusion protector
1.3 Soil removal	1.3.1 Change in topography and groundwater system	1.3.1 Prior elucidation of groundwater system
	1.3.2 Extinct of terrestrial ecosystem	1.3.2 Implementation of valuable species and community
1.4 Generation of surplus soil and waste; dumping on land.	1.4.1 Water/sediment pollution	

[7] *Source: "A Guide to Preparing an Environmental Impact Assessment(Harbour Sector) March 1998", JBIC*

1.5 Employment of workers	1.4.2 Effects on terrestrial ecosystem 1.5.1 Inflow of different culture 1.5.2 Change in economic activities	1.4.1 Programming of treatment facilities plan 1.4.2 Programming of disposal plan 1.5.1 Employment planning; Thorough data/information collection 1.5.2 Employment planning; personnel training
1.6 Convergence of construction vehicles and craft.	1.6.1 Economic loss (traffic congestion) 1.6.2 Drop in the value of fishing area	1.6.1 Construction of new access roads 1.6.2 Set-up of alternative fishing grounds

4.4 Review of Existing Environmental Conditions

4.4.1 Past Related Study (The Moffatt & Nichol Study)

In 2001, according to the Contract (No.CC-3-557) of Architectural and Engineering Services for Engineering Site and Assessment, Conceptual Design and Related Services between ACP and "Moffatt & Nichol Engineers Group", the group provided a "Final Report on Preliminary Study of Island Development at the Pacific Entrance of the Panama Canal" to examine an overview of the technical, environmental and capital costs for the construction of the artificial island using the excavated and dredged material from the expansion project and related Third Lock Project.

Regarding the study results, environmental issues and impacts of the Project are summarized in the following table.

Table 4.4.1 Preliminary Identification of Environmental Impacts[8],

Impact	Description	Recommendations
Interference in artisan & other fishing activities	The access to the island (viaduct) will interfere with the movement of small fishing boats and ships that move along the	Early on, initiate the process of consulting with citizens. Evaluate traffic and operational
Disruption of Hydrodynamic Movement In the bay	Interference caused by the island access complex in the main hydrodynamic complex of the Bay of Panama could affect sediments (erosion - local and zone deposits).	Develop a tri-dimensional hydrodynamic model that addresses issues of sediment and contamination. Calibrate according to the existing studies and
Effects to soil use and potential interference with plans and tourist activities	The potential development of the Howard Canal corridor could affect the protected areas on the coast and the development of the tourist area of Veracruz.	The interests of the tourist sector should be analyzed through surveys, workshops and other forms of citizen participation relating to
Loss of marine habitat	The covering of the marine base by excavated material will signify a loss of approximately 350 hectares of marine habitat	This can be mitigated by some form of land banking, environmental restoration or compensation.
Effects to coastal resources (Coral in Otoque and other	Depending upon effects to the hydrodynamic, the disturbance could reach the coral resources in front of Punta	The hydrodynamic model should be used correctly in order to make a decision regarding this sensitive
Contamination at beaches and western coastline	Some sectors of the pacific coastline west of the canal could suffer from an increase in the concentration of contaminants caused by the island-viaduct complex.	The hydrodynamic model and project design should be sufficiently "robust" in order to answer and address these concerns.
Effects to landscape and visual impacts	The new island and access will visually disrupt the view from shore to the open sea. This could be exacerbated by the nighttime illumination of the island.	The survey process is fundamental to address this concern.
Increase of dredging in the canal	Depending upon the position and shape of the island and its accesses, dredging of the canal could increase.	The mathematic model should be capable of responding to this question.
Interference with maritime traffic and disruption during	The island construction may cause interference with maritime movement west of the canal, in particular ships in transit	This effect can be mitigated but will require a specific study and formulation of detailed processes.

Based on the results, Moffat & Nichol undertook a feasibility study including environmental baseline studies by request of ACP.

[8]) *Source: Final Report on Preliminary Study of Island Development at the Pacific Entrance of the Panama Canal, December 2001*

4.4.2 Tidal Current Analysis

In the construction of an artificial island at the Pacific Entrance to the Panama Canal, the construction may influence the existing oceanographic conditions such as wave and current in the coastal region.

Two cases were simulated, one was the case of an Artificial Island that is planned to construct approximately 3.5 km offshore from the coast and the other was the case of Peninsula that is planned near the Pacific Entrance to the Panama Canal.

The following conclusions have been made as a result of the simulation analysis.

1. Velocities of the tidal flow may be changed at around the Artificial Island and Peninsula compared with the present conditions. However, the change is about 50 percent compared with the present state and the current direction is not changed significantly. Therefore, the influence is supposed to be small.

2. From the tidal residual flow, large variation of the tidal residual flow occurs within a radius of 2 km around the reclamation site.

As a result of the numerical simulation, the tidal flow at the project site in the Gulf of Panama is assumed to be little influenced by the construction of the Artificial Island or Peninsula (For more detailed information, see Appendix A).

4.5 Introduction of Environmental Measure in Japan (Kansai International Airport [9])

The Kansai International Airport Construction Project can be introduced as a sample of an artificial island project in Japan in terms of borrow material for reclamation, access bridges to the artificial island, and huge reclamation volume required for the artificial island. Especially, proposed quay structures and construction method are almost similar to those of the Kansai International Airport case. The following introduces environmental considerations employed for the Kansai International Airport case.

Kansai International Airport, located approximately 5 km offshore in the Osaka Bay, was constructed as the first Japan's airport to operate 24 hours a day taking into consideration noise generated by airport operation.

An EIA was required in advance because a huge amount of reclamation was carried out in the sea. Environmental protection measures and environmental monitoring plans were established to pay special attention to the environmental protection in the Osaka Bay and the surrounding area.

Monitoring was performed in order to grasp possible impacts on the environment such as noise, air, sea water, seabed material, marine biology and oceanography at locations as shown in Fig. 4.5.1. Contents and frequencies of the monitoring are shown in Table 4.5.1.

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[9]) *Source: Construction Technology on Artificial Island, Japan1995.*

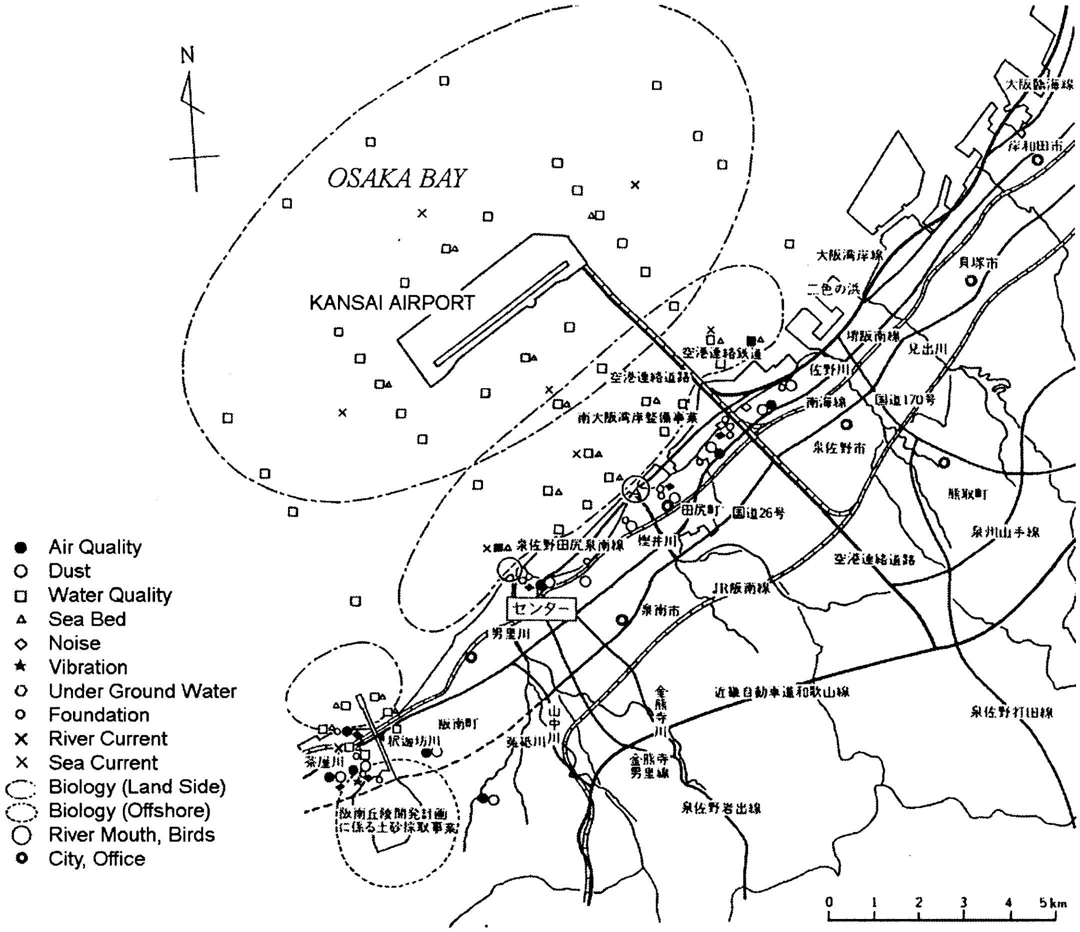

Figure 4.5.1 Location of Environmental Monitoring and Management on Kansai International Airport

Table 4.5.1 Environmental Monitoring and Management on Kansai International Airport

Items	Description	Location/Area	Measurement Points	Frequency
Noise	during Construction	opposite shoreline of artificial airport island	2 points	One(1) time/month, (change by progress)
Air quality, Meteorology	NOx (NO, NO2), SO2, CO, SPM, oxidant, wind (direction, speed), etc	ditto	Meteorological observing station	regular observation
Water quality	Turbidity, pH, DO, oil, current, etc	offshore at around artificial airports island and access bridge	20 points (change by construction progress)	One(1) time/day
	SS (suspended solid), VSS (volatile suspended solids)			One(1) time/week
	Turbidity, SS, VSS, pH, DO, COD, BOD, particle size distribution, etc		8 points	One(1)time/month
Sea bed quality	SS (suspended solid), particle size distribution	offshore at around artificial airports island and access bridge	5 points	Four(4) times/year
	pH, IL (ignition loss), COD, all-nitrogen, sulfide, etc			
Marine biology	Fishes, plankton, sea bed biology	offshore at around artificial airports island	5 points	Four(4) times/year
	Shoreline biology, Seaweed	revetment of artificial airport island	4 points	
	Etc	offshore at around artificial airports island	8 points	
Oceano-graphy	river mouth, shoreline (topography, bathymetric)	opposite shoreline of artificial airport island	2 locations	Two(2) times/year
	current conditions (direction, speed),	offshore at around artificial airports island	4 points	One time /Before and after construction

During the construction of the Artificial Island, predicted environmental impacts such as turbidity and noise were carefully monitored.

In the EIA report, a simulation analysis was conducted to examine dispersal of disposed material. The results of the simulation reveal that turbidities are significant depending on depths in an order of upper, middle and bottom layers. The maximum flash concentration of 2 mg/l in the bottom layer was recorded at locations approximately 1 km far from the construction site and the maximum flash concentration of 10 mg/l was expected at the reclamation site.

The results did not require silt fences at the construction site in accordance with the Japanese fishery regulations. However, considering the unexpected events during the construction works such as turbidities beyond the standardized values, silt fences were employed as shown in Figures 4.5.2 and 4.5.3.

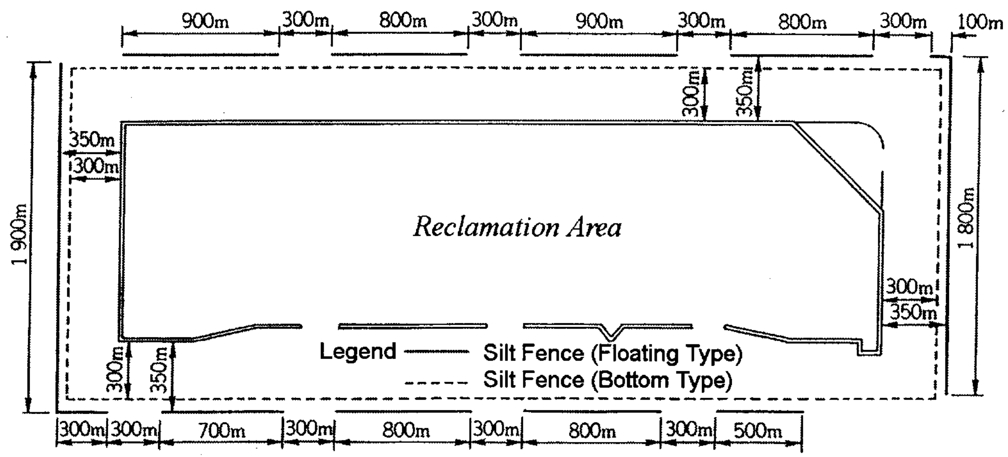

Figure 4.5.2 Layout plan of the silt protection/ fence

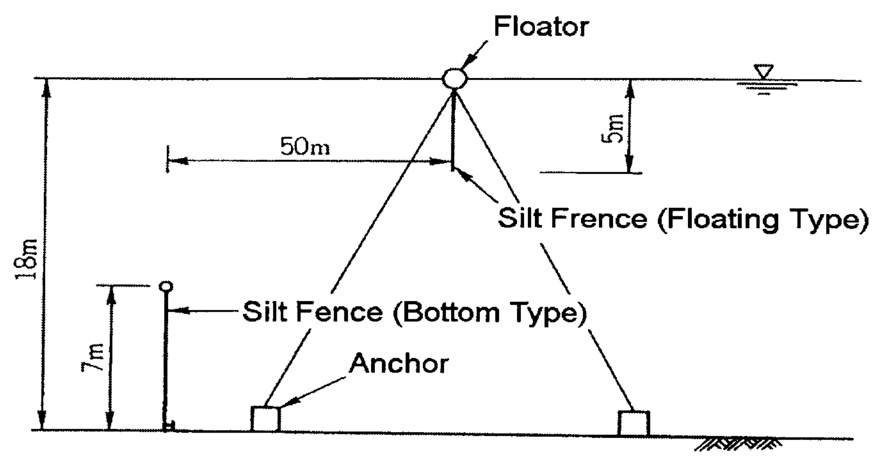

Figure 4.5.3 Install method of the silt protection/ fence

CHAPTER 5 CONCLUSION AND RECOMMENDATIONS

JETRO Preliminary Study on "Land Reclamation Alternatives at the Pacific Entrance to the Panama Canal" was carried out in cooperation with Autoridad del Canal de Panama (ACP) from December 2002 to March 2003. With a view to the beneficial usage of excavated materials coming from Panama Canal Expansion Plan activities, offshore land reclamation alternative (artificial island) and onshore reclamation alternative (peninsula) are proposed in consideration of technical and environmental aspects.

Artificial Island Construction Alternative:

Category	Item	Particular
Artificial Island	Size of Artificial Island	1,400m*1,050m = 147 ha
	Ground Level	M.L.W.S. + 7.00 m
	Volume of Reclamation Soil	28 million m3
Quaywall	Water Depth: Quaywall side Revetment side	M.L.W.S. - 16.75 m M.L.W.S. - 12.00 m
	Type of Structure	Prefabricated steel sheet pile cellular-bulkhead quaywall
	Construction Period	18 months
Land Reclamation	Material Transportation Method	By Barge
	Land Reclamation Method	By Reclaimer
Construction Cost	Quaywall Construction	$ 212 M.
	Land Reclamation	$ 50 M.
	Total	$ 262 M.

[Remarks]

Advantages of adopting prefabricated steel sheet pile cellular-bulkhead quaywall:

(1) Seawater pollution is minimized during land reclamation because the reclamation area is isolated from external seawater by cellular-bulkhead quaywall.

(2) Construction period is short.

(3) Quaywall can be used for container berth.

Peninsula Reclamation Alternative:

Category	Item	Particular
Peninsula	Size of Peninsula Reclamation	690 ha
	Ground Level	M.L.W.S. + 7.00 m
	Volume of Reclamation Soil	54 million m3
Quaywall	Water Depth	Less than M.L.W.S. -4.00 m
	Type of Structure	Double sheet pile wall
	Construction Period	26 months
Land Reclamation	Material Transportation Method	By Barge
	Land Reclamation Method	By Reclaimer and Bulldozer
Construction Cost	Revetment Construction	$ 70 M.
	Land Reclamation	$ 135 M.
	Total	$ 205 M.

Environmental Aspect:

In Chapter 4, environmental aspects are reviewed and recommended, and also an environmental monitoring measure executed in Japan is introduced. In addition, tidal current simulations are implemented.

Recommendations for Further Feasibility Study

In this preliminary study, due to time and data constraints, some bold assumptions have to be adopted to propose offshore/onshore land reclamation alternatives.

In the possible future feasibility study on land reclamation alternatives, following items must be studied for the implementation of this land reclamation project.

1. Review of ocean graphic studies
2. Review of seismic profiles results.
3. Selection of artificial island location.
4. Boring exploration on the sites.
5. Detail design of marine structures, including causeway and breakwater.

6. Detail materials transportation and handling analyses.
7. Detail construction planning.
8. Detail cost estimation.
9. Environmental study, including environmental monitoring plan.

APPENDICES

CONTENTS OF APPENDICES

MINUTES OF MEETING

This MINUTES OF MEETING (hereinafter called the "MOM") is made on this 13th day of December, 2002 as the results of the meetings held during 10th to 13th of the same month between Autoridad Del Canal de Panama (ACP) and the JETRO Study Team (JST) in order to define the Scope of Work for the "Preliminary Study on Land Reclamation Alternatives at the Pacific Entrance of the Panama Canal"(hereinafter the "Study") that Japan External Trade Organization (JETRO) will perform.

1. Objectives of the Preliminary Work
   a. Both parties agreed that the purpose of this study is for the beneficial usage of excavated materials coming from the Panama Canal Expansion Plan.
   b. Both parties agreed to include Peninsula alternative in addition to Artificial Island alternative.

2. Scope of Work

   ACP and JST discussed and agreed on the Terms of Reference as attached hereto.

3. ACP preparation of the Technical and Engineering Data Requested by JST

   ACP provided technical data related to the Study except the following information that could be available within the next dates:
   a. Analysis of Geologic Conditions for new lock site in January 2003.
   b. Seismic Profile Information within January 2003

4. Intellectual Property and Confidentiality

   All data and information provided by ACP and used under this Study are the property of ACP, and shall not be used, released to the public or issued by JST to other sources without written permission of ACP. However, due to the fact that JETRO is a governmental organization, JETRO shall have the right to disclose only the report of the Study to the public, so long as required by the laws of Japan.

IN WITNESS WHEREOF, the Parties have caused this MOM to be signed in their respective names by their duly authorized representatives as of the date first above written.

JETRO Study Team

T. Kokado

Mr. Takeshi Kokado
Team Leader

Autoridad del Canal de Panamá

Mr. Agustín Arias, Director
Engineering and Projects Department

TERMS OF REFERENCE

FOR THE PRELIMINARY STUDY

OF LAND RECLAMATION ALTERNATIVES

AT THE PACIFIC ENTRANCE TO THE PANAMA CANAL

December 13, 2002

AUTORIDAD DEL CANAL DE PANAMA (ACP)

JAPAN EXTERNAL TRADE ORGANIZATION (JETRO)

T.K.

1. OBJECTIVES OF THE PRELIMINARY STUDY

In order to consider the beneficial usage of excavated material coming from Panama Canal Expansion Plan activities, land reclamation at the Pacific entrance to the Panama Canal is under consideration. This Preliminary Study shall cover technical aspects on previously identified land reclamation alternatives, and is considered a preparatory study regarding a possible future full scope feasibility study on constructing an artificial island and/or peninsula at the Pacific entrance to the Panama Canal. This basic study will show the effectiveness of Japanese land-reclamation technology.

2. SCOPE OF WORK

To achieve the objectives mentioned above, the Preliminary Study will cover the following work items:

A. Preparatory Work:

a. Collection and review of the existing engineering studies, surveys and investigations relevant to the dry and wet excavation material disposal site alternatives from Panama Canal Expansion studies for the Pacific side.

b. Identification, collection and analysis of the engineering data necessary for the design of the offshore/onshore land reclamation alternatives, including meteorological, geo-technical, oceanographic, hydrographic, physical constraints (air space, navigation), etc.

c. Field visits to new lock sites and dry excavation disposal site alternatives.

* Regarding data collection for this preliminary study, ACP shall provide any data & information requested by engineers of the JETRO mission and available to the ACP, as much as possible.

B. Basic Proposal for Land Reclamation Alternatives:

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a. Proposal for designing the construction of two land reclamation alternatives: artificial island and peninsula by using Pacific new locks dry excavation material. Due to time and budget constraints, this preliminary study will include basic technical, and cost comparisons between both alternatives.

b. Proposal for optimal artificial island size based on container port operation activities.

c. Proposal for implementing seawall technology in the construction of land reclamation alternatives depending on the conditions of soil and ocean geology. The proposal will include technical, and cost comparisons between seawall and embankment technologies.

d. Proposal for onshore and offshore land reclamation plans with regard to the methods of filling and sea transportation from a designated site. The study will assume the island and peninsula location as proposed by the ACP artificial island preliminary study, and preliminary screening of alternative disposal sites for materials from locks excavation.

e. Recommendations for potential options for island and peninsula developments based on Japanese experience.

f. For both land reclamation alternatives, JETRO will study the land filling and transportation, and will use ACP artificial island preliminary study data as follows:

	Transportation type	Transportation to the site	Reclamation	Take-over point
ARTIFICIAL ISLAND	Railway	ACP data	JETRO design	Delivered at reclamation site
	Barge	JETRO estimate	JETRO design	FOB Loading Point (ACP data) + Loading Cost+ Cost estimation of transportation between Loading Point and Reclamation Point
PENINSULA	Railway	ACP data	JETRO design	Delivered at reclamation site
	Barge	JETRO estimate	JETRO design	FOB Loading Point (ACP data) + Loading Cost + Cost estimation of transportation between Loading Point and Reclamation Point subject to navigation feasibility of Barges.

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g. Proposal for environmental monitoring plans (EMP) during time periods before-construction, during-construction and after construction.

* These plans should be based on Japanese technologies, which can reduce the impacts to the environment.

C. Cost Estimation:

Estimation of approximate costs of constructing seawalls, land reclamation, and dry excavation materials transportation by barges.

D. Preparation of Feasibility Study:

a. Discussion of the study components that may be implemented in a possible future feasibility study by the ACP and the JETRO mission.

b. Preparation of draft of the terms of reference and work program that may be implemented for a future feasibility study. The execution of such future feasibility study is not covered under the existing MOM and will be negotiated under a separate MOM.

3. SCHEDULE OF THE PRELIMINARY STUDY

	Dec. 2002	Jan. 2003	Feb. 2003	Mar. 2003
(1) Collection of Engineering Data				
(2) Proposal for Land Reclamation Alternatives				
(3) Cost Estimation				
(4) Preparing Report				*

Note: ▬▬ Study in Panama, ——— Study in Japan, * Submit final report and technical presentation

4. EXPERTS REQUIRED

Experts required for the implementation of the preliminary study shall include at

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least, but may not be limited to, the following:

1) Team Leader
2) Marine Engineer
3) Geo-Technical Engineer
4) Structural Engineer
5) Construction and Cost Surveyor
6) Environmental Engineer

T.K.

Prediction of the Tidal Current Variations Resulting from the Construction of an Artificial Island in the Gulf of Panama

Report

March 2003

JETRO STUDY TEAM

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LIST OF FIGURES

1. Introduction

1-1. Introduction

The construction of an artificial island at the Pacific Entrance of the Panama Canal may affect the existing oceanographic condition of the region, such as the wave or current pattern. Under such circumstance, it is essential to acquire an understanding of those effects prior to implement the proposed construction.

In this report, the tidal current has been observed and reproduced to predict the tidal current variations using numerical simulation model in order to determine the influence of the construction of an artificial island.

There are two cases of simulation phenomena that have been formulated, i.e., one for an Artificial Island proposed to construct at about 5 kilometers offshore and the other one is a peninsula near the Pacific Entrance of the Panama Canal.

See Figure 1-1 showing the map of Central America and Figure 1-2 showing the proposed locations of the artificial island and peninsula.

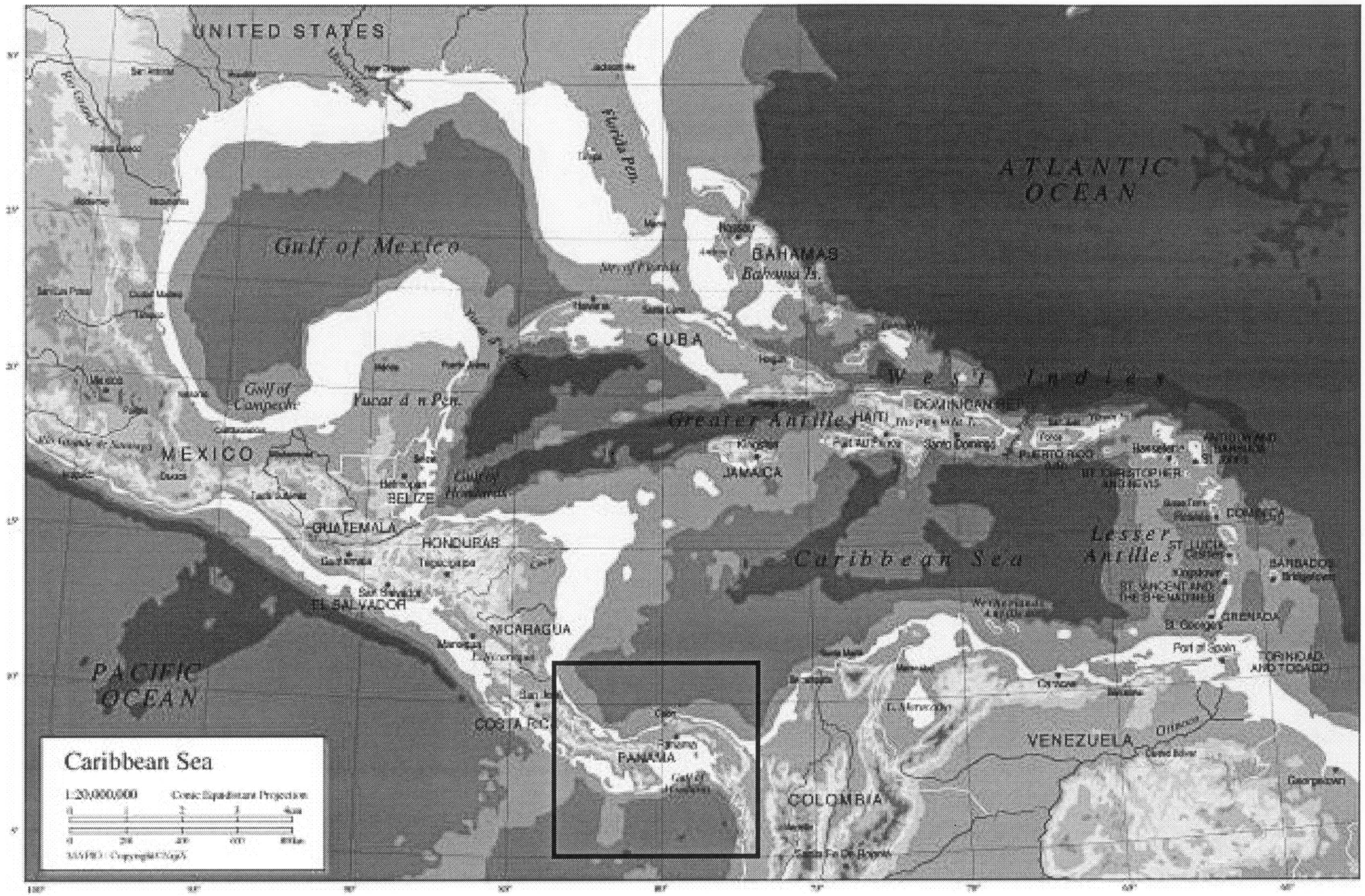


Figure 1-1 : Map of Central America

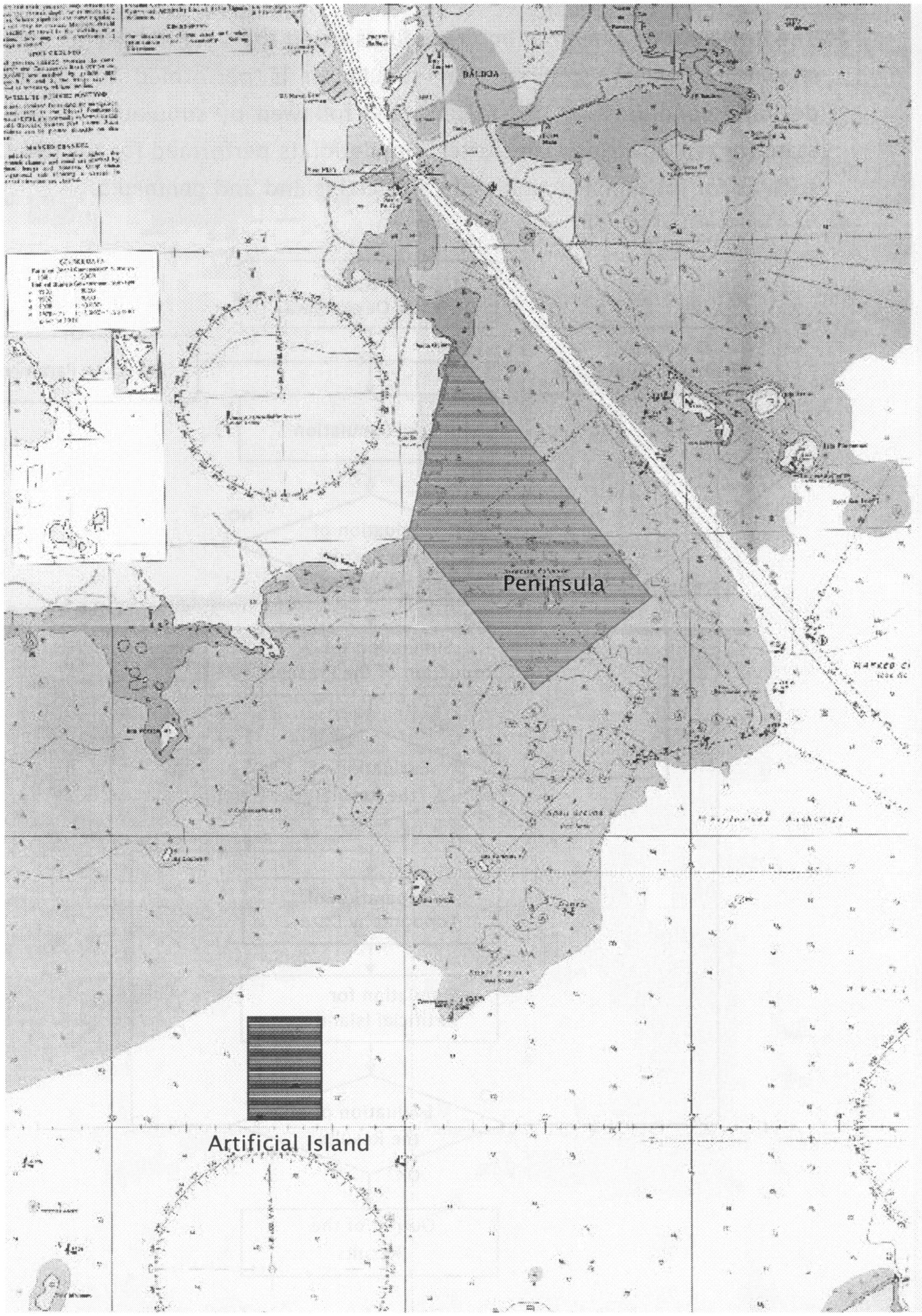


Figure 1-2 :   Location of the Proposed Artificial Island and Peninsula

1-2. Flowchart of Examination Procedures

The flowchart of examination procedures using the tidal numerical simulation is shown in Figure1-3. First, trial simulation is performed to determine the deciding conditions or parameters, and followed by simulation of the present state for reproduction. Thereafter, simulation is performed for the two (2) cases, such as for the construction of an artificial island and peninsula.

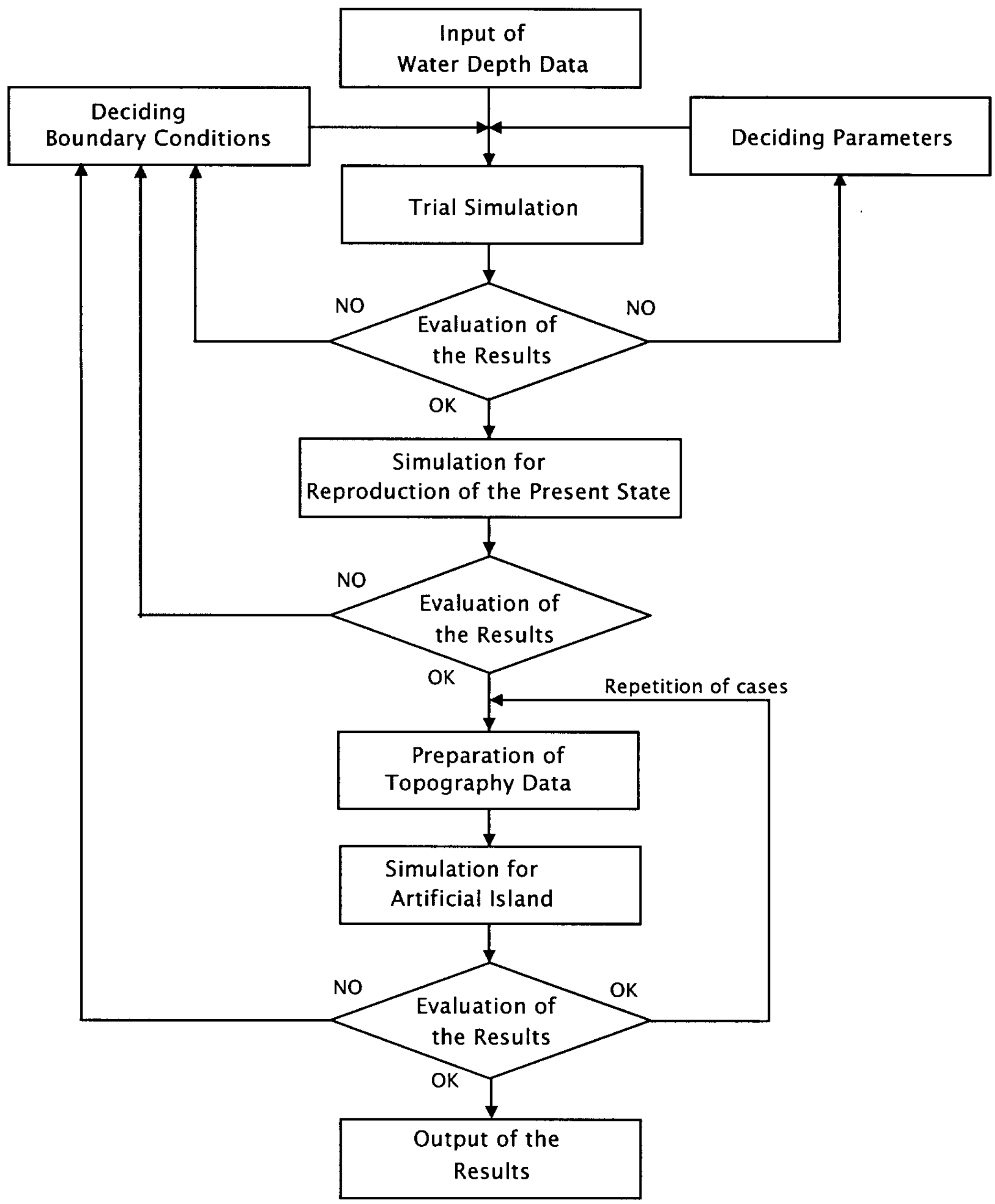


Figure 1-3 : Flowchart of Examination

2. Numerical Model

2-1. Basic Equations

In the coastal area, tidal current is basically supposed to be barotropic, which means water density is uniform in the vertical direction. Because horizontal velocity of tide and tidal current are usually very large compared with vertical velocity, a numerical model for tidal current generally deals with horizontal 2-D simulation.

The basic equations employed are the Navier-Stokes equations to obtain the incompressible fluid, under the shallow water and the Bussinesq assumption and continuity equation. These equations are described as

$$\frac{\partial u}{\partial t}+u\frac{\partial u}{\partial x}+v\frac{\partial u}{\partial y}-fv=-g\frac{\partial \eta}{\partial x}+A_h\frac{\partial^2 u}{\partial x^2}+A_h\frac{\partial^2 u}{\partial y^2}+\frac{\tau_x}{h} \quad \text{-----------(1)}$$

$$\frac{\partial v}{\partial t}+u\frac{\partial v}{\partial x}+v\frac{\partial v}{\partial y}+fu=-g\frac{\partial \eta}{\partial y}+A_h\frac{\partial^2 v}{\partial x^2}+A_h\frac{\partial^2 v}{\partial y^2}+\frac{\tau_y}{h} \quad \text{-----------(2)}$$

$$\frac{\partial \eta}{\partial t}+\frac{\partial (hu)}{\partial x}+\frac{\partial (hv)}{\partial y}=0 \quad \text{-----------(3)}$$

Where:

u, v	:	horizontal velocity
η	:	surface elevation
h	:	water depth
f	:	Coriolis parameter
g	:	Gravitational acceleration
A_h	:	Eddy viscosity
τ_x and τ_y	:	Bottom friction

Due to some difficulties solving those basic equations accurately, the Difference Method is applied in this numerical simulation, which is also one way of solving the approximate values.

2-2. Boundary Conditions

The bottom friction is considered to occur on the bottom of the sea and that model is described in the following equation:

$$\tau_x = -\gamma_b^2 \left(u^2 + v^2\right)^{1/2} u \qquad \text{-----------(4)}$$

$$\tau_y = -\gamma_b^2 \left(u^2 + v^2\right)^{1/2} v \qquad \text{-----------(5)}$$

where, the parameter γ_b stands for the coefficient of bottom friction.

The shoreline has been defined by slip condition where the horizontal friction is equal to zero. And the amplitude of M2 tide has defined the open boundary.

3. Area Description

3-1. Domain for Numerical Simulation

The domain for numerical simulation of tidal current is shown in Figure 3-1. The covered area is 70 kilometers from the east and west direction and 60 kilometers from the south and north direction. The water depth in the offshore is over 60 meters.

As strictly considerable, the entire Gulf of Panama shall be simulated first. Thereafter, the local area shall be simulated using the results obtained from the simulation of the entire gulf in order to determine the open boundary conditions (as shown in Figure 3-1). However, in this analysis, the local area is directly simulated because this is aimed only to acquire the outline of the influences by the proposed construction.

In this simulation, the water depth is defined by grid spacing of 500 meters. Notwithstanding the actual depth of over 60 meters in offshore and below 2.0 meters in nearshore, the assumption used for these figures are taken as approximately 60 meters and 2.0 meters respectively for the stability of the numerical simulation.

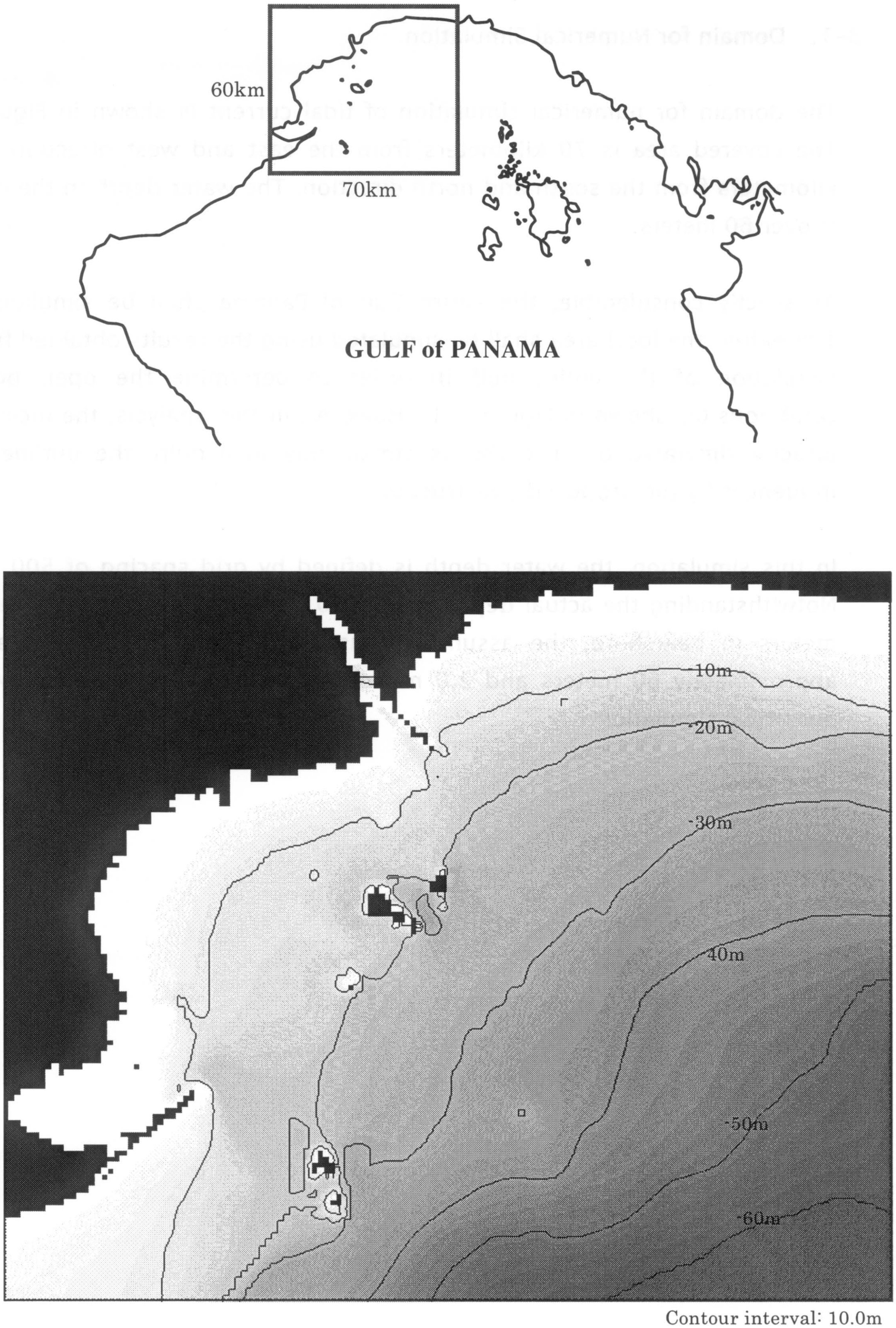
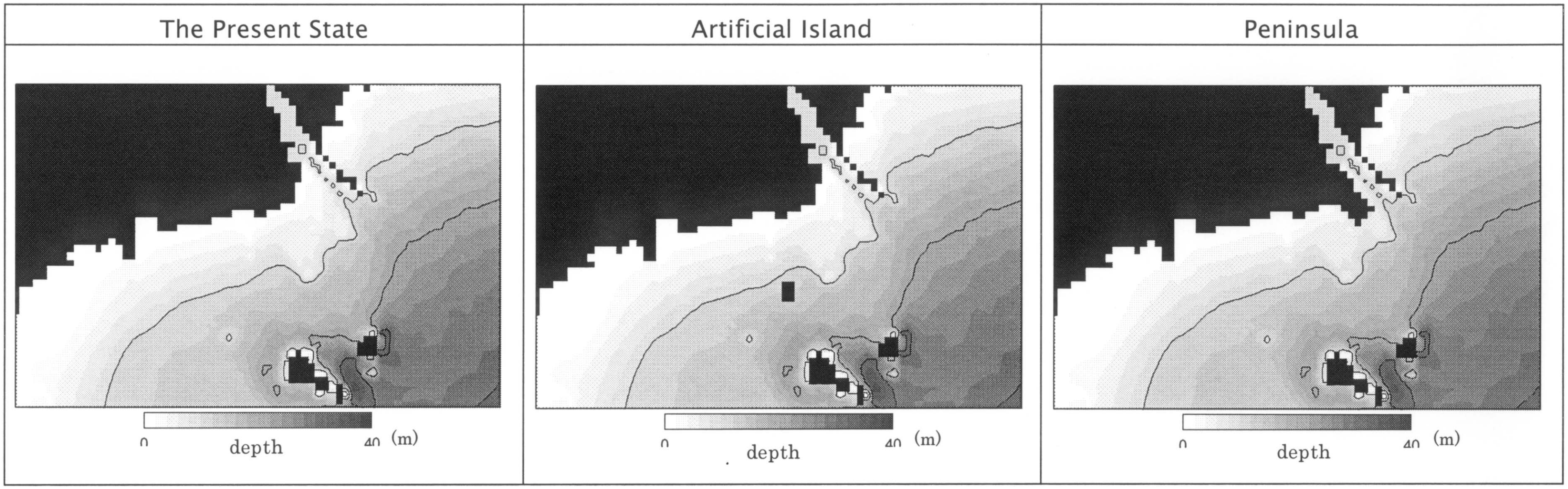


Figure3-1 :　Domain for numerical simulation

3-2. Location of an Artificial Island and Peninsula

In numerical simulation, the artificial island or peninsula are dealt with as a land, see Figure 3-2.



Contour interval: 10.0m

Figure3-2 : Location of the proposed Artificial Island and Peninsula

4. Simulation Conditions

4-1. Simulation Conditions

The objective of this analysis is aimed to acquire an understanding of the outline of influences that may bring by the construction of an artificial island or peninsula. Therefore, only the tidal and oceanic currents have been considered as the driving forces in the numerical simulations. The wind force is not considered and the water density is set uniformly in the entire domain.

The numerical simulation has been performed for five (5) periods of tidal wave using the same tide. The result obtained in the last one period is applied in the analysis. Moreover, the steady state of the tidal currents is confirmed after successive simulation of five (5) tidal periods.

4-2. Open Boundary Conditions

4-2-1. Tidal Currents

The open boundary has been set using the amplitude of M2 tide, which is the most significant tide at the project site with the period of about 12 hours and 25 minutes.

The amplitude of the spring tide of M2 tide has been applied to the open boundaries because of several reasons. First, the influence of the construction of an artificial island or peninsula is seemed to be more significant during spring tide, because the velocity of the tidal currents is faster during spring tide. Secondly, the tidal current in the Gulf of Panama as estimated by Bennett (1965) [3] is considered as the object of the comparison with the results of simulation. Figure 4-1 shows the current patterns by Bennett.

The tide amplitude at the open boundary is defined by the trial simulations in the parameter considering the observed data reported on the Autoridad del Canal de Panama (2002)[2] .

Then, in all open boundaries, the amplitude of the M2 tide is spatially equal and the phase difference has been defined to zero.

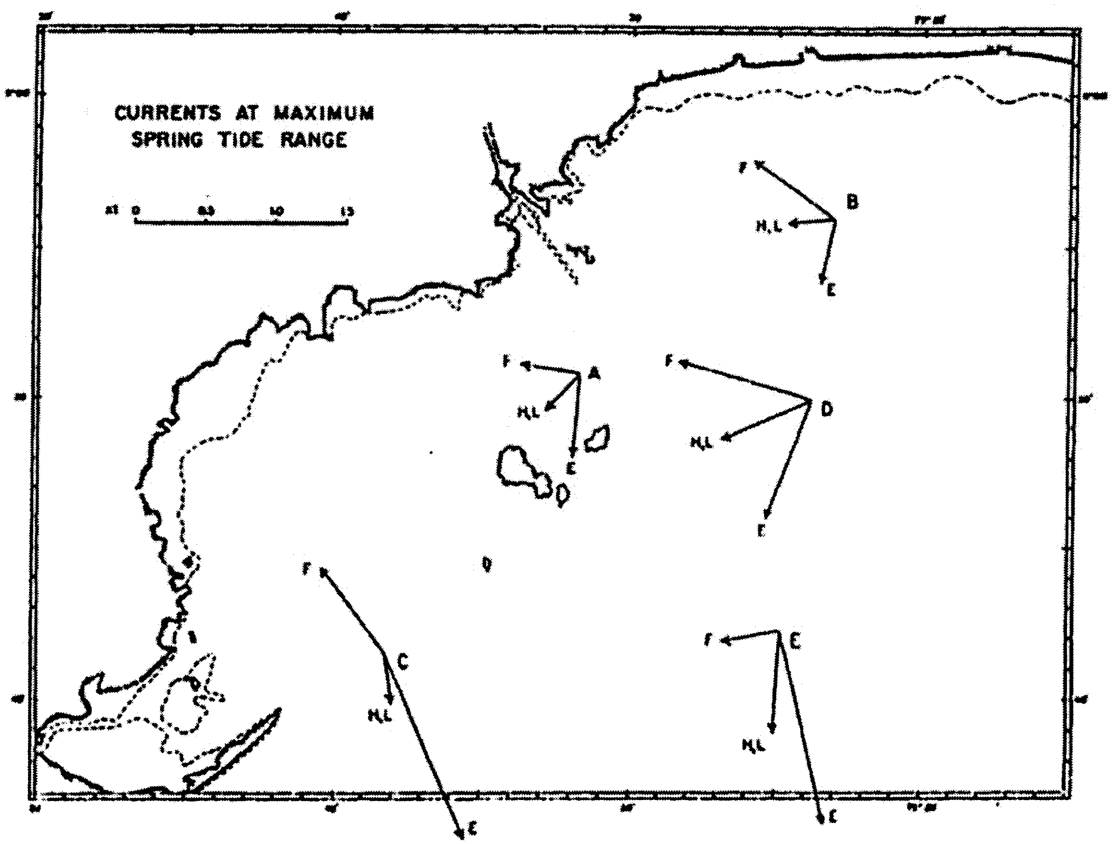


Figure4-1 :   Currents Observation at Panama Bay during spring tide from Bennett (1965)
(where E,F,H and L present Peak Ebb, Peak Flood, High Water and Low Water respectively.
A,B,C,D and E stand for the observation points.)

4-2-2. Oceanic Current

The current in the Gulf of Panama is another significant driving force that is equal to tidal currents. In this analysis, it is tried to bring into the simulation by the fixed difference of surface elevation in the open boundary.

The difference of surface elevation is defined as follows (Figure 4-2). The surface elevation on the C-D line of the open boundary (η_{CD}) is higher than that of the A-B line (η_{AB}) and the B-C line (η_{BC}) at all times. Trial simulations of the parameter define the magnitude of difference.

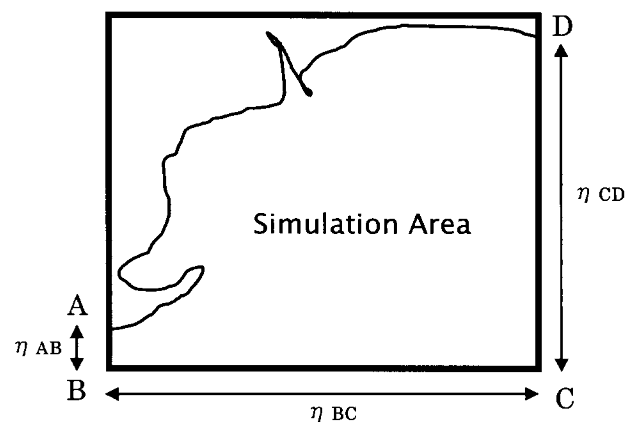


Figure4-2 : Artificial tilting of the boundary

4-3. Model Parameters

The following parameters presented in Table 4-1 are applied in the model. Eddy viscosity, A_h is based on Richardson's 4/3 Law. Coriolis parameter, f is calculated ~~by~~ using the following equation (6).

$$f = 2\Omega \sin\phi \qquad \text{-------------- (6)}$$

where:

Ω : Angular velocity (7.92×10^{-5} (s^{-1}))

ϕ : latitude of the Gulf of Panama (9.0^{o})

Table4-1 : Model Parameters

Horizontal Grid Size	500 (m)
Time Step : dt	60 (s)
Simulation Period	62 hours (about 5 periods of M2 tide)
Water Density : ρ_0	1,020 (kg/ m^3)
Gravitational Acceleration: g	9.78 (m/s^2)
Eddy Viscosity : A_h	10.0(m^2/s)
Coriolis Parameter : f	2.28×10^{-5}(s^{-1})

5. Simulation Results and Issues

5-1. Reproduction of the Present State

In order to verify the ability of the model to simulate the current patterns in the Panama Bay, the results of the model are compared with available data.

Figure 5-1, 5-2, 5-3, 5-4 and 5-5 show the simulated current patterns during spring tide at present state. The flow patterns are presented at four (4) stages of the tide: low water, peak flood, high water and peak ebb.

First, Figure 5-1 and 5-2 show the simulated current patterns during spring tide together with the estimated currents by Bennett (1965)[3]. The vector of red color in the current patterns of Bennett presents a target of comparison at each stage. It can be confirmed that the model reproduces sufficiently the magnitude and direction of the currents at all observation points in Bennett.

Secondly, Figure 5-3 and 5-4 show the comparison of numerical model with the simulation of ACP Feasibility Study (2002)[2] for current patterns during spring tide. It can be confirmed that the model sufficiently reproduces the magnitude and direction of the currents at three stages except for peak flood. Some discrepancies in the direction of currents are observed at the flood current. At the peak flood, the direction of the simulated current seems to be more influenced by the Gulf of Panama current. But, from the comparison with Bennett (1965) as shown in Figure 5-1, the current patterns at the peak flood is reproduced accurately. It can be concluded that the model represents accurately the magnitude and direction of the currents at the present stage.

Figure 5-5 presents the flow patterns in the project area in the four stages. The colored contour shows the magnitude of the velocity. The influence of the current of the Gulf of Panama to the flow patterns in the project area is easily observed, especially during low and high water conditions.

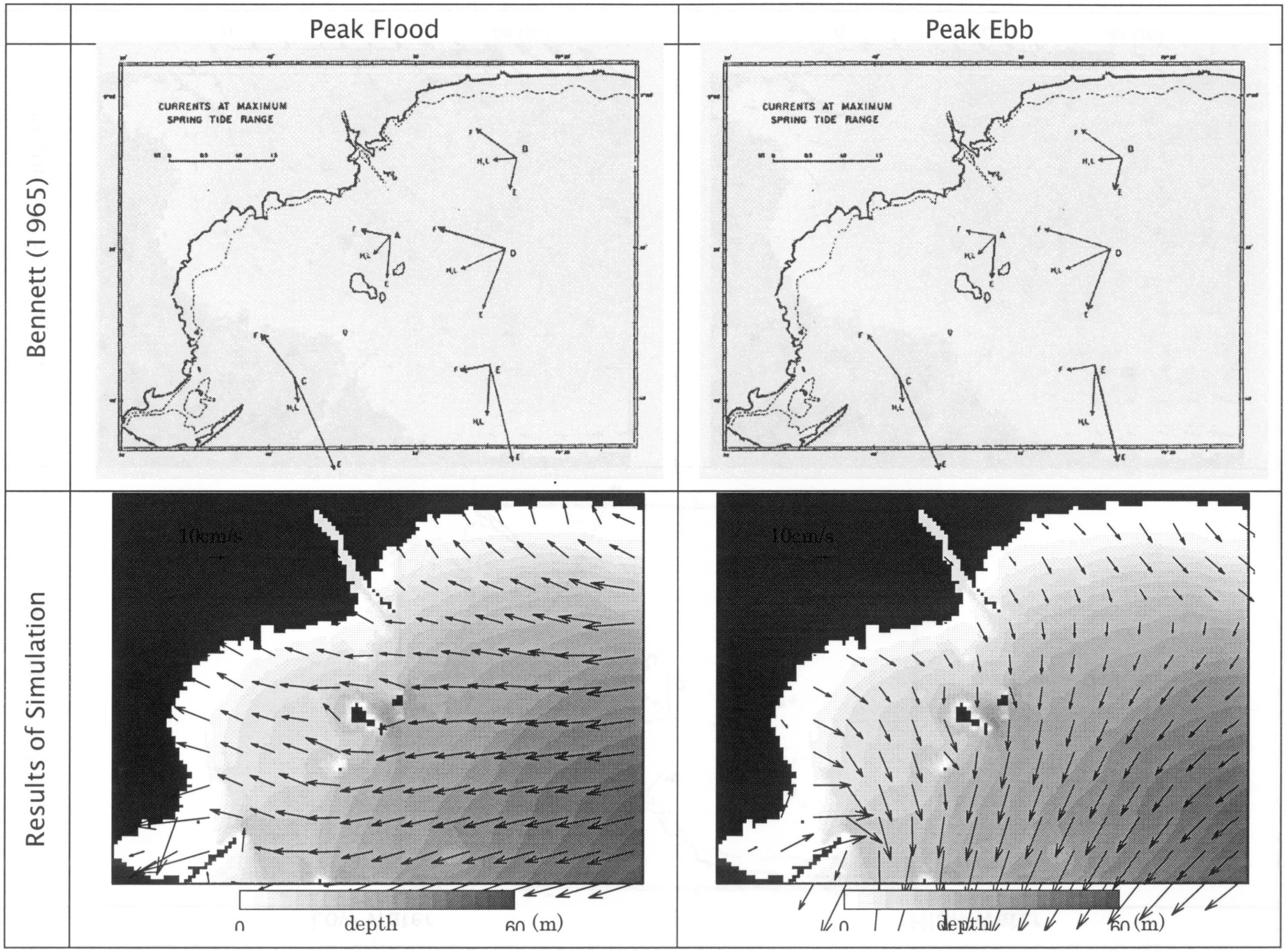


図 5-1 :　Comparison of the numerical model with Bennett (1965) for current patterns during spring tide

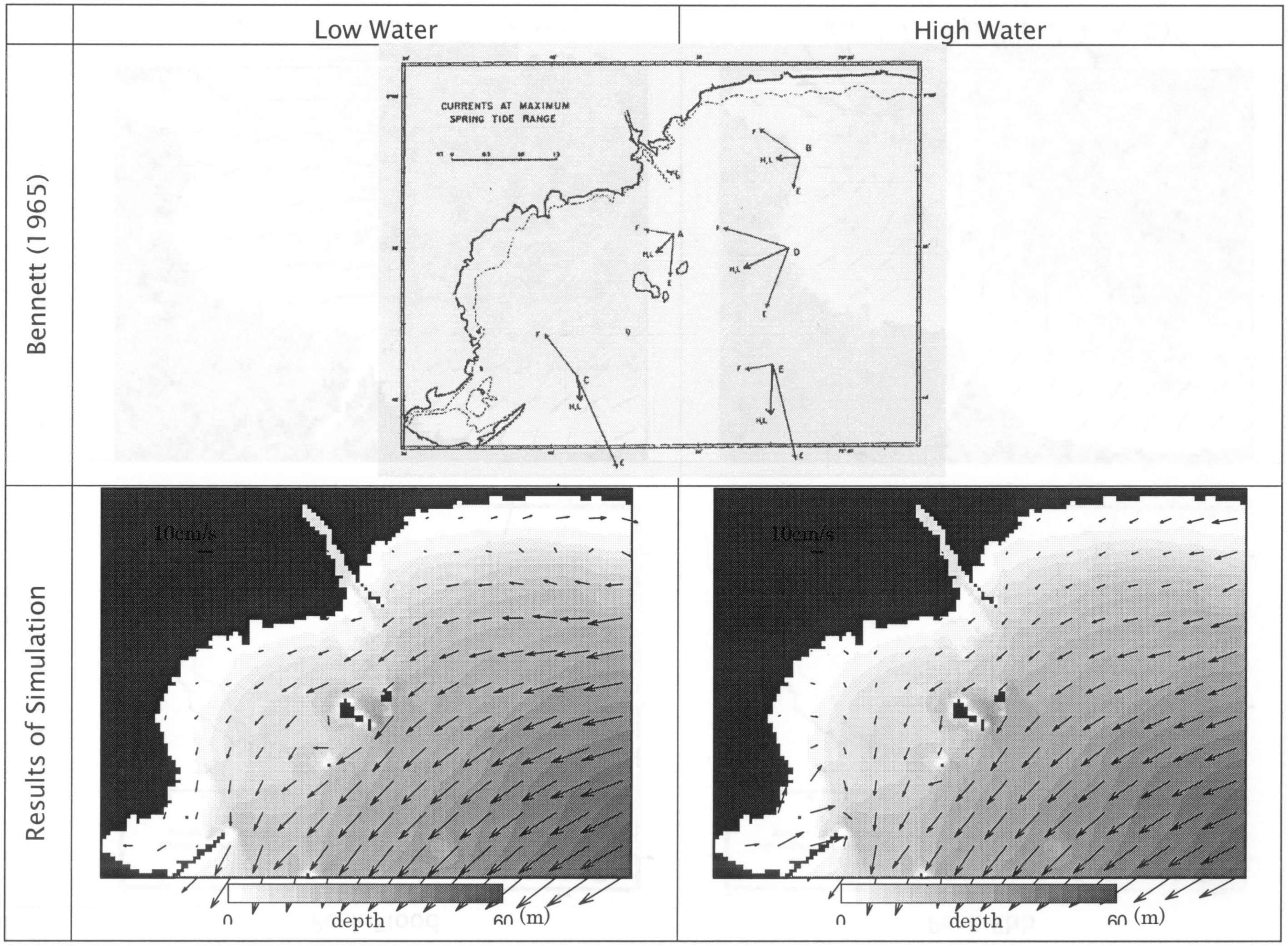


Figure5-2 : Comparison of the numerical model with Bennett (1965) for current patterns during spring tide

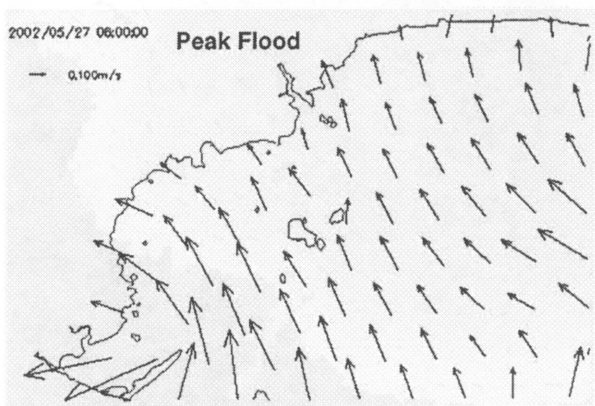
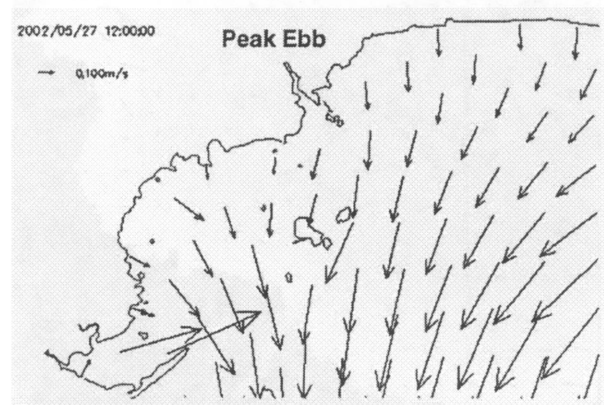
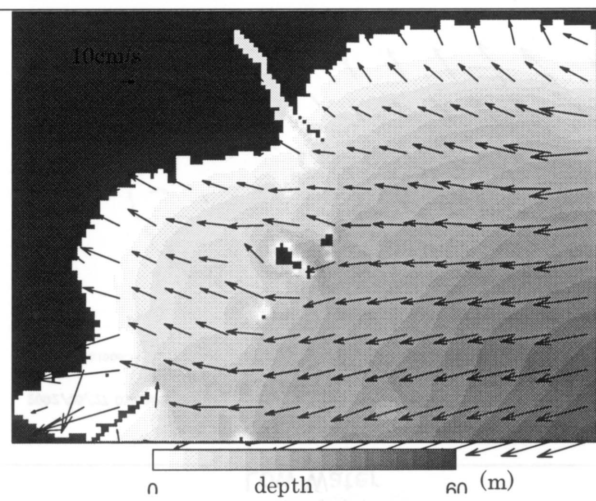
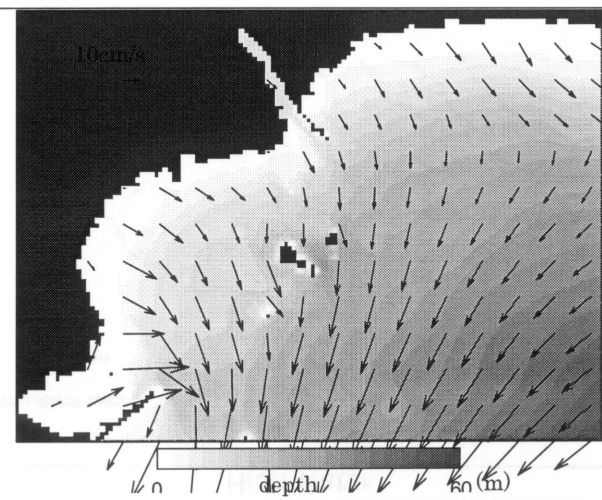
	Peak Flood	Peak Ebb
ACP Feasibility Study (2002)		
Results of Simulation		

Figure5-3 : Comparison of the numerical model with the simulation of ACP Feasibility Study (2002) for current patterns during spring tide

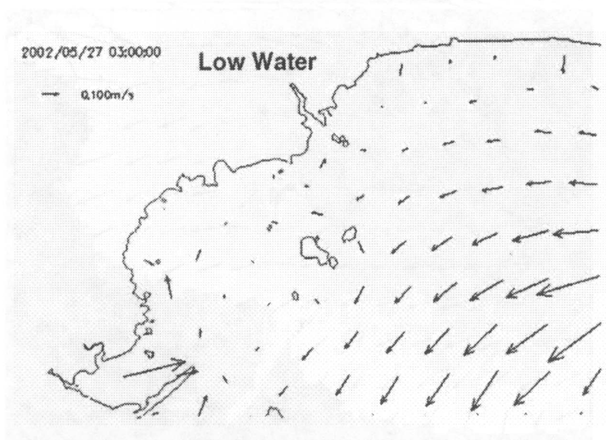
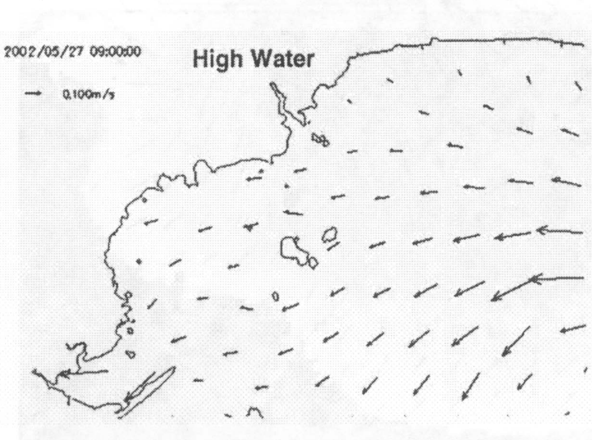
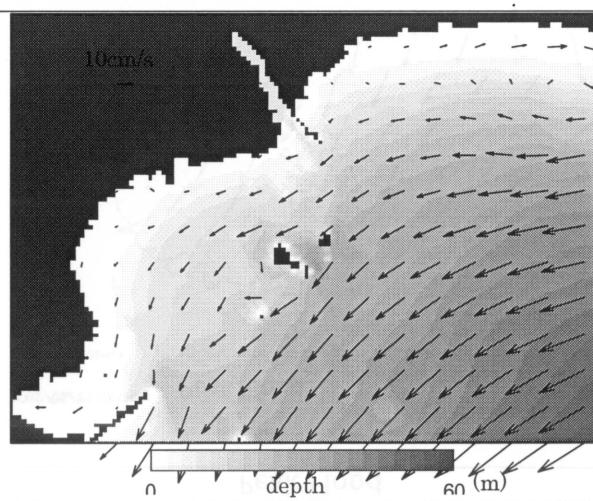
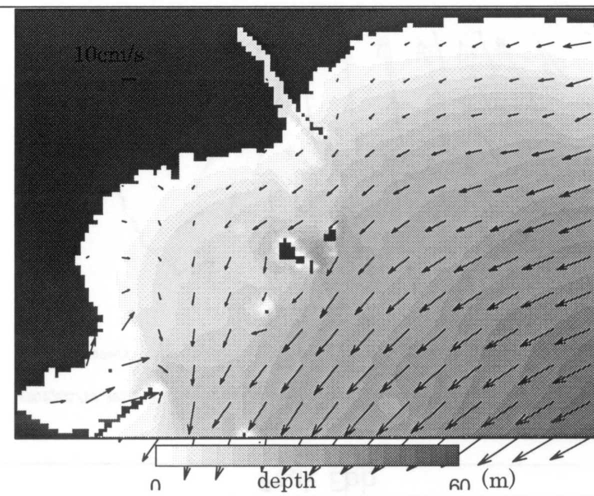
	Low Water	High Water
ACP Feasibility Study (2002)		
Results of Simulation		

Figure5-4 : Comparison of the numerical model with the simulation of ACP Feasibility Study (2002) for current patterns during spring tide

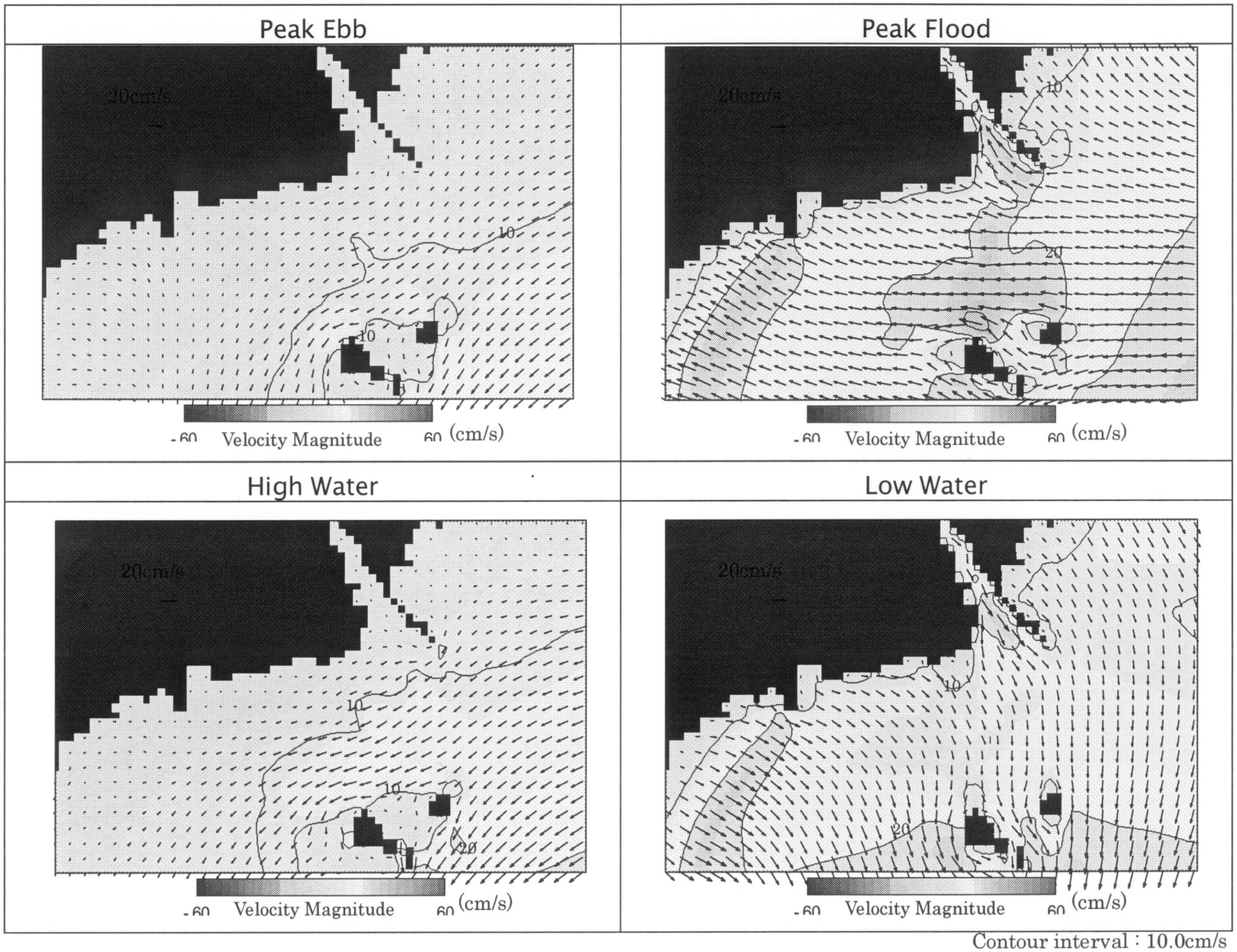


Figure5-5 : Simulated current patterns during spring tide at the project area

5-2. The Case of Artificial Island and Peninsula

Figure 5-6 to Figure 5-11 show the simulated current patterns in both cases of an artificial island and peninsula during spring tide. The flow patterns are presented at four stages of the tide: low water, peak flood, high water and peak ebb. For comparison purpose, the current pattern at the present state is presented herein.

Figure 5-6 and 5-7 present the simulated current patterns for the artificial island and peninsula during spring tide. It turned out that the global flow patterns of the Gulf of Panama are not affected significantly by their construction.

Figure 5-8 and 5-9 show the simulated current patterns at the project area during spring tide. Colored contour indicates the magnitude of the velocity. It can be seen from the figure that the velocity magnitude around the artificial island or peninsula is variable. Clarifying these variations, the contour of current variations is compared with the present state during spring tide as shown in Figure 5-10 and 5-11. Here, the vectors of the velocity in the figures are just the same with those of the figure 5-8 and 5-9. It is predicted that the current patterns around the artificial island and peninsula have changed at four stages of the tide.

This is especially in the case of an artificial island, where the variations of the velocity of the tidal current seemed to be significant at the flood current. The variations have ranged almost from -15 cm/s to +5 cm/s and significant only within a radius of 2 kilometers around the artificial island. On the other hand, in the case of peninsula, the variations of the velocity of the tidal current seemed to occur significantly at the peak flood and peak ebb. The maximum magnitude of the variations seemed to be about +10 cm/s at the Pacific entrance of the Panama Canal and about -10 cm/s around the peninsula. However, these changes can be estimated only at about 50 percent compared with the present state. Furthermore, the direction of the current has not changed significantly in both cases.

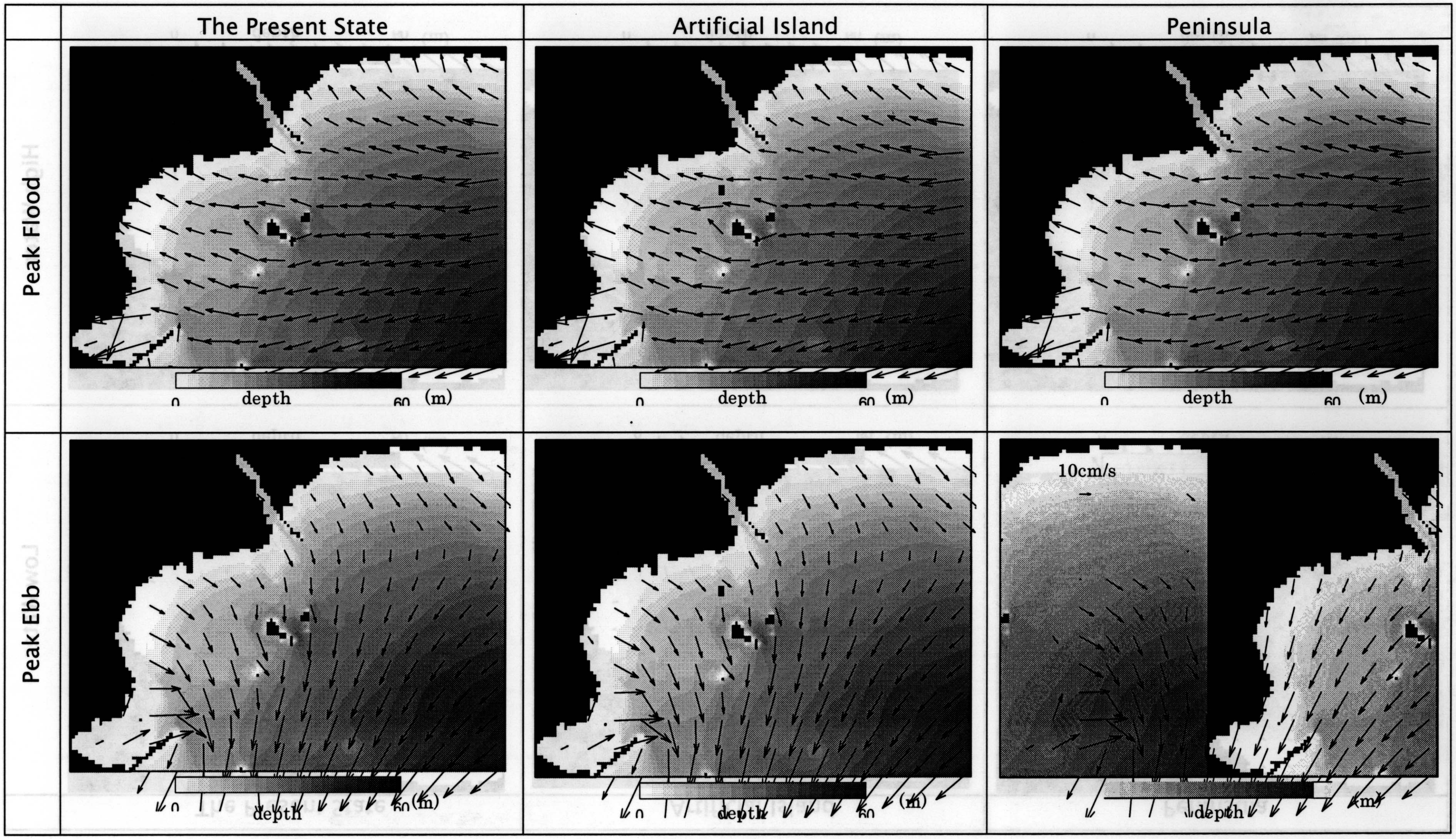


Figure5-6 : Simulated current patterns in the case of an Artificial Island and Peninsula during spring tide

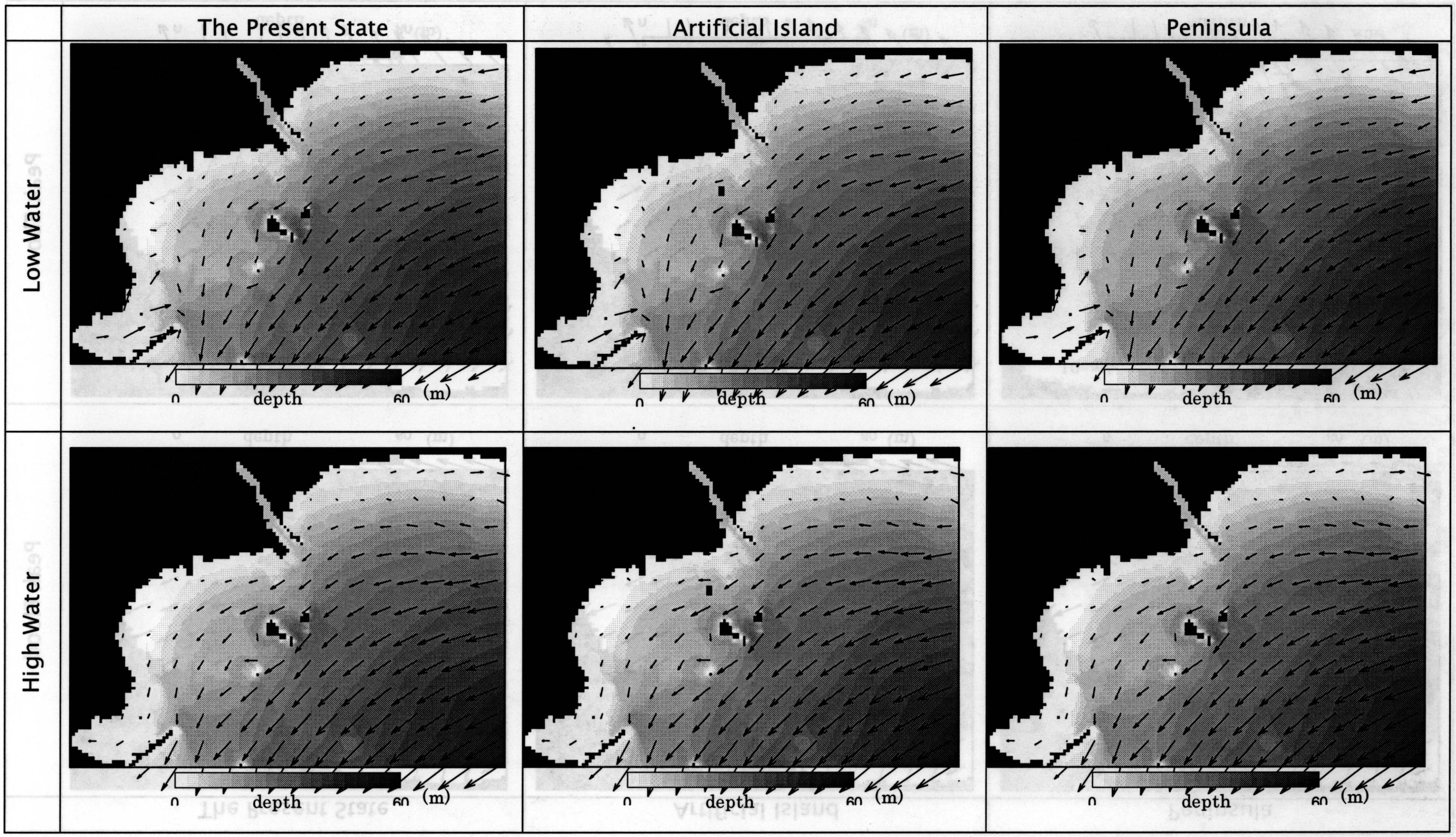


Figure5-7 :　Simulated current patterns in the case of an Artificial Island and Peninsula during spring tide

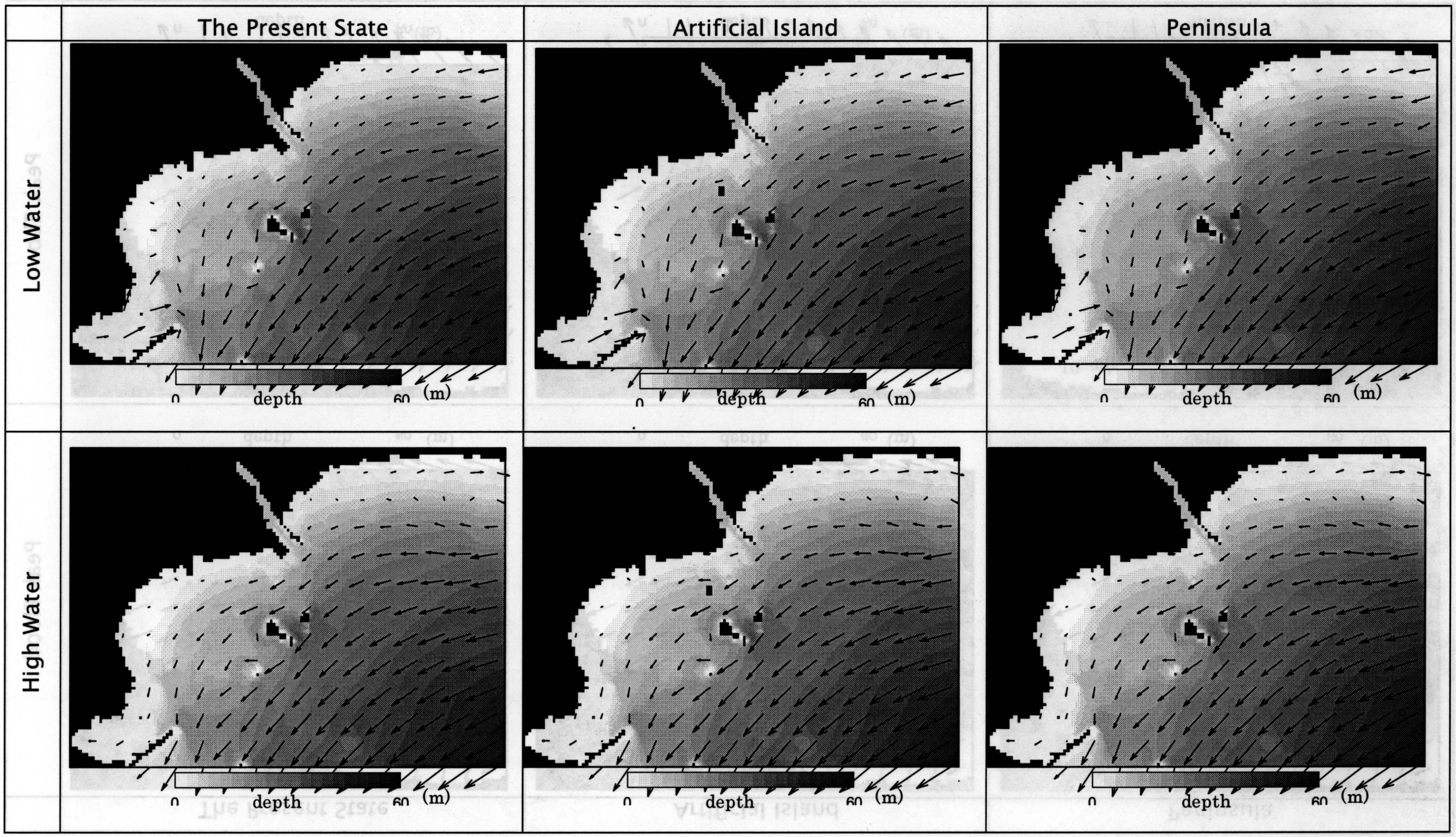
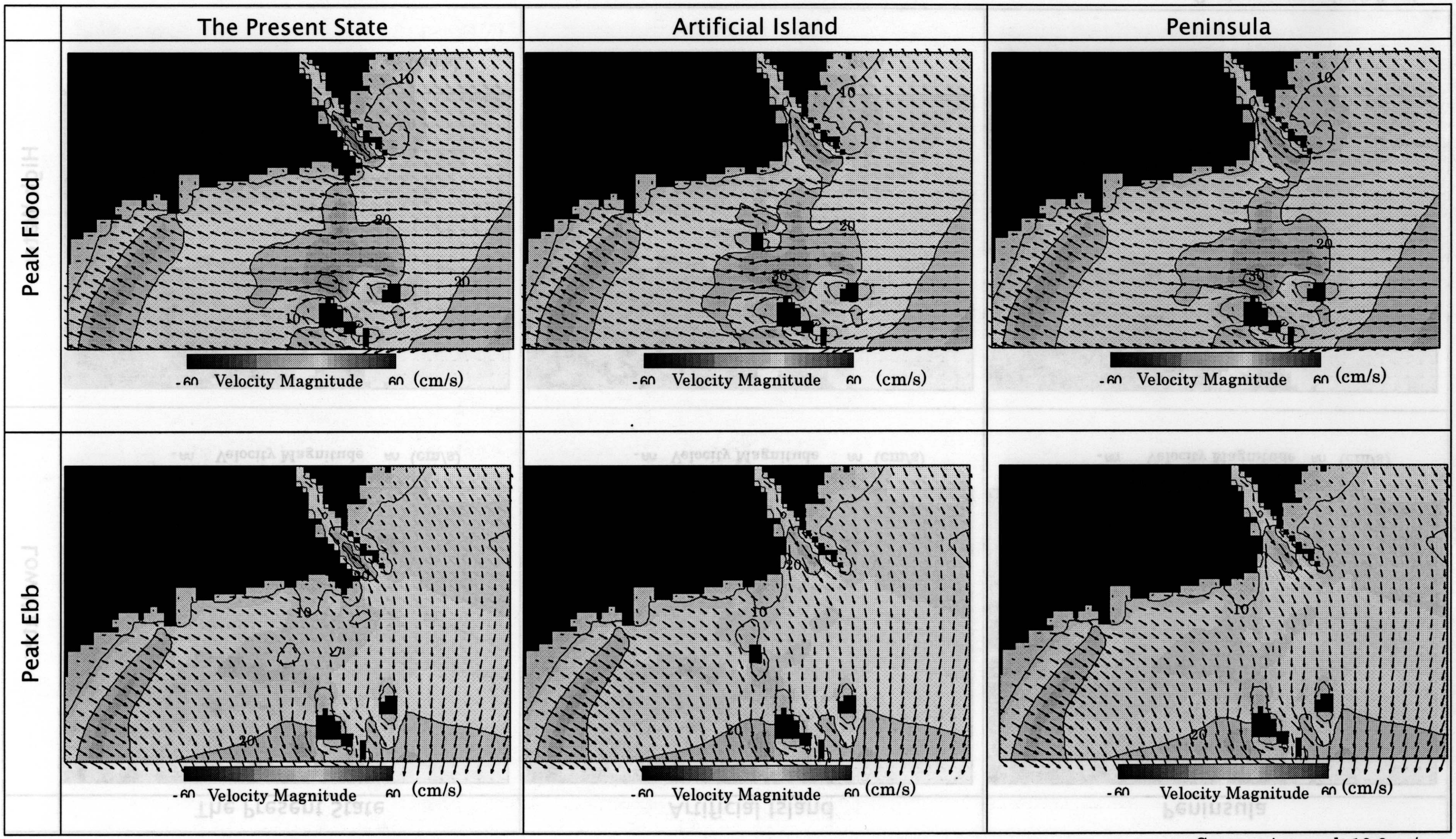
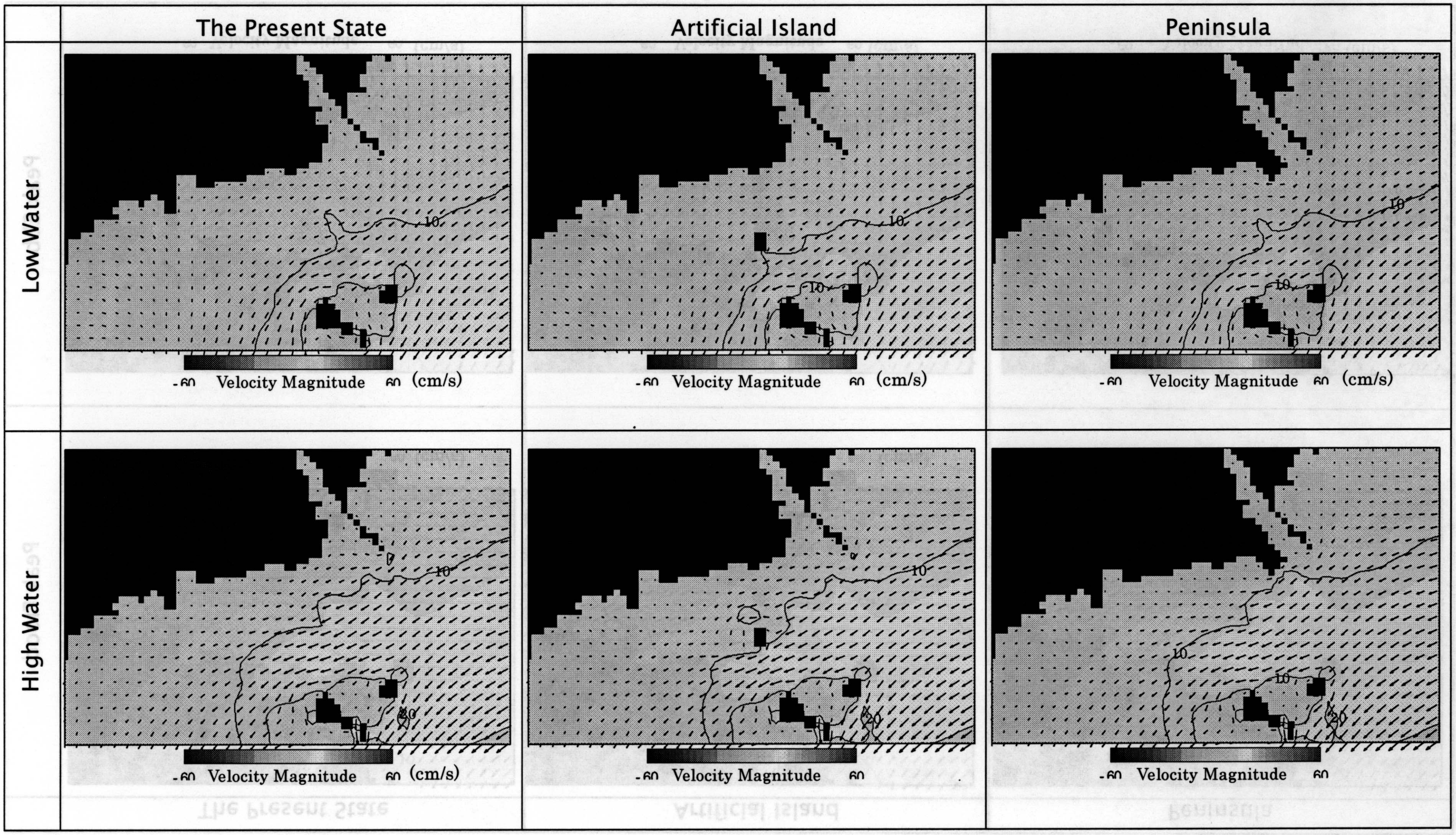


Figure5-7 :　Simulated current patterns in the case of an Artificial Island and Peninsula during spring tide



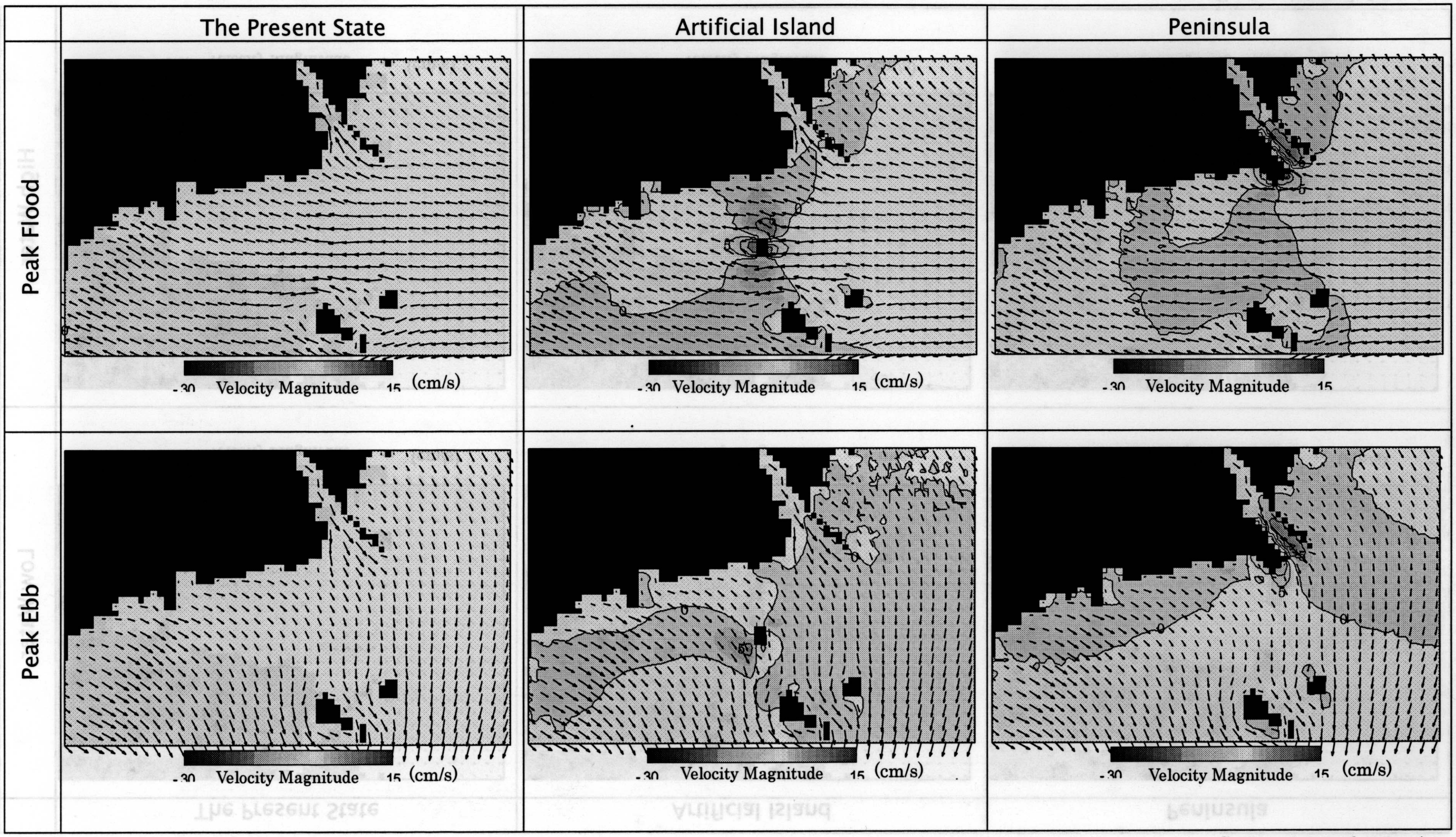
Contour interval: 10.0cm/s

Figure5-8 : Simulated current patterns in the case of an Artificial Island and Peninsula at the project area



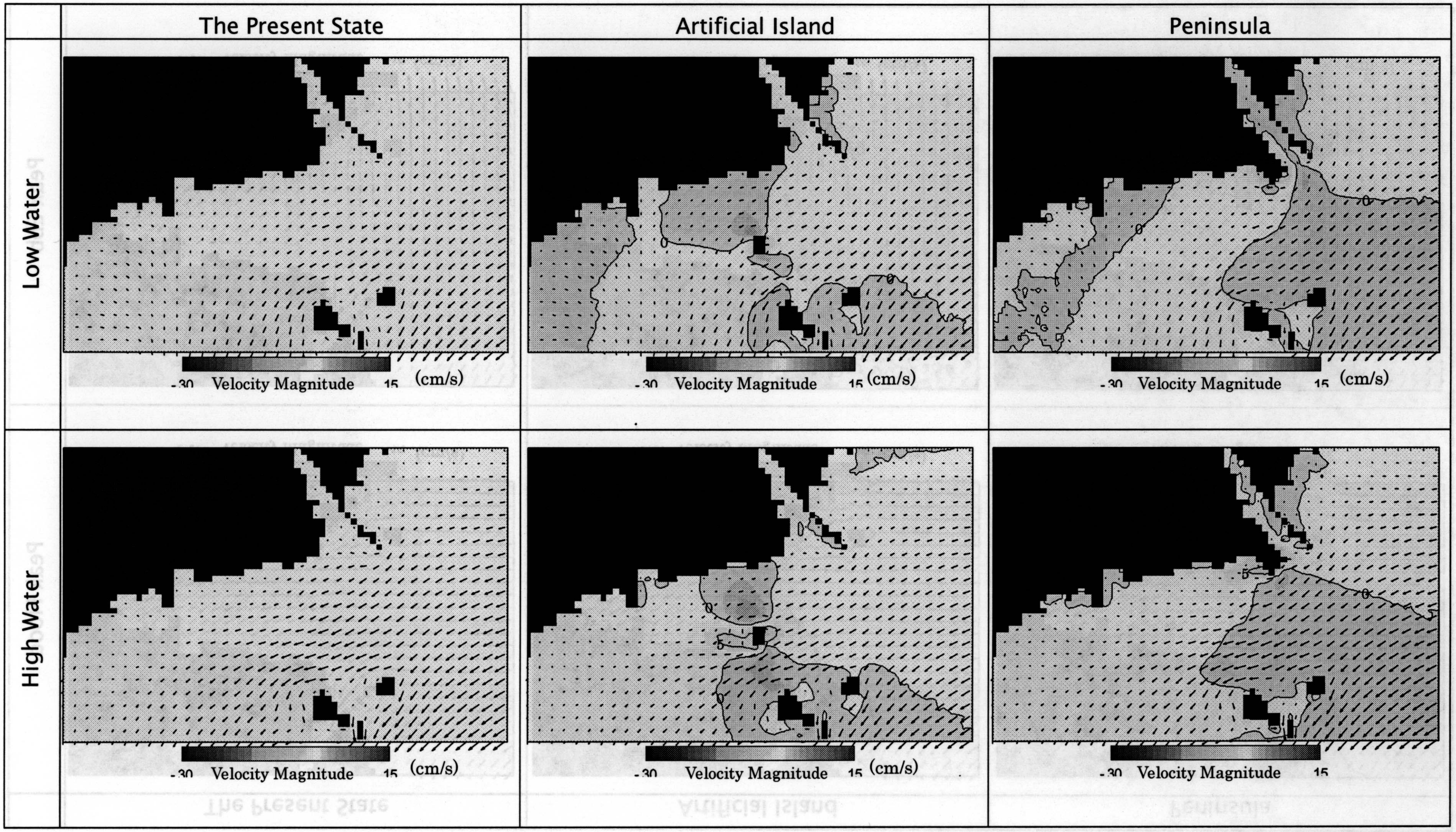
Contour interval: 10.0cm/s

Figure5-9 : Simulated current patterns in the case of an Artificial Island and Peninsula at the project area



Contour interval: 5.0cm/s

Figure5-10 : Simulated current patterns and contour of current variation compared with the present state during spring tide



Contour interval: 5.0cm/s

Figure5-11 : Simulated current patterns and contour of current variation compared with the present state during spring tide

5-3. Residual Flow

The results of the present state and ~~the~~ cases of the construction are shown in Figure 5-12, 5-13 and 5-14, where they present the tidal residual flow in the final period of M2 tide during spring tide.

Figure 5-12 shows the simulated current patterns of tidal residual flow in the Gulf of Panama during spring tide. Figure 5-13 shows the current patterns at the project area. The contour of current variation of the tidal residual flow compared with the present state is shown in Figure 5-14.

It can be seen from figure 5-12 that the residual flow does not change significantly due to the construction of an artificial island or peninsula. From figure 5-14, the velocity variation of the residual flow compared with the present state seemed to be about 10 percent, and the significant changes occurred within a radius of 2 kilometers around the artificial island and peninsula.

In short, the construction of an artificial island or peninsula is supposed to be of little influence to the residual current at the project area.

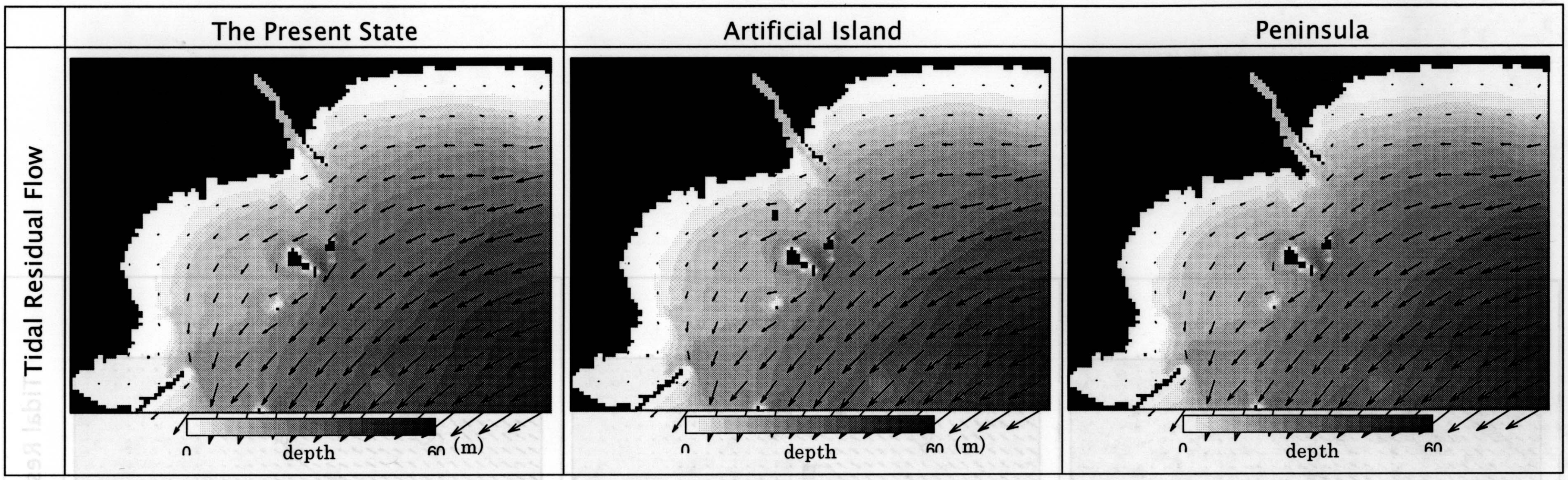


Figure5-12 :   Residual flow patterns during spring tide

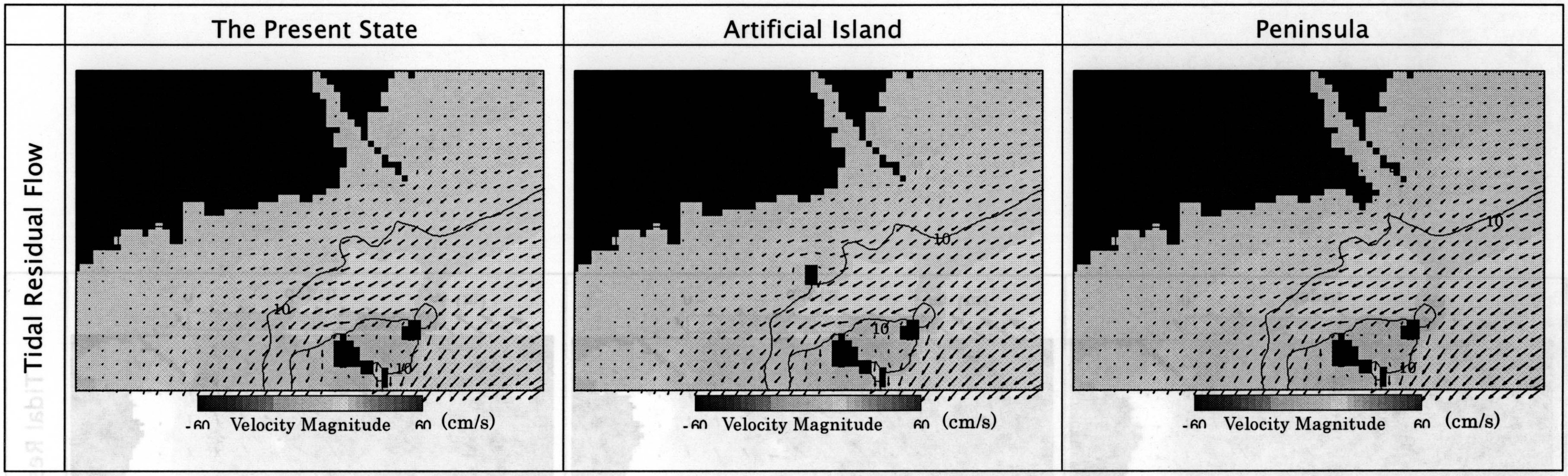
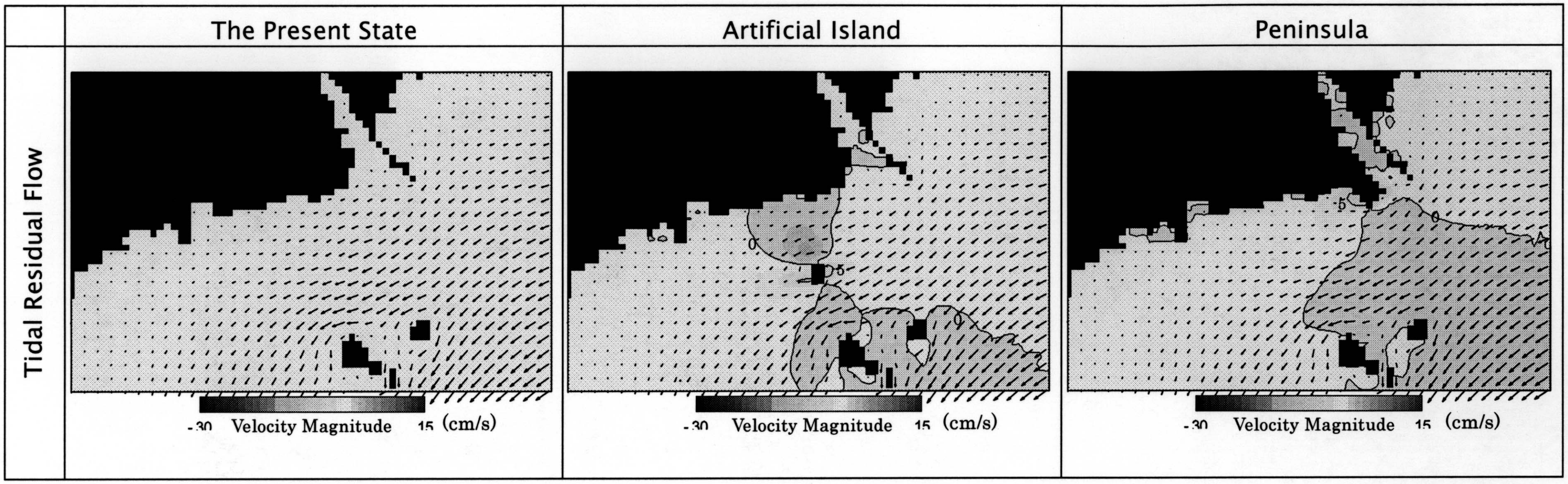


Figure5-13 : Residual flow patterns during spring tide at the project area



Contour interval: 5.0cm/s

Figure5-14 : Residual flow patterns and contour of current variation compared with the present state

6. Conclusion

After careful consideration of the results of analysis based on the model, the concluding phenomena are as follows:

1. The velocity of the tidal flow has changed around the artificial island and peninsula from its present state. However, the change is at about 50 percent of the present state and there is no significant change in the current direction. So the effect is supposed to be minimal.
2. From the tidal residual flow, the large variation of the tidal residual flow occurred only within a radius of 2 kilometers around the artificial island and peninsula.

It is therefore concluded that the construction of artificial island and peninsula has a minimal effect in the tidal flow at the project site in the Gulf of Panama as proven by the tidal numerical simulation.

7. Recommendation

Since the proposed artificial island or peninsula will be constructed in the coastal region, it will influence the movement of sediment and near shore current.

As such, it is recommendable to perform another simulation analysis on the topographic and current condition near the shore.

8. References

1) Autoridad del Canal de Panama (September, 2002): "Panama Canal capacity expansion program"

2) Autoridad del Canal de Panama (November, 2002): "Feasibility Study for the Construction of an Artificial Island at the Pacific Entrance of the Panama Canal"

3) Bennett, B.E..(1965) "Currents observed in Panama Bay during September-October 1958". Inter-American Tropical Tuna Commission. Bulletin Vol.10, No.7. pp.399-457.