Chapter 4 Basic Plan

4.1 Tide embankment

4.1.1 Design concept

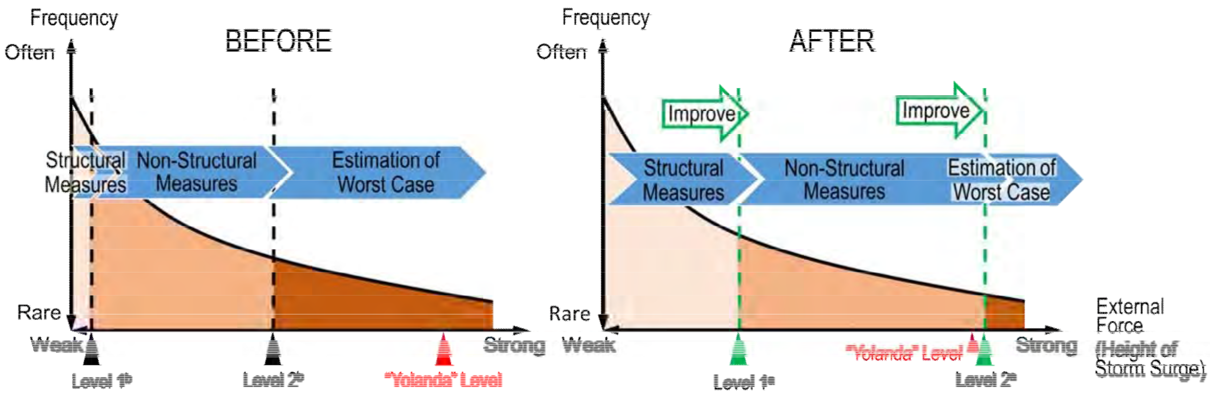
Basic design concept of the protection level of structure measures is to protect properties from storm surge of 50-year-return-period and in this regard, top elevation of the tide embankment was set based on the simulation result for the 50-year-return-period storm surge deviation.

Storm surge of lower frequency, as was the case of Yolanda, shall be protected by non-structural measures, but resiliency of tide embankment needs to be examined so that overtopping water will not immediately break the embankment. Non-structural measures include evacuation planning which will be facilitated by the resiliency of structural measures.

Therefore, design concept of the tide embankment can be summarized as follows,

- To protect properties inside the embankment from the 50 years return period storm surge.

To be resilient enough to resist overtopping water by storm surge of lower frequency, the facilitation of practical use of non-structural measures is expected parallel with the implementation of the DPWH project.



Source: JICA Study Team

Figure 4.1-1 Basic concept of Protection Level of Structure Measures

A direction for disaster risk reduction measures in the area is suggested in Figure 4.1-1 where “BEFORE” indicates the present situation, while “AFTER” describes the situation with sufficient measures installed.

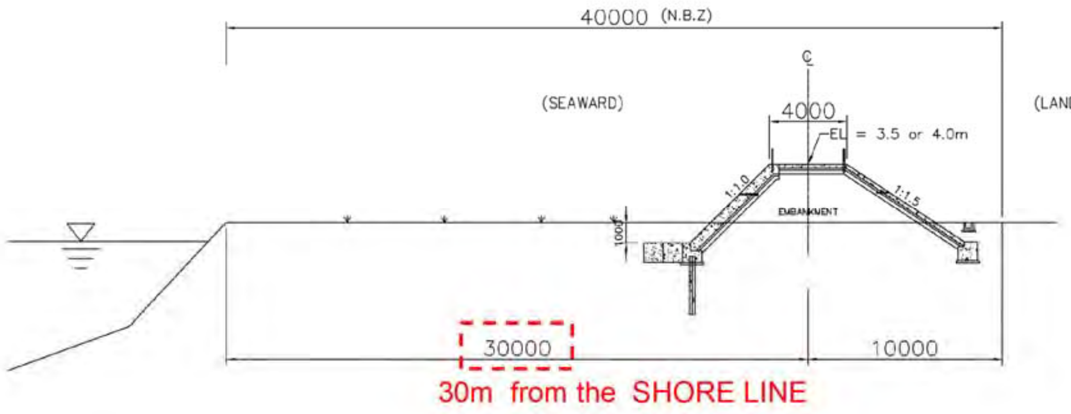
By improving structural measures such as heightening an existing road, urban areas can be protected from the storm surge with stronger external force of Level 1a than the external force of Level 1b. In case of road heightening measures, the appropriate height of the road can be determined through storm surge height analysis.

As for non-structural measures, an evacuation plan can be improved by examining the locations of evacuation centers and evacuation routes. It results in increase of the external force from Level 2b to Level 2a, or in other words, evacuation can save more lives even in the case of much stronger storm surge. As shown in Figure, by improving countermeasures against storm surge, it is possible to prevent disaster even if an enormous typhoon such as "Yolanda" strikes the region again.

4.1.2 Control points in setting up the alignment

(1) Basic policy

Basic policy for setting up an alignment is showed in the figure below. The centerline of the tide embankment shall be set up at 30 meters from the sea shore, so that the embankment falls inside the no-building zone, which is 40 meters from the sea shore.



Source: JICA Study Team

Figure 4.1-2 Basic Policy for Setting up an Alignment

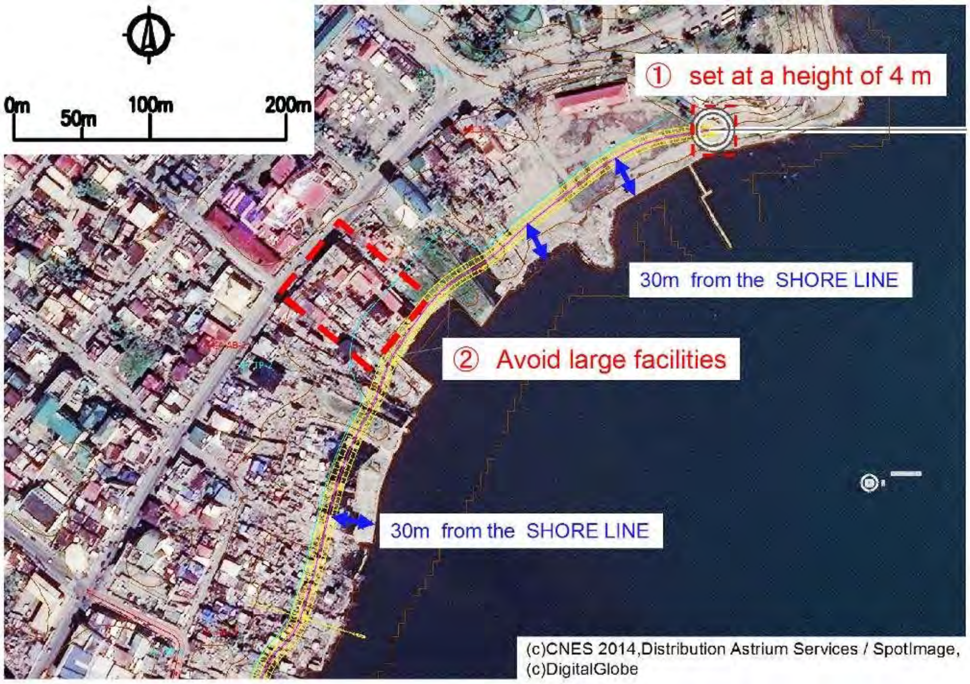
However, some areas are exempt from the basic rule, as described below.

- Existing large facilities (factories) shall be avoided.
- Where there is an existing road within 40 meters from the shore line, the embankment shall be shifted seaward so that the existing road will not be affected.

Even if there are large facilities and existing roads to avoid and tide enbankment needs to be shifted towards the sea, the tide embankment should be constructed on existing land (dry area).

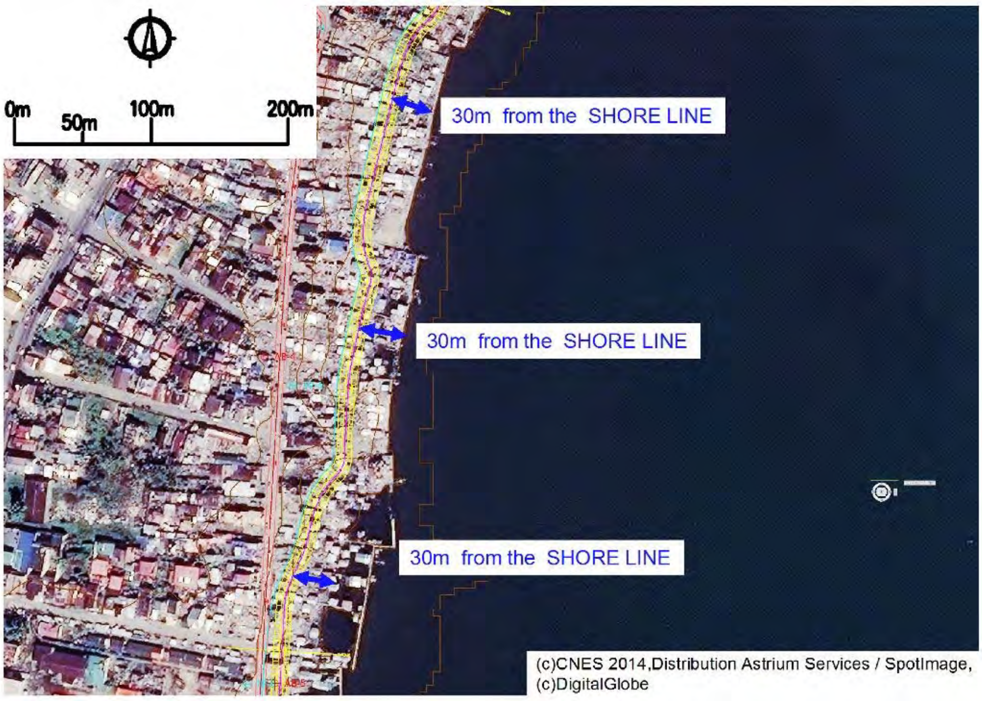
(2) Alignment settings in section 3

Control points in setting up the alignment is shown in following figures.



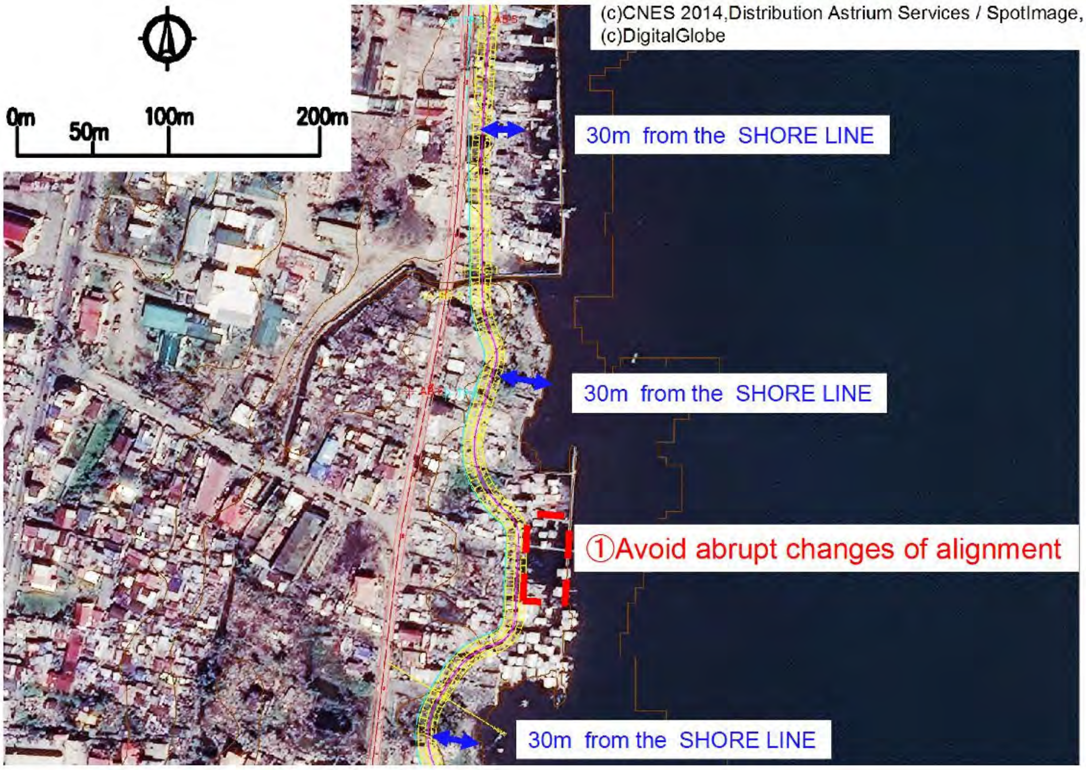
Source: JICA Study Team

Figure 4.1-3 Control points in setting up alignment in section 3 (1)



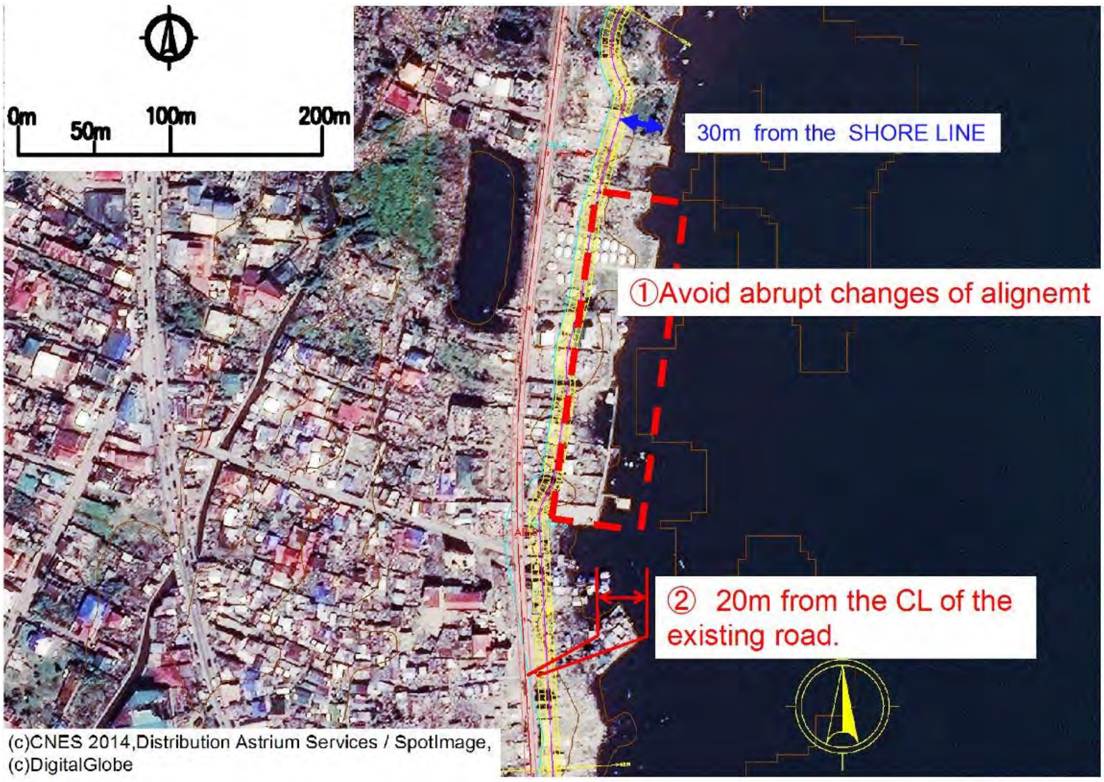
Source: JICA Study Team

Figure 4.1-4 Control points in setting up alignment in section 3 (2)



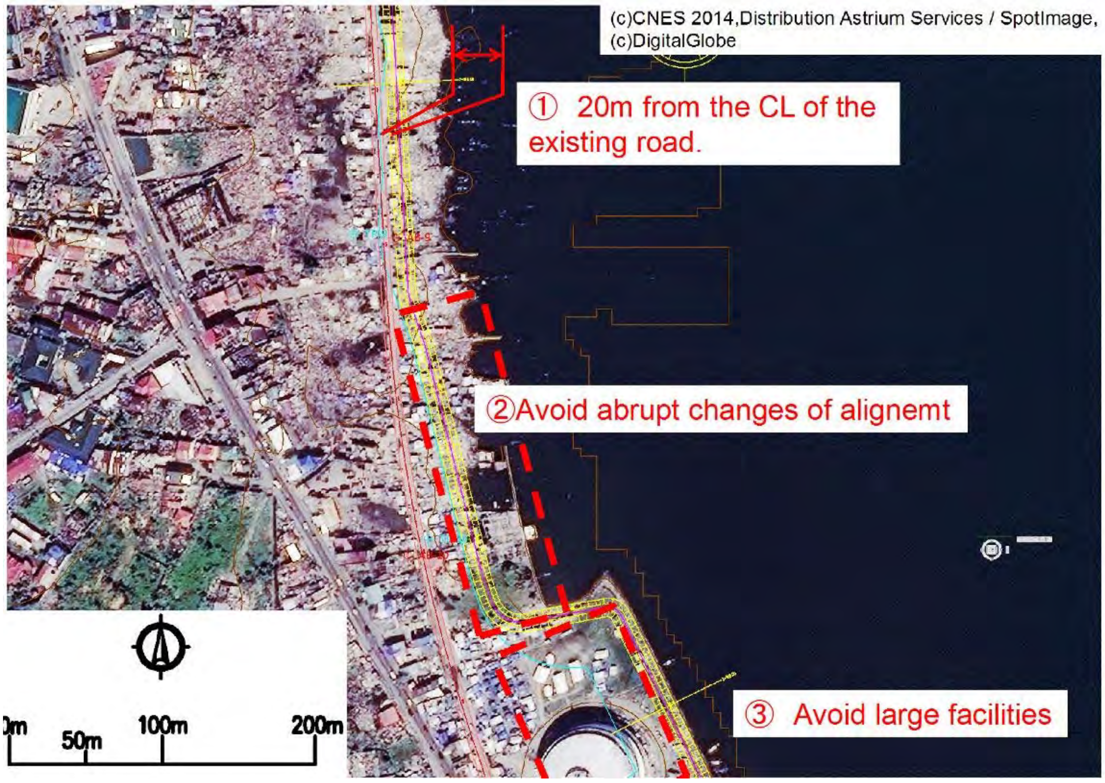
Source: JICA Study Team

Figure 4.1-5 Control points in setting up alignment in section 3 (3)



Source: JICA Study Team

Figure 4.1-6 Control points in setting up alignment in section 3 (4)

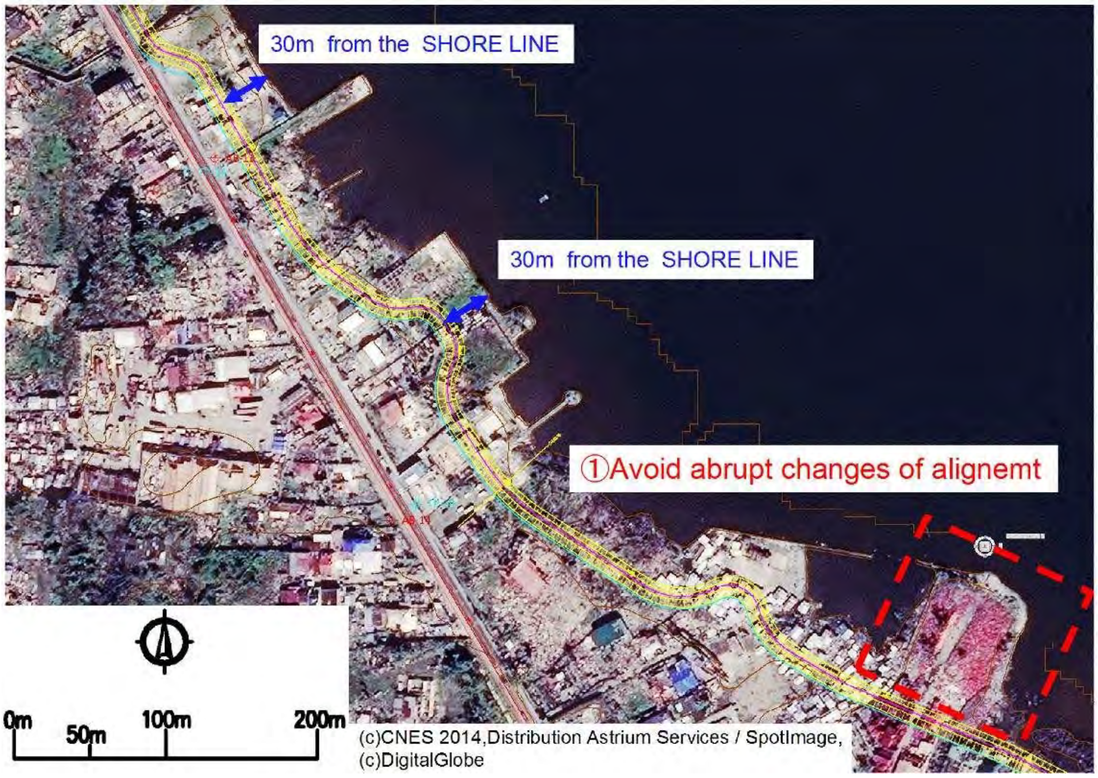


Source: JICA Study Team

Figure 4.1-7 Control points in setting up alignment in section 3 (5)

Source: JICA Study Team

Figure 4.1-8 Control points in setting up alignment in section 3 (6)



Source: JICA Study Team

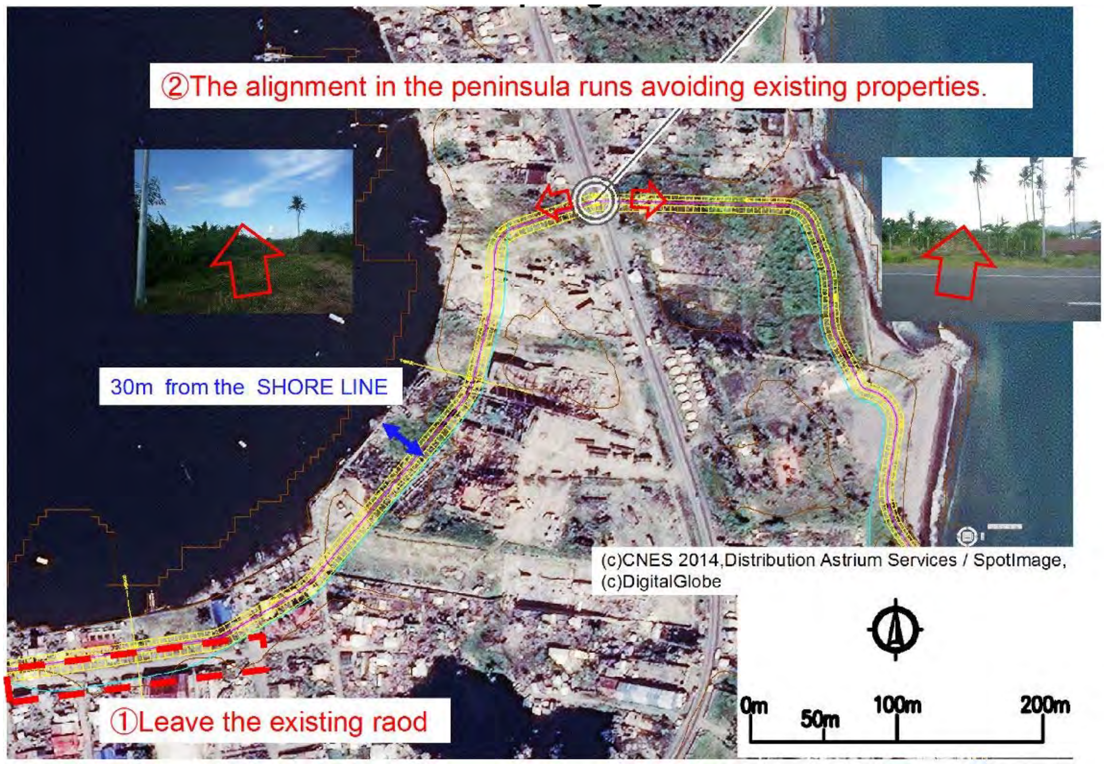
Figure 4.1-9　Control points in setting up alignment in section 3 (7)

Source: JICA Study Team

Figure 4.1-10　Control points in setting up alignment in section 3 (8)

Source: JICA Study Team

Figure 4.1-11 Control points in setting up alignment in section 3 (9)

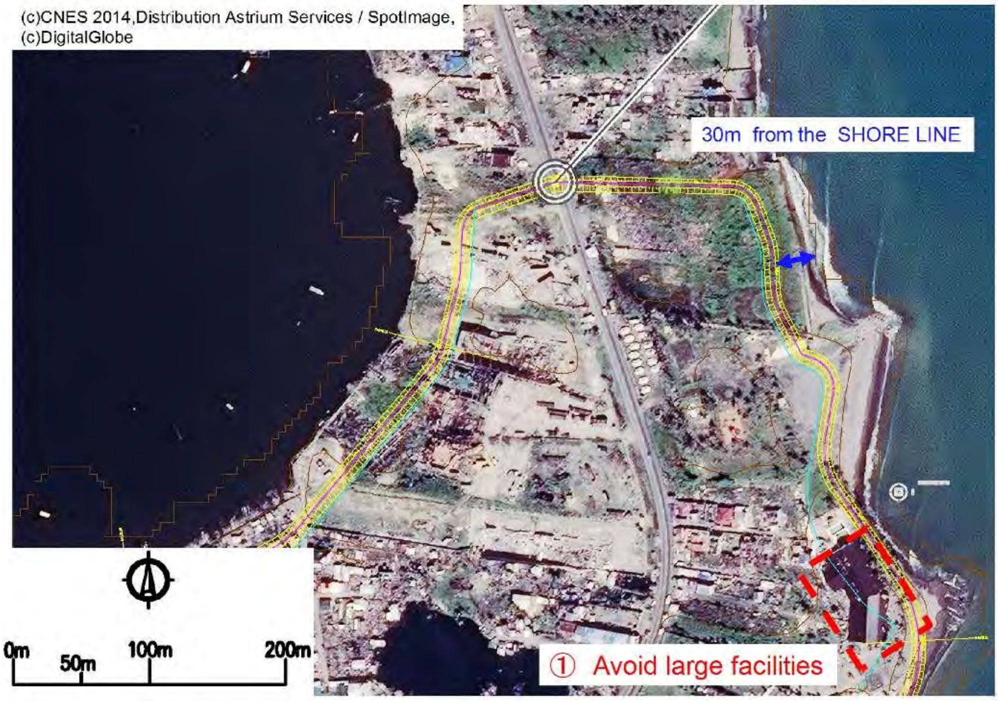


Source: JICA Study Team

Figure 4.1-12 Control points in setting up alignment in section 3 (10)

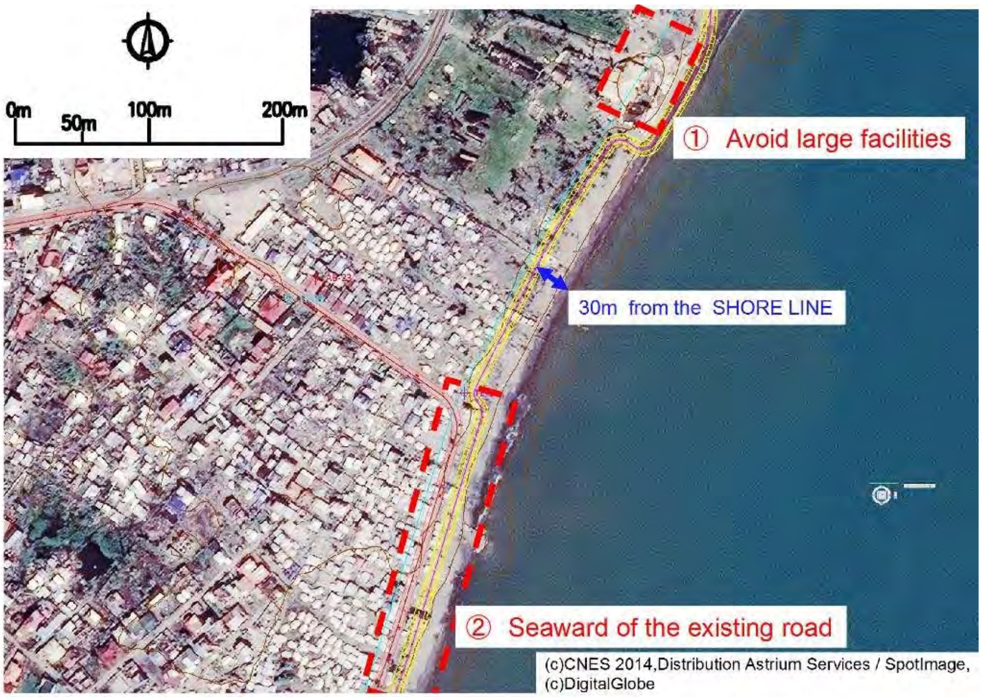
(3) Alignment settings in section 4

Control points in setting up the alignment is shown in following figures.



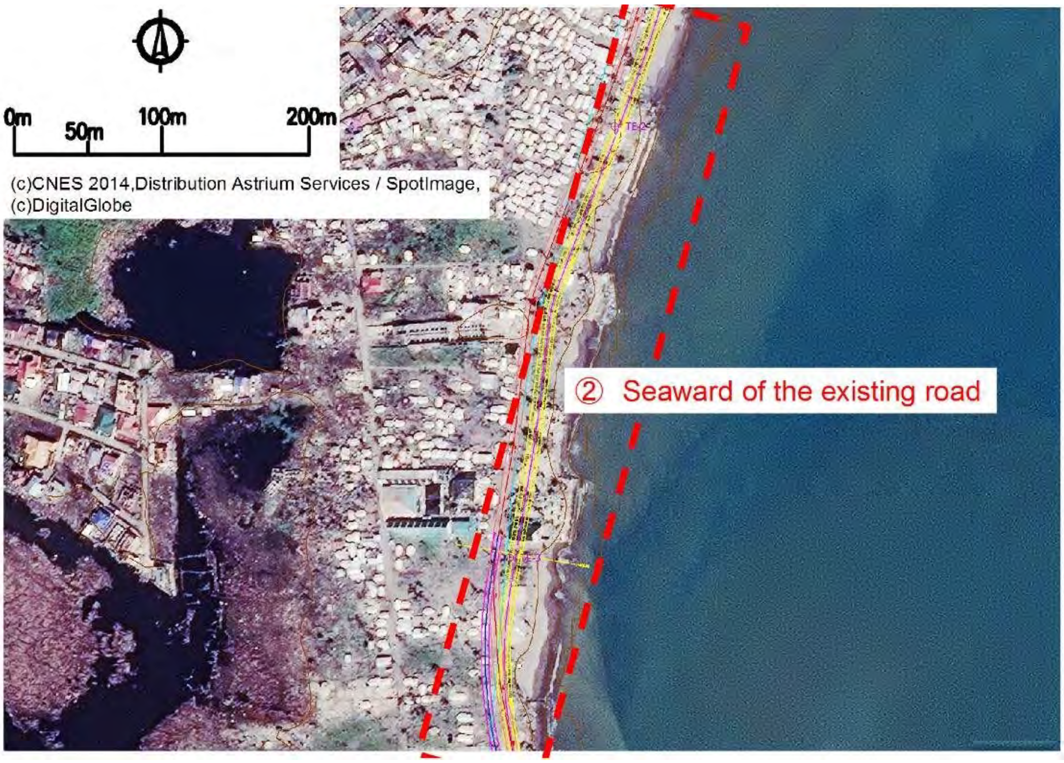
Source: JICA Study Team

Figure 4.1-13 Control points in setting up alignment in section 4 (1)



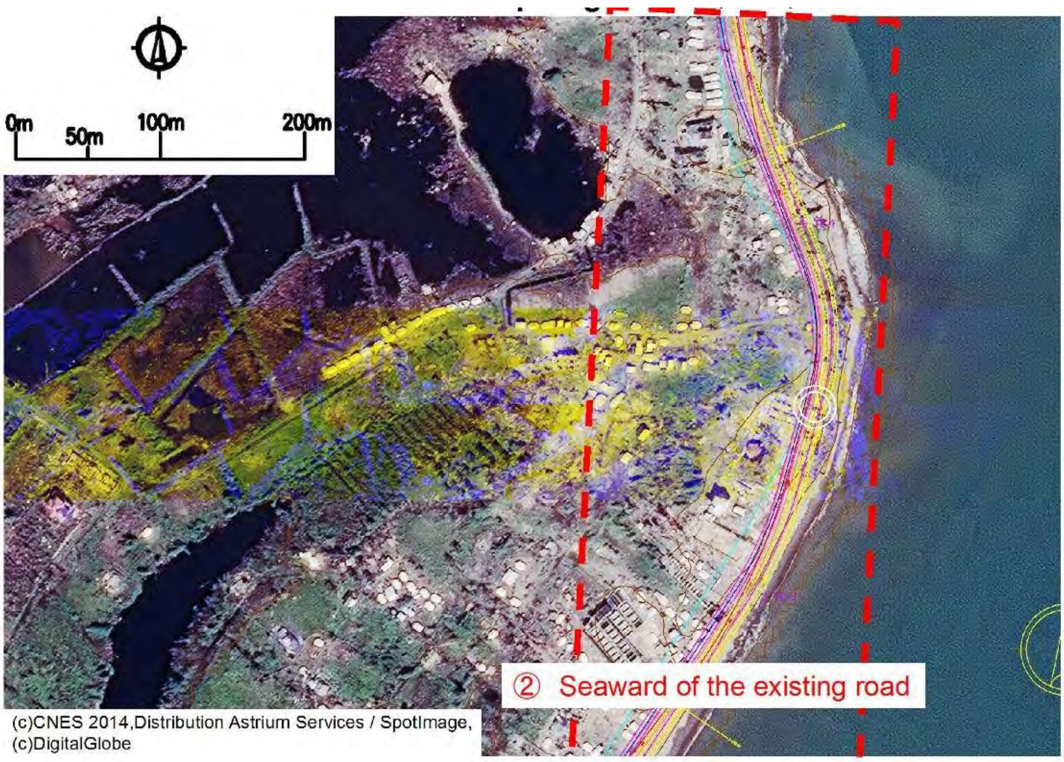
Source: JICA Study Team

Figure 4.1-14 Control points in setting up alignment in section 4 (2)



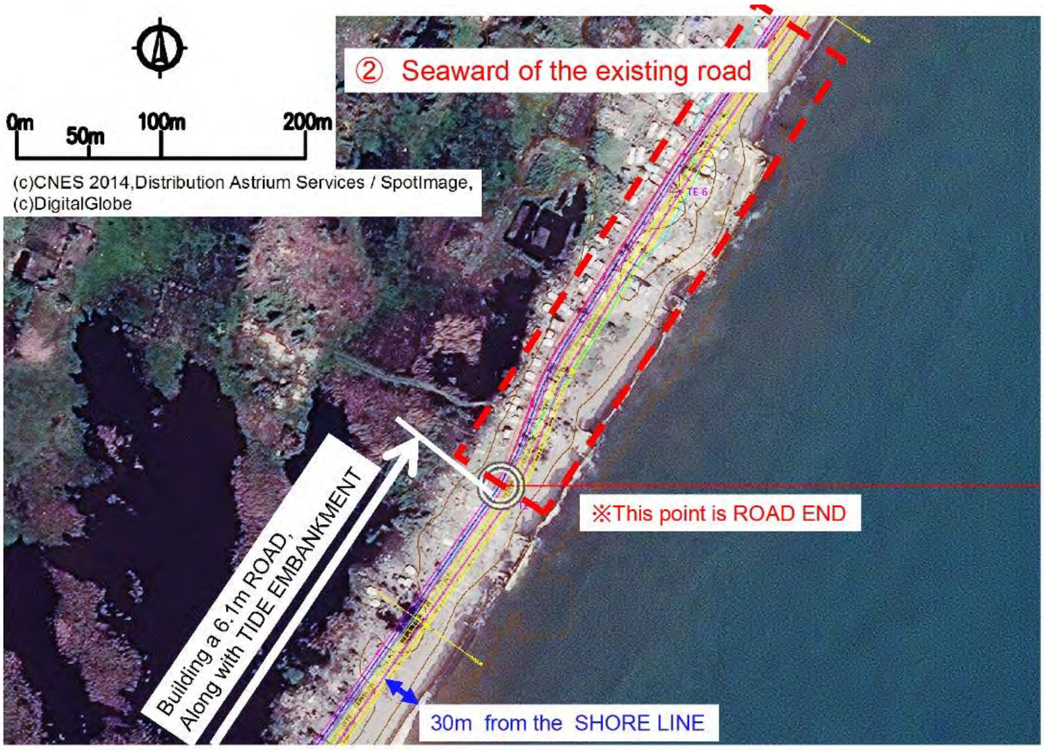
Source: JICA Study Team

Figure 4.1-15 Control points in setting up alignment in section 4 (3)



Source: JICA Study Team

Figure 4.1-16 Control points in setting up alignment in section 4 (4)



Source: JICA Study Team

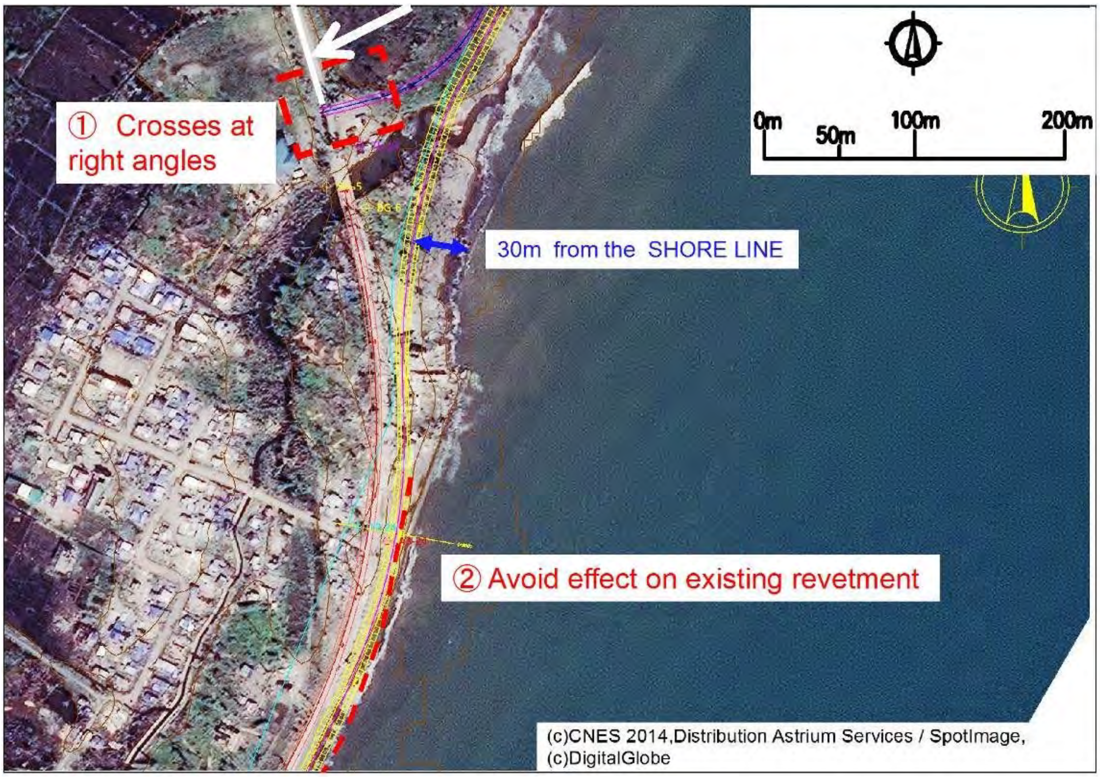
Figure 4.1-17 Control points in setting up alignment in section 4 (5)

Source: JICA Study Team

Figure 4.1-18 Control points in setting up alignment in section 4 (6)

Source: JICA Study Team

Figure 4.1-19 Control points in setting up alignment in section 4 (7)

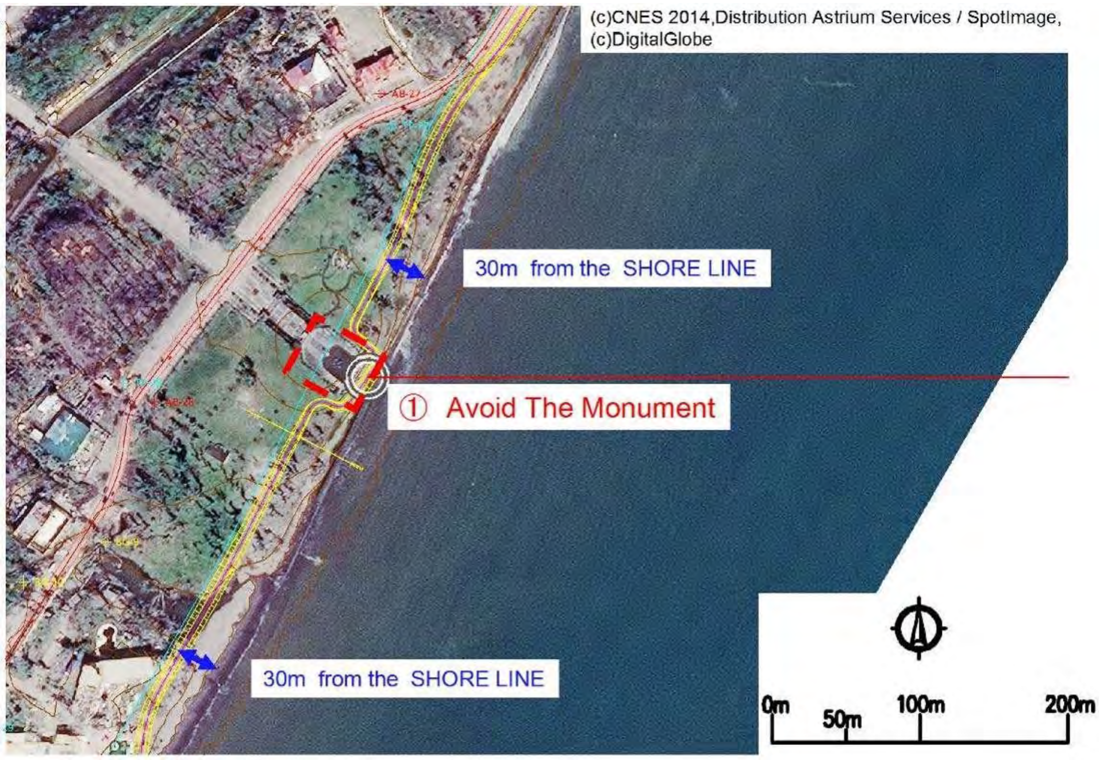


Source: JICA Study Team

Figure 4.1-20 Control points in setting up alignment in section 4 (8)

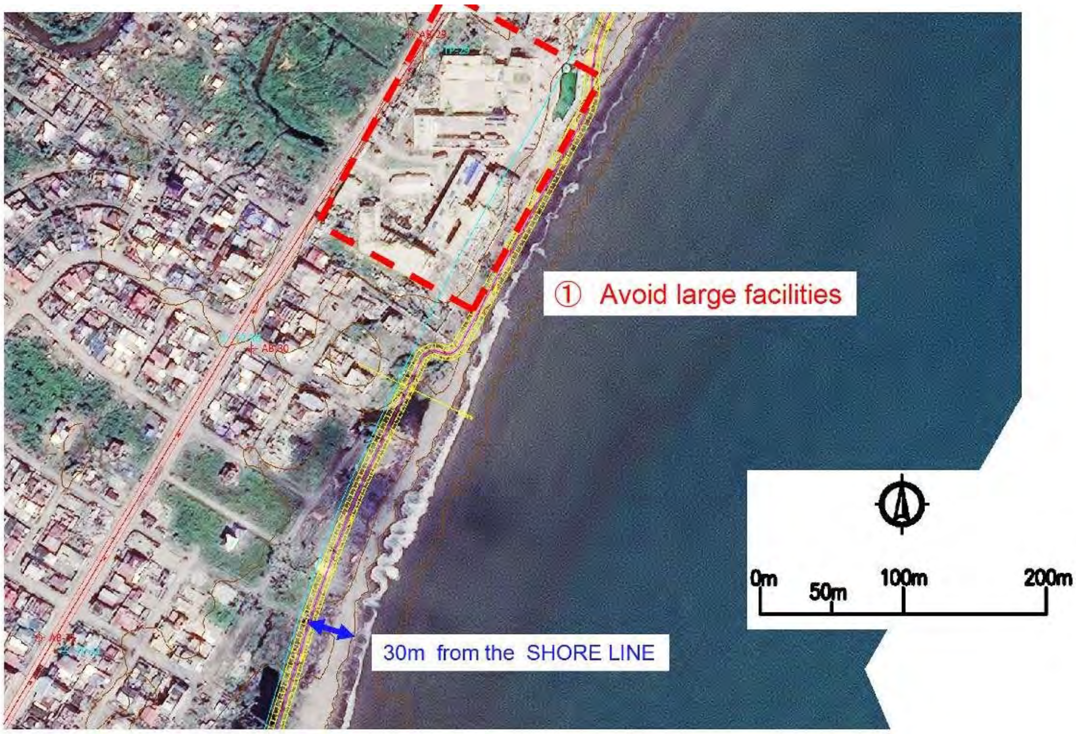
Source: JICA Study Team

Figure 4.1-21 Control points in setting up alignment in section 4 (9)



Source: JICA Study Team

Figure 4.1-22 Control points in setting up alignment in section 4 (10)

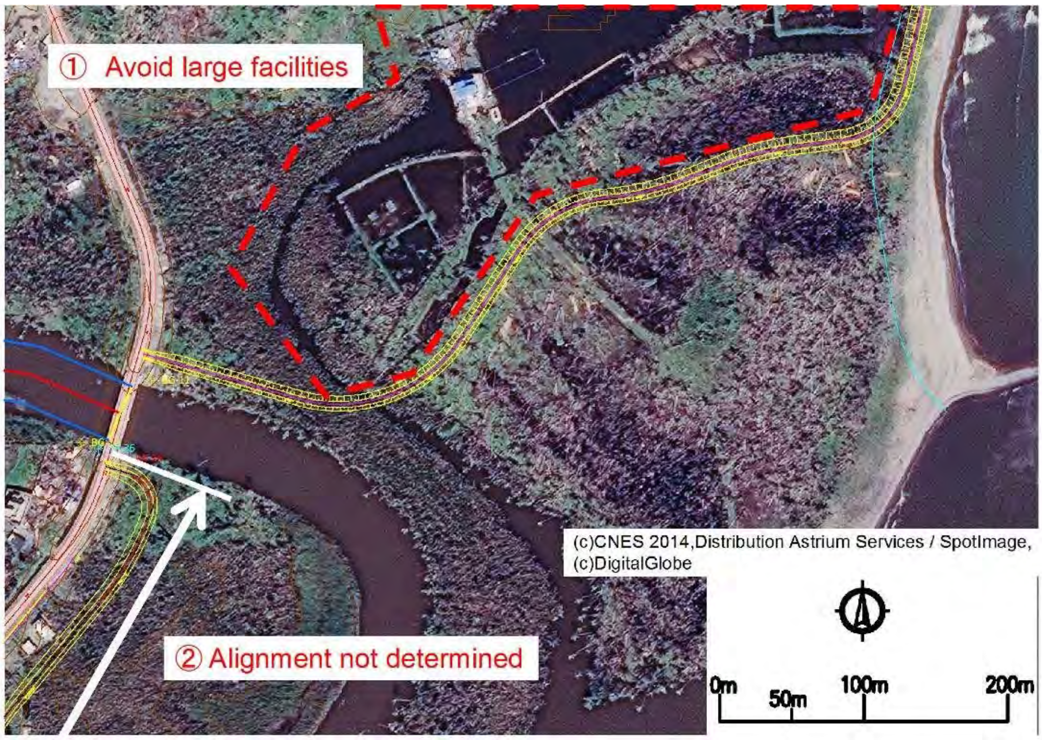


Source: JICA Study Team

Figure 4.1-23 Control points in setting up alignment in section 4 (11)

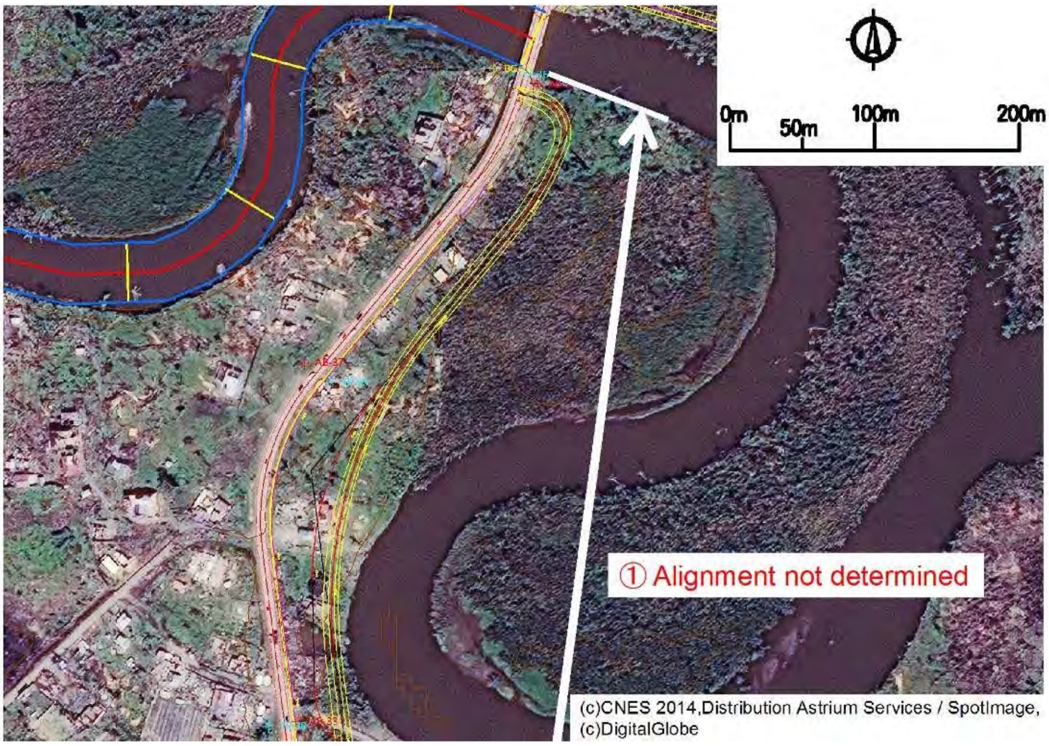
Source: JICA Study Team

Figure 4.1-24 Control points in setting up alignment in section 4 (12)



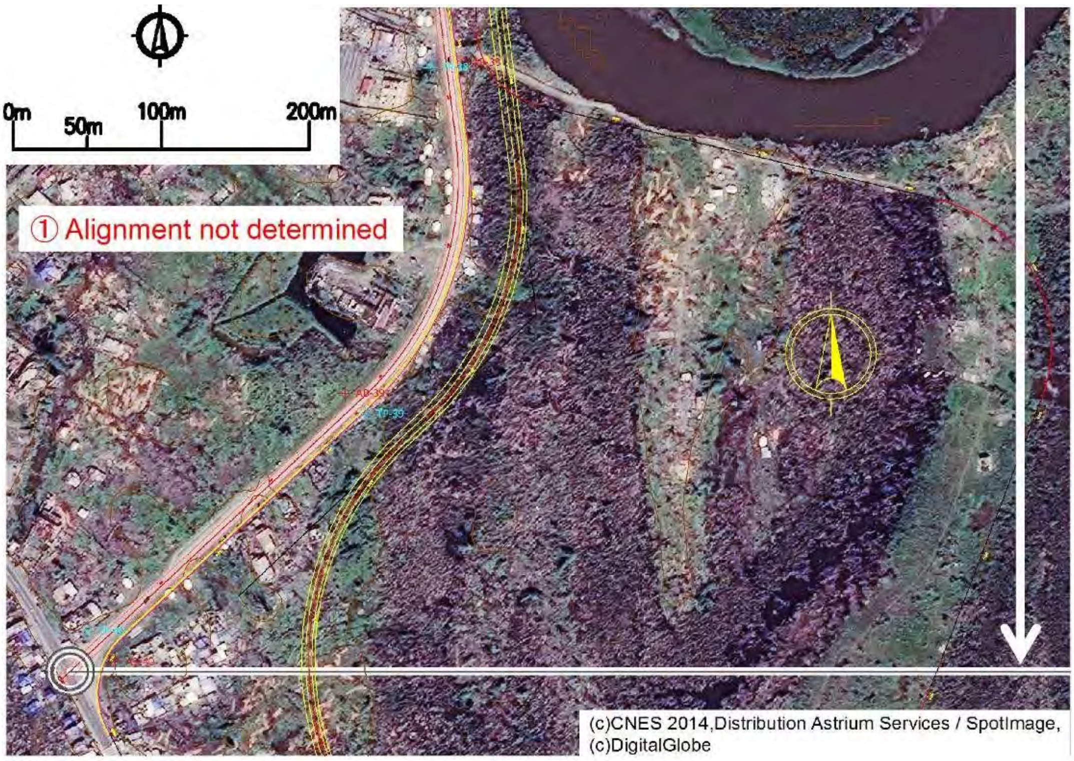
Source: JICA Study Team

Figure 4.1-25 Control points in setting up alignment in section 4 (13)



Source: JICA Study Team

Figure 4.1-26 Control points in setting up alignment in section 4 (14)



Source: JICA Study Team

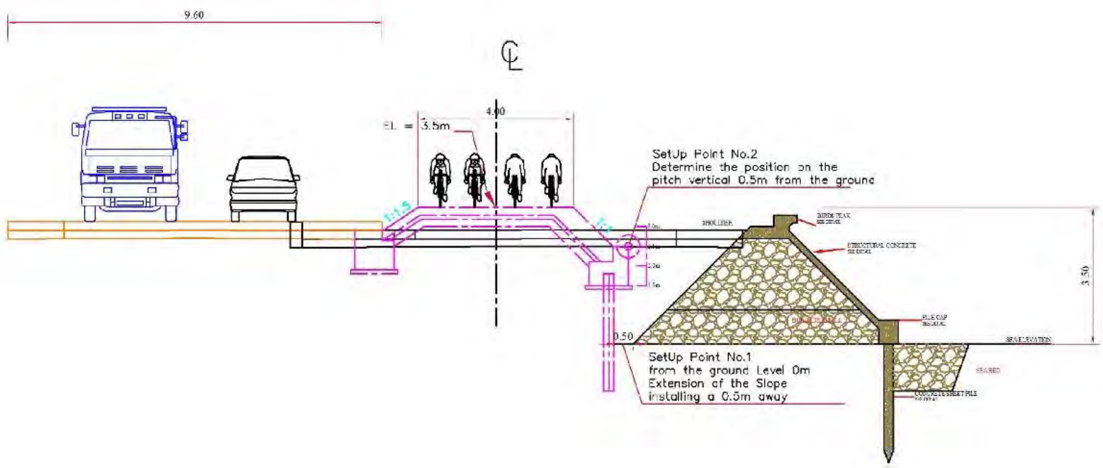
Figure 4.1-27 Control points in setting up alignment in section 4 (15)

Alignment in the right bank of Bangon river, as shown in Figure 4.1-25 to Figure 4.1-27 shall be determined in relation to the alignment in section 5, setting a rule on the extent of protection with regard to existing houses on seaside of the road.

(4) Alignment settings in the north of Macarthur Park

Existing revetment along the road in the north of Macarthur Park was damaged by Yolanda. The restoration work of the revetment is ongoing by DPWH.

The newly constructed tide embankment will be placed behind the existing revetment so that the embankment will not affect the existing one. The road needs to be shifted landward as shown in the figure below.



Source: JICA Study Team

Figure 4.1-28 Alignment settings in the north of Macarthur Park

4.1.3 Structural Studies for tide embankment

(1) Selection of structure type

Tide embankment and tidal wall are the two basic structure type for tide protection. Tide embankment shall be selected where,

- Foundation ground is not solid.
- Procurement of embankment sand is available.
- Procurement of land is available.
- Utilization of the crest road, as promenade or cycling road is desired.
- Embankment is preferred in view of managing landscape.

(2) Design conditions

1) Top elevation

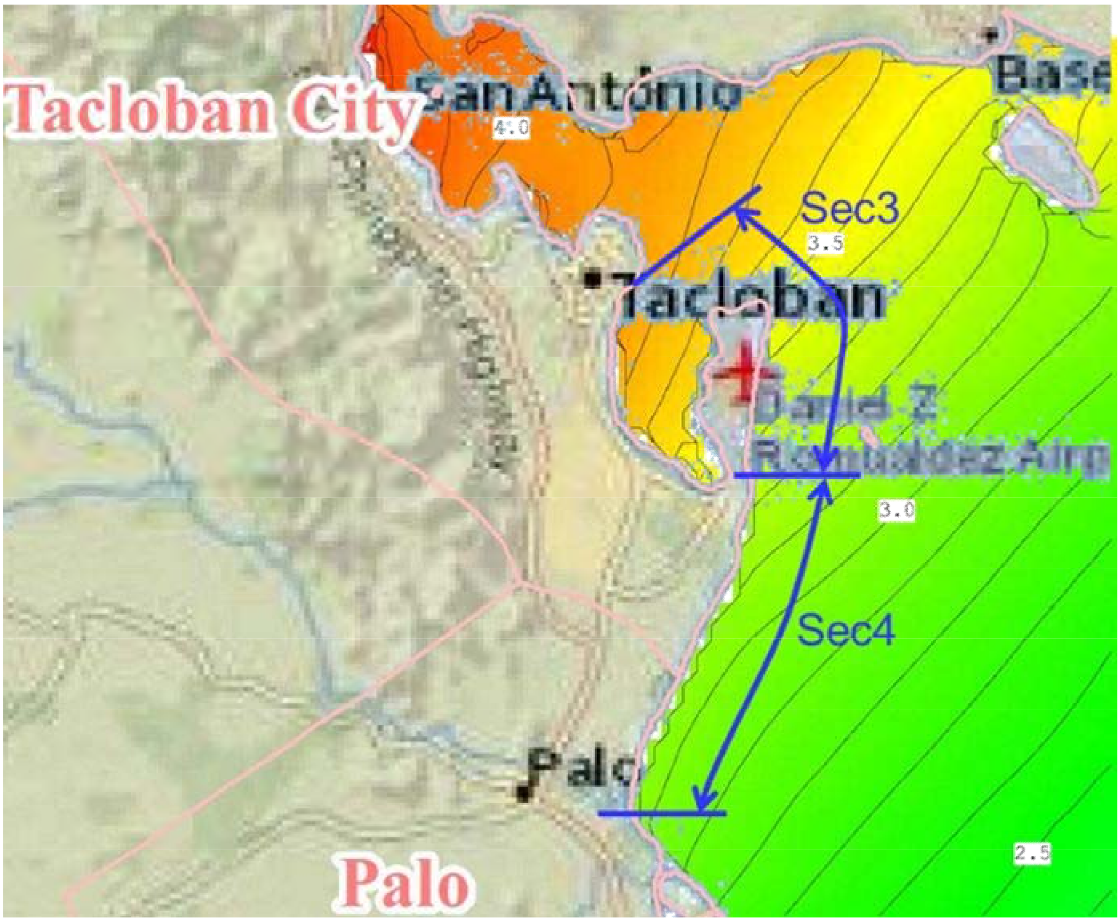
Top elevation of the embankment is set at MSL+4.0m in section 3 and MSL+3.5m in section 4. The height is based on the results of simulated 50 years return period storm surge deviation in each section. The maximum storm surge height in section 3 is MSL+3.8m, which is rounded up to set the design top elevation of MSL+4.0m. The maximum storm surge height in section 4 is MSL+3.2m, which is rounded up to set the design top elevation of MSL+3.5m.

In the storm surge simulation, it is the centric atmospheric pressure which is 50 years return period and track and speed of typhoon was assumed as the same with Yolanda. The conditions for simulation is on the safe side, since initial sensibility analysis demonstrated that the track and speed of Yolanda were the most severe case, as discussed in the appendix.

Table 4.1-1 Top elevation settings

	Simulated storm surge deviation (MSL+m)	Design Top elevation (MSL+m)
Section 3	MSL +3.8m	MSL +4.0m
Section 4	MSL +3.2m	MSL +3.5m

Source: JICA Study Team



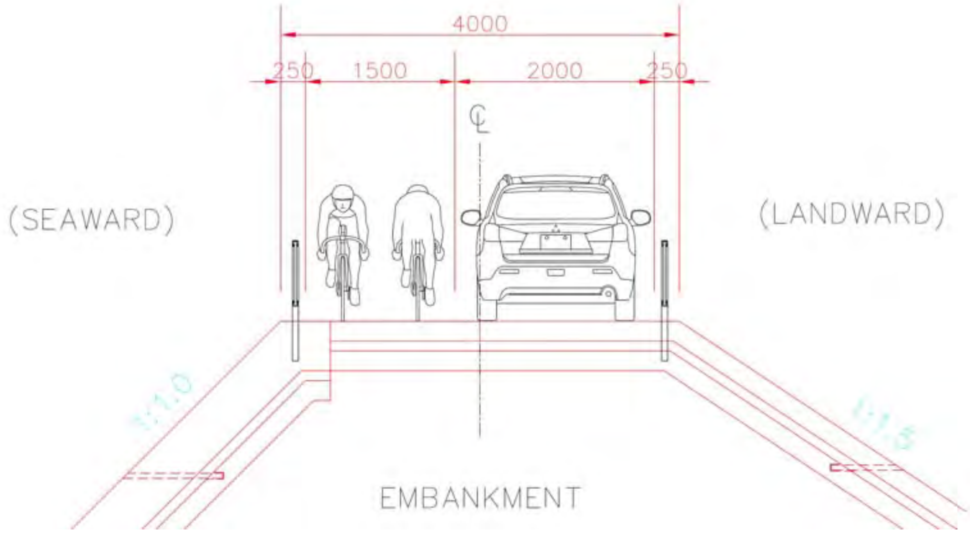
Source: JICA Study Team

Figure 4.1-29 Simulated storm surge deviation (50 years return period)

2) Crest width

The crest width for tide embankment shall be more than 3 meters and it is generally 3 to 4 meters on a case-based study for similar structures.

Here, the width is set 4 meters considering utilization of the crest as well as described below.



Source: JICA Study Team

Figure 4.1-30 Crest width requirement as a cycling and maintenance road.

The crest of the embankment will be utilized as a promenade and cycling road, and it will not be utilized as a residential roadway. In this regard, utilization of normal vehicles shall be regulated.

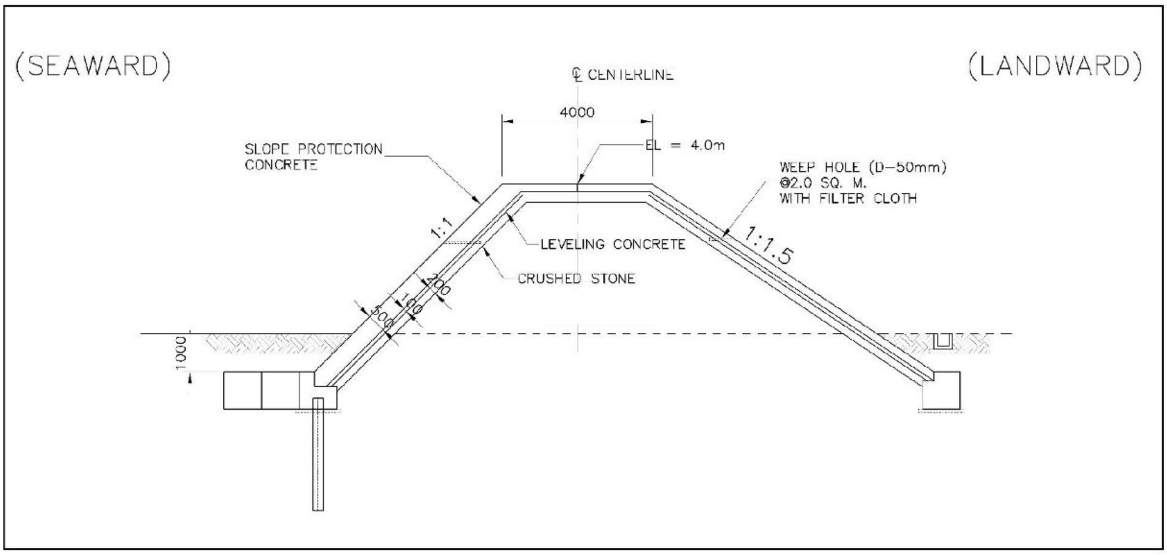
In the meantime, the crest will also be utilized for an access lane for official vehicle, in case tide embankments, gates and other related structure need to be surveyed, maintained or repaired. Such utilization is occasionally.

The crest width is set at 4 meters so that it can be used as a cycling road considering an official vehicle parked on it. The width of 2.0 meters and 1.5 meter for an official vehicles and bicycles, respectively. In addition, 0.25 meters for shoulders where fall prevention net will be installed at both sides(shoulders) as shown in Figure 4.1-30Figure 4.1-30.

(3) Structural Studies

1) Structural specifications

Structural specifications for tide embankment is organized in the table below.



Source: JICA Study Team

Figure 4.1-31 Standard structure of the embankment

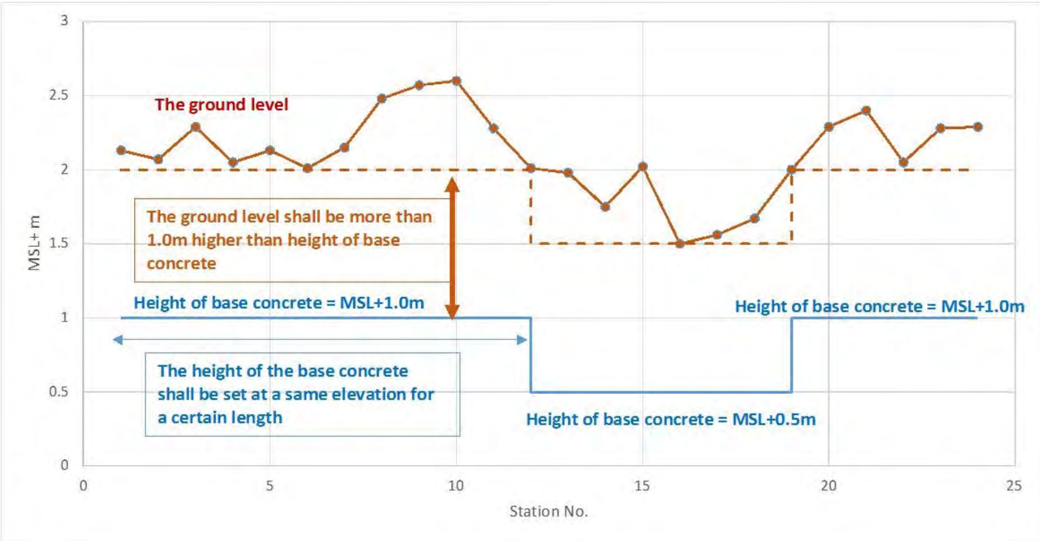
Table 4.1-2 Structural specifications of tide embankment

Items	specifications	explanation
Seaward slope gradient	1 : 1.0	Steepest case-based gradient for concrete-protected embankment. Lowering the gradient facilitate overtopping thus unsafe.
Landward slope gradient	1 : 1.5	Typical case-based gradient for concrete-protected embankment.
Seaward slope protection	Concrete (50cm) Lean concrete (10cm) Crushed stone (20cm)	Typical case-based protection for tide embankment.
Landward slope protection	Concrete (20cm) Lean concrete (10cm) Crushed stone (20cm)	Typical case-based protection for tide embankment.
Crest protection	Concrete (20cm) Lean concrete (10cm) Crushed stone (20cm)	Typical case-based protection for tide embankment.
Sheet pile (seaward)	L=3.0m	Standard length needed for erosion protection, water cutoff and soil draw-out prevention.

Base concrete (seaward)	1m x 1m (1 unit)	Typical case-based protection for tide embankment.
Embedded depth (seaward)	D=1.0m	Standard length needed for erosion protection
Embedded depth (landward)	D=1.0m	Standard length needed for erosion protection
Foot protection	2 lines of base concrete (1m x 1m)	Typical case-based protection for tide embankment.

Source: JICA Study Team

The top elevation of the base concrete shall be set at a level elevation for a certain section, depending on the tendency of the ground elevation profile. The base concrete shall always be embedded for more than one (1) meter beneath the ground level and the top elevation of the base concrete shall be set at 0.50 meter interval. The concept applies for seaward base concrete as well as landward base concrete.



Source: JICA Study Team

Figure 4.1-32 Example of setting the height of base concrete

2) Stability analysis

Stability of the foundation ground shall be analyzed utilizing the result of geotechnical survey. Consolidation of the foundation ground and liquefaction shall be also analysed.

3) Protection against erosion by overflow water

The tide embankment should be resilient to some extent to resist overtopping water against storm surge of lower frequency, as was the case of Yolanda. The resiliency of tide embankment means that the structure will not collapse even if the storm surge water overtop its crest.

The overflow discharge is estimated here, based on an empirical equation with the simulated height of storm surge cause by Yolanda. Then, the depth of erosion on the foot of landward slope is estimated to evaluate the settings of embedded depth and the size of base concrete is enough.

a) Overflow discharge

The simulated maximum storm surge height during Yolanda in section 3 is MSL+5.0m and that of section 4 is MSL+4.0m. Given the design top elevation of MSL+4.0m in section 3 and MSL+3.5m in section 4, the overtopping depth is calculated as 1.00m in section 3 and 0.50m in section 4.

Table 4.1-3 Estimated overtopping depth for Yolanda

Section	Simulated maximum Storm surge height	Top elevation	Overtopping depth
	(MSL+m)	(MSL+m)	(m)
Section 3	MSL+5.0m	MSL+4.0m	1.00m
Section 4	MSL+4.0m	MSL+3.5m	0.50m

Source: JICA Study Team

The overflow discharge is calculated with following equation (Hon-ma equation).

$$Q = 0.35 \times h1 \times \sqrt{(2 \times g \times h1)} \times B$$

Given 1.0m for h1 (overtopping height), 9.8 m/s^2 for g and 1 m (unit meter) for B, Q (the discharge) will be 1.55 m^3/s.

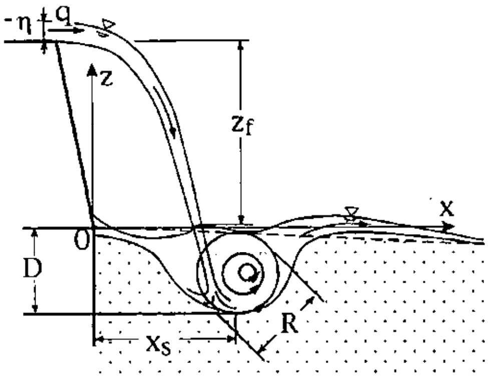
b) Estimation of erosion depth

The erosion depth (D in Figure 4.1-33) will be calculated using a method and equation proposed by Noguchi et al (1997)[1]. As shown in Figure 4.1-34, erosion depth caused by model experiment (D) and calculated diameter of a steady vortex (R) are correlated, the value of D being 2.1 times larger than that of R. The proposed method firstly calculate the size of R with following equation.

[1] Kenji Noguchi, Shinji Sato and Shigenobu Tanaka (1997): Large-scale model experiment of revetment wave overtopping and front erosion caused by tsunami run-up. Proceedings of Coastal Engineering, JSCE No.44 p.296-300 (in Japanese)

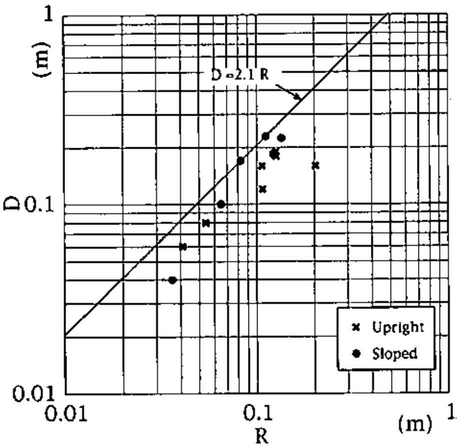
$$R = g^{-1/4} \times Q^{1/2} \times zf^{1/4}$$

zf is the height of embankment above the ground level, which is assumed 3.5 meters at most in this study case. Using the value of Q as 1.55 m^3/s, R is calculated to be 0.96 meter. Thus D is estimated to be 2.0 meters.



Source: Kenji Noguchi, Shinji Sato and Shigenobu Tanaka (1997)

Figure 4.1-33 Notion of calculating erosion depth



Source: Kenji Noguchi, Shinji Sato and Shigenobu Tanaka (1997)

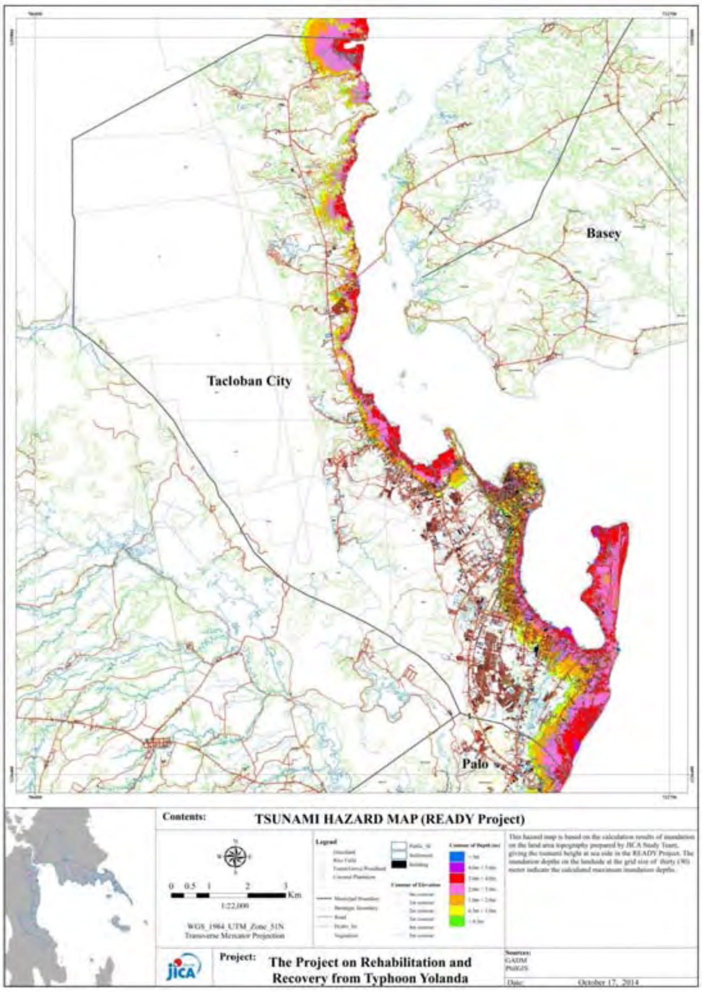
Figure 4.1-34 Experimented relation between D (erosion depth) and R (steady vortex)

c) Setting of embedded depth and size of base concrete

The size of base concrete and its embedded depth is designed so that the embankment will not be destabilized even if an erosion of 2.0 meters depth takes place at the foot of it. Subsequently, the size of base concrete shall be 1.0 meter, the maximum size of base concrete and the embedded depth shall be 1.0 meter, which will cover the depth of 2.0 meters as a whole.

4) Protection against Tsunami

Protection against Tsunami is not in the scope of the project but inundation map made by JICA study team based on the calculation of tsunami height studied in the Ready project gives a tsunami level of about 4.5 meters above mean sea level. This tsunami height gives an overtopping depth of about 1.0 meters, which equals to the overtopping depth for Yolanda. As long as the overtopping depth is the same, protection for tide embankment is deemed efficient for tsunami as well.



Source: JICA Study Team

Figure 4.1-35 Tsunami Hazard Map and Tsunami Level (MSL+ m) by READY project

Table 4.1-4 Simulated storm surge height and Tsunami height

Section	Simulated maximum Storm surge height (MSL+m)	Tsunami level By Ready project (MSL+m)
Section 3	MSL+5.0m	About MSL+4.5m
Section 4	MSL+4.0m	About MSL+4.5m

Source: JICA Study Team

4.1.4 Structural Studies for road heightening

Structural studies for road heightening is organized in case the road heightening will be selected in future planning, although the case-C (tide embankment) was selected for Section 3 and Section 4.

(1) Selection of structure type

Road heightening shall be selected where,

- Existing road will be heightened so that it will serve as tide embankment.
- Existing properties in seaside of the road are few or can be relocated.

(2) Design conditions

1) Top elevation

Top elevation is same as the tide embankment which is related in 4.1.3(1).

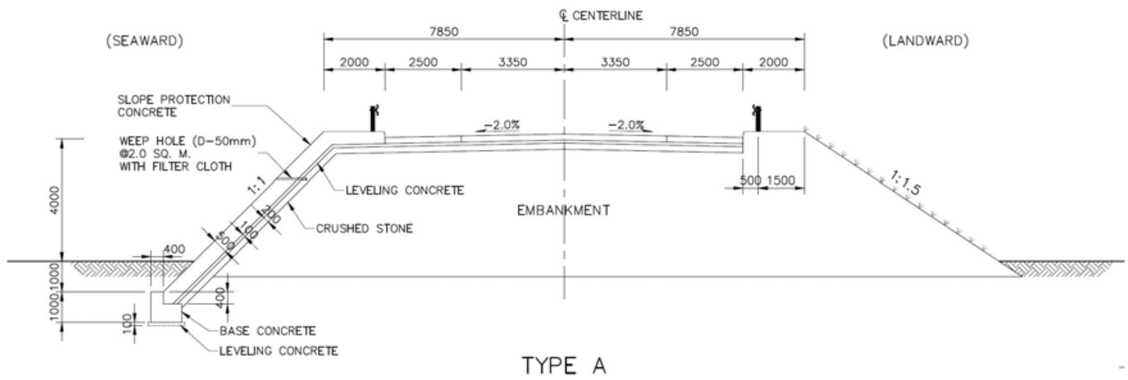
2) Crest width

Crest width of the heightened road shall be determined in relation to road functions.

(3) Structural Studies

1) Structural specifications

Structural specifications for tide embankment is organized in the table below.



Source: JICA Study Team

Figure 4.1-36 Standard structure of road heightening

Table 4.1-5 Structural specifications of road heightening

Items	specifications	explanation
Seaward slope gradient	1 : 1.0	Steepest case-based gradient for concrete-protected embankment. Lowering the gradient facilitate overtopping thus unsafe.
Landward slope gradient	1 : 1.5	Typical case-based gradient for concrete-protected embankment.
Seaward slope protection	Concrete (50cm) Lean concrete (10cm) Crushed stone (20cm)	Typical case-based protection for tide embankment.
Landward slope protection	Grass sodding	Based on structural study in 3)
Crest protection	Concrete pavement	Based on structural study in 3)
Base concrete (seaward)	1m x 1m	Typical case-based protection for tide embankment.
Embedded depth (seaward)	D=1.0m	Standard length needed for erosion protection
Embedded depth (landward)	D=1.0m	Standard length needed for erosion protection

Source: JICA Study Team

2) Stability analysis

Stability of the foundation ground shall be analyzed utilizing the result of geotechnical survey. Consolidation of the foundation ground and liquefaction shall be also analysed.

3) Protection against erosion by overflow water

The heightened road should be resilient enough to resist overtopping water against storm surge of lower frequency, as was the case of Yolanda. The resiliency of tide embankment means that the structure will not collapse even if the storm surge water overtop its crest.

The overflow discharge is estimated here, based on the storm surge simulation result for the Yolanda case.

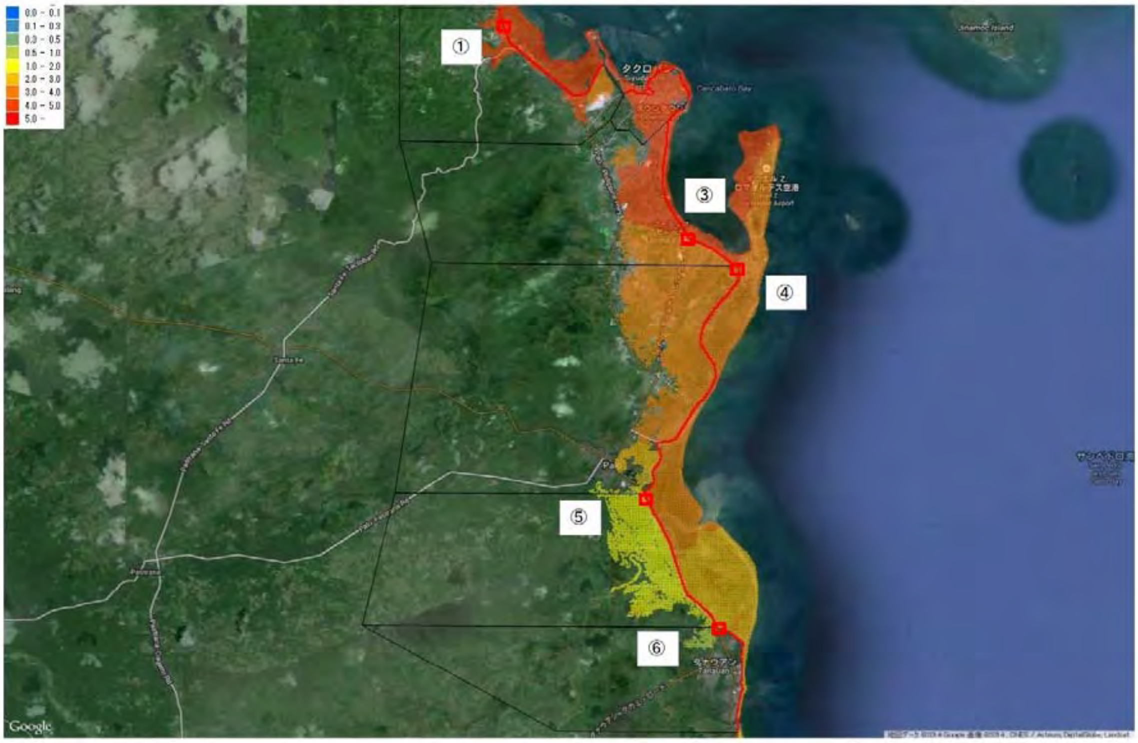
a) Overflow discharge

The maximum overflow discharge per unit width for each Section is shown in Table 4.1-1 below. Note that this study assume the alignment to be along the existing road and not the alignment of the tide embankment.

Table 4.1-6 Maximum Overflow Discharge per unit width for each Section

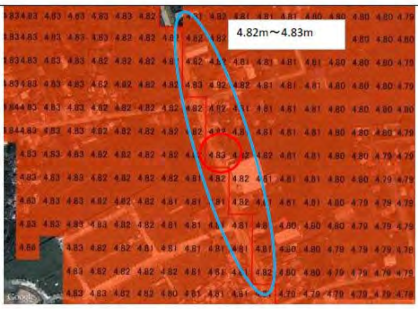
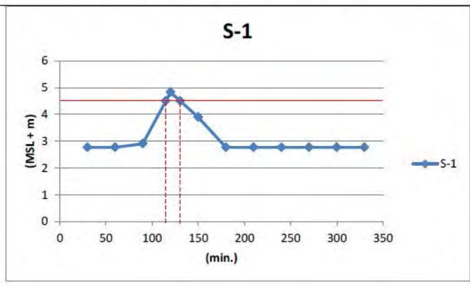
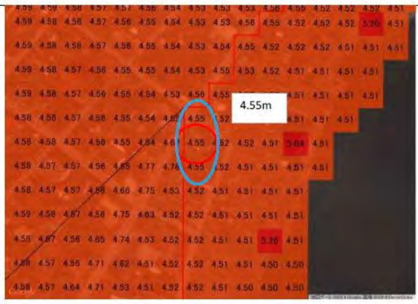
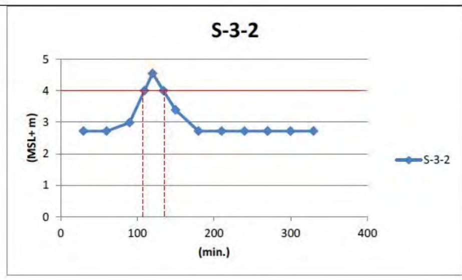
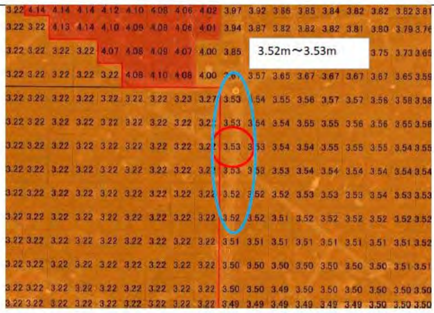
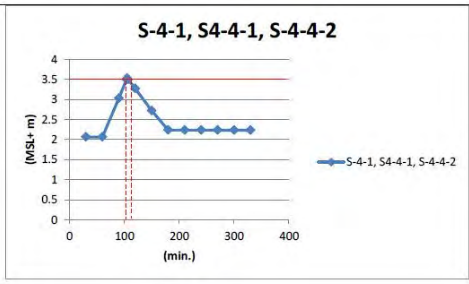
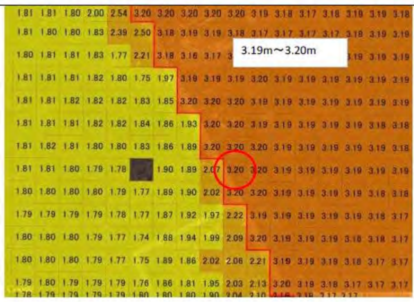
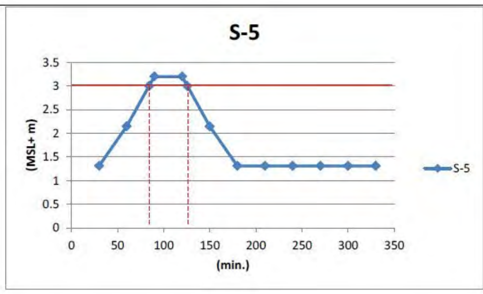
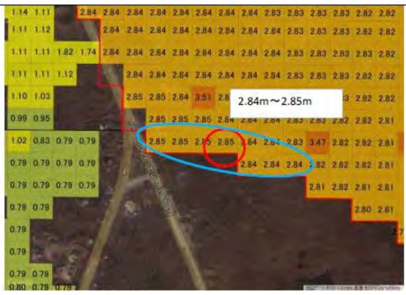
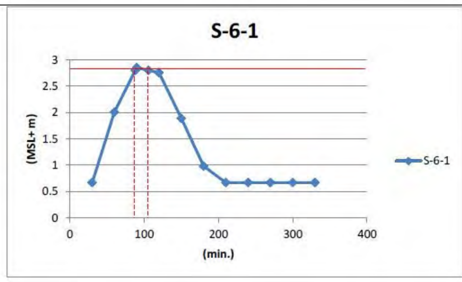
Location	Ground Elevation (m) (1)	Required Dike Elevation(m) (2)	Height of Storm Surge*1(m) (3)	Overflow Depth(m) (4)=(3)-(2)	Overflow Discharge ($m^3/s/m$) (5)	Duration of Overflow (min.) (6)	Total Overflow Discharge (m^3/m) (7)=60x(5)x(6)
S-1	2.77	4.50	4.83	0.33	0.29	16	139
S-3-2	2.72	4.00	4.55	0.55	0.63	25	469
S-4-1	2.07	3.50	3.53	0.03	0.01	3	1
S-4-4-1	2.07	3.50	3.53	0.03	0.01	3	1
S-4-4-2	2.07	3.50	3.53	0.03	0.01	3	1
S-5	1.31	3.00	3.20	0.20	0.14	41	297
S-6-1	0.67	2.80	2.85	0.05	0.02	17	9

Source: JICA Study Team



Source: JICA Study Team

Figure 4.1-37 Location of Maximum Inundation Level Occurrence for each Section (Distant)

Maximum Inundation Level	Time series of Inundation Level
	
	
	
	
	

Source: JICA Study Team

Figure 4.1-38 Max. Inundation Level and Time series of Inundation Level for each Section

b) Failure progress of grass sodding and crown asphalt

According to Civil Engineering Technical Bulletin 27-7 (1985) "Experimental Study on Levee Reinforcement against Failure by Overtopping", the relationship between overflow discharge per unit width and embankment failure was analyzed by experiments. Some results show below.

In the case of grass sodding, immediately after the overflow the surface slope erosion occurs. When the overflow discharge reaches 100～500m3/m, the crown is eroded. When the overflow discharge reaches 200～800m3/m, the dike breach occurs. It is mentioned in the report that the combination of Kanto loam (volcanic cohesive soil) and grass sodding is most resistant to erosion.

Also it is pointed out that according to the experimental result, the erosion on the landside slope is caused by sheet flow and the erosion near the crown is caused by the slope failure itself due to the slope surface erosion. This phenomenon contributes to the delay of the crown failure, however, once one location of the crown failures, afterwards the dike loses the resistance substantially.

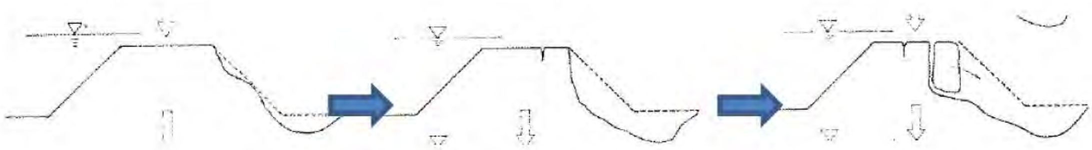


Figure 4.1-39 Failure Progress of Grass sodding Dike

Regarding the case in which the crown top is covered by asphalt, the result shows the wide variety.

Figure 7.5-9 shows the result that when the overflow discharge exceeds 3,000m3/m the dike breach occurs. Figure 7.5-10 shows when the overflow discharge exceeds 600m3/m the crown starts the erosion and when exceeds 3,000m3/m the entire breach occurs.

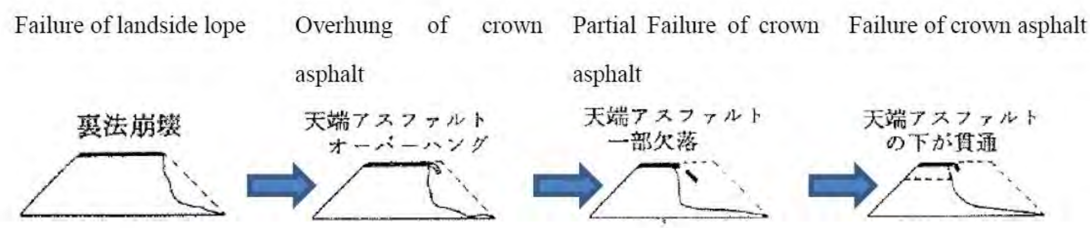


Figure 4.1-40 Failure Progress of Crown Asphalt Dike

（source: Civil Engineering Technical Bulletin 27-7 (1985) "Experimental Study on Levee Reinforcement against Failure by Overtopping"）

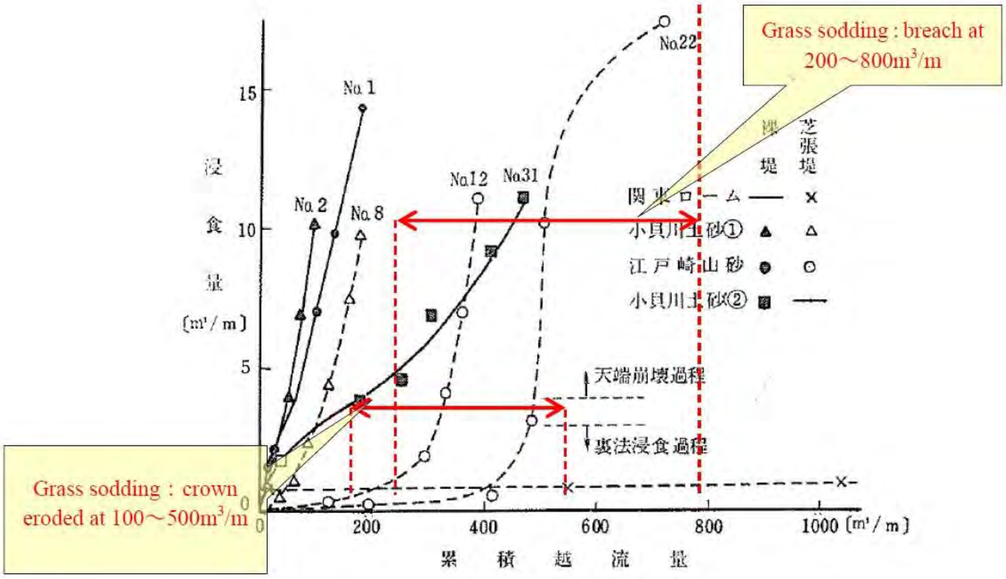


Figure 4.1-41　Relation between Overflow Discharge and Erosion

（source: Civil Engineering Technical Bulletin 27-7 (1985) "Experimental Study on Levee Reinforcement against Failure by Overtopping"）

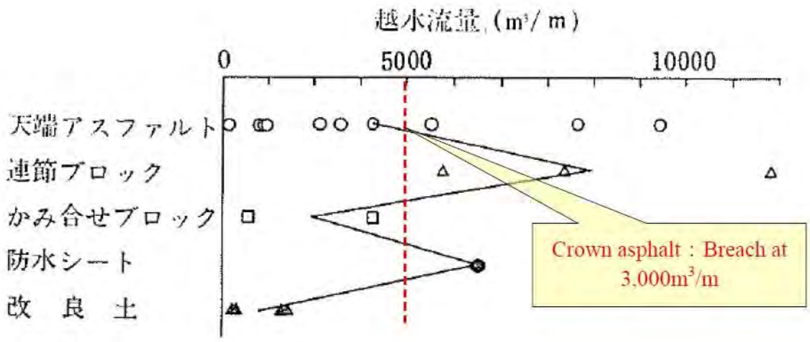


Figure 4.1-42　Total Overflow Volume per Unit Width until Dike Breach

（source: Civil Engineering Technical Bulletin 27-7 (1985) "Experimental Study on Levee Reinforcement against Failure by Overtopping"）

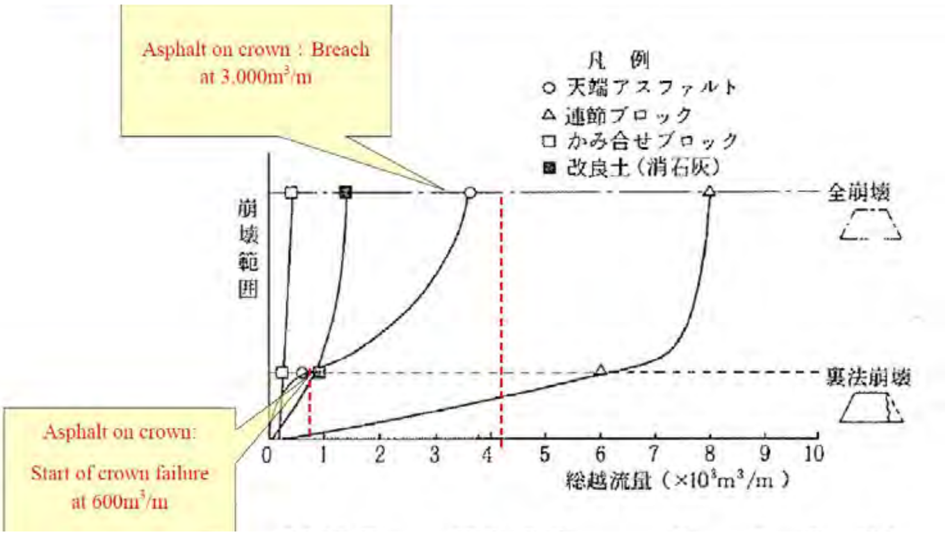


Figure 4.1-43　Extent of Dike Failure according to Overflow Volume

（Civil Engineering Technical Bulletin 27-7 (1985) "Experimental Study on Levee Reinforcement against Failure by Overtopping"）

c) Application to the study case

While the experimental result shows the wide variety, the resistant effect to overflow by asphalt crown and grass sodding is large.

The maximum overflow discharge per unit width according to the storm surge simulation on typhoon Yolanda is 1 to 469m3/m, and if the local erosion on landside slope and of crown is allowed, the combination of crown asphalt and grass sodding should be recommended as slope protection of the dike. However, in the referred report the experiment model used the asphalt for the crown, in Philippine concrete crown pavement shall consider.

4) **Protection against Tsunami**

Protection against Tsunami is not in the scope of the project but inundation map made by JICA study team based on the calculation of tsunami height studied in the Ready project gives a tsunami similar or less than that of storm surge as shown in Figure 4.1-35 in 4.1.3(3)4). As long as the overtopping depth is the same, protection for road heightening is deemed efficient for tsunami as well.

4.1.5 Structural Studies for tidal wall

(1) Selection of structure type

Tide embankment and tidal wall are the two basic structure type for tide protection. Tidal wall shall be selected where,

- Foundation ground is solid.
- Procurement of land is unavailable.
- Tidal wall is preferred in view of connection to existing structures

(2) Design conditions

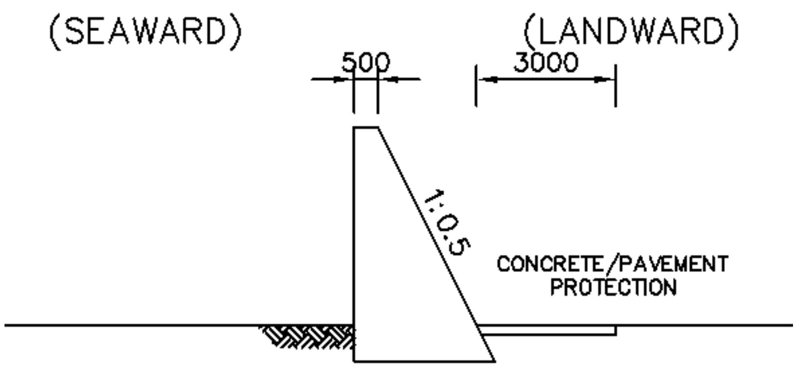
1) Top elevation

Top elevation is same as the tide embankment which is related in 4.1.3(1).

(3) Structural Studies

1) Structural specifications

Structural specifications for tidal wall is organized in the table below.



Source: JICA Study Team

Figure 4.1-44 Standard structure of road heightening

Table 4.1-7 Structural specifications of road heightening

Items	specifications	explanation
Seaward wall gradient	upright	Steepest case-based gradient for tidal wall. Lowering the gradient facilitate overtopping thus unsafe.
Landward slope gradient	1 : 0.5	Typical case-based gradient for tidal wall
Embedded depth	D=1.0m	Standard length needed for erosion protection

Source: JICA Study Team

2) Stability analysis

Stability of the foundation ground shall be analyzed utilizing the result of geotechnical survey. The structure is not applicable in case the foundation ground is not solid.

3) Protection against erosion by overflow water

The tide wall should be resilient enough to resist overtopping water against storm surge of lower frequency, as was the case of Yolanda. The resiliency of tide embankment means that the structure will not collapse even if the storm surge water overtop its crest.

Overflow water will cause erosion on the foot of landward slope, as is the case with tide embankment related in 4.1.3(3)3). In contrast to the tide embankment, embedded depth for 2 meters cannot be ensured for tide wall. Therefore, the landside ground should be covered by concrete or pavement protection.

4) Protection against Tsunami

Protection against Tsunami is not in the scope of the project but inundation map made by JICA study team based on the calculation of tsunami height studied in the Ready project gives a tsunami level of about 4.5 meters above mean sea level as organized in 4.1.3(3)4). This tsunami height gives an overtopping depth of about 1.0 meters, which equals to the overtopping depth for Yolanda. As long as the overtopping depth is the same, protection for tide embankment is deemed efficient for tsunami as well.

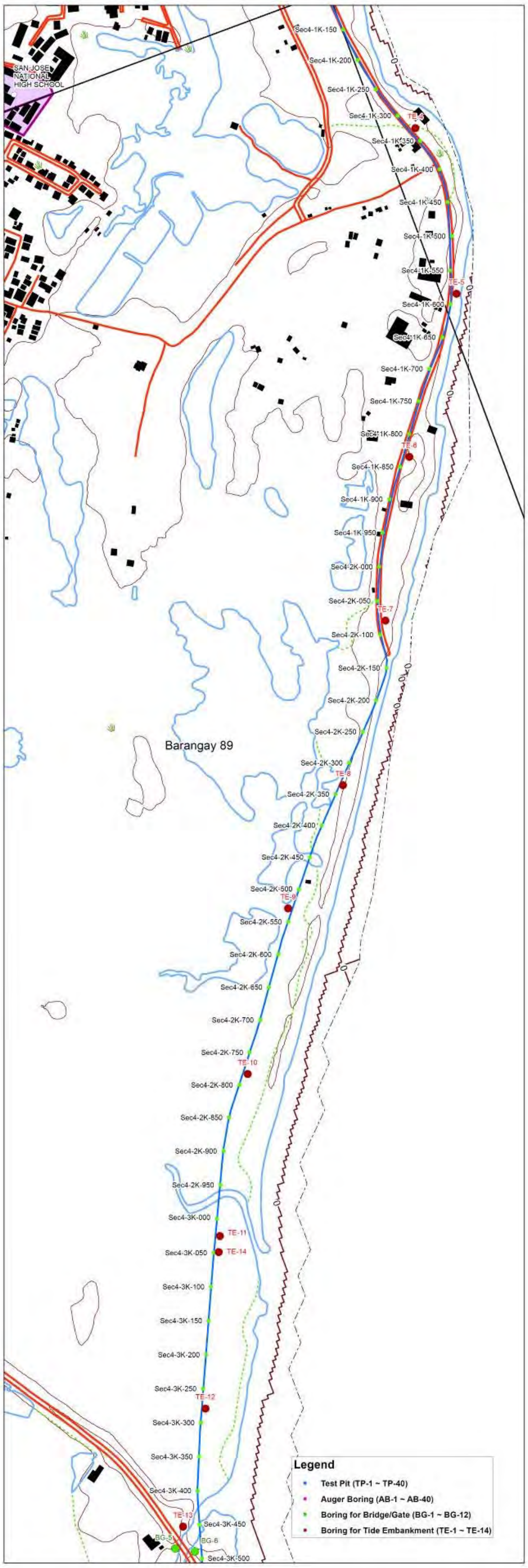
4.1.6 Geological conditions

Geological conditions of the foundation ground shall be surveyed to analyze the stability of the structure, potential risk of liquefaction and settlement possibly caused by consolidation.

(1) Location of geological survey

Geological survey was conducted along the initially proposed alignment. In section 3, since the selected alignment for Case-C is distant from the initially proposed alignment, additional geological survey must be conducted to evaluate the geological conditions for detailed design.

Location of geological survey in section 4, north of Payapay River is shown in the following figures. Additional survey is needed in the south of Payapay river along the alignment.



Source: JICA Study Team

Figure 4.1-45 Location of geological survey in Section 4

(2) Geological conditions

Table 4.1-8 shows the preliminary result of geological survey conducted along the tide embankment in section 4. Sand can be seen from the ground surface to the depth of around 10 meters. From 10 meter-depth to 20 meter-depth, clay is predominant with N value less than 5.

(3) Geotechnical analysis

Geotechnical analysis shall be made to evaluate the stability of the structure, potential risk of liquefaction and settlement possibly caused by consolidation.

Distribution of N Value in each Drilling Station

Tide Embankment

Depth	BH-4 6+600 Color	BH-4 Soil type	BH-4 N value	BH-5 6+850 Color	BH-5 Soil type	BH-5 N value	BH-6 7+100 Color	BH-6 Soil type	BH-6 N value
0	Gray	Poorly Graded Fine Sands with shell							
-1			32	Brown	Sandy Silt	65			5
-2	Dark Gray		32			29		Poorly Graded Fine Sands	7
-3	Gray	Silty Fine Sands	29		Silty Gravel with Sand	10			9
-4	White	Coral Stone	lime stone			7			11
-5		Gravels with sands and shell	> 50		Silty Fine Sands with Gravel	10			11
-6			20			6		Poorly Graded Fine Sands with silts	14
-7	Light Gray	Gravelly Poorly graded Sands with silts and coral stone	[illegible]			27			17
-8			10	Light Gray		15			18
-9		Gravelly Poorly graded Sands with silts and shell	14			11		Silty Fine Sands	20
-10			15		Silty Gravel with Sand	24			27
-11	White Gray	Gravelly Poorly graded Fine Sands with silts and shell	15			26			U.D.S.
-12			13			10		Fat Clays with sands	[illegible]
-13		Silty Fine Sands with shell	15			9		Fat Clays with sands & shell	[illegible]
-14			15		Sandy Lean Clays	10			[illegible]
-15		Sands with Silts, gravel & shell	11	Brown		11	Gray	Fta Clays with shell	[illegible]
-16	Gray		7		Lean Clays with sand	32		Fat Clays with sands & shell	[illegible]
-17	Brown	Sandy Lean Clays with shell	[illegible]			>50		Silty Fine Sands with clays & shell	[illegible]
-18		Sandy Silts with shell	36						[illegible]
-19	Light Gray	Sandy Lean Clays with shell	20					Fat Clays with few sands & shell	[illegible]
-20		Fat Clays with shell	16					Fat Clays with shell	5
-21	Gray		18	Gray	highly weathered Sandstone			Fat Clays with few sands & shell	[illegible]
-22		Fat Clays	7					Fat Clays with sands & shell	5
-23		Fat Clays with shell	8						[illegible]
-24		Silty Sands with Clays, gravel & shell	12					Lean Clays with sands & shell	[illegible]
-25	White Gray	Lean Clays with Sands & shell	10					Sandy Silts with shell	[illegible]
-26			18					Lean Clays with sands & shell	[illegible]
-27		Lean Clays with Sands	17						12
-28	Gray	Sandy Silts with shell	33					Silty Fine Sands with shell	18
-29	White Gray	Silty Fine Sands with Clays & shell	38						22
-30			37						30
-31	Light Brown	Sandy Lean Clays with shell	18						
-32		Lean Clays with sands & shell	79						
-33	Grayish Brown		80						
-34		Fat Clays with shell	80						
-35			88						
-36	Gray		100						
-37									
-38									
-39									
-40									

Depth	BH-7 7+350 Color	BH-7 Soil type	BH-7 N value	BH-8 7+600 Color	BH-8 Soil type	BH-8 N value	BH-9 7+850 Color	BH-9 Soil type	BH-9 N value
0					Poorly Graded Fine Sands with silts				
-1		Silty Fine Sands	12			4			12
-2			15			4			12
-3			13		Poorly Graded Fine Sands	8			14
-4		Silty Fine Sands with shell	13			9		Silty Fine Sands	12
-5			15		Silty Fine Sands with silts	10			14
-6			17		Poorly Graded Fine Sands	11			15
-7			14			14			17
-8		Silty Fine Sands	19		Poorly Graded Fine Sands with silts	15			21
-9			23			16			25
-10			31		Silty Fine Sands	22		Silty Fine Sands with shell	26
-11			6			[illegible]		Lean Clays with Sand	[illegible]
-12			7		Lean Clays with sands	[illegible]		Fat Clays with shell	[illegible]
-13			5			[illegible]			U.D.S.
-14		Fat Clays	6		Fat Clays	[illegible]	Gray	Lean Clays with Sand	5
-15	Gray		U.D.S.	Gray	Fat Clays with few sands	[illegible]		Silty Sands with shell	5
-16		Fat Clays with Sand	6			[illegible]			10
-17			5		Lean Clays with sands & shell	[illegible]		Fat Clays with shell	5
-18			7		Lean Clays with sands	[illegible]			6
-19		Lean Clays with Sand	5		Sandy Lean Clays with Few gravel	[illegible]			6
-20			7		Sandy Silts with few gravel	[illegible]		Fat Clays	6
-21		Fine Sands with shell	5		Lean Clays with few sands	[illegible]		Lean Clays with Sand	5
-22			6		Sandy Lean Clays with shell	[illegible]			5
-23			6		Lean Clays with sands & shell	[illegible]		Fat Clays with shell	5
-24			7		Fat Clays with few sands & shell	5			6
-25		Fat Clays with shell	7		Fat Clays with sands & shell	5			6
-26			8		Fat Clays with few sands & shell	5			7
-27			7		Lean Clays with sands & shell	5		Lean Clays with Sand and shell	5
-28			8		Fat Clays with few sands & shell	6	Light Brown		6
-29		Lean Clays with shell	7			6	Brown	Sandy Lean Clays	5
-30			8	Grayish Brown	Fat Clays with sands & shell	30	Grayish Brown		5

Depth	BH-10 8+100 Color	BH-10 Soil type	BH-10 N value	BH-11 8+350 Color	BH-11 Soil type	BH-11 N value
0						
-1			3			12
-2			5		Poorly Graded Fine Sands	13
-3			6			10
-4		Silty Fine Sands	8			14
-5			12			16
-6			18			19
-7			20		Silty Fine Sands	17
-8			22		Poorly Graded Fine Sands with silts	24
-9	Gray	Silty Fine Sands with shell	26		Silty Fine Sands	27
-10			29		Poorly Graded Fine Sands with silts	28
-11		U.D.S.	[illegible]		Fat Clays with few sands	[illegible]
-12		Fat Clays with sand	5			[illegible]
-13			6		Sandy Lean Clays	[illegible]
-14		Fat Clays with sand	9			[illegible]
-15			[illegible]	Gray	Fat Clays with little sands&shell	[illegible]
-16		Fat Clays with shell	[illegible]		Fat Clays with shell	[illegible]
-17		Sandy Silt with shell	6		Fat Clays with little sands&shell	[illegible]
-18		Fat Clays with shell	9		Fat Clays with sands&shell	[illegible]
-19		Lean Clays with sand	14		Sandy Lean Clays with shell	[illegible]
-20		Fat Clays	18			[illegible]
-21			20		Lean Clays with sands	[illegible]
-22	Grayish Brown		22			[illegible]
-23			23		Fat Clays with little sands	[illegible]
-24		Fat Clays with shell	24		Fat Clays with few sands	5
-25			8		Fat Clays	5
-26			10		Fat Clays with few sands&shell	[illegible]
-27	Light Brown	Lean Clays with sand	5			[illegible]
-28		Sandy Lean Clays	9		Fat Clays with little sands&shell	5
-29	Gray	Silty Fine Sands with few gravel	45			5
-30			50		Fat Clays with shell	5

Depth	BH-11-A 8+375 Color	BH-11-A Soil type	BH-11-A N value	BH-12 8+600 Color	BH-12 Soil type	BH-12 N value	BH-13 8+850 Color	BH-13 Soil type	BH-13 N value
0					Silty Fine Sands				
-1				Grayish Brown		2		Silty Fine Sands with Gravel	5
-2					Silty Clays	4	Brown		12
-3					Silty Fine Sands	10			21
-4			2		Silty Sands	16		Sandy Clays	[illegible]
-5	Gray	Silty Fine Sands	7			21			13
-6			8			26			22
-7			29			28		Silty Fine Sands	25
-8			14		Silty Fine Sands	32			33
-9			18			36			28
-10			26	Gary		10			8
-11			21		Silty Clays	[illegible]	Gray		[illegible]
-12			3			[illegible]			6
-13			6		Silty Clays with shell	[illegible]			[illegible]
-14					Silty Clays	[illegible]			[illegible]
-15						5			[illegible]
-16					Silty Clays with shell	[illegible]		Silty Clays	[illegible]
-17						[illegible]			[illegible]
-18		U.D.S.			Silty Clays	5	Light Gray		[illegible]
-19						[illegible]	Grayish Brown		[illegible]
-20						[illegible]	Gray		[illegible]
-21					Silty Clays with shell	[illegible]	Grayish Brown		[illegible]
-22						[illegible]	Brown	Sandy Clays	5
-23				Light Gray		[illegible]			8
-24						5	Grayish Brown	Silty Sands	13
-25						[illegible]			25
-26		Fat	3			6		Sandy Clays	21
-27	Gray		4		Silty Clays	8	Light Brown		23
-28			5	Gary		12		Clayey Sands with Gravel	19
-29			6	Light Gray		18	Brown	Silty Fine Sands with Gravel	40
-30			7	Dark Gray	Sandy Clays with shell	32	Grayish Brown	Sandstone	100

: 8 ≦ N ≦ 15 (Stiff)   : 15 ≦ N ≦ 30 (Very Stiff)   : N > 30 (Hard)   : CLAYS classified by Grain Suze Analysis

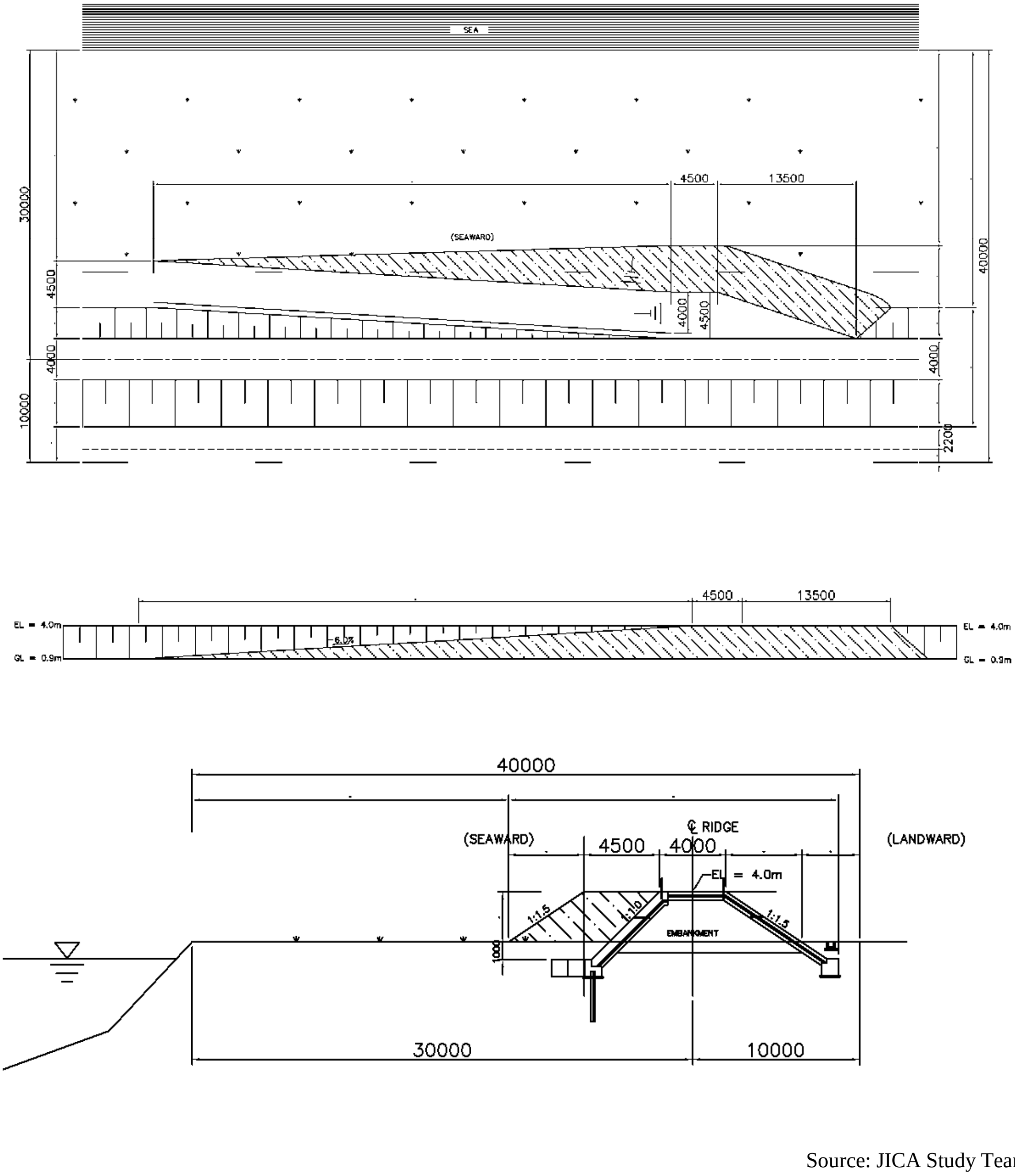
Table 4.1-8 Geological conditions along the tide embankment in section 4

Source: JICA Study Team

4.1.7 Related structures

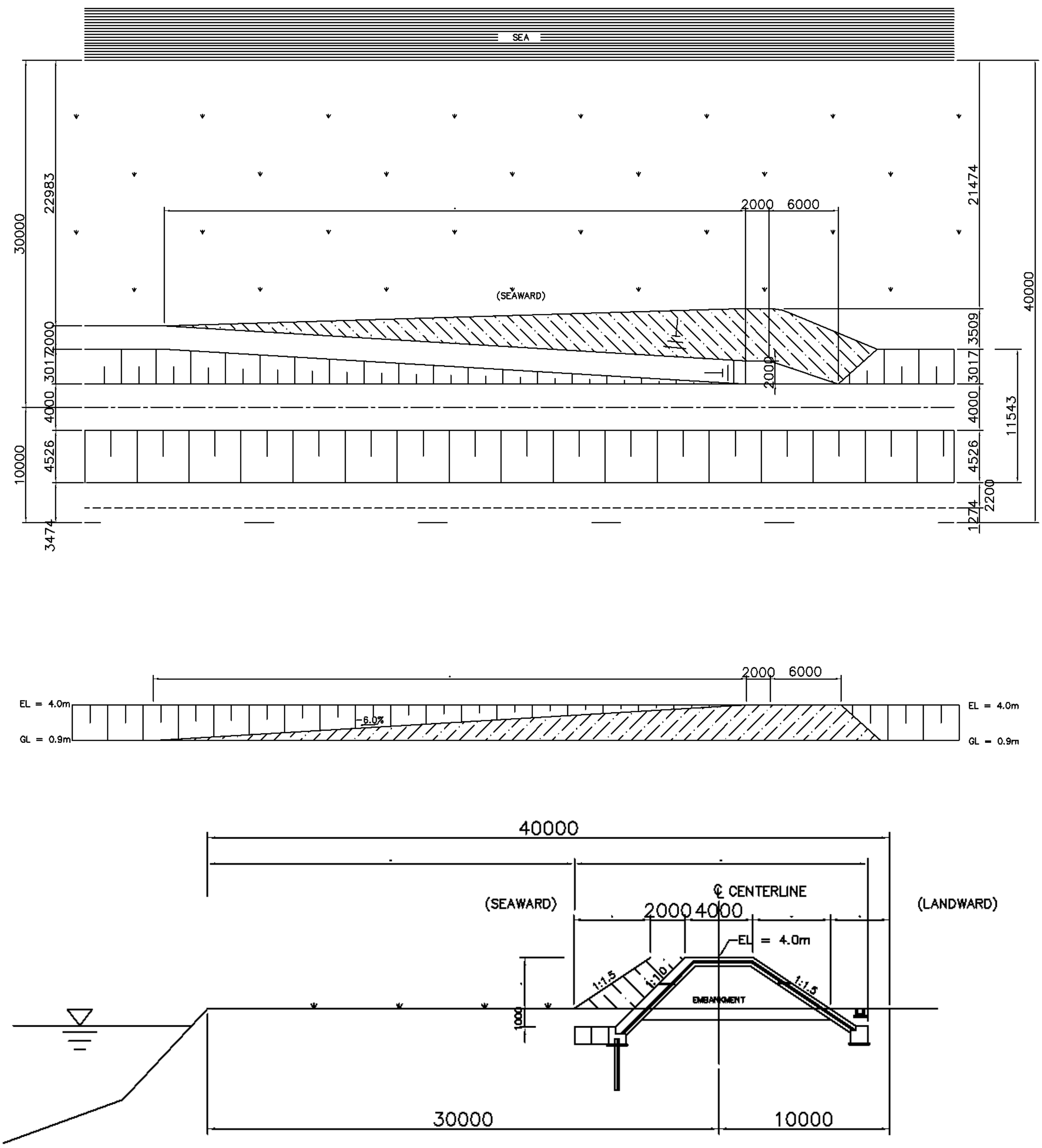
(1) Access road

Access road to the top of embankment as well as to the other side of the embankment shall be secured by installing an access road wherever necessary. Depending on its usage, whether it's for vehicle (for official use for maintenance), bicycles or pedestrians, the size of the slope must be chosen.



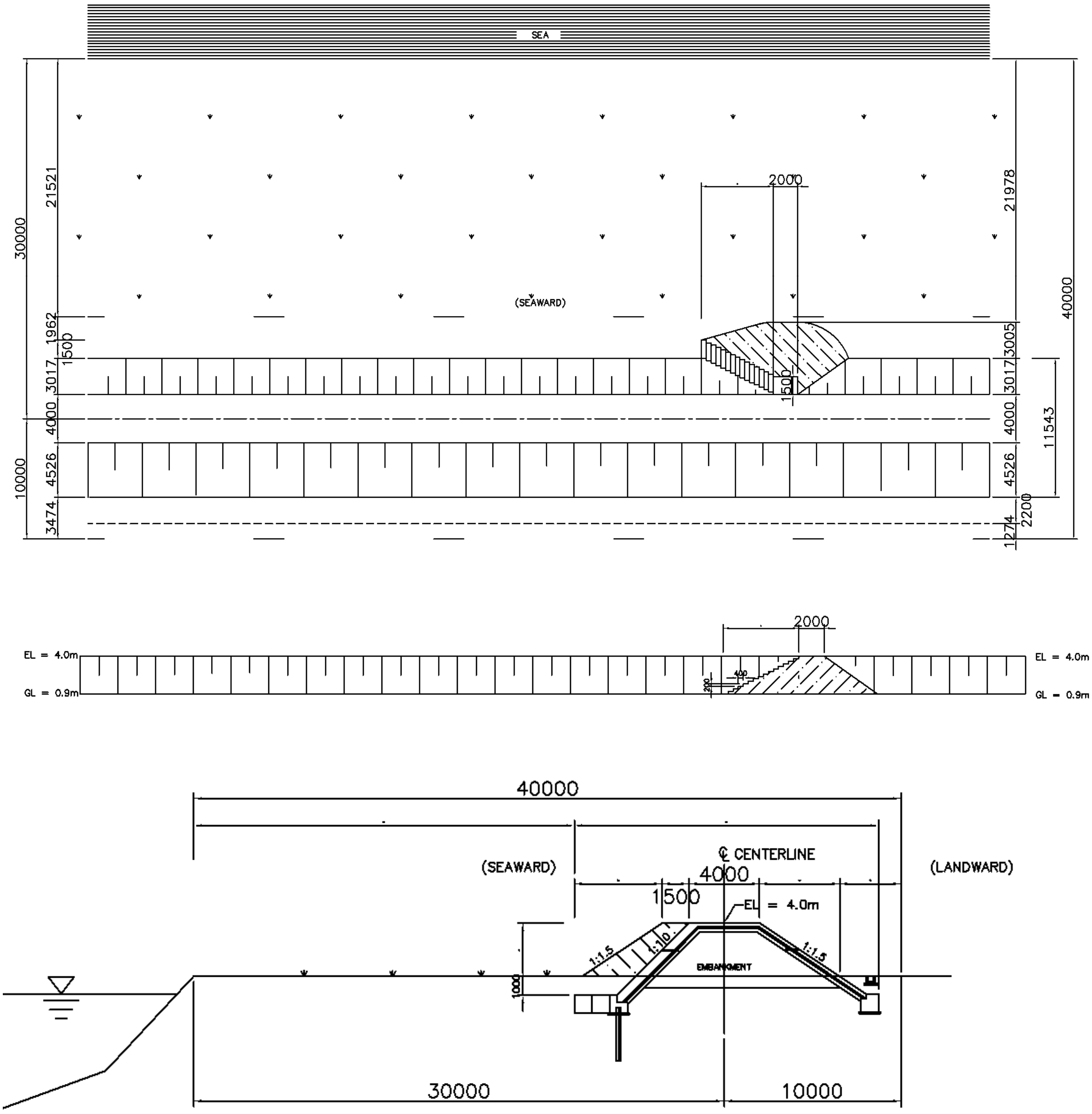
Source: JICA Study Team

Figure 4.1-46 General image of access road for vehicles



Source: JICA Study Team

Figure 4.1-47 General image of access road for bicycles



Source: JICA Study Team

Figure 4.1-48 General image of access road for pedestrians

4.2 Road

4.2.1 Traffic Volume Analysis

(1) Traffic Survey

The JICA Study Team reviewed traffic count survey data information from the DPWH and pertain to traffic along existing road for under study area of the Tidal Protection Dike alignment and these roads are one of the alternative routes for Road Heightening Scheme.

Traffic surveys were undertaken in order to establish baseline information as a basis to traffic forecast for existing highway in study area. Normally traffic forecasts aid in the design of the pavement and number of lane for the highway. However this traffic survey did not conduct Origin-Destination (OD) survey, therefore this study is not include traffic distribution in Tacloban and Palo by OD matrix.

1) Traffic Classification Count Survey

In the traffic count surveys, vehicles were counted for each direction of traffic flow and classified into nine (9) categories, namely: car/jeep/taxi, jeepneys, pick-up/van, mini-bus, big bus, 2-axle truck, 3-axle truck, 4-axle truck or more, tricycle/motorcycle. The traffic counts were conducted at twelve (12) stations for one full week, over a period full day of 24 hours. The location and date of the surveys are indicated below in Table 4.2-1.

Table 4.2-1 Traffic Classification/Intersection Count Survey

Road Name	Station No.	Date	Day	Location
Magsaysay Blvd.	S00001LT	4/10/2015 ～ 4/16/2015	Friday ～ Thursday	Lions Den - Real St. - Jct. Burgos St
Real St.	S00002LT	4/10/2015 ～ 4/16/2015	Friday ～ Thursday	Pampango St.- -Jct. Imelda St.-Magallanes St
Esparas Ave.	S00003LT	4/17/2015 ～ 4/23/2015	Friday ～ Thursday	Magallanes St.- Pericohon S
Esparas Ave.	S00004LT	4/17/2015 ～ 4/23/2015	Friday ～ Thursday	Old Road Sagkahan-Astrodome
Esparas Ave.	S00005LT	4/24/2015 ～ 4/30/2015	Friday ～ Thursday	Astrodome-CocaCola Jct (Natasha)
Real St.	S00006LT	4/24/2015 ～ 4/30/2015	Friday ～ Thursday	Coca-Cola Jct - San Jose Rotonda (INC)
San Jose DZR Airport Road	S00007LT	4/24/2015 ～ 4/30/2015	Friday ～ Thursday	San Jose Rotonda INC - Baybay Jct.
San Jose DZR	S00008LT	4/24/2015 ～	Friday ～	Baybay Jct (Yolanda) - Brgy Payapay

Road Name	Station No.	Date	Day	Location
Airport Road		4/30/2015	Thursday	
Bay Bay Rd.	S00009LT	5/1/2015 ～ 5/7/2015	Friday ～ Thursday	Macarthur Park (Payapay Bridge) - Brgy. San Fernando, Palo
Manlurip Rd.	S00010LT	5/1/2015 ～ 5/7/2015	Friday ～ Thursday	Brgy. San Fernando Baras - Jct. Guindapunan-Salvacion Road
Manlurip Rd.	S00011LT	5/1/2015 ～ 5/7/2015	Friday ～ Thursday	National Highway - Brgy. Guindapunan
Palo East ByPass Road,	S00012LT	5/1/2015 ～ 5/7/2015	Friday ～ Thursday	Palo East By-Pass Road,

Source: JICA Study Team

(2) Analysis of Traffic Survey Results

1) Annual Average Daily Traffic

The One week 24 hour counts were converted into Annual Average Daily Traffic (AADT) by applying the seasonal adjustment factors were taken from the 1999 DPWH National Traffic Counting Programme. For this study the following seasonal factors were adopted below Table 4.2-2.

Table 4.2-2 Seasonal Adjustment Factors

Vehicle Type	Seasonal Factors
Cars Jeep, Taxi	1.040
Jeepney	1.000
Pick-Up,Van	1.040
Mini Bus	1.200
Large Bus	0.950
Truck 2-3 Axle	1.110
Truck 4 or more Axle	1.750
Tricycle/M'cycle	0.890

Source: 1999 DPWH National Traffic Counting Programme

The traffic volume and the vehicle fleet composition on the along sea side existing road in the Tacloban and Palo area and AADT volumes are shown in Table 4.2-3.

Table 4.2-3 Summary of Existing AADT Volumes (2015)
(Existing Road Network)

Road Sections	Sta. No.	Car, Taxi, Jeep	Jeepney	Pickup/ Van	Mini Bus	Big Bus	2-Axle Truck	3-Axle Truck	4-Axle Truck or more	Sub-Total	Motorcycle / Tricyce	TOTAL
Mabsaysay Blvd	S00001LT	4,762	125	5,861	4	6	249	72	113	**11,192**	4,477	**15,669**
Real St.-1	S00002LT	810	42	2,727	2	1	76	3	0	**3,660**	2,922	**6,582**
Esparas Ave.-1	S00003LT	925	36	2,485	0	0	46	0	0	**3,492**	3,026	**6,519**
Esparas Ave.-2	S00004LT	699	53	2,731	0	0	168	13	1	**3,665**	2,473	**6,138**
Esparas Ave.-3	S00005LT	8,290	285	6,491	6	1	344	97	71	**15,584**	7,627	**23,211**
Real St.-2	S00006LT	5,643	184	3,075	3	3	660	105	403	**10,076**	6,851	**16,927**
San Jose DZR Airport Rd.-1	S00007LT	2,938	164	2,707	3	0	473	86	42	**6,414**	4,047	**10,460**
Baybay Rd.	S00008LT	120	12	29	0	0	33	2	1	**196**	647	**843**
Manlurip Rd.-1	S00009LT	3,171	149	98	2	1	309	161	25	**3,915**	5,457	**9,372**
Manlurip Rd.-2	S00010LT	1,910	150	98	2	0	369	94	23	**2,646**	4,285	**6,931**
Manlurip Rd.-3	S00011LT	1,995	111	177	2	2	510	110	11	**2,918**	4,216	**7,134**
Palo East By-Pass Road	S00012LT	2,563	142	170	2	2	724	175	149	**3,926**	3,836	**7,762**

Source: Traffic Count Survey Data by DPWH

Based on the results of the traffic counts, traffic volume on the existing road varies from 23,211 to 843 vehicles per day. Among them, the largest traffic volume was observed along the Astrodome-Coca Cola Junction (Natasha). In most case traffic along this road is characterized by the high composition of Car/Taxi/Jeep which is about 36% of the total traffic volume and second case traffic is characterized by Motorcycle/ Tricyce which is about 32% of the traffic volume.

(3) Traffic Demand Forecast

1) Methodology

The estimated normal and diverted traffic will serve as either the base year or opening year traffic for each road. The above traffic was projected by vehicle type throughout the 20-years assumed economic life of each road by employing traffic growth rates.

The traffic growth rates were calculated based on the traditional method presented in the DPWH Highway Planning Manual, Volume 3. This method considers three (3) factors namely: i) population growth rate ii) Gross Regional Domestic Product (GRDP) Per Capita Growth Rate and iii) elasticity of demand for transport, all within the Region VIII.

2) Present Population and Forecast

Region VIII of Eastern VISAYAS has a total population forecast of 4,911,500 based on the Year 2000 Survey of National Statistics Office.

Population development was assessed on the basis of the past performance and the future prospects as estimated by NSO population projection.

As shown in Table 4.2-4 is the population forecast in Region VIII of Eastern VISAYAS by NSO.

From 2015 to 2020 the population within the Region VIII is increased at an annual growth rate of 2.00% and each year of population growth rate is shown in Table 4.2-5.

Table 4.2-4 Population Forecast for Region VIII

Age/Sex	2000	2005	2010	2015	2020	2025	2030	2035	2040
REGION VIII – EASTERN VISAYAS									
Both sexes	3,629,400	4,020,900	4,447,500	4,911,500	5,406,300	5,914,700	6,417,500	6,906,600	7,381,300
Male	1,854,200	2,049,600	2,264,900	2,501,200	2,753,800	3,011,300	3,265,000	3,510,800	3,749,500
Female	1,775,200	1,971,300	2,182,600	2,410,300	2,652,500	2,903,400	3,152,500	3,395,800	3,631,800

Source: National Statistics Office

Table 4.2-5 Population Growth Rate

Region	**Y E A R**				
	2015-2020	**2020-2025**	**2025-2030**	**2030-2035**	**2035-2040**
Region VIII Eastern Visayas	1.0200	1.0001	1.0181	1.0165	1.0148

Source: JICA Study Team's Estimate

3) Gross Regional Domestic Product (GRDP) Estimate and Forecast

The latest available data on Gross Regional Domestic Product (GRDP) is presented in the 2013 by Philippine Statistics Authority. In the absence of GRDP by Region VIII, GRDP per capita in the Region VIII is Php 35,535 and growth rate of 2012-2013 is 4.5% based on at constant year 2000 price estimated by Philippine Statistic Authority (PSA). The GRDP per capita has been projected following the GRDP forecasts for Region VIII.

The GRDP per capita of corresponding growth rates for the Region VIII is shown in Table 4.2-6 respectively.

Table 4.2-6 GRDP Per Capita Growth Rate (%)

	2015-2019	**2020-2024**	**2025-2029**	**2030-2034**	**2035-2039**
Region VIII East VISAYAS	4.90	5.42	5.67	5.91	6.26

Source: JICA Study Team's Estimate

4) Traffic Growth Rate

Based on the existing traffic forecasting methodology, traffic growth factors were established. The basic inputs are population growth, GRDP per capita growth and consumer demand elasticity factors for transport usage. The analysis on elasticity considered both public and private passenger transport.

Population was estimated by the project area of the Region VIII. The population forecast was based on the NSO's projection for base year of 2000.

The projection of traffic volume was carried out by using Traffic Growth Rate (TGR). The TGR is derived by using the traditional method found in the DPWH Highway Planning Manual Volume 3. The major inputs in the derivation of traffic growth rates are projected population growth rate, GRDP per capita growth rate, and the transport demand elasticity.

Mathematically, the TGR formula is shown below and each factor is shown in Table 4.2-7 and Table 4.2-8.

TGR = 1 + (PCIGR x Elasticity) /100 x CPGR –1,

*Where;*PCIGR = GRDP Per Capita Growth Rate

E = Elasticity of demand for transport

CPGR = Compounded Population Growth Rate

Table 4.2-7 GRDP Per Capita and Population Growth Rates

Period Year to Year	GRDP Per Capita Growth Rate	Compounded Population Growth Rate
2015-2020	4.90	1.0200
2020-2025	5.42	1.0001
2025-3030	5.67	1.0181
2030-2035	5.91	1.0165
2035-2040	6.26	1.0148

Source: JICA Study Team

Table 4.2-8 Traffic Demand Elasticity (%)

Car/Jp/Van	Jeepney	Buses	Trucks	Motorcycle	Tricycle
1.8	1.5	1.5	1.0	1.1	1.1

Source: NEDA-MIDP

The forecast for Traffic Growth Rate is estimated based on the above mentioned inputs as shown in Table 4.2-9.

Table 4.2-9 Traffic Growth Rates (%)

Year to Year	Car/Jp/Van	Jeepney	Buses	Trucks	Motorcycle	Tricycle
2015-2020	11.20	9.33	9.33	6.22	6.84	6.84
2020-2025	12.14	10.11	10.11	6.74	7.42	7.42
2025-2030	12.94	10.79	10.79	7.19	7.91	7.91
2030-2035	13.47	11.22	11.22	7.48	8.23	8.23
2035-2040	14.24	11.87	11.87	7.91	8.70	8.70

Source: JICA Study Team

5) **Result of Traffic Demand Forecast**

Traffic Assignment Model results are presented as AADT volumes by vehicle type along each homogeneous link. The detailed calculation for future traffic demand per type of vehicle for each road section is presented in the Appendix of this report. Result of Traffic Demand Forecast of each road is shown in Table 4.2-10.

Table 4.2-10 Traffic Demand Forecast

	Road Section	AADT/PCU	2015	2020	2025	2030	2035	2040
1	Lions Den - Real St. - Jct. Burgos St	AADT	15,669	22,352	32,962	50,086	77,790	124,468
		PCU	16,307	23,474	34,917	53,485	83,660	134,668
2	Pampango St.- -Jct. Imelda St.-Magallanes St.	AADT	6,582	9,149	13,109	19,307	29,018	44,841
		PCU	6,310	8,931	13,036	19,560	29,931	47,053
3	Magallanes St.- Pericohon St.	AADT	6,519	9,060	12,986	19,135	28,786	44,540
		PCU	6,055	8,581	12,545	18,858	28,920	45,580
4	Old Road Sagkahan- Astrodome	AADT	6,138	8,542	12,255	18,070	27,188	42,049
		PCU	6,213	8,773	12,777	19,124	29,193	45,778
5	Astrodome-CocaCola Jct (Natasha)	AADT	23,211	33,143	48,955	74,570	116,192	186,682
		PCU	22,616	32,681	48,842	75,233	118,415	191,975
6	Coca-Cola Jct - San Jose Rotonda (INC)	AADT	16,927	23,833	34,712	52,153	80,227	127,388
		PCU	16,140	22,922	33,685	51,078	79,276	126,954
7	San Jose Rotonda INC - Baybay Jct.	AADT	10,460	14,720	21,413	32,107	49,252	77,914
		PCU	10,245	14,557	21,385	32,381	50,138	80,008
8	Baybay Jct (Yolanda) - Brgy Payapay	AADT	843	1,135	1,572	2,236	3,248	4,855
		PCU	573	783	1,103	1,599	2,373	3,634
9	Macarthur Park (Bridge Payapay) - Brgy. San Fernando, Palo	AADT	9,372	13,015	18,695	27,710	42,097	66,107
		PCU	7,279	10,267	15,007	22,672	35,131	56,305
10	Brgy.San Fernando, Baras - Jct. Guindapunan-Salvacion Road	AADT	6,931	9,534	13,547	19,846	29,778	46,158
		PCU	5,413	7,538	10,865	16,175	24,695	38,998
11	National Highway - Brgy. Guindapunan	AADT	7,134	9,815	13,951	20,444	30,687	47,581
		PCU	5,858	8,135	11,690	17,347	26,397	41,542
12	Palo East By-Pass Road	AADT	7,762	10,761	15,432	22,842	34,658	54,367
		PCU	7,173	9,963	14,326	21,286	32,447	51,179

Source: JICA Study Team

6) **Required Number of Lane**

Calculation of required number of lane is used target AADT of traffic demand forecast in Year 2035 which is 20 years from this year.

DDHV = AADT x K x D = Directional design hour volume
K = Peak-hour traffic factor = 8.8%
D = Directional distribution factor = 60%
K x D = 5.28%

Service Flow Rate (SF) in one direction is calculated using the following equation:
SF = MSF x fw x fHV
where:
MSF = Maximum Service Flow = 1,700 pcu/hr/lane (from HCM)
Fw = adjustment factor for lane width and or lateral clearance, = 0.94
Fhv = adjustment factor for heavy vehicle, = 0.93
SF = 1,700 x 0.94 x 0.93 = 1,486 pcu/hr./lane

Table 4.2-11 Lane Requirement for CNCRP

Road Section	Traffic Projection for Year 2035		No. of Required Lanes/Direction
	AADT pcu/day	Peak Hour = 5.28% of AADT	
Magsaysay Rd.	83,660	4,417	2.97 – (3 lanes)
Real St-1	29,931	1,580	1.06 – (2 lanes)
Esparas Ave.-1	28,920	1,527	1.03 – (2 lanes)
Esparas Ave.-2	29,193	1,541	1.04 – (2 lanes)
Esparas Ave.-3	118,415	6,525	4.39 – (5 lanes)
Real St.-2	79,276	4,186	2.82 – (3 lanes)
San Jose Airport Rd.-1	50,138	2,647	1.78 – (2 lanes)
Baybay Rd.-1	2,373	125	0.08 – (1 lane)
Manlurip Rd.-1	35,131	1,855	1.25 – (2 lanes)
Manlurip Rd.-2	24,695	1,304	0.88 – (1 lane)
Manlurip Rd.-3	26,397	1,394	0.94 – (1 lane)
Palo East By-Pass Road	32,447	1,713	1.15 – (2 lanes)

Source: JICA Study Team

Based on above result, in most case 5 lanes per direction are required in Year 2035 for Esparas Ave at location of Astrodome-CocaCola Jct (Natasha) and mostly needs more than 2 lanes per direction in the Year 2035. It is meaning needs road network improvement in Tacloban City within 10 years period. If heightening of existing road is chosen, construction cost becomes very high to accommodate required number of lanes in the future. If not it is very difficult for future improvement.

4.2.2 Road Alignment

(1) Geometric Design Standard

The Geometric Design Standard is to be based on Design Guidelines, Criteria and Standards for Public Works and Highways Volume I and II Department of Public Works and Highways (DPWH) and AASHTO policies.

The proposed modified Geometric Design Standard is shown in Table 4.2.12 and Table 4.2.13 is shown DPWH Geometric Design Standards.

The engineering design was made in order to come up with a reasonable estimate of the project cost with an accuracy of + or –10%.

1) Anticipated Traffic Volume

The design of a highway or any part thereof should be based on factual data among those related to traffic. The service for the improvement is indicated by present and future demands of traffic. It directly affects the geometric features of design such as number of lane, width, grade, alignment and type of pavement. Similarly, all roads should be designed to accommodate almost all types of vehicles with provision for safety and convenience

2) Design Speed

The value of a highway is evaluated by the convenience and economy that it affords in transporting goods and people in a safe and expeditious manner. The design speed should be the maximum safe speed that can be maintained over a specified section of a highway where conditions are so favorable that the design features of the highway govern. Table 4.2-12 shows the recommended design speed for each type of road for flat and rolling terrain

3) Horizontal Alignment

The horizontal alignment is a series of tangents and circular curves, connected by transition curves. The factors considered are safety, grade profile, type of facility, design speed, topography and construction cost.

4) Vertical Alignment

The vertical alignment is the series of connected gradients and vertical curves. General controls for vertical alignment are the following:

- Smooth grade line with gradual changes;

- The roller coaster or hidden dip type of profile should be avoided;
- Undulating grade lines involving substantial lengths of momentum grades should be appraised for their effect upon traffic operations since they may result in undesirably high downgrade speeds of trucks;
- A broken back grade line should be avoided;
- On long grades, it is preferable to lighten the grades near the top of the ascent, particularly on low design speed highways;
- Gradients through the intersections should be reduced;
- Climbing lanes should be considered where the critical length of grade is exceeded. The DHV exceeds the design capacity on the grade by 30% in case of multilane highways;
- Cross-section Elements;

These comprise the types of surface, the width of pavement, the cross slopes, the shoulders, drainage channels and side slopes.

Table 4.2-12 Recommended Geometric Design Standards

Design Element	**Unit**	**Design Speed (kph)**			
		40	**50**	**60**	**80**
1. Minimum Radius of Curvature	m	55	85	120	220
2. Minimum Clothoid Parameter	A	45	65	80	120
3. Maximum Grade	%	8	8	7	6
4. Maximum Super-elevation	m/m	0.10	0.10	0.10	0.10
5. Minimum Stopping Sight Distance	m	50	65	80	110
6. Minimum Passing Sight Distance	m	150	200	250	325
7. Lane Width (4 lane traffic)	m	3.25	3.25	3.35	3.50
8. Length of Vertical Curves:					
- Crest (Desirable Value)	m	5A	8A	14A	32A
- Sag (Desirable Value)	m	6A	10A	15A	25A
9. Embankment Side Slope	m	1:1.5	1:1.5	1:1.5	1:1.5
10. Normal Cross Slope					
- Concrete Pavement	%	2.0	2.0	2.0	2.0
- Asphalt Pavement	%	2.0	2.0	2.0	2.0
11. Sidewalk					
- Slope	%	4	4	4	4
- Width	m	1.5	1.5	1.5	1.5
12. Right-of-Way Width	m	30	30	30	30

Source: JICA Study Team

Table 4.2-13 Minimum Design Standard Philippine Highways

ADT AVERAGE DAILY TRAFFIC ON OPENING	UNDER 200	200 – 400	400 – 1000		1000 - 2000		MORE THAN 2000	
			MINIMUM	DESIRABLE	MINIMUM	DESIRABLE	MINIMUM	DESIRABLE
DESIGNED SPEED (km/h)								
FLAT TOPOGRAPHY	60	70	70	90	80	95	90	100
ROLLING TOPOGRAPHY	40	50	60	80	60	80	70	90
MOUNTAINOUS TOPOGRAPHY	30	40	40	50	50	60	60	70
RADUIS (metre)								
FLAT TOPOGRAPHY	120	160	160	280	220	320	260	350
ROLLING TOPOGRAPHY	55	65	120	220	120	220	160	280
MOUNTAINOUS TOPOGRAPHY	30	50	50	80	80	120	180	160
GRADE (PERCENT)								
FLAT TOPOGRAPHY	6.0	6.0	5.0	3.0	4.0	3.0	4.0	3.0
ROLLING TOPOGRAPHY	8.0	7.0	6.0	5.0	5.0	5.0	5.0	4.0
MOUNTAINOUS TOPOGRAPHY	10.0	9.0	8.0	6.0	7.0	6.0	7.0	5.0
PAVEMENT WIDTH (m)	4.0	5.5 ; 6.0	6.10		6.70		6.70	7.30
SHOULDER WIDTH (m)	0.50	1.0	1.50	2.00	2.50	3.00	3.00	
RIGHT OF WAY (m)	20	30	30		30	30	30	
SUPERELEVATION (m / m)	0.10 (MAX.)		0.10 (MAX.)		0.10 (MAX.)		0.10 (MAX.)	
NON PASSING SIGHHT DISTANCE (metre)								
FLAT TOPOGRAPHY	70	90	90	135	115	150	135	160
ROLLING TOPOGRAPHY	40	60	70	11.5	70	115	90	135
MOUNTAINOUS TOPOGRAPHY	40	40	40	60	60	70	70	90
PASSING DISTANCE (metre)								
FLAT TOPOGRAPHY	420	490	490	615	645	645	615	675
ROLLING TOPOGRAPHY	270	350	350	560	560	560	490	615
MOUNTAINOUS TOPOGRAPHY	190	270	270	350	420	420	420	490
TYPE OF SURFACING	GRAVEL, CRUSHED GRAVEL OR CRUSHED STONE BIT, PRESERVATIVE TREATMENT, SINGLE OR DOUBLE BIT, SURFACE TREATMENT, BITUMINOUS MACADAM PAVEMENT		BITUMINOUS MACADAM PAVEMENT, DENSE OR OPEN GRADED PLANT MIX SURFACE COURSE, BITUMINOUS CONCRETE SURFACE COURSE		BITUMINOUS CONCRETE SURFACE COURSE		BITUMINOUS CONCRETE SURFACE COURSE PORTLAND CEMENT CONCRETE PAVEMENT	

Source: DPWH Design Standard and Criteria

(2) Road Alignment for Extension of Service Road (Baybay Road) and Road Widening for Manlurip Road in Section 4

1) Extension of Service Road (Baybay Road) in Section 4

Existing road of Baybay road at around Sta. 2+400 in Section 4 was washed out during Typhoon Yolanda. Rehabilitation of damage by Typhoon Yolanda, Tidal Protection Dike will be constructed 40m of no built zone from shore line and beside for landside of this dike will be constructed 6.1m width concrete pavement extension road to connect Manlurip Road as a Service Road (Barangay Road). Basically road alignment will be same as Tidal Protection Dike.

2) Road Widening in Section 4

Existing road (Manlurip Rd.) of 760m in length at Sta. 3+500～Sta. 4+260 based on stationing of tide protection dike in Section 4 will be affected new dike construction. Therefore existing road shall be sifted about 8m to landside and road alignment is basically same as existing road.

4.2.3 Intersections

(3) General

One of the important highway elements, limiting capacity and often interrupting the flow of vehicular traffic, especially in an urban area, is the intersection-at-grade. Proper channelization designs at intersection will improve traffic movements and safety, increase capacity, and instill drivers confidence. Channelized intersection will also directly influence the capacity of the road to provide services to the vehicles and pedestrian traffic.

(4) Objective

The main objective is to design the intersection that would provide a maximum service volume for individual isolated intersection approaches and thereby maximize the capacity for each intersection.

As a function of the preliminary design process the Consultant ensures that the location of intersection shall be chosen to avoid steep profile grades for driver safety (less than 2.5%).

An intersection is the general area where two or more highways join or cross for traffic movements. It is an important part of a highway since much of the efficiency, safety, speed, cost of operation, and capacity of the carriageway are dependent upon its design.

General Consideration for design

Following are the guidelines adopted in designing at-grade intersections:

- Provide sight distance at least equal to the stopping distance for the design speed of the road. Avoid if possible intersection in cuts or near the crest of vertical curves.
- If possible avoid placing the intersection where the superior road is on a sharp horizontal curve.
- Avoid intersections where road is on a steep grade.
- Try to make the intersection as nearly right angled as possible. It is operationally safer and cheaper to construct.

The major intersections are proposed to be signalized and all minor crossings which require only a simple road connection by improving the corner radius.

The pertinent data for the traffic flow forecast for the intersections shall be the basis for the design of the channelized at-grade intersections and has be undertaken during the detailed design phase.

The geometric design criteria were based on AASHTO, A Policy on Geometric Design of Highways and Streets, Series of 2001 and Highway Capacity Manual Series 2000.

(5) Design of Minor Intersections

The design of minor intersections is standardized for the entire stretch of the project. A standard layout is prepared for a typical T, Y, and 4-legged crossing indicating a range of pavement edge radius that could be adopted, as shown in Table 4.2-14.

A small radius layout is accepted on the basis that:

- turning vehicles are not significant in number;

- there are few occasional trucks to turn; and/or
- a parking lane is provided and shall not be permitted with in safe
- appropriate distances from the crossing.

Table 4.2-14 Design Pavement Edge Radius for Minor Intersection

TYPE	Turn < 90º	Turn = 90º	Turn > 90º
T - Intersection	--	6 – 10 m.	--
y - Intersection	6 m.	--	8 – 18 m.
+ - Intersection	--	6 – 15 m.	--

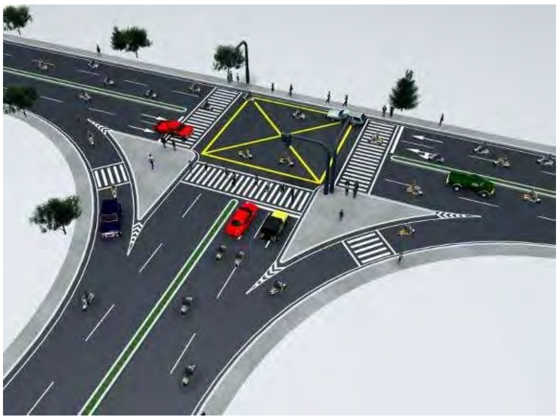
Source: JICA Study Team

(6) Design of Major Intersections

All types of intersections are designed and conducted with the characteristics of simplicity and uniformity. Intersection movements should appear obvious to the driver. To accomplish this effect, intersections are designed that permissible vehicle paths are easily driven while undesirable paths relocated. Complex designs that may confuse a driver are eliminated.

The need for uniformity of design should be directed to overcome the driver's deficiencies. Most drivers tend to drive by habit and generally do not devote full attention in the driving task. Confronted with unusual highway environment or different operational conditions, most drivers tend to seek solutions based on their previous experience. Likewise, the infrequent users of the highway facility can easily become confused when confronted with unexpected situations.

Typical Three-Leg and Four-Leg Channelized Intersection is presented in Figure 4.2-1.



Source: JICA Study Team

Figure 4.2-1 Typical Three-Leg and Four-Leg Channelized Intersection

(7) Geometric Design Standard

Geometric design elements have a strong influence on the safety and efficiency of operation of intersection-at-grade. The elements for which uniformity is important are, the design speed, corner radii, intersection angles, vehicle turning paths, auxiliary lanes, median end treatment and channelized island

(8) Design Speed

Generally, the design speed is determined during the design of the road horizontal alignment. Design speed is adopted 60 kph for common section, and 40 kph for at-grade intersections. This is a reduction by 20 kph from the adopted design speed for common section

(9) Corner Radii

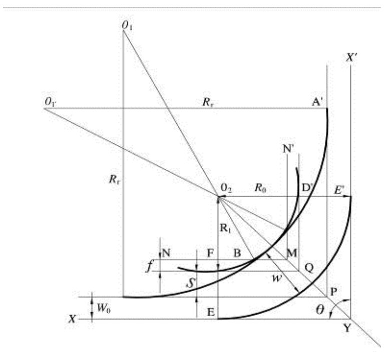
The turning roadway was designed using a simple curve and asymmetrical compound curves. The radius for the simple curve was limited to a minimum of 10 meters to suit operational requirements and to minimize ROW acquisition cost. Where smooth passing is required for a right turn vehicle, either a tapered section precedes the curve or asymmetrical compound curve (radius of a flatter curve is twice the radius of a sharper curve). The width was based on two-lane operation with stalled vehicle passing for SU design vehicle.

(10) Intersection Angle

The angle of intersection is the difference of Azimuths of two intersecting road. Azimuths of the road are established in the design of the horizontal alignment. The azimuths of the crossing road are determined from the coordinates of its centerline profile

(11) Turning Path

The turning path is recommended to use 3-centered compound curve. Outside radius (Ro) and turning path width (w) will be selected from Table 4.2-15 Transition curve radius will be set 3~4 times the inner radius (Ri). Margin width requires less than 0.5m from the edge of pavement to the edge of turning path.



Source: JICA Study Team

Figure 4.2-2 Turning Path Design

Table 4.2-15 Turning Lane Width

<table>
<tr><th colspan="2"></th><th>Large Semi-Trailer</th><th>Truck/Bus</th><th>Passenger Car</th></tr>
<tr><td colspan="2"><= R <</td><td></td><td></td><td></td></tr>
<tr><td>7</td><td>9</td><td>-</td><td>-</td><td>3.5</td></tr>
<tr><td>9</td><td>13</td><td>-</td><td>-</td><td rowspan="12">3</td></tr>
<tr><td>13</td><td>14</td><td>8.5</td><td rowspan="2">5.5</td></tr>
<tr><td>14</td><td>15</td><td>8</td></tr>
<tr><td>15</td><td>16</td><td>7.5</td><td rowspan="3">5</td></tr>
<tr><td>16</td><td>17</td><td>7</td></tr>
<tr><td>17</td><td>19</td><td>6.5</td></tr>
<tr><td>19</td><td>21</td><td>6</td><td rowspan="2">4.5</td></tr>
<tr><td>21</td><td>25</td><td>5.5</td></tr>
<tr><td>25</td><td>30</td><td>5</td><td rowspan="2">4</td></tr>
<tr><td>30</td><td>40</td><td>4.5</td></tr>
<tr><td>40</td><td>60</td><td>4</td><td rowspan="2">3.5</td></tr>
<tr><td>60</td><td></td><td>3.5</td></tr>
</table>

Source: JICA Study Team

(12) Channelizing Island

All major intersections are designed with channelized islands. Channelization is also important because it can be used to reduce impedance by separating conflicting flow from each other.

The basic considerations adopted in the design are:

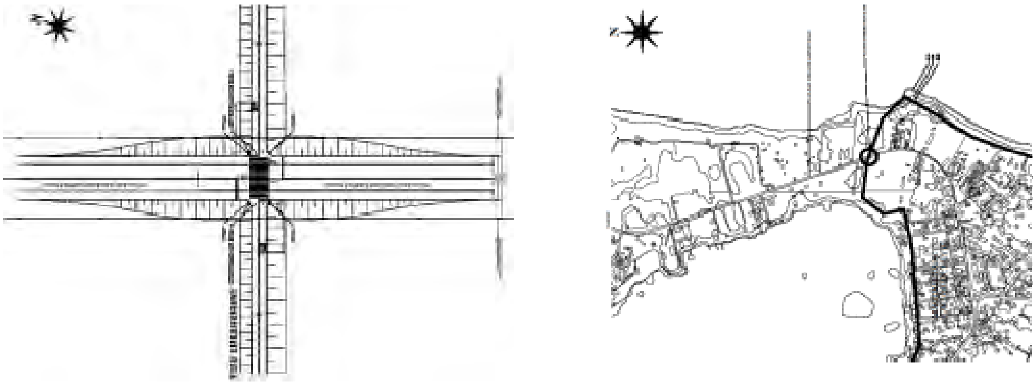
- to separate and regulate the right turn traffic;
- to increase capacity of the intersection;
- to improve safety and driver’s confidence;
- to serve as pedestrian refuge;
- to reduce paved area thereby narrowing conflicting areas for the vehicle; and
- to provide space for any traffic control devices as necessary.

The dimensions of the island were the product of the right turns radii and the left turn radii, approach pavement width and the geometry of the islands provided.

(13) New Intersection for Starting Point of Section 4 at San Jose Airport Road

Existing San Jose Airport Road will be crossing Tidal Protection Dike alignment at starting point of section 4. Therefore existing San Jose airport road height shall be adjusted to Tidal Protection Dike elevation to make new intersection, but normally public car cannot enter dike road. This dike road is for only pedestrian, bicycle and maintenance car use.

Alignment of San Jose Airport road is same as existing alignment. Below Figure 4.2-3 shows schematic plan for intersection of San Jose Airport road and Tidal Protection Dike.

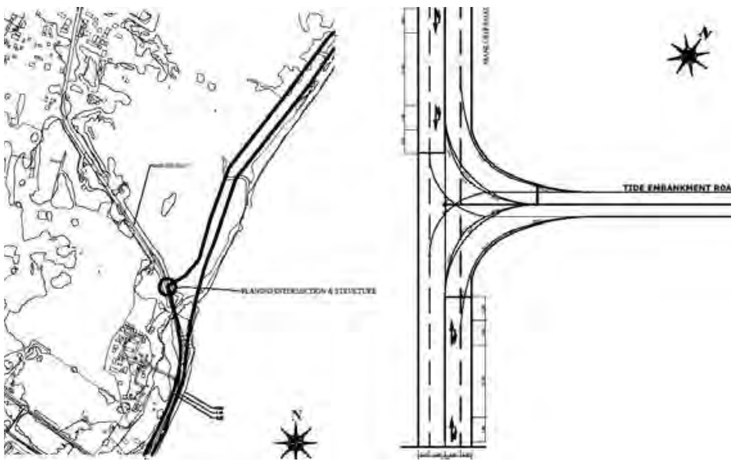


Source: JICA Study Team

Figure 4.2-3 Intersection of San Jose Airport Road and Tidal Protection Dike

(14) New Intersection for Extension of Baybay connected to Manlurip Road in Section 4

Existing Baybay road from Sta. 2+400 in Section 4 will be extended to Manlurip Road at Sta. 3+700 and this connection point has been new T type intersection. Below Figure 4.2-4 shows schematic plan for intersection of Baybay road and Manlurip Road.



Source: JICA Study Team

Figure 4.2-4 Intersection of Baybay road and Manlurip Road at Section 4

4.2.4 Pavement Structure

The Design Standards and Criteria will adopt DPWH and AASHTO Guidelines 2004 edition in the design of the pavement for the road project. Parameters/data for input shall be taken from soils survey (CBR), traffic surveys (computation of ESAL and ESWL); Modulus of Resiliency (from Laboratory Test results) and demand forecast.

The Pavement Design will use of the result of the Life-cycle Cost Analysis for the Road Project.

(1) Design Life Period

The pavement design life is as follows;

Rigid Pavement:

- Highway……………… 30 years
- Service Road…………. 20 years

Flexible Pavement:

- Highway……………… 20 years
- Service Road…………. 20 years (Total Extend Life)
  {10 years (Initial period) + 10 years (Overlay)}

(2) Selection of Pavement Type

Basically Pavement type is divided broadly into two categories which are Rigid Pavement and Flexible Pavement. One of the differences between these two types of pavement lies in the ability to adopt stage construction to optimize the investment in the project implementation. Adoption of multi-stage is initial project construction for 10-year design life with periodic overlays to extend the performance period, is common in the case of Flexible pavement, while single-stage construction is normally adopted in the case of Rigid pavement.

1) Rigid Pavement

The rigid pavement structure consisting of a prepared roadbed underlying layer of granular sub-base and plain concrete slab is assumed in calculation the required thickness.

The traffic load is estimated based on the result of traffic study. Such design input as environmental impact and effective modulus of sub-grade reaction are estimated from the results of soil survey and by referring to available data and information.

2) Flexible Pavement

The flexible pavement structure consisting of a prepared underlying layer of sub-base and base course and 5 cm asphalt binder course, 5 cm asphalt surface course is assumed in calculation the required thickness. The traffic load estimation is same as rigid pavement. The pavement structural number (SN) requirements are determined from design charts for flexible pavement shown in the AASHTO design guide.

(3) Pavement Type for Service Road (Baybay Rd.) in Section 4

Extend of Baybay road which is say Service Road from Sta. 2+400 to connect Manlurip Road in Section 4. Existing road is 5m width concrete pavement and traffic volume is very small (AADT=843). Therefore 21 cm minimum thickness of concrete pavement and 20 cm of aggregate sub-base course is proposed

(4) Pavement Type for Road Widening of Manlurip Road in Section 4

Existing concrete pavement road (Manlurip Rd.) of 760m in length at Sta. 3+500～Sta. 4+260 in section 4 will be affected new dike construction. Therefore existing road shall be sifted 4m toward to land side and pavement type and thickness for widening is proposed 25 cm minimum thickness of concrete pavement and 20 cm of aggregate sub-base course

(5) Reconstruction of San Jose Airport Road at Crossing of Tidal Protection Dike

Existing San Jose Airport Road will be crossing Tidal Protection Dike alignment at starting point of section 4. Therefore existing road height shall be adjusted to Tidal Protection Dike elevation.

Approach length of existing road is about 75m both side.

Existing Pavement type is asphalt pavement and 5cm thickness of asphalt surface course, 5cm thickness of asphalt binder course, 15cm of asphalt treated base course, 20cm aggregate base course and 30cm aggregate sub-base course is proposed.

4.3 River Crossing

4.3.1 Selection for the River Crossing Structure (River Gate / Backwater Dike)

(1) Purpose of comparing

Opening of river should be kept to maintain the discharge capacity while it is necessary to prevent storm surge from coming in from there. In general, there are two methods for treating the opening to cope with this case.

1. Close an opening with floodgate to prevent storm surge from coming into the river.
2. Constructing river dike (back water dike) to prevent storm surge from flowing into the land, although it will allow storm surge to come into the river.

Comparison was made for each river for the two methods above in terms of coherence with existing plan, economical efficiency and constraints of construction.

Notes for consideration are as follows.

- Floodgates and backwater dikes shall secure a height for the storm-surge prevention of the 50 yrs. return period that has been a target in this project.
- Floodgates and backwater dikes shall not obstruct the safety downward flow of flooding.

When opting for floodgates and backwater dikes, the dimension of the gate shall be designed appropriately so that the cofferdam effect by the construction of tide embankment doesn't not aggravate inundation inland.

(2) Target river

The following four rivers (creak) within the Section 3 and Section 4 were selected for comparison. These target rivers are those with a certain river width near the river mouth, where tide embankment will cross, and have a relatively large basin.

- Tanghas lirang creak (Aslum Creek)
- Burayan R.
- Payapay Creak
- Bangon R.

Table 4.3-1 Selection of River Crossing structure(1)

Name of river		Tanghas lirang creek (Aslum Creek)	Burayan R.
Section		Section 3	
Planned height of tide embarkment		4.0m＋M.S.L	
Specification of cross section（as-built）		 ・Width：6m ・Extension：8,400m	 ・Width：10m ・Extension：8,600m
Candidates of cofferdam construction method		Floodgate plan / Back water dike plan ◆ Floodgate plan ・Width：3.0m×3=9.0m ・Height：2.6m ◆Back water dike plan With the road raising point as the starting point, back water dikes are established on the upstream side up to the point satisfying 4.0m＋M.S.L of ground level.	Floodgate plan / Back water dike plan ◆Floodgate plan ・Width：3.5m×3=10.5m ・Height：2.6m ◆Back water dike plan With the road raising point as the starting point, back water dikes are established on the upstream side up to the point satisfying 4.0m＋M.S.L of ground level.
Cost performances	Floodgate	P61M	P71M
	Back water dikes	P495M	P596M
Workability	Floodgate	Downstream side of the bridge	Downstream side of the bridge
	Backwater dike	・The relocation of houses will be necessary for the raising newly implemented since the river runs through the conurbation district. ・It will be obstructions of drainage in watershed area. ・There is a case of which rebuilding of bridge might be necessary.	

Source: JICA Study Team

Table 4.3-2 Selection of River Crossing structure(2)

<table>
<tr><td colspan="2">Name of river</td><td>Payapay R. (Kilot creek)</td><td>Bangon R.</td></tr>
<tr><td colspan="2">Section</td><td colspan="2">Section 4</td></tr>
<tr><td colspan="2">Planned height of tide embankment</td><td colspan="2">3.5m＋M.S.L</td></tr>
<tr><td colspan="2">Specification of cross section (as-built)</td><td></td><td>
・Width：50m
・Extension：2200m</td></tr>
<tr><td colspan="2">Candidates of cofferdam construction method</td><td>This river is originally a branch of the Bangon river delta part.
The back water dike method is not a rational way of construction since houses and properties to be protected are almost not existed in watershed area.
There is an anxiety of occurrences of flood damage to other watershed areas, on the other hand, once high tide flows from an opening part.
Thus the floodgate (flap gate) is a rational way for the cofferdam construction method due to which the cofferdam is necessary.

◆Floodgate plan
Width：3.5m×1
As to open width, agreed with Ms. Palo Mayer that the size shall be approximately as large as a small boat can be passed through.
Height：2.6m</td><td>While the floodgate plan and back water dike plan can be considered, the back water dike plan is to be determined since the function as the back water dike can be secured with the utilization of embankments DPWH is planning and constructing as the flood measure.

◆Back water levee plan
In the stretch from the 1st bridge through the 3rd bridge of the Bangon river, there is a plan of the waterway replacement and river banks and some have already been started.
Since the height of all the stretch is 3.5m＋M.S.L and over, the planned height of tide embankment has been satisfied so the function securing as the back water dike is capable.
For the bank surfaces already constructed, the concrete coated revetment has been completed already.</td></tr>
<tr><td>Cost performances</td><td>Floodgate</td><td>・P24M</td><td>Currently the construction of back water dike has been in progress.</td></tr>
</table>

Source: JICA Study Team

4.3.2 Exiting Flood Condition

Current condition and characteristics regarding flood in the Project Area were briefly reviewed and an analyzed based on collected data and reports listed below.

Table 4.3-3 Collected Data and Reports

	Item	Objectives	Reference / Collected from
1	Observed Rainfall	Rainfall and Runoff Analyses	Philippine Atmospheric Geophysical Astronomical Service Administration (PAGASA)
2	Probable Discharge, Rainfall Intensity	Rainfall Analysis	Inventory Survey and Basic Analysis of Hydrological Data for Department of Public Works and Highways Technical Standards and Guidelines (WOODFIELDS CONSULTANTS INC., March, 2002)
4	Land cover	Runoff Analysis	National Mapping and Resources Information Authority (NAMRIA)
5	Soil Map	Runoff Analysis	Bureau of Soil and Water Management (BSWM)
6	DEM	Runoff Analysis	Generated from the result of LiDAR survey conducted by JICA Study Team
7	Report	Review of Flood Control Plan in Tacloban City	Study on the Flood Control for Rivers in the Selected Urban Centers, Final Report (JICA, 1995)
8	Drawings	Review of Flood Control Plan in Tacloban City	Drainage Plan in Tacloban City
9		Review of Flood Control Plan for Bangon River	Flood Control Plan for Bangon River

Source: JICA Study Team

(1) Review of Flood Control Plan

1) Tacloban City

In Tacloban City, drainage Master Plan Study (M/P) which targeted to hamper ten (10) years return period flood was carried out by JICA in 1995, which was aimed to formulate master plans on flood control for rivers in the four (4) cities considered as priority areas, such as Iloilo City, Cebu, Ormoc and Tacloban City. Finally two (2) areas, Iloilo and Ormoc Cities were selected as prioritized target area for further design study. According to city engineer of Tacloban, the M/P was used only as an example model for drainage improvement. Drainage improvement in Tacloban City has not been conducted in accordance with the M/P so far.

Besides, the drainage plan has been prepared by Tacloban City during 2011-2012 though technical report has not been prepared. Dimensions of drainage channel proposed in the plan were determined based on the observed flood depth and velocity during flood event. The target sites for improvement plan such as channeling and riprap construction are locally focused on upstream and improvement doesn’t seem to be implemented for downstream.

Also in the inventory study carried out by WoodField Consultant Inc. (hereinafter called “WCI”) in 2002, probable daily rainfall and rainfall duration curve for return periods were estimated and generated, respectively. Though probable rainfall for WCI study for twenty (20) to a hundred

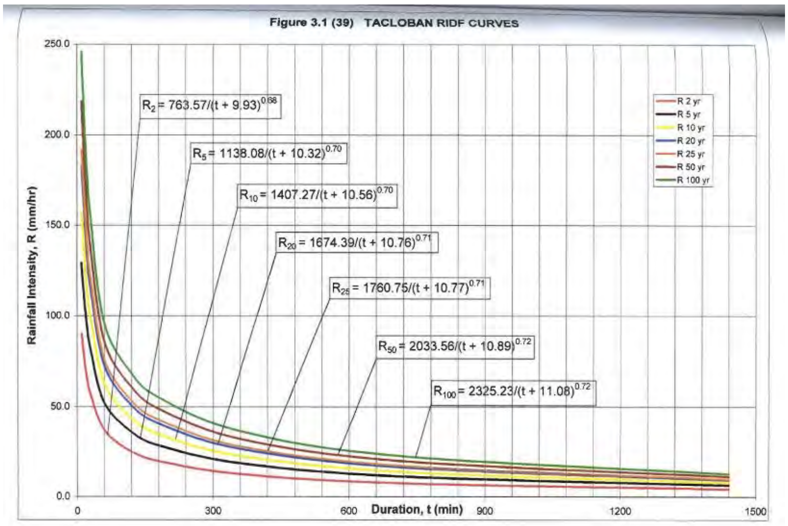
(100) years are larger than that of M/P carried out by JICA, there is no difference between both studies for short return period such as two (2) to ten (10) years (as shown in Table 4.3-4).

Table 4.3-4 Estimated Probable Daily Rainfall in JICA Drainage MP in 1995 and Rainfall Analysis by WCI in 2002

Unit: mm

	Case	Probable function	1/2	1/5	1/10	1/20	1/50	1/100
1	1995 JICA MP (1961-1991, N=31)	Iwai	119.9	146.1	161.2	174.5	190.2	201.3
2	2002 WCI Study (1961-1991, N=41)	Log Normal	119	145	161	216	247	206

Source: JICA Study Team



Source: JICA Study Team

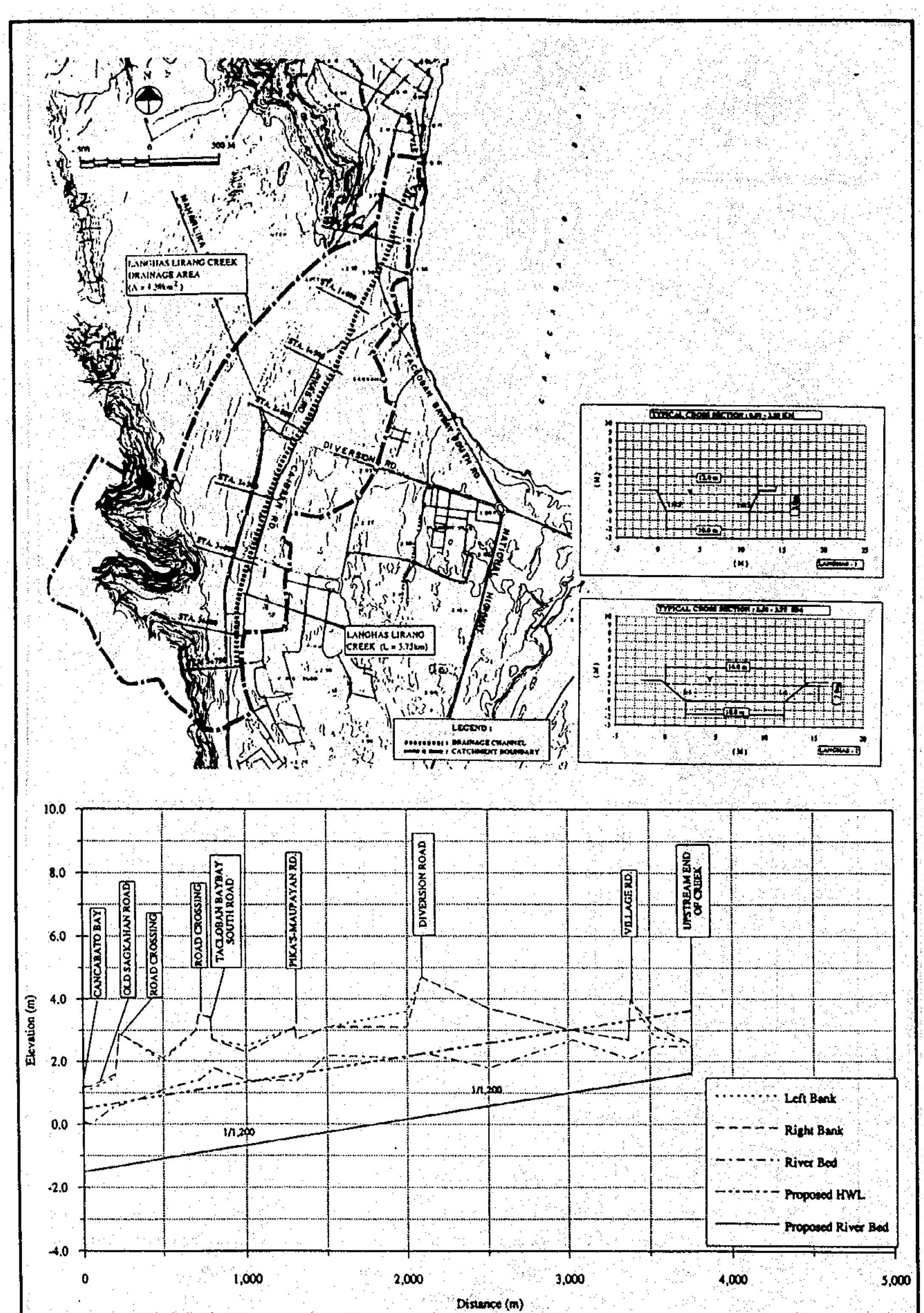
Figure 4.3-1 Rainfall Duration Curve of Tacloban GS

Table* Comparison of Rainfall Intensities for 2-10 years of return periods between 1995JICA MP and 2002WCI Study

mm/hr

Case	Applied Formula	Return Period (year)	Equation	Duration Time (minute)								
				10	20	30	60	120	180	360	720	1440
1995 JICA MP	Talbot	2	R_2=4839/(t+34)	110	90	76	51	31	23	12	6	3
		5	R_5=6762/(t+44)	125	106	91	65	41	30	17	9	5
		10	R_{10}=8048/(t+49)	136	117	102	74	48	35	20	10	5
2002 WCI	Unknown *R=a/(t+b)^c*	2	R_2=763.57/(t+9.93)^$^{0.68}$	100	76	62	43	28	22	14	9	5
		5	R_5=1138.08/(t+10.32)^$^{0.70}$	138	104	86	58	38	29	18	11	7
		10	R_{10}=1407.27/(t+10.56)^$^{0.70}$	170	128	105	72	46	36	22	14	9

Source: JICA Study Team

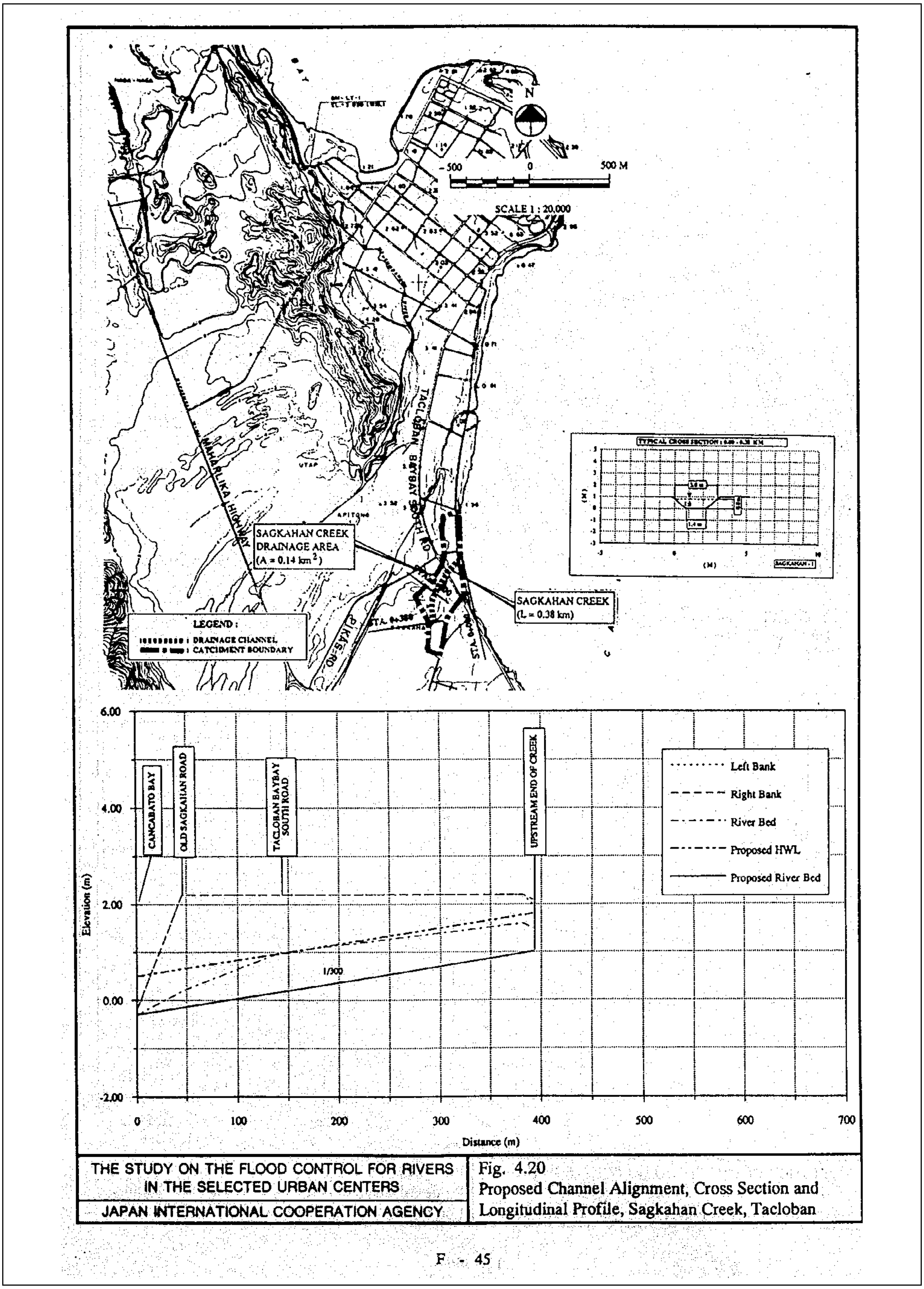


THE STUDY ON THE FLOOD CONTROL FOR RIVERS IN THE SELECTED URBAN CENTERS	Fig. 4.19
JAPAN INTERNATIONAL COOPERATION AGENCY	Proposed Channel Alignment, Cross Section and Longitudinal Profile, Langhas-Lirang Creek, Tacloban

F - 44

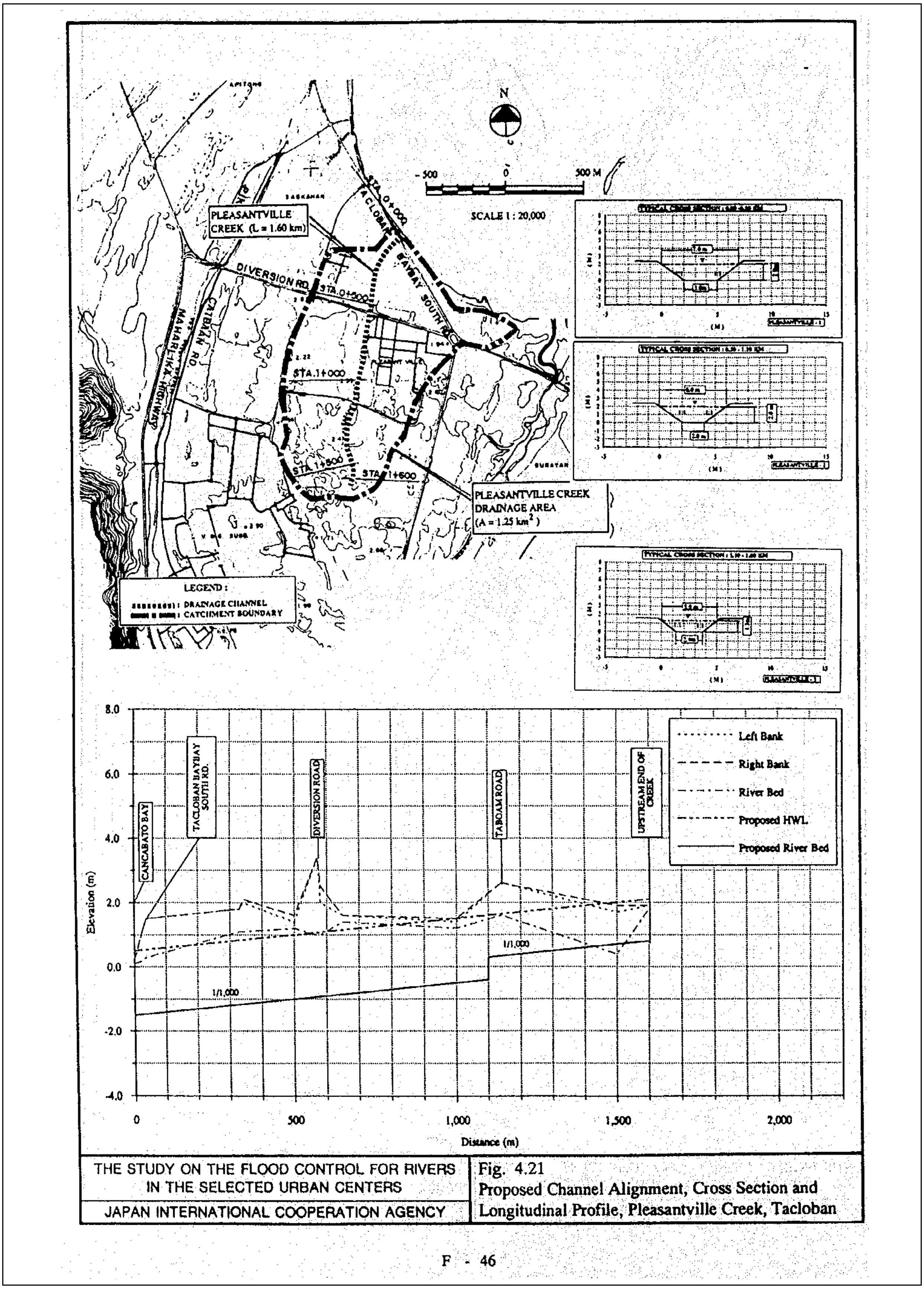
Source: Study on the Flood Control for Rivers in the Selected Urban Centers, Final Report (JICA, 1995)

Figure 4.3-2 Drainage Master Plan in Tacloban City by JICA(1/4)



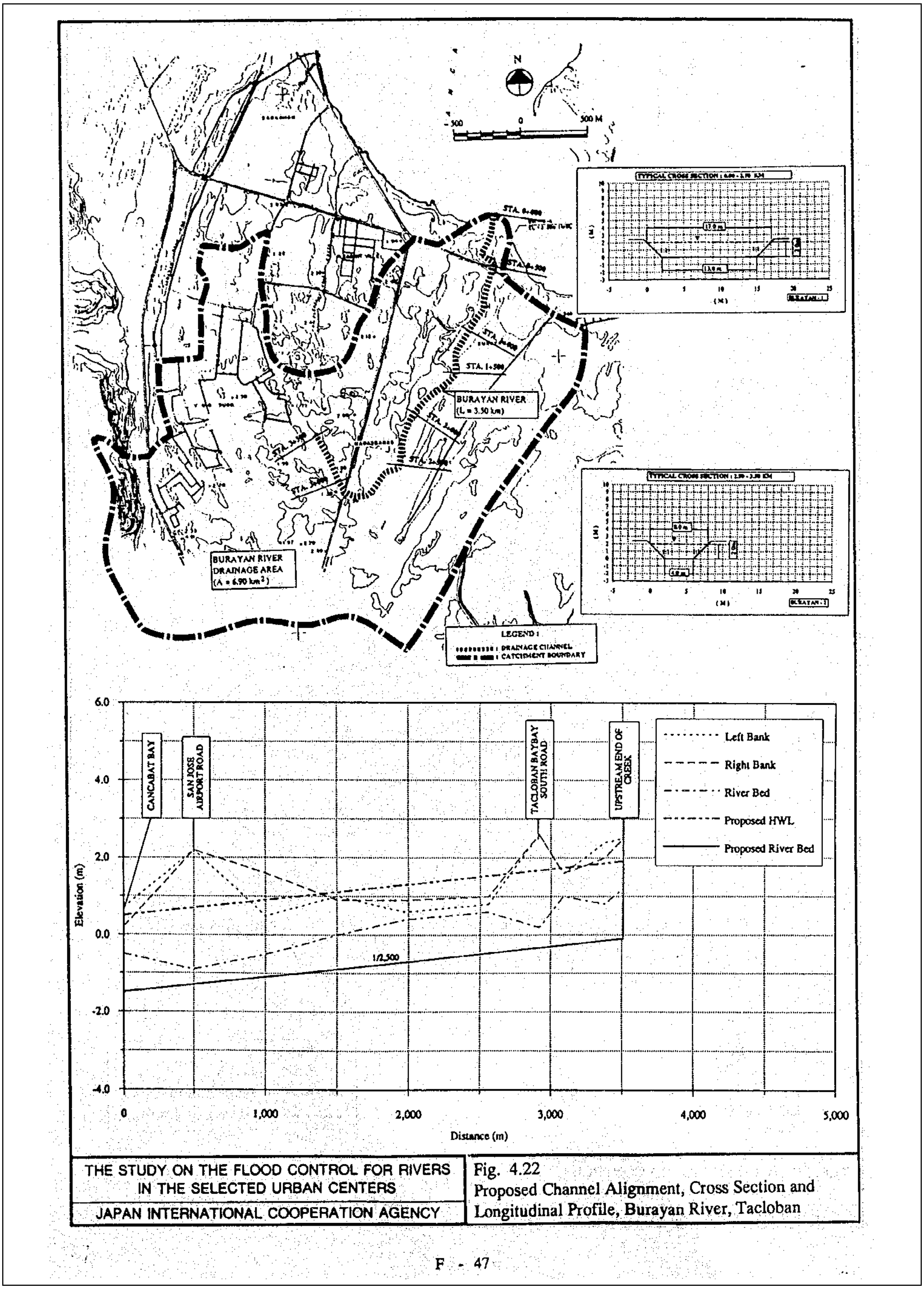
Source: Study on the Flood Control for Rivers in the Selected Urban Centers, Final Report (JICA, 1995)

Figure 4.3-3 Drainage Master Plan in Tacloban City by JICA (2/4)



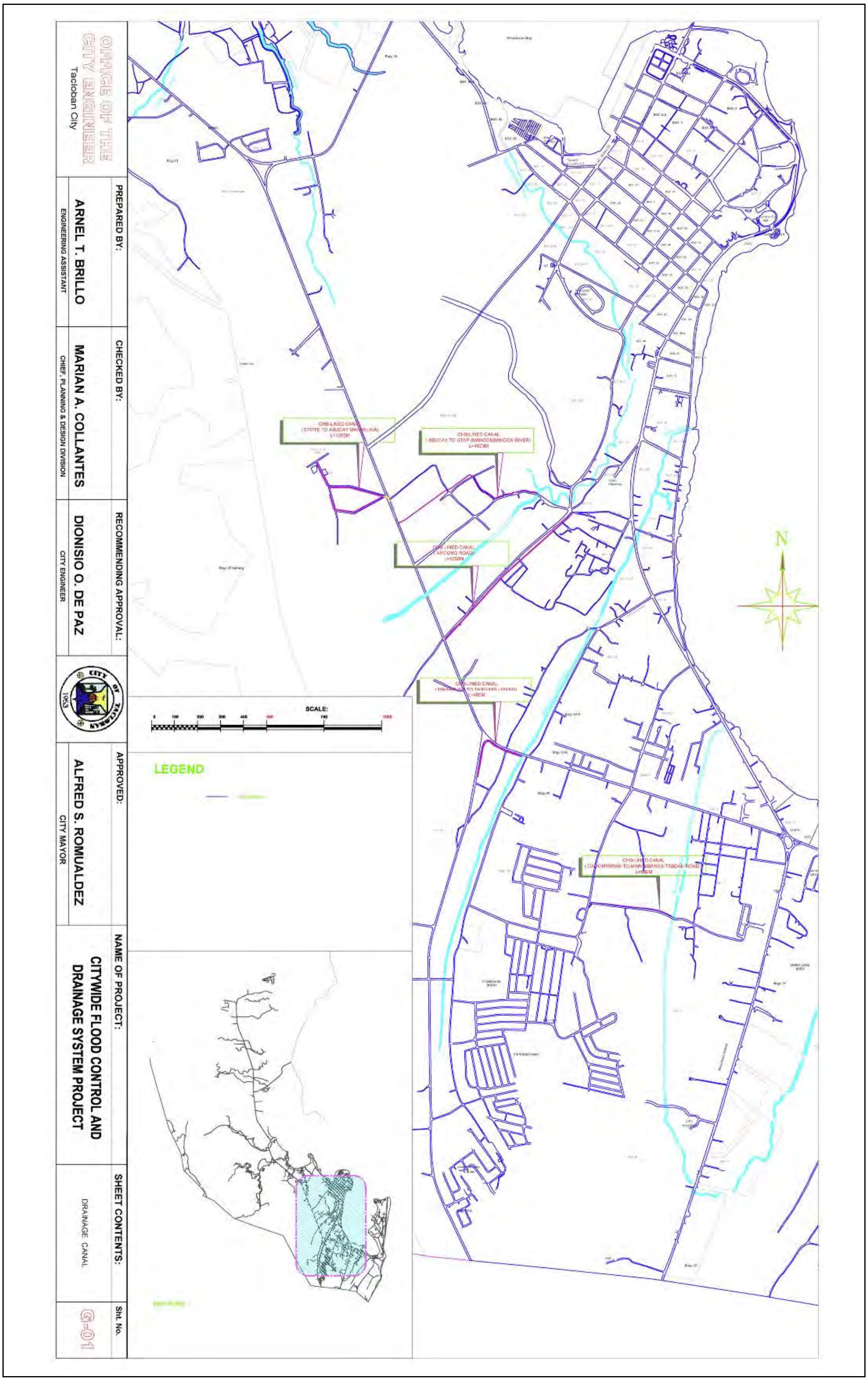
Source: Study on the Flood Control for Rivers in the Selected Urban Centers, Final Report (JICA, 1995)

Figure 4.3-4 Drainage Master Plan in Tacloban City by JICA (3/4)



Source: Study on the Flood Control for Rivers in the Selected Urban Centers, Final Report (JICA, 1995)

Figure 4.3-5 Drainage Master Plan in Tacloban City by JICA (4/4)



Source: Tacloban City

Figure 4.3-6 Horizontal Plan on Flood Control in Tacloban City (Proposed by Tacloban City)

2) **Bangon River**

The conclusive study of flood control for Bangon River has not been carried out so far but flood control countermeasure has been partly implemented under the concept of river improvement "site protection". Recently flood protection dike is under construction by DPWH for downstream between Bernard Reed Bridge and Bernard Reed 2 Bridge which is a combination of dike construction and shortcut by rechanneling, which targets to prevent overflow by flood on the scale of the flood at March 17 of 2011.

Source: JICA Study Team

Figure 4.3-7 Bangon River (Partly Constructed flood protection dike)

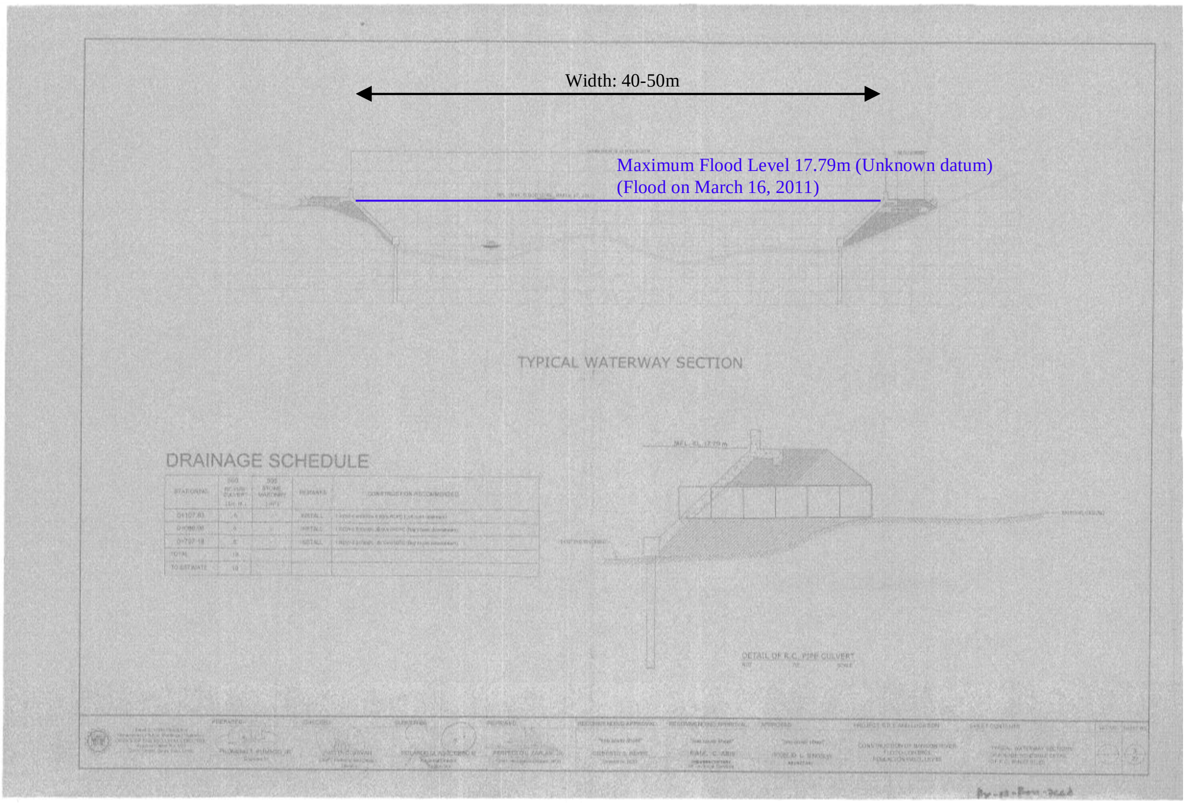


Figure 4.3-8 River Cross Section Designed by DPWH

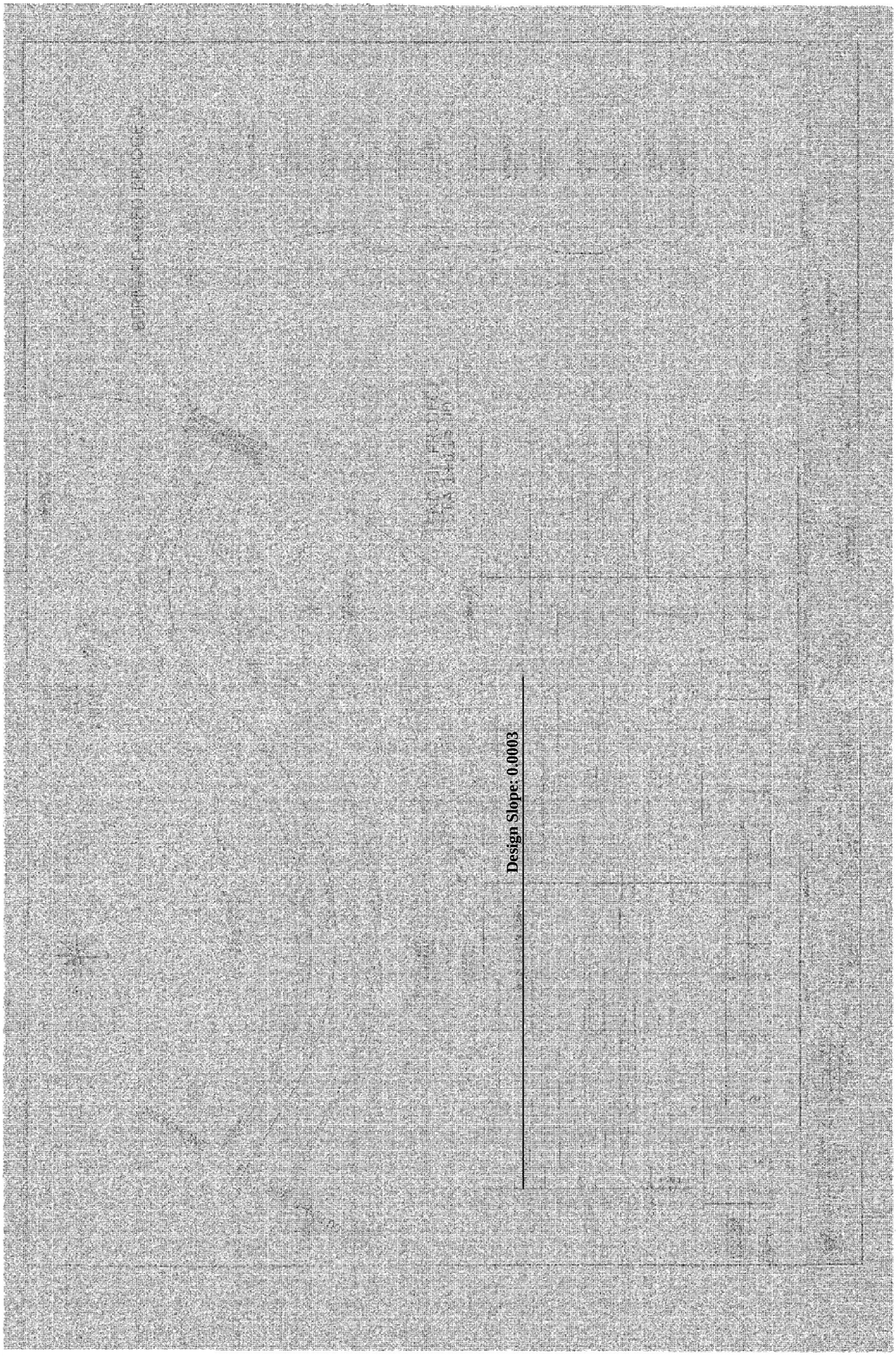


Figure 4.3-9 Horizontal and Longitudinal Plans Designed by DPWH

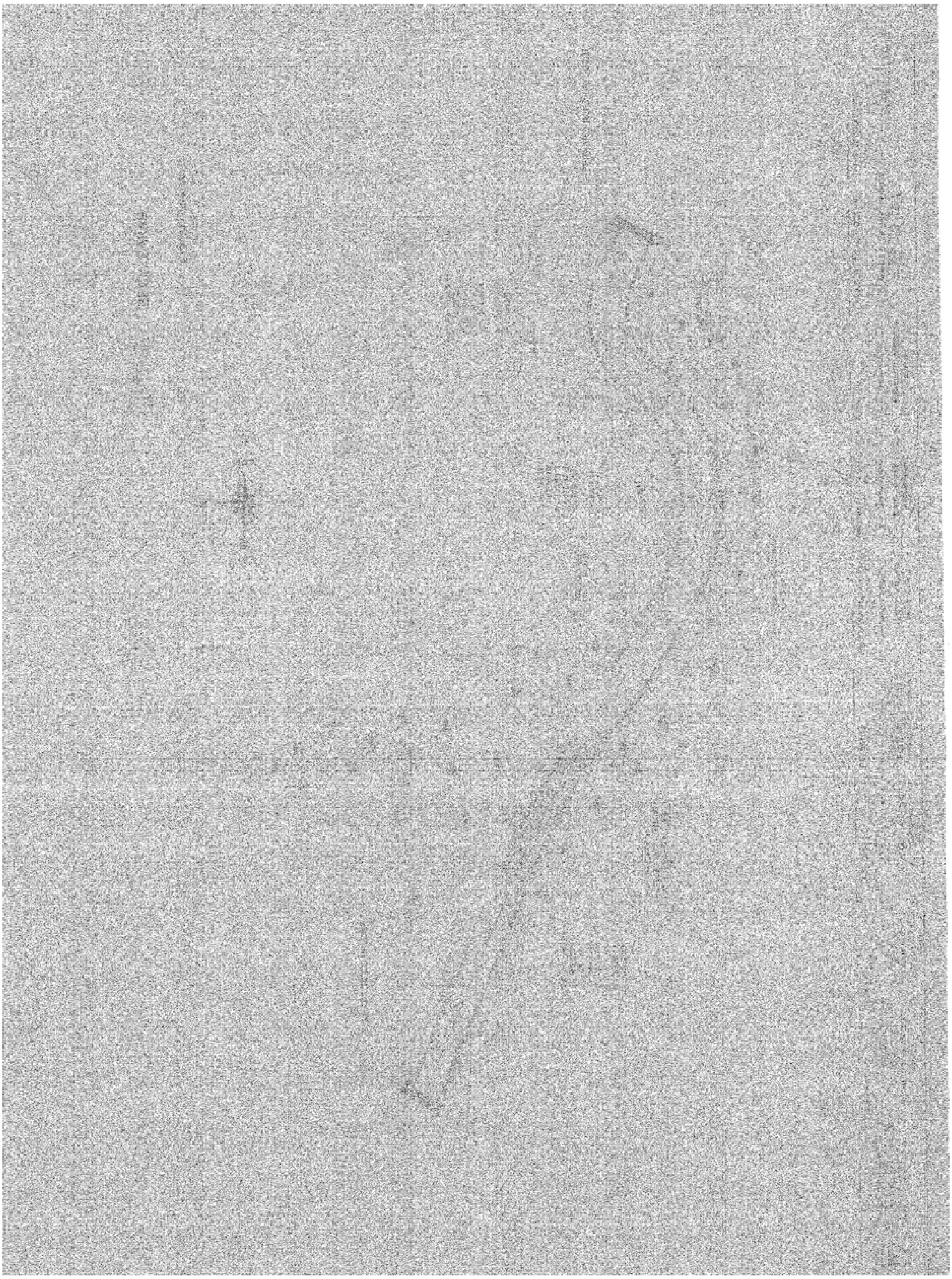
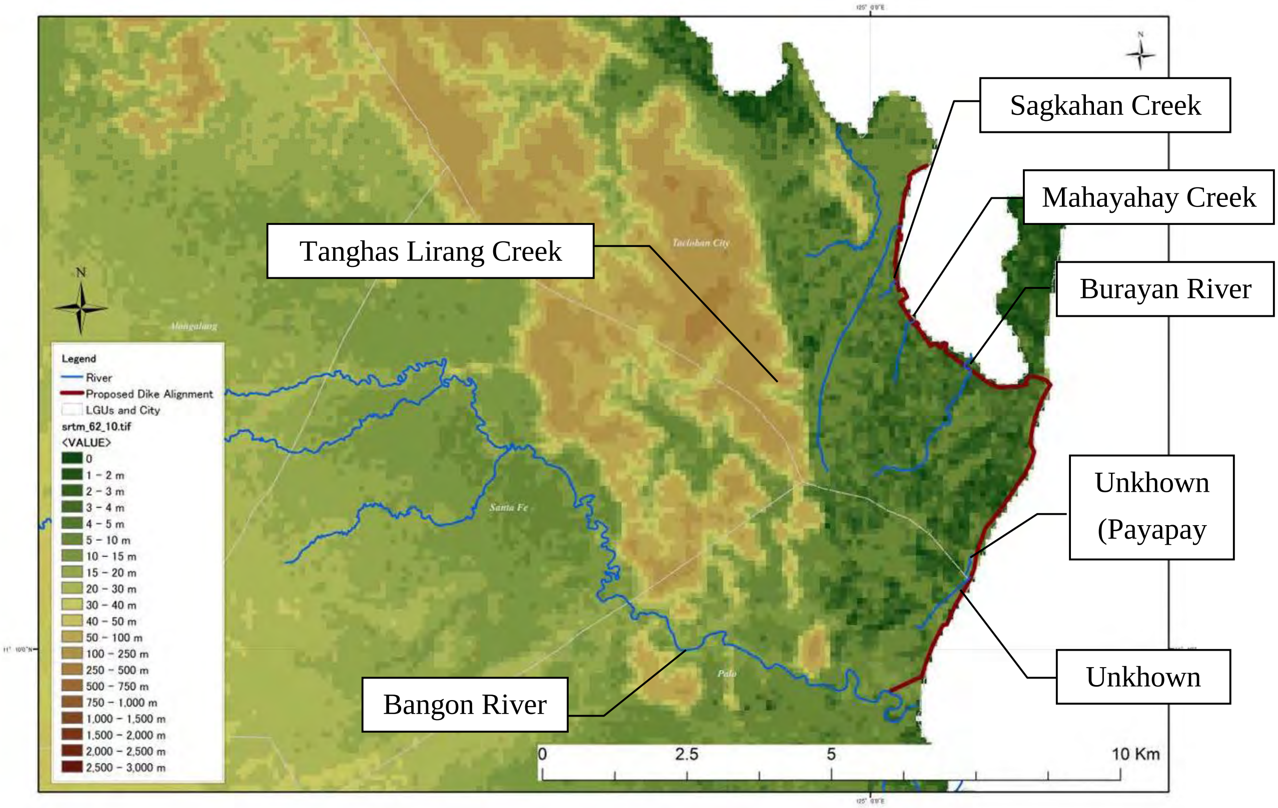


Figure 4.3-10 Horizontal Plan of Rechanneling Designed by DPWH

(2) Topographic Condition

1) Horizontal Condition

In this study rivers and creeks on the tidal structure proposed by DPWH are identified as shown in Figure 4.3-11 and information of their outlets are summarized in Table 4.3-5)



Source: JICA Study Team

Figure 4.3-11 Rivers and Creeks in the Project Area

Table 4.3-5 Summary on Crossing Structure on the Tidal Structure proposed by DPWH

No.	St. No.	River/Creek name	Location of outlet		Structure type of existing outlet	Dimension of existing outlet	Overburden of existing outlet
			Lon (Degree)	Lat (Degree)			
1	Sec3-1K-200	Tanghas-Lirang Creek	125.00428	11.23257	Bridge	H= 1.1 m W= 4 m	380mm
2	Sec3-2K-516	Sagkahan Creek	125.00470	11.22085	Box Culvert	H= 1.1 m W= 2.6 m	300mm
3	Sec3-2K-930	Mahayahay Creek	125.00653	11.21756	Box Culvert	H= 1 m W= 1 m x 2	300mm
4	Sec3-4K-268	Burayan River	125.01544	11.20939	Bridge	H= 3 m W= 10 m	1,500mm
5	Sec3-4K-800	Unknown (Payapay Brd.)	125.01589	11.18140	Bridge	H= 4.2 m W=26.5 m	Unknown
6	Sec4-4K-250	Unknown	125.01536	11.17799	Box Culvert	H= 2 m W= 4 m	500mm
7	Sec4-*K-***	Bangon River	125.00317	11.15982	Bridge (Bernard Reed Brd)	H= Unknown W= Unknown	-

Source: JICA Study Team

 Tanghas Lirang Creek (Facing upstream)	 Tanghas Lirang Creek (Under girder)
 Sagkahan Creek (Facing upstream)	 Sagkahan Creek (Box culvert)
 Mahayahay Creek (Nallow channel and pipe culvert)	 Mahayahay Creek (Box Culvert)

Figure 4.3-12 Pictures of rivers and creeks (1)


Bureyan River (Facing downstream)


Bureyan River (Under girder)


Unknown (Payapay Brd)


Unknown (Box culvert)


Bangon River (Facing river mouth)

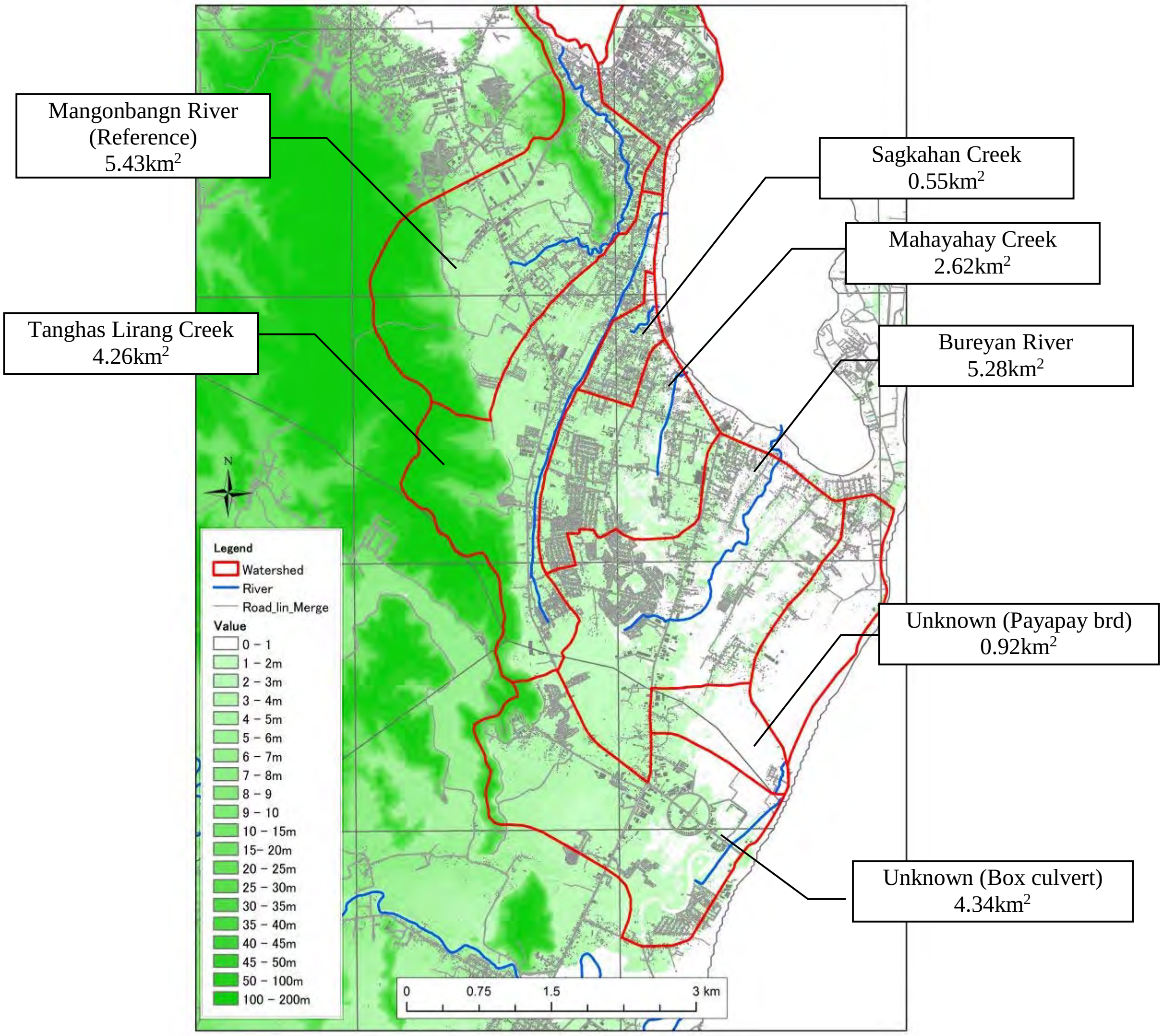

Bangon River (Partly Constructed flood protection dike)

Source: JICA Study Team

Figure 4.3-13 Pictures of rivers and creeks (2)

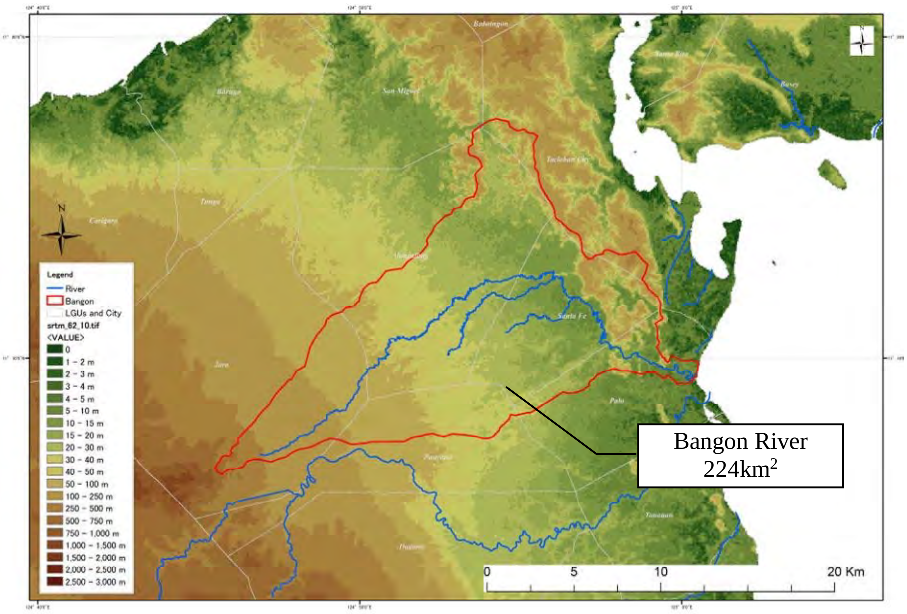
2) **Catchment Area**

Based on collected information (e.g. JICA M/P in 1995, topographic condition, existing drainage coverage, proposed drainage plan, existing heightened road), catchment area for each river/creek and dominant drainage channel in the Project Area was delineated as shown in Figure 4.3-14.



Source: JICA Study Team

Figure 4.3-14 Catchment Area (1/2)



Source: JICA Study Team

Figure 4.3-14 Catchment Area (2/2)

3) **Rivers and Creeks**

Based on catchment area of rivers and creeks prepared by JICA STUDY TEAM, the two (2) rivers and one (1) creek were selected for river study, whose topographic information are summarized as below.

Table 4.3-6 Summary on Target Rivers and Creek for Runoff Analysis

No.	St. No.	River/Creek name	Catchment Area (km^2)	Channel Length (km)	Topographic Slope	Flood concentration time** (min)
1	Sec3-1K-200	Tanghas-Lirang Creek	4.26	4.6	1,120	54
2	Sec3-2K-516	Sagkahan Creek*	0.55	0.47	340	15
3	Sec3-2K-930	Mahayahay Creek*	2.62	1.17	460	36
4	Sec3-4K-268	Burayan River	5.27	3.3	1,250	54
5	Sec3-4K-800	Unknown (Payapay Brd.)	0.2	0.5	-	-
6	Sec4-4K-250	Unknown	4.34	0.8	320	36
7	-	Bangon River	224	38	50 - 800	117

*Source: JICA STUDY TEAM
** Kraven's formula was employed.

(3) Flood Characteristics

1) **Tacloban City**

Flood and inundation issues caused by insufficiencies of drainage channel distribution and lack of drainage capacity are one of the most concerned by DPWH and Tacloban City. Flood/inundation prone area are distributed in Tacloban City, such as where swamp area has originally been located (e.g. Barangay 78-80) and adjacent area to existing/proposed by-path way along mountainous area located Westside of Tacloban City (e.g. Barangay Apitong, area along Pan-Philippine Highway). City Engineers of Tacloban mentioned that some structures for drainage improvement are already constructed and been proposed, such as the channels connecting between upstream of Tanghas-Lirang Creek and Burayan River, and diversion channels from downstream of Mangonbangon River into Tanghas-Lirang Creek.

2) **Bangon River**

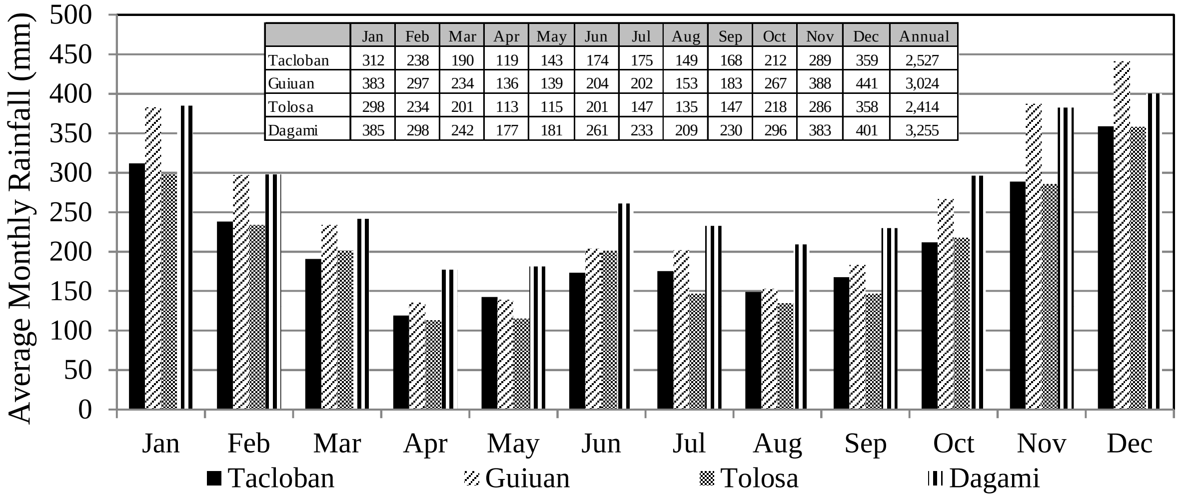
Bangon River is characterized as three types of river characteristics based on topological conditions; a) alluvial area in the upper to middle reaches with steep slope, b) plain and irrigated area in the middle to lower reaches with moderate slope and c) intermixed area with lowland, swampy and residential in the lower reaches, where the river channel is meandering.

Judging from topographic condition and flood condition survey conducted by JICA STUDY TEAM, in the Project area, flood frequently occurs that was not as severe as people’s lives lost. Flood occurred at which the river slope changes between the upper to middle reaches, and at lower reaches with low discharge capacity of river channel because of meandering. As for the lower reaches rainfall inundation chronically occurs at lowland besides flood.

(4) Rainfall Analysis

1) Rainfall Characteristics in the Project Area

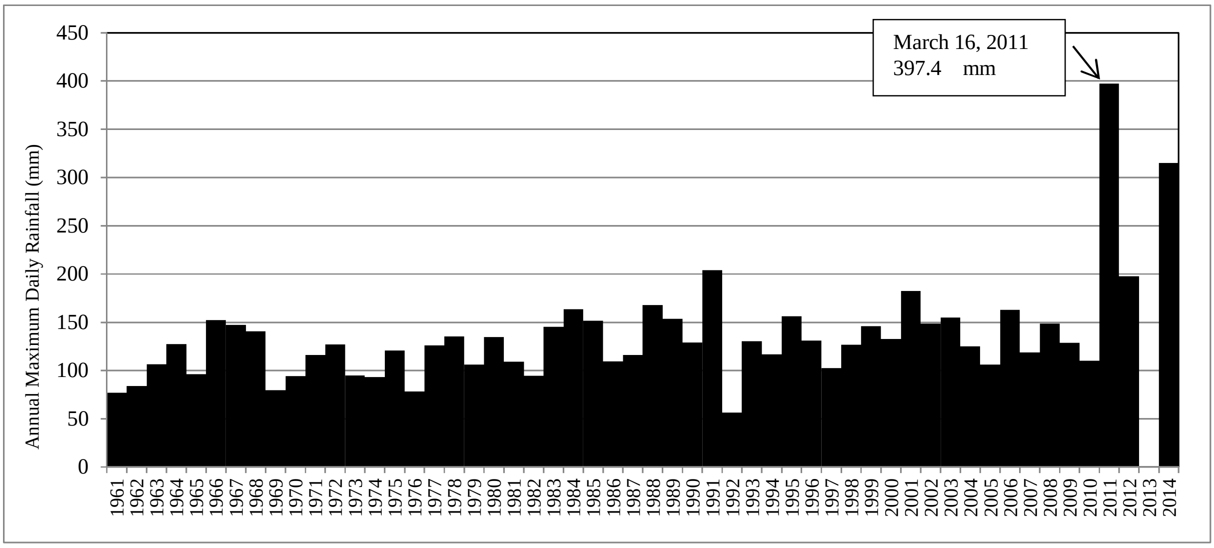
The Project area, Eastern Leyte is classified into "Type4" in Philippines Climate Type, with unclear difference between dry and rainy seasons. Rainfall occurs through a year (as shown in Figure 4.3-15), and increases between November to February. The biggest rainfall event in Tacloban is a continuous rainfall recorded on March 16, 2011, with daily rainfall of 397mm (as shown in Figure 4.3-16). Its return period is beyond 400 years. According to LGUs Staff of Palo and Tanauan, it was the most serious flood occurred.



	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	Annual
Tacloban	312	238	190	119	143	174	175	149	168	212	289	359	2,527
Guiuan	383	297	234	136	139	204	202	153	183	267	388	441	3,024
Tolosa	298	234	201	113	115	201	147	135	147	218	286	358	2,414
Dagami	385	298	242	177	181	261	233	209	230	296	383	401	3,255

Source of data: PAGASA, organized by JICA STUDY TEAM

Figure 4.3-15 Average Monthly Rainfall



Source of data: PAGASA, organized by JICA STUDY TEAM

Figure 4.3-16 Annual Maximum Daily Rainfall in Tacloban

Table 4.3-7 Annual Maximum Daily Rainfall in Tacloban

Year	1day	Date	2day	Date	3day	Date	Year	1day	Date	2day	Date	3day	Date
1961	77.2	1961/10/17	96.8	1961/7/24	124.7	1961/7/25	1988	167.9	1988/10/23	258.8	1988/12/17	309	1988/12/18
1962	84.1	1962/1/11	108	1962/9/10	114.9	1962/9/10	1989	153.7	1989/2/14	172.5	1989/2/15	175.8	1989/2/15
1963	106.7	1963/8/12	150.4	1963/8/12	153.2	1963/8/13	1990	129.1	1990/1/8	196.7	1990/1/8	206.4	1990/1/9
1964	127.5	1964/11/18	252.2	1964/11/19	258	1964/11/19	1991	204	1991/3/12	276.7	1991/3/13	317.6	1991/3/14
1965	96.1	1965/12/15	150.5	1965/12/16	168.6	1965/12/16	1992	56.4	1992/12/17	79.4	1992/7/19	100.8	1992/7/19
1966	152.2	1966/5/15	157.5	1966/5/16	164.7	1966/5/15	1993	130.2	1993/11/20	186.6	1993/11/20	218	1993/12/28
1967	147.1	1967/1/13	237.2	1967/1/19	343.9	1967/1/19	1994	116.8	1994/12/21	149.1	1994/12/6	179.8	1994/12/6
1968	140.7	1968/11/23	145.3	1968/11/24	168.9	1968/11/25	1995	156.4	1995/9/29	191.2	1995/12/27	202.4	1995/12/27
1969	79.7	1969/7/16	97.1	1969/12/22	99.1	1969/12/23	1996	131	1996/3/2	162.2	1996/3/3	257.8	1996/3/2
1970	94.3	1970/10/12	156	1970/2/21	175.8	1970/2/22	1997	102.7	1997/1/27	159.3	1997/1/28	159.6	1997/1/28
1971	116	1971/6/24	152.4	1971/6/25	171.4	1971/6/26	1998	126.8	1998/12/6	173.8	1998/12/6	203.6	1998/12/7
1972	127	1972/1/18	218.4	1972/1/18	273.9	1972/1/18	1999	146.1	1999/2/9	188.8	1999/11/7	242.7	1999/2/9
1973	94.8	1973/12/26	104.4	1973/12/26	124.3	1973/12/26	2000	132.6	2000/11/13	169	2000/11/30	205.3	2000/2/2
1974	93.3	1974/2/13	131.9	1974/12/14	154	1974/5/24	2001	182.6	2001/1/15	244.5	2001/11/6	415.8	2001/1/17
1975	120.9	1975/12/13	148.1	1975/12/13	184.2	1975/12/13	2002	148.4	2002/1/2	259	2002/1/1	267.2	2002/1/3
1976	78.2	1976/1/23	119.1	1976/1/23	153.8	1976/1/24	2003	155	2003/10/1	189.2	2003/9/30	212.8	2003/6/14
1977	126.2	1977/2/16	190	1977/2/17	209.6	1977/2/17	2004	125.1	2004/1/23	143.5	2004/1/23	239.6	2004/1/25
1978	135.4	1978/4/20	251.9	1978/4/20	291.9	1978/4/21	2005	106.2	2005/12/25	136.1	2005/12/15	166.6	2005/12/12
1979	106.2	1979/6/17	121.4	1979/5/12	130.8	1979/5/13	2006	163	2006/5/11	252.3	2006/2/11	352	2006/2/12
1980	134.9	1980/11/11	155.5	1980/1/16	190.6	1980/1/16	2007	118.7	2007/11/19	206.3	2007/11/18	207.7	2007/11/20
1981	109.3	1981/9/24	131.9	1981/12/3	202	1981/12/4	2008	148.6	2008/6/20	228.4	2008/2/16	294.6	2008/2/18
1982	94.7	1982/3/25	165.3	1982/3/26	188.7	1982/3/26	2009	128.6	2009/2/6	151.2	2009/6/22	175.8	2009/12/16
1983	145.3	1983/7/13	233	1983/12/25	299.9	1983/12/26	2010	110.2	2010/1/16	203.2	2010/1/15	223.4	2010/1/17
1984	163.6	1984/12/30	243.9	1984/12/31	243.9	1984/12/31	2011	397.4	2011/3/16	437.1	2011/3/16	561.1	2011/3/18
1985	151.5	1985/1/16	208.6	1985/1/16	226.9	1985/1/17	2012	197.7	2012/12/25	212.9	2012/12/25	223.1	2012/12/26
1986	109.7	1986/1/25	153.4	1986/1/25	170.5	1986/4/7	2013	-	-	-	-	-	-
1987	116	1987/8/12	198	1987/8/12	198	1987/8/12	2014	314.8	2014/12/29	368.8	2014/12/29	379.5	2014/12/30

Data Source: Daily and six(6) hourly Rainfall observed by PAGASA, organized by JICA STUDY TEAM

2) **Probable Rainfall Estimation**

Probable daily rainfall of return periods were estimated using annual maximum daily rainfall in Tacloban Rainfall Station. Suitable probable function was selected based on the value of SLSC that is less than 0.004.

Table 4.3-8 Observation Period and Statistical Parameter

Station Name	Period	Number of Samples
Tacloban	1961-2014	51

Source: JICA Study Team

Table 4.3-9 Estimated Probable Rainfall

Unit: mm

	Case	Probable function	1/2	1/5	1/8	1/10	1/20	1/30	1/50	1/100	1/200	1/400
1	1995 JICA MP (1961-1991, N=31)	Iwai	119.9	146.1	-	161.2	174.5	-	190.2	201.3	-	-
2	2002 WCI Study (1961-1991, N=41)	Log Normal	119	145	-	161	216	-	247	206	-	-
3	2015 JICA Study (1961-2014, N=51*)	Gumbel	122	150	164	169	187	197	210	227	245	262

*Daily Rainfall in 2011 Mar (397.4mm) and 2014 Dec (314.8mm) were rejected based on outlier rejection check.

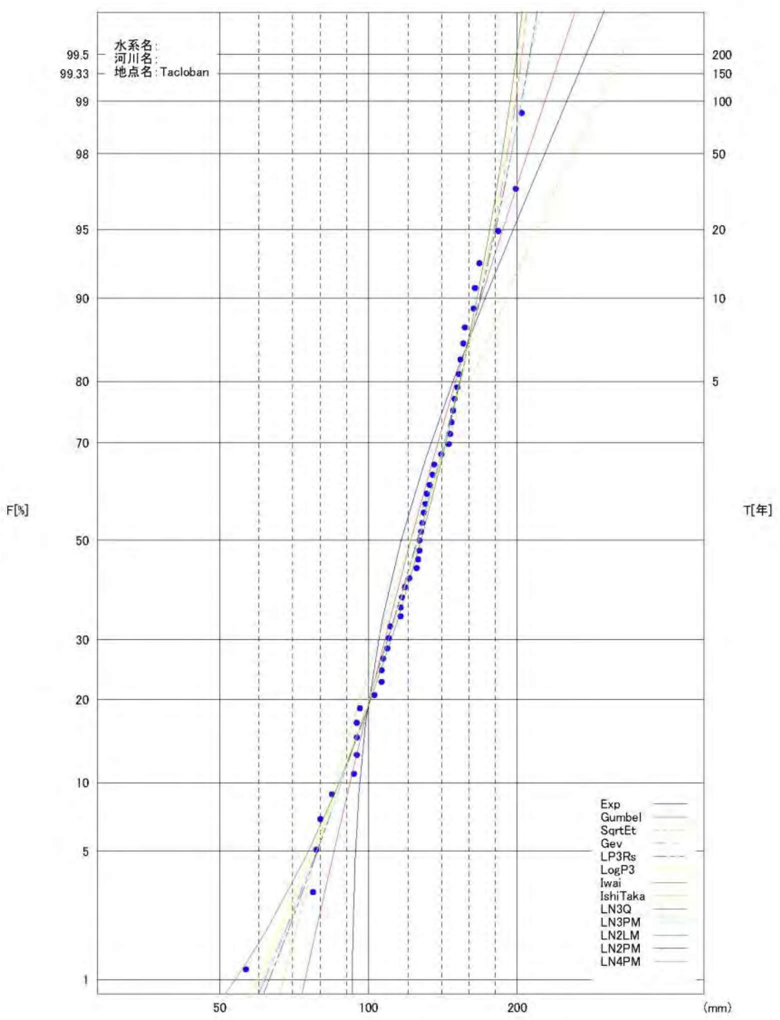
Source: JICA Study Team

Table 4.3-10 Result of Statistical Analysis (Tacloban, 1day rainfall)

Unit: mm

Item		Maximum						
		Gumbel	Gev	LP3Rs	LogP3	IshiTaka	LN3Q	LN3PM
Samples		51	51	51	51	51	51	51
Return Period	1/1.5	110	112	112	113	112	114	112
	1/2	122	126	125	126	126	128	126
	1/3	135	140	139	140	139	141	139
	1/5	150	153	153	153	152	153	152
	1/10	169	168	168	167	167	166	167
	1/20	187	179	181	178	180	176	180
	1/30	197	185	187	184	186	181	186
	1/50	210	191	195	190	194	186	194
	1/80	222	196	201	195	201	191	201
	1/100	227	198	204	197	204	193	204
	1/150	237	202	209	201	209	197	209
	1/200	245	204	212	203	213	199	213
	1/400	262	209	219	208	222	205	222
SLSC		0.038	0.033	0.024	0.023	0.021	0.022	0.021
Probable Function		●						

Source: JICA Study Team



Source: JICA Study Team

Figure 4.3-17 Statistical Distribution (Tacloban, 1day rainfall)

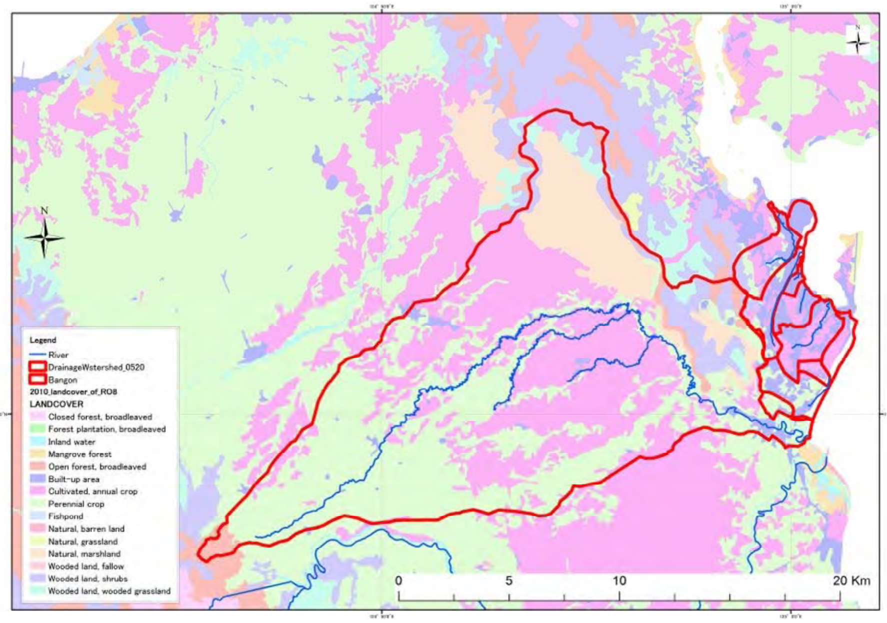
(5) Runoff Analysis

In order to determine hydraulic dimensions for outlets of rivers and creeks, probable discharge at their outlets based on runoff analyses which were conducted with three (3) methodologies described below. Analysis for Mangonbangon River was also conducted for reference because this is also one of major rivers in Tacloban City.

As for Bangon River, calculation result was basically applied for evaluation of backwater dike described in 4.3.4, which was obtained in the flood inundation analysis conducted in “The Urgent Development Study on the Project on Rehabilitation and Recovery from Typhoon Yolanda (JICA, March 2015)” (hereinafter called as “2015 JICA Study”).

1) **Existing Land Cover**

Existing land cover condition was analyzed using collected land cover map in 2010 provided by NAMRIA as shown in Figure 4.3-18. Area of land cover for each catchment is summarized in Table 4.3-11. For the rivers and creeks in Tacloban, urban area is dominantly distributed in the downstream. Cultivated area is distributed in their upstream, which is mostly identified as swamp area in site investigation.



Source: JICA Study Team prepared using NAMRIA data

Figure 4.3-18 Land Cover Map of NAMRIA

Table 4.3-11 Status of Land Cover for Target Rivers/Creeks

Unit: km^2

	River	Open forest, broadleaved	Other land, built-up area	Other land, cultivated, annual crop	Other land, cultivated, perennial crop	Other land, fishpond	Other wooded land, shrubs	Total
1	Tanghas-Lirang Creek	0.04	2.21	0.90	0.67	0.00	0.44	4.26
2	Sagkahan Creek	0.00	0.53	0.02	0.00	0.00	0.00	0.55
3	Mahayahay Creek	0.00	1.80	0.81	0.00	0.00	0.00	2.62
4	Burayan River	0.00	2.67	2.58	0.02	0.00	0.00	5.27
5	Unknown (Payapay Brd)	0.00	0.19	0.65	0.05	0.00	0.00	0.92
6	Unknown	0.00	2.29	0.16	1.87	0.03	0.00	4.34
7	Bangon River	5.42	3.44	81.41	93.64	0.06	7.19	224

Source: JICA Study Team

2) **Estimation of Probable Discharge**

a) Rational Formula

Probable flood discharge for the drainage area is computed using the Rational Formula. Maxmum flood discharge are given by the following formula,

$$Qp = 0.2778frA$$

Where;

Qp : maximum flood discharge (m^3/s)

f : runoff coefficient

R : rainfall intensity within the flood concentration time (mm/hr)

A : catchment area (km^2)

Runoff coefficient for each catchment area was calculated with weighted average method based on their areas of land cover types. Applied runoff coefficient for each land cover type is listed in Table 4.3-12. The calculation results were obtained as shown inTable 4.3-13. Due to difficulty to identify river channel, probable discharge at No.5 was estimated based on the average of specific peak discharges of rivers/creeks that rational formula was employed for runoff calculation.

Table 4.3-12 Applied Runoff Coefficient

Land Cover Type	Applied Runoff Coefficient
Open forest, broadleaved	0.75
Other land, built-up area	0.8
Other land, cultivated, annual crop	0.45
Other land, cultivated, perennial crop	0.45
Other land, fishpond	0.7
Other wooded land, shrubs	0.45

Source: JICA Study Team

Table 4.3-13 Calculation Result (Rational Formula)

No.	River/Creek name	Flood arrival time (min)	Rainfall Intensity (10 years) (mm/hr) **	Rainfall Intensity (5years) (mm/hr) **	Applied Runoff Coef.	**Estimated 10yrs peak discharge (m^3/s)**	**Estimated 5yrs peak discharge (m^3/s)**	Specific Peak Discharge (10years) ($m^3/s/km^2$)	Specific Peak Discharge (5years) ($m^3/s/km^{3)}$
1	Tanghas Lirang Creek	54	76	62	0.63	**57**	**46**	13	11
2	Sagkahan Creek	15	144	117	0.79	**17**	**14**	31	26
3	Mahayahay Creek	36	96	78	0.69	**48**	**39**	18	15
4	Burayan River	54	76	62	0.63	**70**	**57**	13	11
5	Unknown (Payapay Brd.)	-	-	-	-	**4**	**3**	-	-
6	Unknown	36	95	77	0.64	**73**	**59**	17	14

** Following rainfall intensity formula at Tacloban Synoptic Station was applied, which are referred to "Inventory Survey and Basic Analysis of Hydrological Data for Department of Public Works and Highways Technical Standards and Guidelines" (WOODFIELDS CONSULTANTS INC., March, 2002).

R_5=1138.08/(t+10.32)$^{0.70}$

R_{10}=1407.27/(t+10.56)$^{0.70}$

Source: JICA Study Team

b) SCS Unit-hydro graph Method

In 2015 JICA Study, the Soil Conservation Service (SCS) curve number method was employed as a loss model for rainfall-runoff process. The SCS method can reflect the effect of land cover and surface soil condition. The SCS curve number was estimated based on land cover and soil type. Rainfall pattern of rainfall event on 16 Mar, 2011 were applied, whose amount of twenty four (24) hours rainfall is modified with probable daily rainfall of 5 years and 10 years of return periods (details are referred to 5.4.5 in 2015 JICA Study). The calculation results were obtained as shown in Table 4.3-14.

Table 4.3-14 Calculation Result (SCS Unit-hydro graph Method)

No.	River/Creek name	Flood arrival time (min)	SCS No.**	Impervious rate (%)	**10yrs probable peak discharge (m^3/s)**	**5yrs probable peak discharge (m^3/s)**	Specific Peak Discharge (10years) ($m^3/s/km^2$)	Specific Peak Discharge (5years) ($m^3/s/km^{3)}$
-	Mangonbangon River	52	80	21	**28**	**24**	5.16	4.42
1	Tanghas Lirang Creek	54	80	28	**22**	**19**	5.16	4.46
2	Sagkahan Creek*	15	-	-	**3**	**2**	-	-
3	Mahayahay Creek*	36	-	-	**14**	**12**	-	-
4	Burayan River	54	79	49	**29**	**25**	5.51	4.75
5	Unknown (Payapay Brd.)*	-	-	-	**1**	**1**	-	-
6	Unknown	36	-	-	**23**	**20**	-	-

* Probable discharge were estimated based on the average of specific peak discharges of Mangonbangon River, Tanghas-Lirang Creek and Bureyan River that SCS method was employed for runoff calculation.

** SCS Number is defined based on land cover provided by NAMRIA and surface soil map provided by BSWM

Source: JICA Study Team

c) SCS Method plus Water Storage by Swamp and Paddy Areas

In addition to SCS method, retarding function by swamp and paddy field were taken into account in hydrological process. Retarding volume such as by swamp and paddy field were estimated based on site investigation (as shown in Source: JICA Study Team

Figure 4.3-20), inundation survey in 2014 JICA Study and satellite image which are apparently identified as water retarding area such as paddy and swamp area shown in Figure 4.3-21.

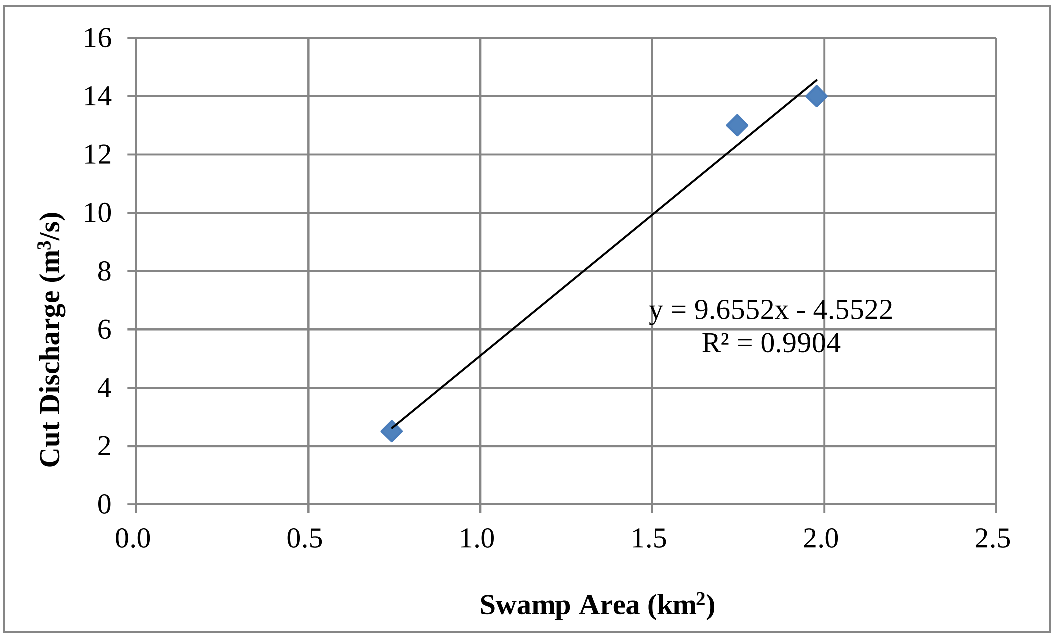
Table 4.3-15 Calculation Result (SCS Unit-hydro graph Method)

No.	River/Creek name	SCS No.	Imper vious rate (%)	w/o Retarding by swamp		Cut Discharge by swamp storage** (m3/s)	**w/ Retarding by swamp**	
				10yr probable peak discharge (m3/s)	5yr probable peak discharge (m3/s)		**10yrs probable peak discharge (m3/s)**	**5yrs probable peak discharge (m3/s)**
-	Mangonbangon River	80	21	28	24	13	**15**	**11**
1	Tanghas-Lirang Creek	80	28	22	19	3	**20**	**17**
2	Sagkahan Creek*	-	-	3	2	0	**3**	**2**
3	Mahayahay Creek*	-	-	14	12	3	**11**	**9**
4	Burayan River	79	49	29	25	14	**15**	**11**
5	Unknown (Payapay Brd.)*	-	-	-	-	0	**1**	**1**
6	Unknown	-	-	23	20	10	**13**	**10**

* Probable discharge were estimated based on the average of specific peak discharges of Mangonbangon River, Tanghas-Lirang Creek and Bureyan River that SCS method was employed for runoff calculation.

**0.3 m of retarding depth was applied based of site investigation and physical inundation survey. As for No2, 3, 5 and 6, amount of cut discharge by swamp storage were estimated based on correlation between calculated swamp area and cut discharge on the three (3) rivers od No.1, 4 and Mangonbangon River (as shown in Figure 4.3-19).

Source: JICA Study Team



Source: JICA Study Team

Figure 4.3-19 Correlation between swamp area and cut discharge

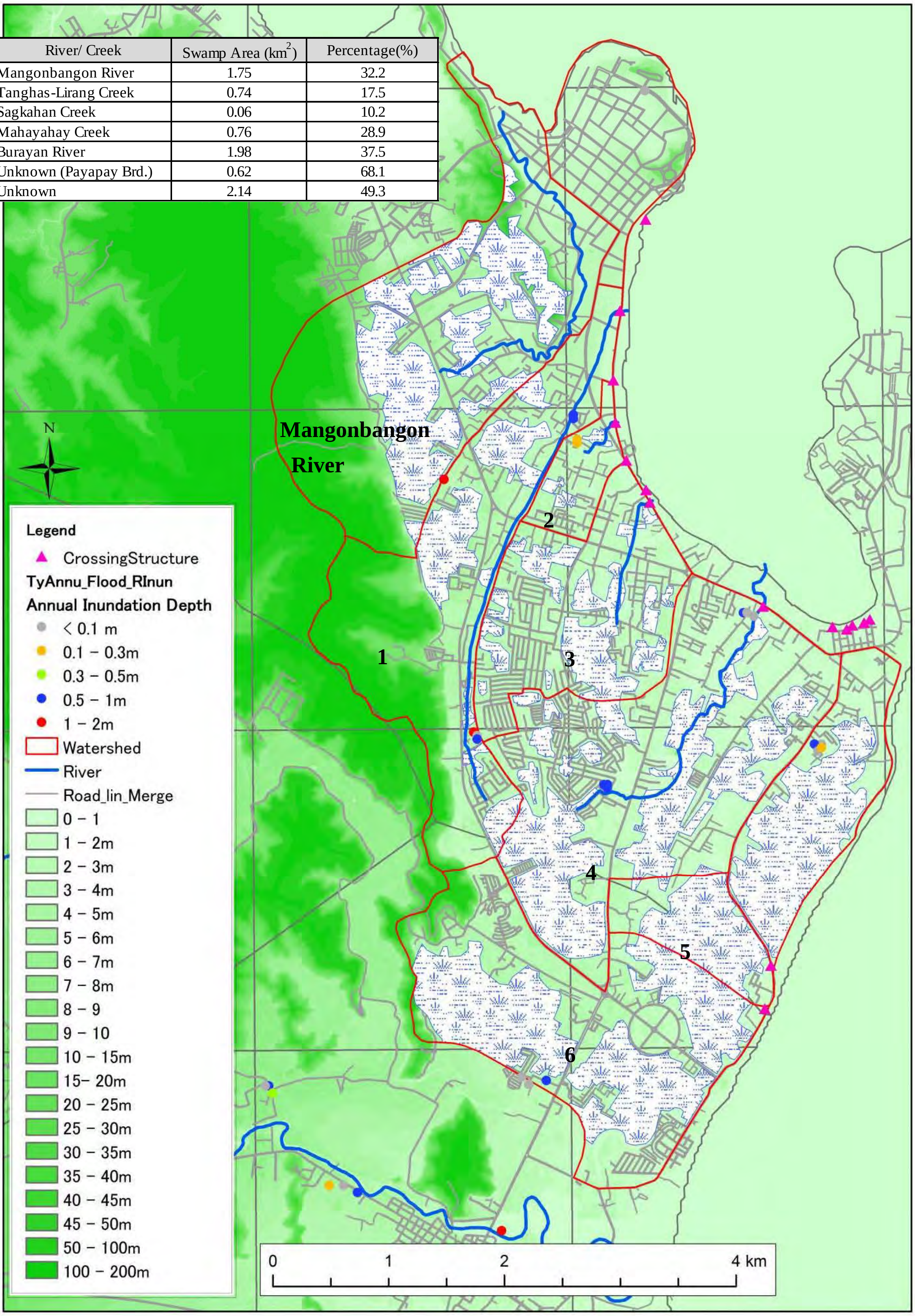
Latitude 11.198442°N
Longitude 124.992017° E

Latitude 11.190190°N
Longitude 124.994008°E

Source: JICA Study Team

Figure 4.3-20 Swamp Area along Mahalika Highway

No.	River/ Creek	Swamp Area (km^2)	Percentage(%)
-	Mangonbangon River	1.75	32.2
1	Tanghas-Lirang Creek	0.74	17.5
2	Sagkahan Creek	0.06	10.2
3	Mahayahay Creek	0.76	28.9
4	Burayan River	1.98	37.5
5	Unknown (Payapay Brd.)	0.62	68.1
6	Unknown	2.14	49.3



Source: JICA Study Team

Figure 4.3-21 Identified Water Retarding Area (Swamp Area and Paddy Field)

c) Summary

The calculation results of three (3) methodologies explained above are summarized below.

Table 4.3-16 Summary of Runoff Analysis

Unit: m^3/s

No.	River/Creek name	Rational formula		SCS		SCS plus Water Retarding	
		10 years	5 years	10 years	5 years	10 years	5 years
1	Tanghas-Lirang Creek	57	46	22	19	20	17
2	Sagkahan Creek	17	14	3	2	3	2
3	Mahayahay Creek	48	39	14	12	11	9
4	Burayan River	70	57	29	25	15	11
5	Unknown (Payapay Brd.)	4	3	1	1	1	1
6	Unknown	73	59	23	20	13	10

Source: JICA Study Team

(6) Evaluation of Discharge Capacity of Existing Outlets

Discharge capacities of outlets of the rivers and creeks analyzed in this study were estimated. Manning's uniform flow was employed for calculation of discharge capacity. The calculation results are summarized in Table 4.3-17.

Table 4.3-17 Discharge Capacity of Outlets

No.	River/Creek name	Probable Discharge* (10years) (m^3/s)	Probable Discharge* (5years) (m^3/s)	Structure type of existing outlet	Dimension of existing outlet	Slope	Discharge (m^3/s)	for 10 years return period flood	for 10 years return period flood
1	Tanghas Lirang Creek	20	17	Bridge	H=1.1m, W=4m	1,120	3	No	No
2	Sagkahan Creek	3	2	Box Culvert	H=1.1m, W=2.6m	340	3	No	Yes
3	Mahayahay Creek	11	9	Box Culvert	(H=1m, W=1m) x 2	460	2	No	No
4	Burayan River	15	11	Bridge	H=3m, W=10m	1,250	29	Yes	Yes
5	Payapay Brd.	1	1	Bridge	H=2.8m, W=25.6m	500	286	Yes	Yes
6	Unknown*	13	10	Box Culvert	(H=2m, W=4m) x 2	320	31	Yes	Yes

* SCS plus water retarding was applied.
**Applied roughness coefficient was 0.03

Source: JICA Study Team

(7) Recommended Design

1) Recommended Hydraulic Dimension to be Secured for Outlet Treatment

In order to drain the discharges estimated above, the hydraulic dimensions listed in Table 4.3-18 are recommended for their outlets, which are obtained by try and error method using Manning's uniform flow formula so as calculated discharge of the dimension are larger than estimated discharge. Considering current situation that has a little tide difference between high and low tides, gate height was fixed with 2m. It is confirmed by Non-uniform flow calculation that the obtained hydraulic dimension can discharge 10 years return period flood without reaching top of their dimensions (as shown in Figure 4.3-22 to Figure 4.3-25).

Table 4.3-18 Recommended Hydraulic Dimension

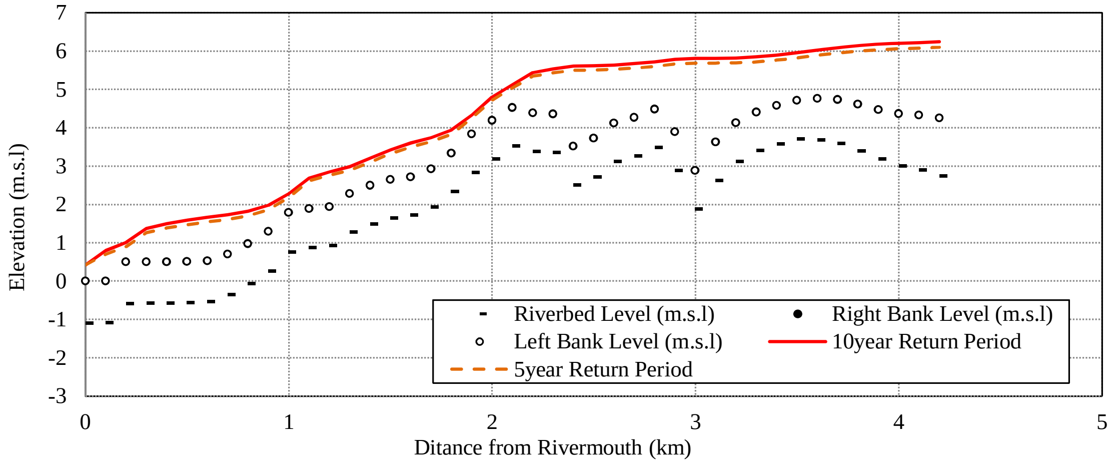
No.	River/Creek name	Structure type of existing outlet	Dimension of existing outlet	Design Slope	Probable Discharge* (m^3/s)		Recommended Hydraulic Dimension			
							10 years		5 years	
					10 years	5 years	Width (m)	Height (m)	Width (m)	Height (m)
1	Tanghas Lirang Creek	Bridge	H=1.1m, W=4m	1,000	20	17	9.0	2.0	8.0	2.0
2	Sagkahan Creek	Box Culvert	H=1.1m, W=2.6m	1,500	3	2	3.0	2.0	2.0	2.0
3	Mahayahay Creek	Box Culvert	(H=1m, W=1m) x 2	250	11	9	3.0	2.0	3.0	2.0
4	Burayan River	Bridge	H=3m, W=10m	2,000	15	11	10.0	2.0	9.0	2.0
5	Unknown (Payapay Brd.)	Bridge	H=2.8m, W=25.6m	2,000	1	1	2.0	2.0	1.0	2.0
6	Unknown	Box Culvert	(H=2m, W=4m) x 2	1,500	13	10	8.0	2.0	6.0	2.0

* SCS plus water retarding was applied.
**Applied roughness coefficient was 0.03

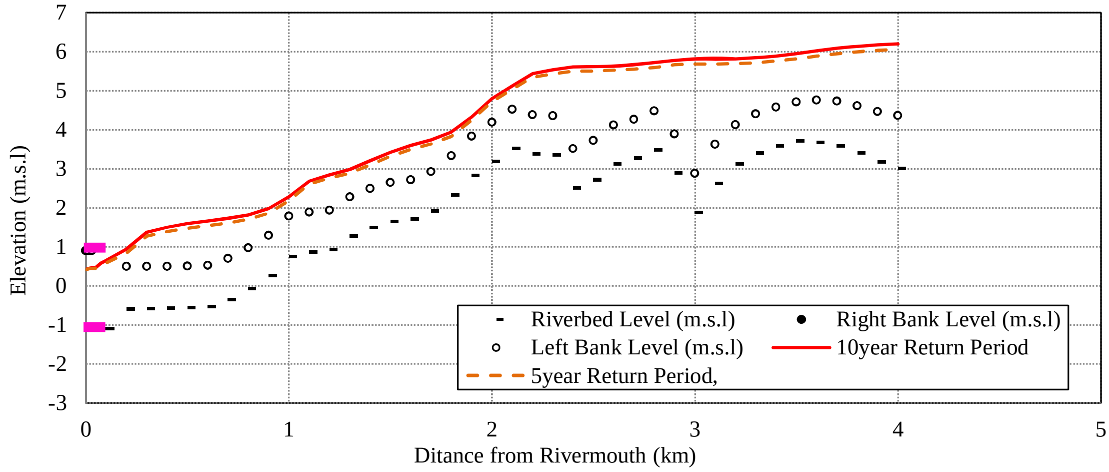
Source: JICA Study Team

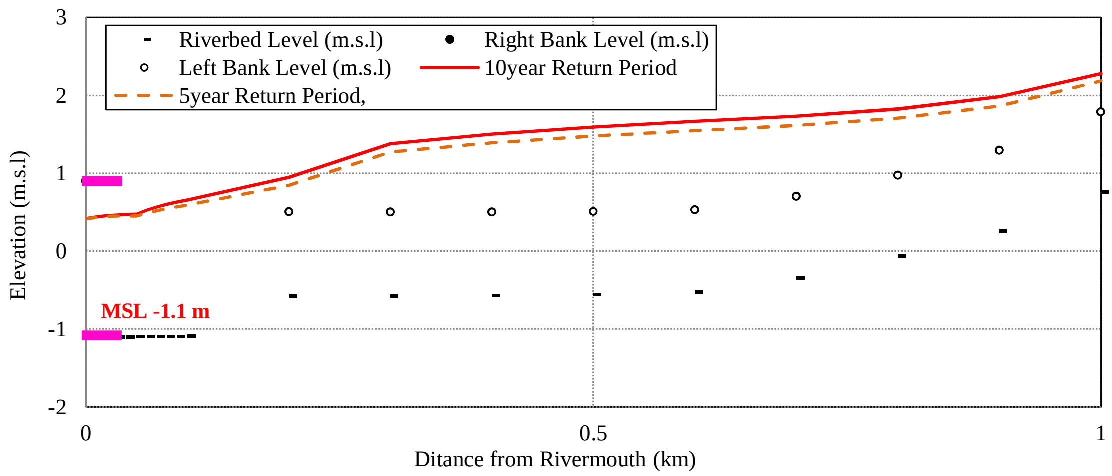
a) Tanghas Lirang

- Existing (extracted from Lidar)



- With flood gate (H=2m, W=9m)



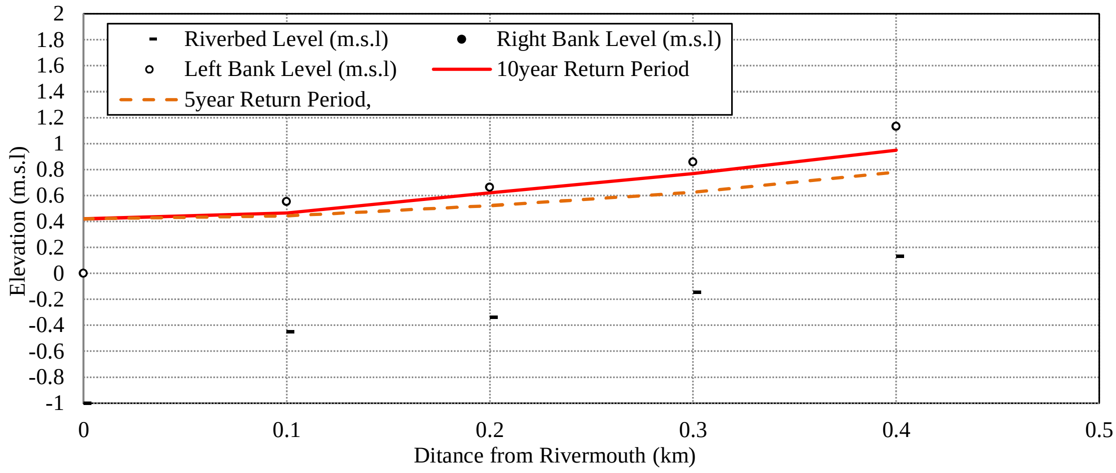


Source: JICA Study Team

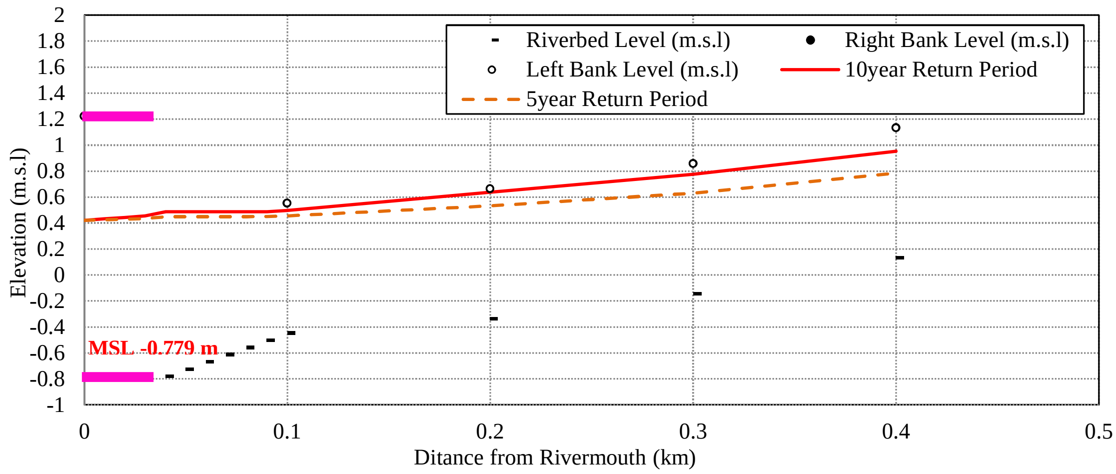
Figure 4.3-22 Longitudinal Profile (Tanghas Lirang Creek)

b) Sangkahan

- Existing (extracted from Lidar)



- With flood gate Flood Gate (H=2m, W=3m)

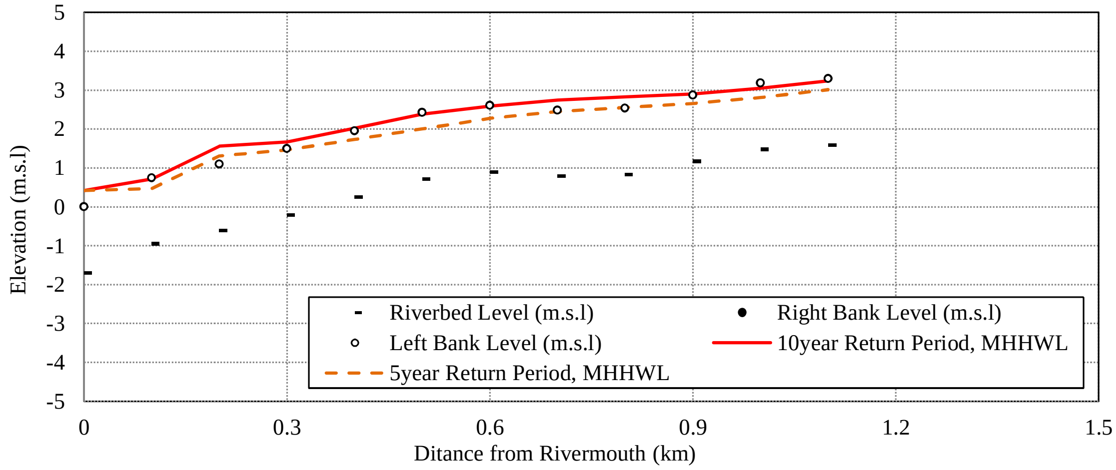


Source: JICA Study Team

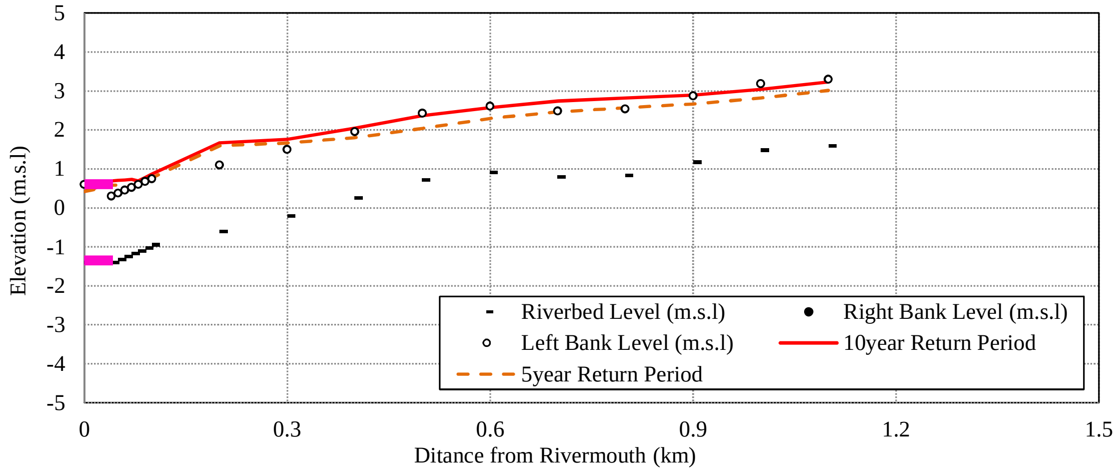
Figure 4.3-23 Longitudinal Profile (Sagkahan Creek)

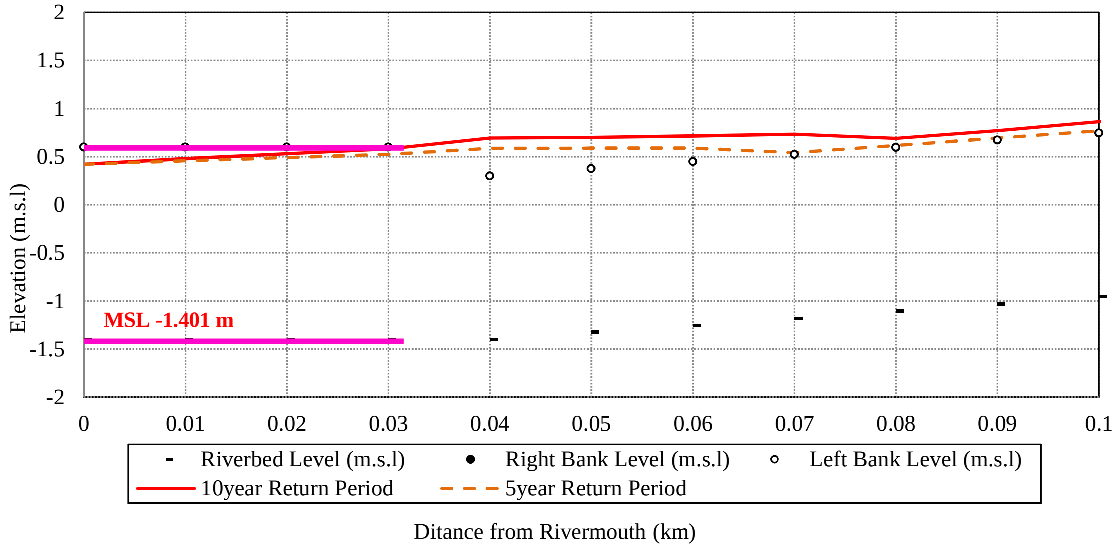
c) Mahayahay Creek

- Existing (extracted from Lidar)



- With flood gate Flood Gate (H=2m, W=3m)



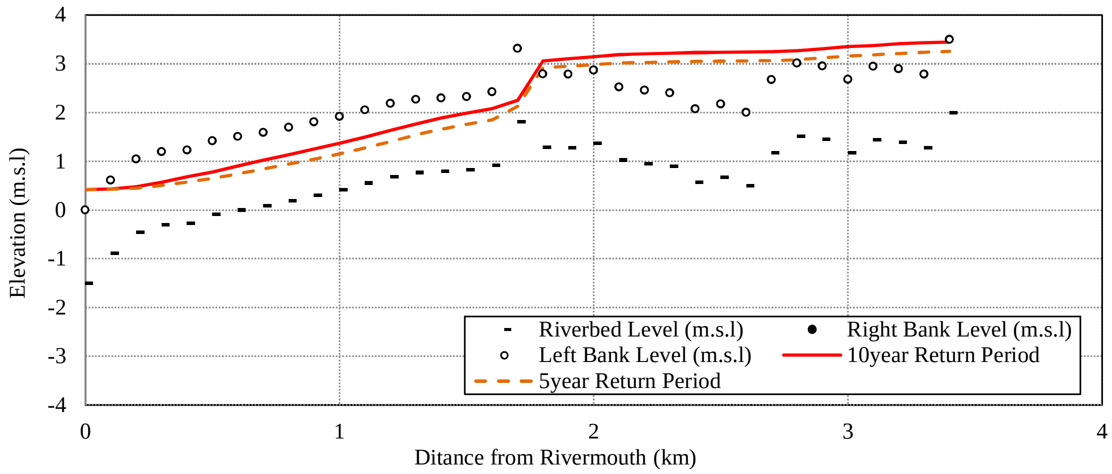


Source: JICA Study Team

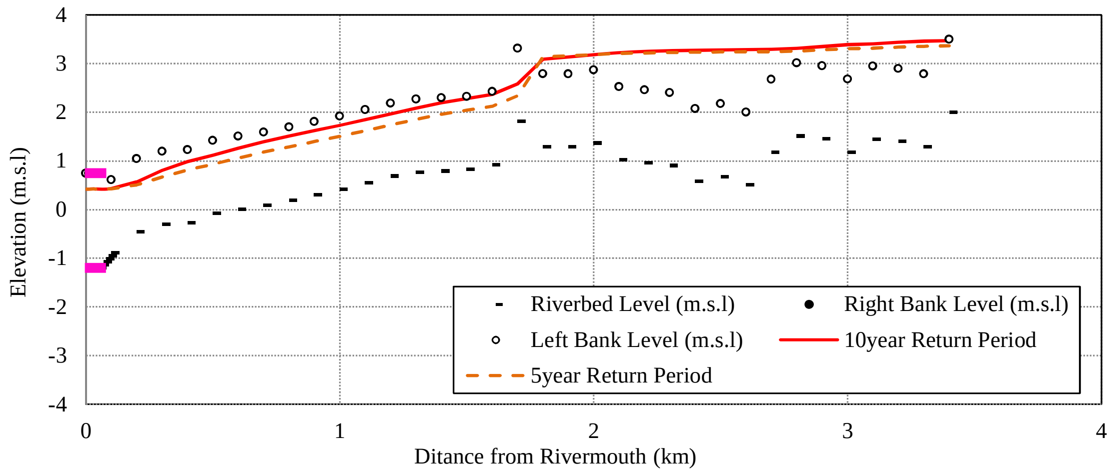
Figure 4.3-24 Longitudinal Profile (Mahayahay Creek)

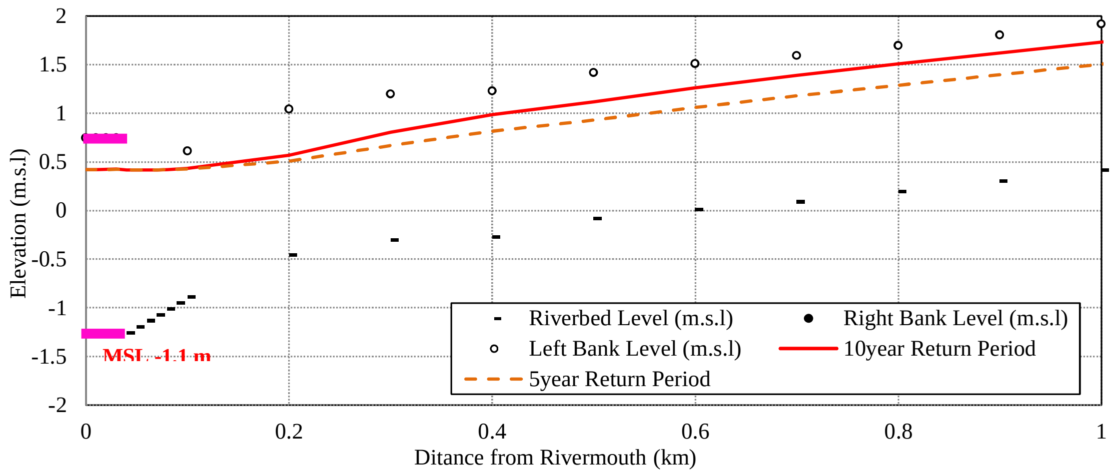
d) Burayan River

- Existing (extracted from Lidar)



- With flood gate Flood Gate (H=2m, W=10m)





Source: JICA Study Team

Figure 4.3-25 Longitudinal Profile (Burayan River)

4.3.3 River Gate Design

(1) Proposed Locations of river gate

River gate facilities are proposed to build at the spots where the rivers/creeks would cross the proposed tide embankment for the purpose of protecting the landside against the sea water. The proposed locations of the river gate facilities are shown in Figure 4.3-26.

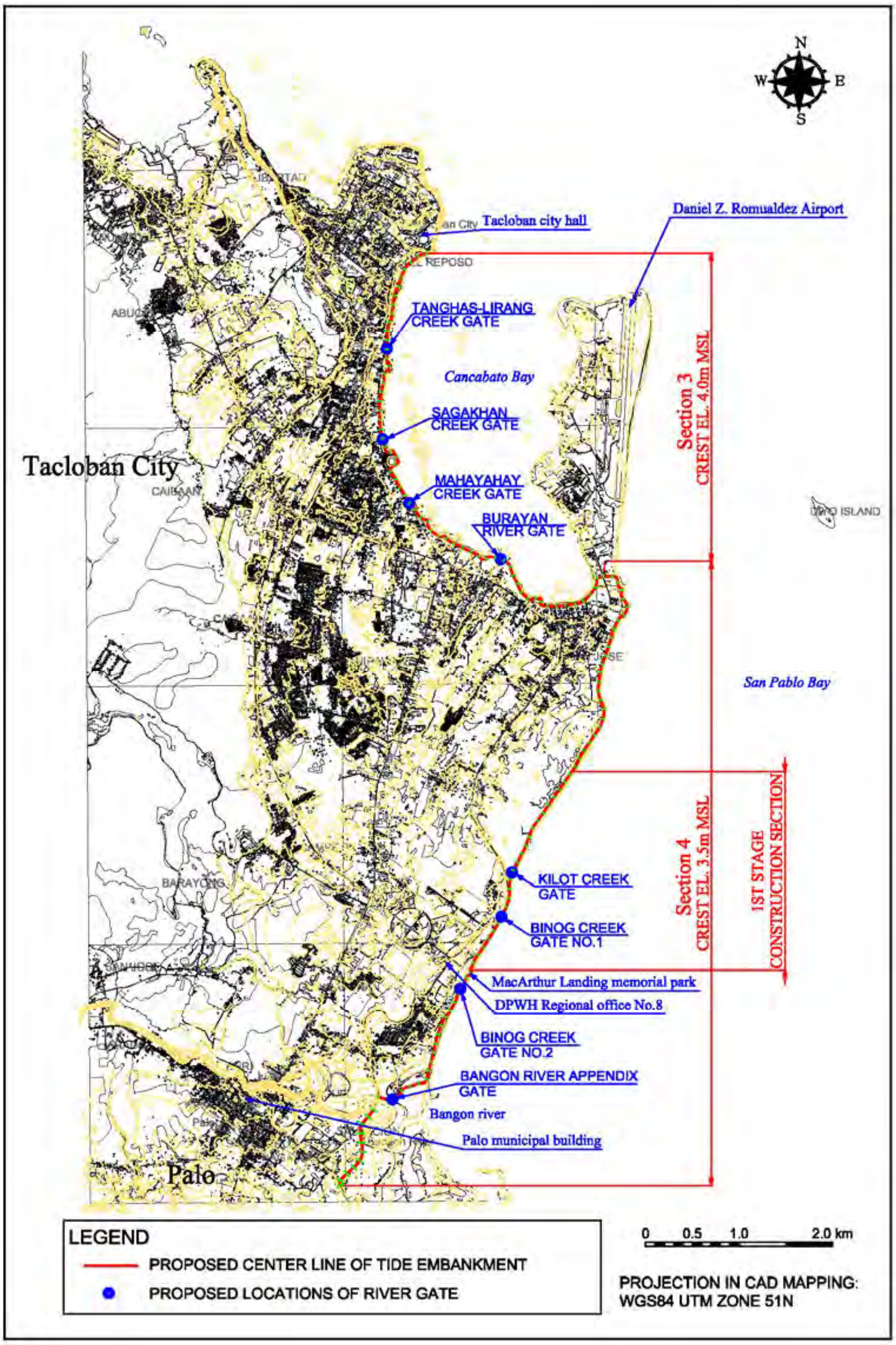


Figure 4.3-26 Proposed Locations of River Gate

(2) Design and operational water levels and depths

River gate facilities will be built against the sea water with the levels not exceeding "Required heights at MSL of the tide embankment" tabulated at the Table 4.3-19 below. The river gate facilities shall be designed not to hinder the river/creek flows at the time of those high water levels or lower, and also not to hydraulically damage the conjunctive tide protection work and river/creek structures.

Table 4.3-19 Required Heights of the Tide Embankment

Sections	Section3	Section 4
Required heights at MSL of the tide embankment	+4.0m	+3.5m

Source: JICA study team

"Required heights at MSL of tide embankment" are being determined by this Study inclusive of a certain allowance, and so the proposed river gate facilities shall be designed to be structurally safe against the sea water with the levels of "Required heights at MSL of the tide embankment".

In the sections 3 and 4, the reevaluated flow discharges conducted by this Study at the existing bridges and culverts closest to the outfalls are summarized as following Table 4.3-20.

Table 4.3-20 Reevaluated Flow Discharges at the Existing Bridges and Culverts

No.	Sections	Rivers/ Creeks	Located LGUs	Current opening size of the existing bridges and culverts closest to the outfalls	1/10 year estimated discharge at outfalls (m^3/s)	Recommended hydraulic dimensions at the locations of the existing bridges and culverts		
						Width (m)	Height (m)	Total area (m^2)
1	S-3	Tanghas-Lirang creek	Tacloban city	Aslum bridge Beam type B4.5m×H1.5m	20	9.0	2.0	18.0
2	S-3	Sagakhan creek	Tacloban city	Box culvert B3.5m×H1.0m	3	3.0	2.0	6.0
3	S-3	Mahayahay creek	Tacloban city	Box culvert B1.0m×H0.8m×2	11	3.0	2.0	6.0
4	S-3	Burayan river	Tacloban city	Burayan bridge 1 span I girder type B10.0m×H2.0m	15	10.0	2.0	20.0
5	S-4	Kilot creek (Payapay)	Palo municipality	Payapay bridge 3 span I girder type B(7.8m+10.0m+7.8 m) ×H3.5m	1	2.0	2.0	4.0
6	S-4	Binog creek	Palo municipality	Box culvert No.1 B2.5m×H1.8m×2	13	5.0	2.0	10.0
7	S-4	Binog creek	Palo municipality	Box culvert No.2 B2.7m×H1.7m	As auxiliary of No.1	-	-	To be not smaller than current
8	S-4	Bangon river appendix	Palo municipality	No structure	-	-	-	To be not smaller than current

Source: JICA study team

Heights, clear spans and the numbers of the gate leaves will be determined so that the reevaluated flow discharges and heights, done by this Study, of the rivers/creeks can pass smoothly when those gate leaves are fully opened.

Freeboard will be added 0.60m, for design discharge of less than 200m^3/s, on the top of the water level reevaluated by this Project in conformity with "Design Guidelines Criteria and Standards for Public Works and Highways, Volume II, page 468, DPWH" as following description.

Item	Design Discharge (m^3/s) Q	Value to be added to design water-level (m)
1	Less than 200	0.60
2	200 to less than 500	0.80
3	500 to less than 2000	1.0
4	2 000 to less than 5 000	1.20
5	5 000 to less than 10 000	1.50
6	More than 10 000	2.00

The proposed gate heights will not be lower than the freeboard 0.60m + the reevaluated flow heights.

In discussion with Palo municipal mayor and her officers which took place in June 11th 2015, the boats having locally common size shall be accommodated landward through the proposed gate opening at Kilot creek.This is because Kilot creek (otherwise called Payapay) is being currently used as waterway.

Regardless of the reevaluated flow discharge for the proposed gate sites, size of single gate leaf will be determined taking into consideration the manufacturing capacity and experience even of Philippine manufactures. So far, information on Philippine manufactures of hydraulic gates has not been obtained enough, however as the result of some hearings from the officers of DPWH Regional office No.8 and NIA Regional office No.8, the manufacturing capacity of Philippine manufactures does not sound higher. Therefore, the maximum size of single gate leaf will be to be 10m^2 as small gate leaf as stipulated in Japan. From terms of the quality assurance, overseas procurement of the hydraulic gate facilities will not be excluded.

Sill elevations of the gate leaves will be determined in the Basic Design stage complying with the current river/creek bed elevations.

Thus in sections 3 and 4, the design & operational water depths and the dimensions of gate leaves are proposed as Table 4.3-21 in order to proceed to the Basic Design stage for conducting mechanical, electrical, structural and foundation design.

Table 4.3-21 Design & Operational Water Depths and Dimensions of Gate Leaves

No.	Sections	Provisional names of gates	Proposed dimensions of gate leaves								
			Design sea water level	Tentative (To be determined in B/D)			Width (Clear span)	Height	Nos.	Total area	Type
				Gate sill elevation	Design water depth	Opera-tional water depth					
			(m MSL)	(m MSL)	(m)	(m)	(m)	(m)	-	(m^2)	
1	S-3	Tanghas- Lirang creek Gate	+4.0	-0.5	4.5	4.5	3.0	2.6	3	23.4	Fixed wheel
2	S-3	Sagakhan creek Gate	+4.0	-0.5	4.5	4.5	3.0	2.6	1	7.8	Fixed wheel
3	S-3	Mahayahay creek Gate	+4.0	-0.5	4.5	4.5	3.0	2.6	1	7.8	Fixed wheel
4	S-3	Burayan river Gate	+4.0	-1.0	5.0	5.0	3.5	2.6	3	27.3	Fixed wheel
5	S-4	Kilot creek Gate	+3.5	-0.5	4.0	4.0	3.5	2.6	1	9.1	Fixed wheel
6	S-4	Binog creek Gate No.1	+3.5	+0.3	3.2	3.2	3.0	2.0	2	12.0	Fixed wheel
7	S-4	Binog creek Gate No.2	+3.5	-0.5	4.0	4.0	3.0	2.6	1	7.8	Fixed wheel
8	S-4	Bangon river appendix Gate	+3.5	-1.0	4.5	4.5	3.0	2.6	2	15.6	Fixed wheel

Source: JICA study team

(3) Design concept

The proposed river gate facilities will be composed of hydraulic gate system (gate leaves, hoisting devices and gate guides), box culvert, piers, columns, hoisting deck, wing walls, breast walls and stairs. Each structure excluding the hydraulic gate system and the staircase shall be of reinforced concrete.
Transition dike will be provided in order to connect the existing channel with the upstream (landside) wing wall of the structure.

The transition and training dikes will be of grouted riprap (wet stone masonry), and the channel bed between dikes will be protected by placing riprap (cobble stones) with length of the estimated high water depth for the landside and with length of 6×(+0.42m MSL as MHHWL - channel elevation) for the seaside.

Hoisting device will be installed on the hoisting deck. The hoisting devices of the gate leaves will be selected so as to be running-cost-minimum and highly reliable motorized if solo manual operation hoisting is judged to be unpractical in terms of usual operation and to be unavailable from manufacturing viewpoint because of the size and weight limitation of gate leaf.
The motorized hoisting device shall equip manual hoisting system just in case for emergent situations such as power failure and/or motor failure.

Power source of the motorized hoisting device will be of 230V × 60Hz single and/or three phases commercial electricity from Leyte II Electric Cooperative, INC. (Leyeco II). However, if the proposed locations of river gates are not available for commercial electricity, engine generator will be provided for supplying electricity.

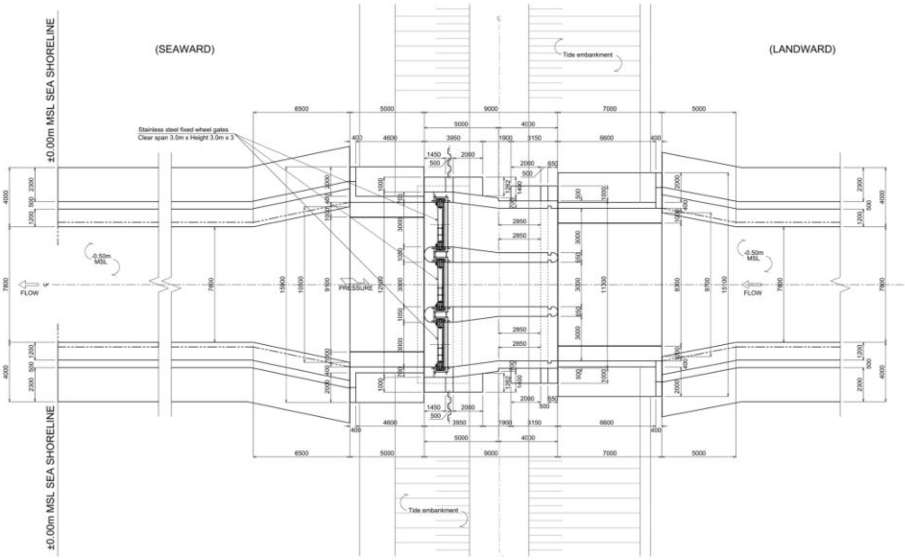
The structures will be borne with foundation piles unless the foundation ground is enough sounder for employing spread foundation method. The bearing capacity of the foundation ground will be analyzed with N values of the standard penetration tests conducted by the DPWH officers. The N values for the pile end bearing require 30 or more for sandy & sandy gravel layer and 20 or more for clayey layer having respectively 3m or more consecutive layer thickness.

At the Basic Planning stage, rough layouts of the proposed river gates are shown as follows.

(4) Layout of river gates

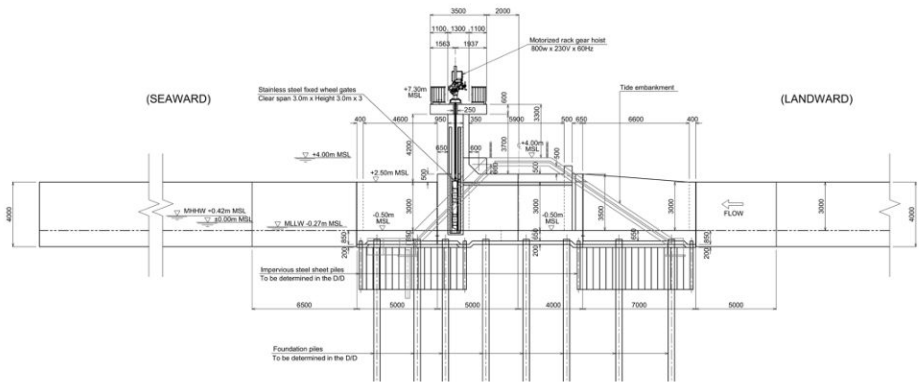
At the Basic Planning stage, rough layouts of the proposed river gates are shown as follows.

1) **Tanghas- Lirang creek Gate in Tacloban city**



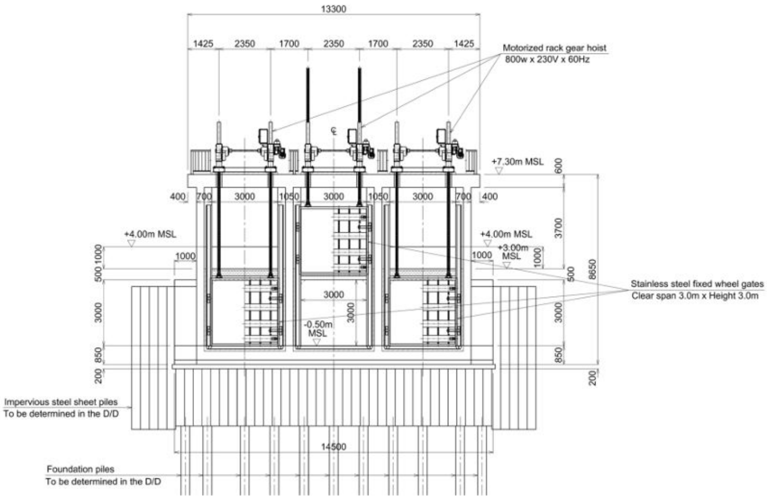
Source: JICA Study Team

Figure 4.3-27 Plan of Tanghas- Lirang creek Gate



Source: JICA Study Team

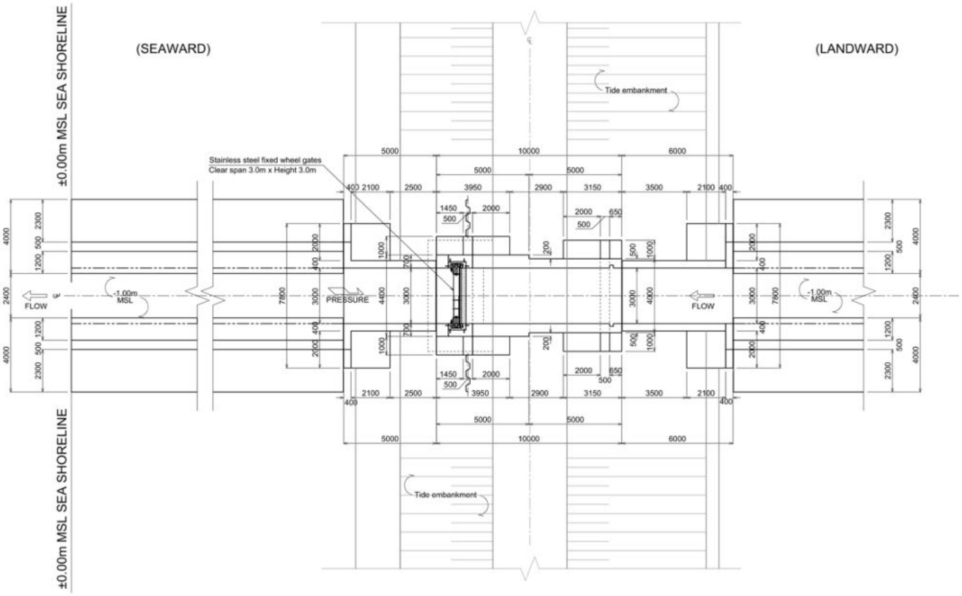
Figure 4.3-28 Profile of Tanghas- Lirang creek Gate



Source: JICA Study Team

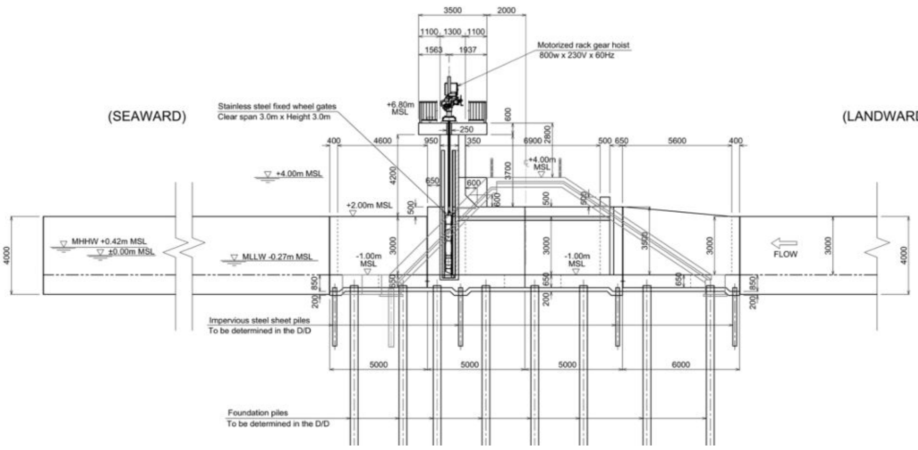
Figure 4.3-29 Front View of Tanghas- Lirang creek Gate from Seaside

2) **Sagakhan creek and Mahayahay creek Gates in Tacloban city**



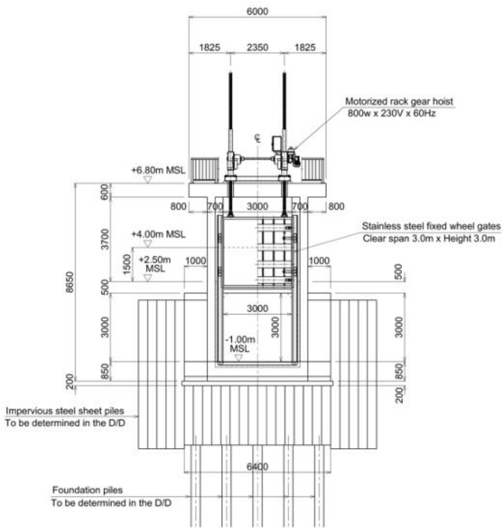
Source: JICA Study Team

Figure 4.3-30 Plan of Sagakhan creek and Mahayahay creek Gates



Source: JICA Study Team

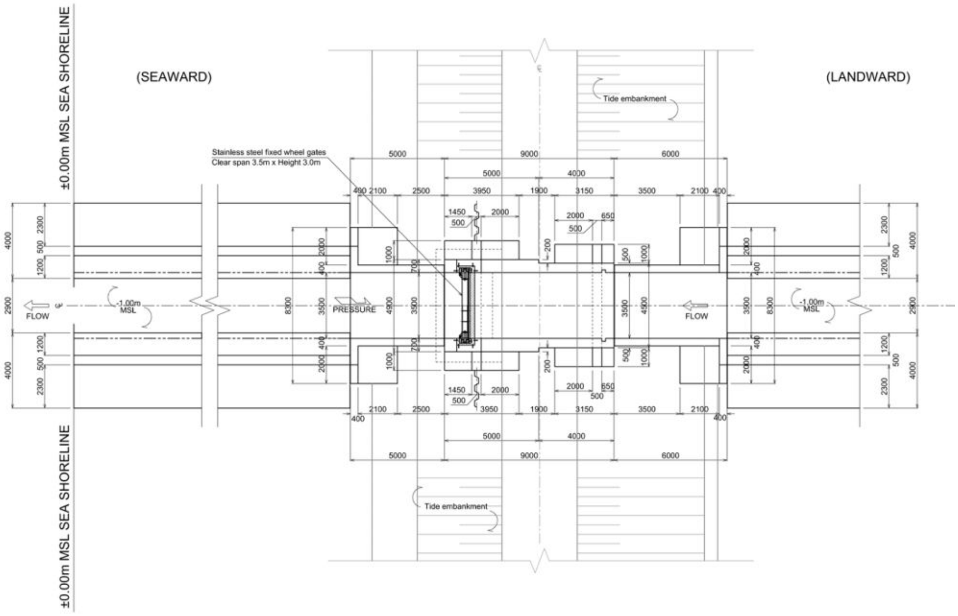
Figure 4.3-31 Profile of Sagakhan creek and Mahayahay creek Gates



Source: JICA Study Team

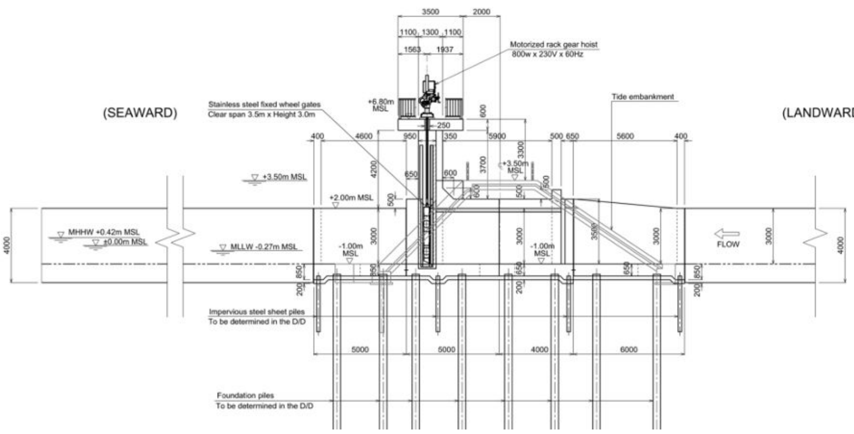
Figure 4.3-32 Front View of Sagakhan creek and Mahayahay creek Gates from Seaside

3) **Kilot creek Gate in Palo Municipality**



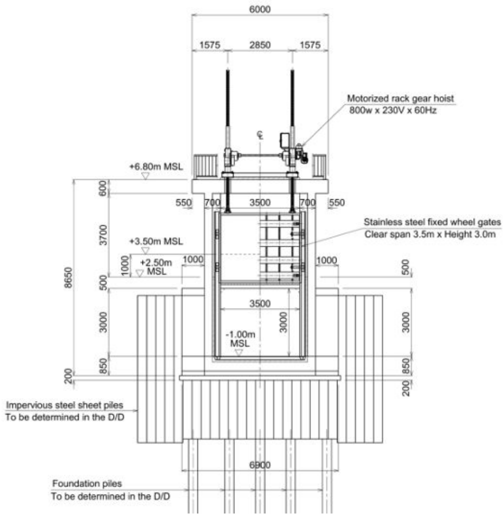
Source: JICA Study Team

Figure 4.3-33 Plan of Kilot creek Gate



Source: JICA Study Team

Figure 4.3-34 Profile of Kilot creek Gate



Source: JICA Study Team

Figure 4.3-35 Front View of Kilot creek Gate from Seaside

(5) Design conditions

1) Codes, standards, guidelines and manuals for designing

The following codes, standards, guidelines and manuals will be used for conducting the Basic and Detailed Designs for the river gate facilities.

a) Philippine codes, standards, guidelines and manuals

- National Building Code of the Philippines
- National Structural Code of the Philippines, Volume 1 and 2
- DPWH Design Guidelines Criteria and Standards for Public Works and Highways, Volumes I, II, III and IV
- DPWH Standard Specifications for Highways, Bridges and Airports
- DPWH Ready Check
- Other Codes, Standards, Guidelines and Manuals Spread in the Philippines

b) International codes and standards

- International Organization for Standardization (ISO)
- International Electrotechnical Commission (IEC)
- Other Codes, Standards, Guidelines and Manuals Spread Worldwide

c) American codes, standards, guidelines and manuals

- American National Standards Institute (ANSI)
- ASTM International (former American Society for Testing and Materials)
- American Association of State Highway and Transportation Officials (AASHTO)
- American Concrete Institute (ACI)
- American Institute of Steel Construction (AISC)
- U.S. Army Corps of Engineers, Engineering and Design
- Other Codes, Standards, Guidelines and Manuals Spread in the USA

d) Japanese codes, standards, guidelines and manuals

- Japanese Industrial Standards (JIS)
- Japan Electrical Manufacturers' Association (JEM)
- Japanese Electrotechnical Committee (JEC)
- Japan Association of Dam & Weir Equipment Engineering Standards
- Technical Standards for River and Sabo Works, River Association of Japan
- Specification for Highway Bridges, Part I ~ V, Road Association of Japan
- Other Codes, Standards, Guidelines and Manuals Spread in Japan

2) Parameters for structural and foundation design

Parameters for the structural and foundation design of the river gate facilities are described as follows.

a) Unit weights of materials

Unit weights of materials are tabulated as follows.

Table 4.3-22 Unit Weights of Materials

Materials	Unit weight (kN/m^3)
Structural Steel	77.0
Stainless steel	77.8
Reinforced concrete	24.0
Prestressed concrete	24.0
Plain concrete	23.5
Cement mortar	21.0
Grouted riprap/Wet stone masonry	22.0
Sand/Gravel/Crushed stone	19.0
Water	9.8
Timber	8.0

Source: Ministry of Land, Infrastructure, Transport and Tourism of Japan

Unit weights of soil shall be determined by the results of laboratory tests, however if there is no data available at hand, the following standards of unit weights will be used.

Table 4.3-23 Unit Weights of Soil

<table>
<tr><th colspan="2" rowspan="2">Type of soil</th><th colspan="2">Wet (kN/m^3)</th></tr>
<tr><th>Loose</th><th>Compacted</th></tr>
<tr><td rowspan="3">Natural Foundation</td><td>Sand /Gravel</td><td>18</td><td>20</td></tr>
<tr><td>Sandy soil</td><td>17</td><td>19</td></tr>
<tr><td>Clayey soil</td><td>14</td><td>18</td></tr>
<tr><td rowspan="3">Embankment</td><td>Sand /Gravel</td><td colspan="2">20</td></tr>
<tr><td>Sandy soil</td><td colspan="2">19</td></tr>
<tr><td>Clayey soil</td><td colspan="2">18</td></tr>
</table>

Source: Ministry of Land, Infrastructure, Transport and Tourism of Japan

3) Parameters of concrete

a) Class

Concrete is classified according to28th day compressive strength, maximum size of aggregates and application as shown in the following table.

Table 4.3-24- Class of Concrete

Class of concrete	28th day compressive strength f'c (N/mm^2)	Maximum size of aggregates (mm)	Application
A	20.7	50	General use, reinforced concrete members with thickness more than 20cm
B	16.5	50	Plain concrete for structure
C	20.7	19	General use, reinforced concrete members with thickness less than 20cm, secondary concrete
P	37.7	-	Prestressed concrete structures and members
E	29.4	25	Precast reinforced concrete pile
F	11.8	40	Plain concrete for leveling
Seal	20.7	37.5	Concrete deposited in water

b) Allowable stress

Allowable stresses for plain and reinforced concretes are obtained from the values of the 28th day compressive strength f'c of concrete.　The following table shows formulas for obtaining those allowable stresses.

Table 4.3-25　Allowable Stresses of Concrete

Type of stress			Plain concrete (N/mm^2)	Reinforced concrete (N/mm^2)
Flexure	Compression		0.40f'c	0.40f'c
	Tension		0.21f'c	—
	Rapture		$0.70f'c^{1/2}$	$0.70f'c^{1/2}$
	Bearing		0.30f'c	0.30f'c
Shear stress	Beam, one-way slab and foundation		$0.08f'c^{1/2}$	$0.08f'c^{1/2}$
	Axial force	Compression	$0.08f'c^{1/2}$	$0.08f'c^{1/2}$
		Tension	$0.075[1+0.6(N/Ag)]\ f'c^{1/2}$	$0.075[1+0.6(N/Ag)]\ f'c^{1/2}$

Note: N = Axial Force、Ag = Area of reinforcement

4) Parameters of reinforcing steel bars

a) Grade and allowable stresses

Reinforcing steel bars shall be of Grade 275 or Grade 415. The following table shows the yield strength and allowable stress of the reinforcing steel bars.

Table 4.3-26- Allowable Stress of Reinforcing Steel Bars

Grade	Minimum yield strength (N/mm^2)	Allowable tensile stress (N/mm^2)
Grade 275 reinforcement	275	140
Grade 415 reinforcement	415	168

Dimensions of reinforcing steel bars used in the Philippines are shown in the following table.

Table 4.3-27- Dimensions of Reinforcing Steel Bars

Nominal diameter (mm)	Nominal perimeter (mm)	Nominal cross-sectional area (cm^2)	Unit mass (kg/m)
8	25.1	0.503	0.387
10	31.4	0.785	0.617
12	37.7	1.131	0.888
16	50.3	2.011	1.578
20	62.8	3.142	2.466
25	78.6	4.909	3.853
28	88.6	6.158	4.834
32	100.5	8.043	6.313
36	113.1	10.179	7.990

b) Modulus of elasticity

The modulus of elasticity for concrete (Ec) is obtained as 4,700 f'c1/2 (N/mm2) for normal weight concrete.

The modulus of elasticity for reinforcing steel bar (Es) is obtained as 200,000 (N/mm2).

Therefore, in case of f'c = 20.7 (N/mm2), modular ratio is obtained as n = Es / Ec = 9.

5) Parameters of soil

a) Cohesion of clayey soil

Cohesion of clayey soil shall be determined by tri-axial test or unconfined compression test. When using an unconfined compression test, the following formula can be used to estimate the cohesion of soft clay.

$$c = \frac{q_u}{2}$$

Where,

c : cohesion (kN/m2)

qu : unconfined compression strength (kN/m2)

If there is no data available, cohesion can be estimated by using N-value as follows.

c = 6N ~ 10N

b) Coefficient of lateral reaction of foundation ground

For the design of foundation piles, coefficient of lateral reaction of soil can be estimated by using the following method.

$$K_H = K_{H0}\left(\frac{B_H}{0.3}\right)^{-3/4}$$

Where,

KH : coefficient of lateral reaction of soil (kN/m3)

KHo : coefficient of lateral reaction of soil (kN/m3) equals to a value of plate bearing test with rigid circular plate of a diameter of 30cm, and it can be estimated by the following equation with a modulus of deformation: E0 obtained by various soil tests and investigations:

$$K_{H0} = \frac{1}{0.3}\alpha\, Eo$$

Where,

α: coefficient given by the table below

Eo : modulus of deformation of soil for design obtained by soil test or equation as shown in table below.

BH : converted loading width of foundation in load action direction (m)

Table 4.3-28- Relation between Eo and α

Modulus of Deformation Eo (kN/m^2)	α value	
	Normal, Storm	Seismic
A half of modulus of deformation obtained by repeating curves in plate bearing test with rigid circular plate of a diameter of 30cm	1	2
Modulus of deformation measured in borehole horizontal loading test	4	8
Modulus of deformation obtained by test pieces in unconfined compression test or tri-axial test	4	8
Modulus of deformation estimated by Eo=2800N with N-value in Standard Penetration Test	1	2

c) Compression index

Compression index of soil or foundation ground shall be determined by compression test, but if there is no data available, it can be estimated by the following formula.

Cc=0.009 (LL -10)

or

Cc=0.0054 (2.6 w - 35)

Where,

Cc : compression index

LL : liquid limit (%)

w : natural water content (%)

6) Seismic load

Seismic load is basically computed using Seismic Coefficient Method. Seismic coefficient shall be one-half (1/2) of the acceleration coefficient (Kh = A/2 = 0.4/2 = 0.2).

7) Seismic load

Seismic load is basically computed using Seismic Coefficient Method. Seismic coefficient shall be one-half (1/2) of the acceleration coefficient (Kh = A/2 = 0.4/2 = 0.2).

8) Wind forces

Minimum wind pressure to be used in this project shall be computed by the following formula.

P = Ce Cq qs I

Where,

P = Design wind pressure

Ce = Combined height, exposure and gust factor coefficient

Cq = Pressure coefficient for the structure or portion of structure

qs = Wind stagnation pressures at height of 10 meters

I = Importance factor

9) Load combination and increase in allowable stresses

All structures shall be designed for the largest stresses resulting from the worst combination of loads that may act on the structure at any given condition. For safety reasons, each component of the structure shall be in proportion to bear all combinations of these forces.

Group I : Normal condition : D + L + I + E + H + U + F

Group II : Wind condition I : D + E + H + U + F + W

Group III : Wind condition II : Group I + 0.3W + WL + LF

Group IV : Seismic condition : D + Ee + H + U + V + De

Where,

D=dead load

L=live load

I=impact/dynamic effect of live load

E=earth pressure

H=hydrostatic pressure

U=uplift

W=wind load on structure

WL=wind load on live load

LF=longitudinal force from live load

V=seismic load

F=flowing water pressure

Ee=earth pressure due to earthquake

De=Dynamic water pressure due to earthquake

The following extra factors in allowable stresses shall be applied to the load combinations listed above.

Table 4.3-29- Extra Factors in Allowable Stresses

Philippines code and standard	Group I / Condition	None
	Group II	25%
	Group III	25%
	Group IV	33%
Japanese code and standard	Normal	None
	Seismic	50%

(6) Types of Hydraulic Gates

Small gate leaves themselves, whose size is 10m2 or less, is classified as slide gate type and fixed wheel (otherwise called as roller) gate type.

Slide gate type is to do water pressure supporting and water sealing by attaching a bearing plates on the contact surface with the gate guide. That structure is very simple. However the lifting load gets greater because of greater friction generated between the bearing plates and gate guide when water pressure is higher. So, this type is used for low water protection.

Fixed wheel/roller gate type is to do water pressure supporting and lifting by attaching wheels/rollers in the both side the gate leaf itself. That water sealing is made by attaching the rubber seals on the four sides of the gate leaf. The lifting load gets smaller because of smaller friction generated between the wheels/rollers and gate guide even when water pressure is higher. So, this type is used commonly even for higher water protection.

Hoisting device is generally classified as rack gear type, screw spindle type and wire rope winch type. In Japan, the up and down hoisting speed requires 0.3m/min except for emergent use gates. Manual hoisting device is normally manufactured up to 40kN of lifting loads. Lifting loads more than 40kN require motorized hoisting device. Screw spindle type and rack gear type is commonly used for small gates. The screw spindle type is used for higher hoisting speed for emergency use gates. However the device requires higher power motor. The rack gear type is used for normal hoisting speed for general use gates and so the device requires smaller power motor.

In conclusion, the combination of fixed wheel/ roller gate with motorized rack gear is highly recommended for this project implementation.

Table 4.3-30 and Table 4.3-31 show the comparison gate types and hoisting devices.

Table 4.3-30 Proposed Gate Types and Hoisting Devices

No.	Sections	Provisional names of gates	Proposed gate types and hoisting devices								
			Design sea water level	Tentative (To be determined in B/D)			Availability of gate leaves judged by rack gear hoist			Rack gear Hoist	
				Gate sill elevation	Design water depth	Operational water depth	Arrangement	Slide type	Fixed wheel type	Manual 40kN or less	Motorized
			(m MSL)	(m MSL)	(m)	(m)					
1	S-3	Tanghas-Lirang creek Gate	+4.0	-0.5	4.5	4.5	B3.0m×H 2.6m×3	N/A	A	N/A	100kN
2	S-3	Sagakhan creek Gate	+4.0	-0.5	4.5	4.5	B3.0m×H 2.6m×1	N/A	A	N/A	100kN
3	S-3	Mahayahay creek Gate	+4.0	-0.5	4.5	4.5	B3.0m×H 2.6m×1	N/A	A	N/A	100kN
4	S-3	Burayan river Gate	+4.0	-1.0	5.0	5.0	B3.5m×H 2.6m×3	N/A	A	N/A	100kN

5	S-4	Kilot creek Gate	+3.5	-0.5	4.0	4.0	B3.5m×H 2.6m×1	N/A	A	N/A	75kN
6	S-4	Binog creek Gate No.1	+3.5	+0.3	3.2	3.2	B3.0m×H 2.0m×2	N/A	A	N/A	50kN
7	S-4	Binog creek Gate No.2	+3.5	-0.5	4.0	4.0	B3.0m×H 2.6m×1	N/A	A	N/A	75kN
8	S-4	Bangon river appendix Gate	+3.5	-1.0	4.5	4.5	B3.0m×H 2.6m×2	N/A	A	N/A	75kN

Source: JICA study team

Table 4.3-31 Comparison of Hoisting Devices

Item	Type of hoisting device		
	Rack gear	Screw spindle	Wire rope winch
Self-weight closing	Possible	Impossible	Possible
Pressing-down force	Available	Available	Not available
Self-locked	Possible	Possible	Depends on speed reducer type
Mechanical efficiency	High	Low	Depends on speed reducer type
Motor capacity	Small	Large	Depends on speed reducer type
Opening degree control	Possible	Possible	Possible
Opening & closing speed	0.3m/min	Approx.3m/min	0.3m/min
Mechanical complexity	Fair	Fair	Complex
Applicable gate leaf size	Small	Small and medium	Medium and large
Operationability	Easy	Easy	Easy
Maintenance	Easy	Fair	Much required
Recommendability	Recommendable	Unrecommendable	Unrecommendable

Seawater contains great deal of chloride as much as 19,000ppm and has higher electric conductivity as much as 270 ohm-cm that are respectively 100 times or higher than fresh water. Higher concentration of chloride breaks electrochemically a passive film created on the material surface. As the result, surface metal comes to dissolves continuously and bring about corrosion of material surface. SUS304 is representative of austenite stainless steel. SUS 316, which is also austenite stainless steel, is added molybdenum comparing SUS304 for higher corrosion resistance. Stainless steel is made adding nickel and chrome into steel materials in the manufacturing process. Chrome added in the materials with oxygen in the surrounding air forms a dense and strong oxide film giving performance of corrosion resistance. Table 4.3-32 and Table 4.3-33 show the comparison of stainless steel and common steel.In the project implementation, the stain less steel gate leaves and gate guides are highly recommended.

Table 4.3-32 Comparison of Materials for Gate Leaves and Guides

Evaluation Item	Materials for gate leaves and guides	
	Stainless steel (SUS304, SUS316)	Steel (SS400, SM400)
Strength	Very good	Very good
Corrosion resistance against seawater	Very good	Very poor
Maintenance	Easy	Fair
Aesthetics	Good	Fair (Very poor when oxidized)
Marketability	Good	Very good
Erection	Good	Good
Workability	Fair	Very good
Initial cost	High	Cheap
Repainting	Unrequired	Required (every 4 to 7 years)
Running cost	Low	High
Recommendability	**Highly recommendable**	Unrecommendable

Table 4.3-33 Stress of Gate Materials

Item \ Materials	SUS 304	SS400 and SM400 Thickness $\leq$ 40 mm
Axial tensile stress (for net sectional area)	100 N/mm^2	120 N/mm^2
Axial compressive stress (for gross sectional area)	On condition of $(l/r) \leq 19$, 100 N/mm^2	On condition of $(l/r) \leq 20$, 120 N/mm^2
Compressive members	On condition of $19 < (l/r) \leq 96$, $100– 0.53 ((l/r)-19)$	On condition of $20 < (l/r) \leq 93$, $120 – 0.75 ((l/r)-20)$
	On condition of $96 < (l/r)$, $980,000/(7,200 + (l/r)^2)$	On condition of $93 < (l/r)$, $10,000,000/(6,700 + (l/r)^2)$
	where; l: buckling length of member (mm) r: radius of gyration of gross sectional area of member (mm)	
Compressive splice member	100 N/mm^2	120 N/mm^2
Bending stress at tensile side (for net sectional area)	100 N/mm^2	120 N/mm^2
Bending stress at compressive side (for gross sectional area)	On condition of $(l/b) \leq (10/K)$, 100 N/mm^2	On condition of $(l/b) \leq (9/K)$, 120 N/mm^2
	On condition of $(l/b)> (10/K)$, $100– 0.9 ((Kl/b) – 10)$	On condition of $(9/K) < (l/b) \leq 30$, $120 – 1.1((Kl/b) – 9)$
	where; l: distance between fixed points of compressive flange (mm)	

	b: width of compressive flange (mm) $K = \sqrt{3 + \frac{Aw}{2Ac}}$ Aw: gross sectional area of web plate (mm^2) Ac: gross sectional area of compressive flange (mm^2) In case of (Aw/Ac) < 2, K is taken as 2.	
On condition that compressive flange is directly welded to skin plate, etc.	100 N/mm^2	120 N/mm^2
Shear stress (for net sectional area)	60 N/mm^2	70 N/mm^2
Bearing stress	150 N/mm^2	180 N/mm^2

Notes:

In case the thickness exceeds 40 mm, the allowable stresses for normal loading condition of the structural steel members shall be adjusted that the stress is 0.92 time that of the allowable stress as mentioned above in the case of steel materials SS 400 and SM 400.

4.3.4 Back Water Dike Design

In terms of the tide embankment design, Outlet treatment at river mouth has several options (e.g. flood gate, backwater dike), that will be determined based on river scale, construction cost, workability, social background and so on. As for Bangon River, back water dike will be applied at downstream because DPWH has already started constructing flood protection dike between Bernard Reed Bridge 2 and Bernard Reed Bridge. Basically JICA STUDY TEAM follows the concept design by DPWH. This flood protection dike targets 50 years return period flood.

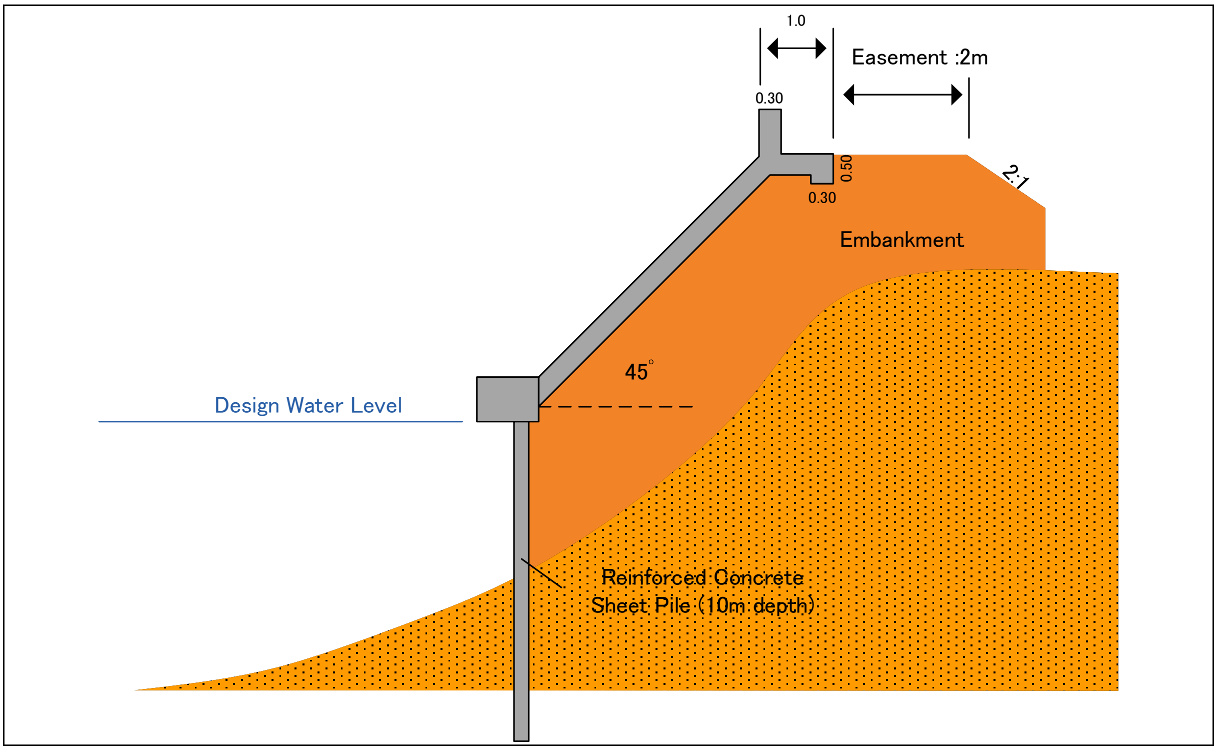
(1) Design Concept

1) Target Return Period

According to the collected drawing proposed by DPWH, designed flood protection dike aims to mitigate the flood impact that on the scale as such that occurred on March 16 of 2011. DPWH also mentioned that target return period is 50 years.

2) Structure Type

The structure type is earth embankment (unknown height) with forty five (45) degree of slope which is covered with revetment and 60 cm parapet of concrete. Reinforced concrete sheet pile is installed into river bed with 10m depth.

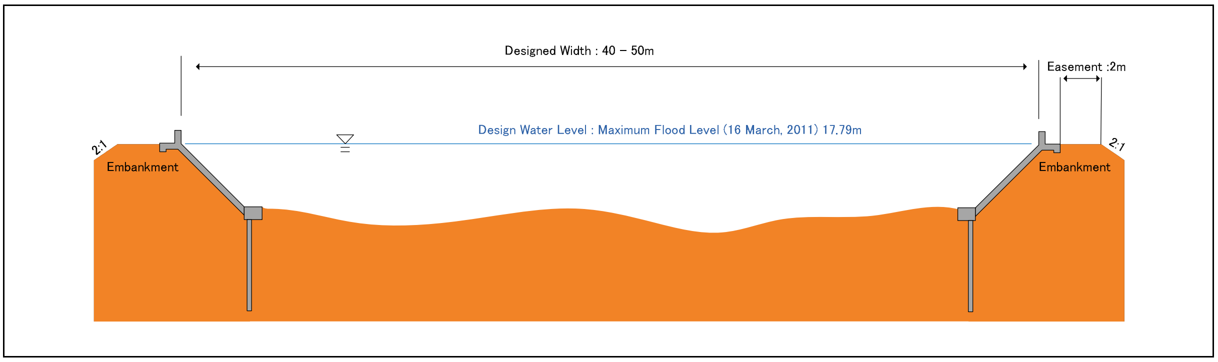


Source: DPWH

Figure 4.3-36 Structure Type of Backwater Dike

(2) Cross Section Design

Figure 4.3-37 shows general design cross section. Basically the river improvement plan designed by DPWH doesn't assume river widening. According to designed plan, river width ranges between forty (40) and fifty (50) meters, which seems to depend on the width of the existing channel. Flood maximum level is fixed with 17.79m though its datum is unknown.



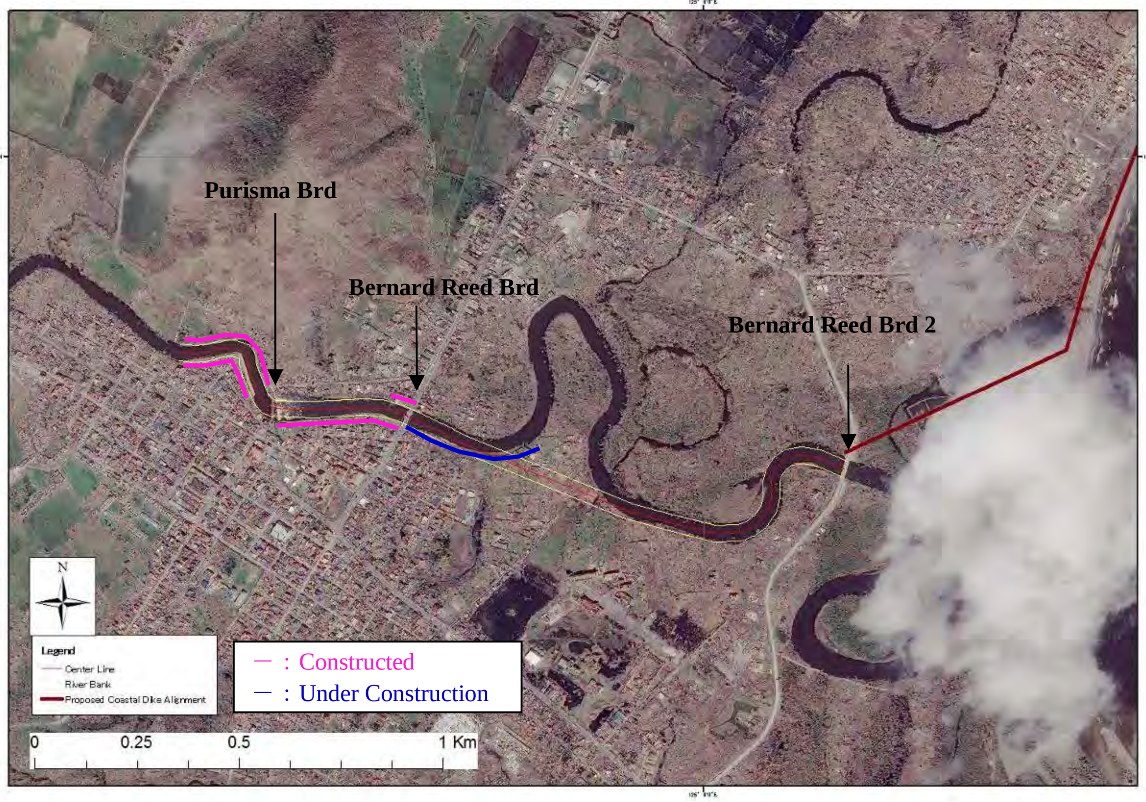
Source: DPWH

Figure 4.3-37 River Cross Section Designed by DPWH

(3) Alignment Plan Designed by DPWH

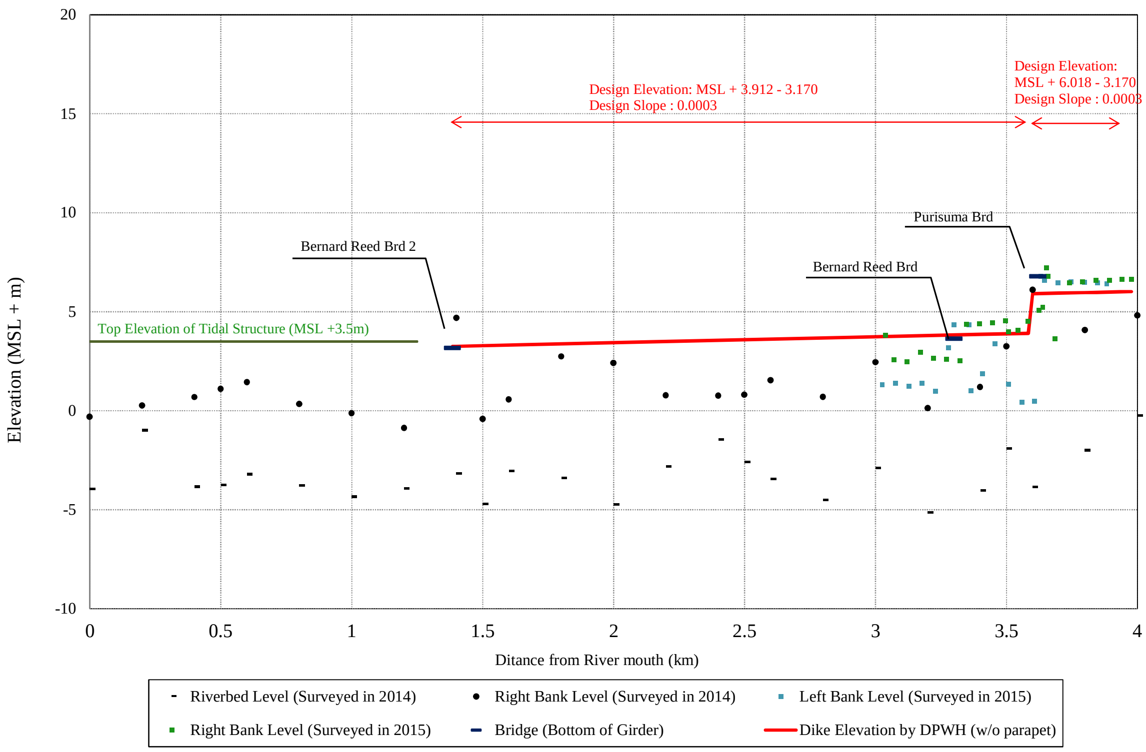
Figure 4.3-38 shows horizontal alignment plan was designed by DPWH. The designed alignment has length of 1.9 km ranging from Bernard Reed Bridge 2 (passing point of coastal dike proposed by DPWH) to upstream of Purisima Bridge. Construction of embankment has been partly completed at 1) both side banks of upstream of Pursima Bridge, 2) right bank between Pursima Bridge and Bernard Reed Bridge and 3) left bank with a length of 50m from Bernard Reed Bridge.

Figure 4.3-39 shows longitudinal profile of the alignment plan. In the longitudinal plan designed by DPWH, there are not specific values of top elevation of the dike but only a description of design slope (0.0003), which was used for alignment plan. It was confirmed that the elevation of dike at Bernard Reed Bridge 2 is supposed to be lower than elevation of the coastal dike proposed by DPWH.



Source of Alignment: DPWH
(c)CNES 2014,Distribution Astrium Services / SpotImage,
(c)DigitalGlobe

Figure 4.3-38 Horizontal Alignment Plan Designed by DPWH



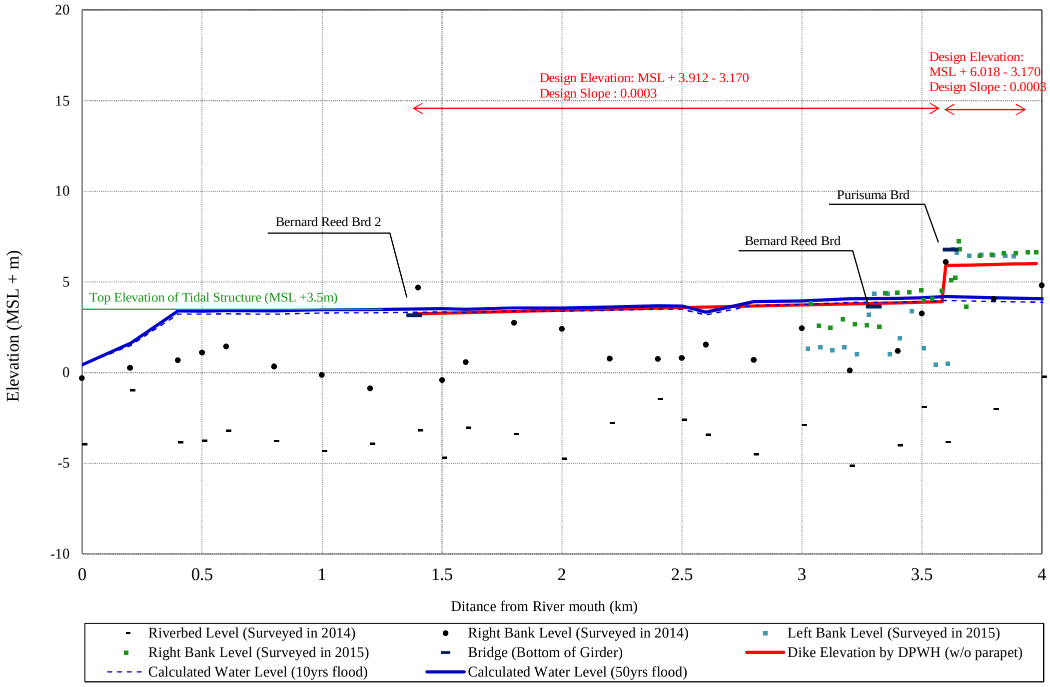
*Dike elevation was obtained by subtracting the height of paraet (60cm) from the survryed top elvation of existing dike (including parapet), and by appling design slope of 0.0003 basically adopted by DPWH.

Source: JICA Study Team

Figure 4.3-39 Longitudinal Alignment Plan Designed by DPWH

(4) Evaluation of Effect of Flood Protection Dike Designed by DPWH

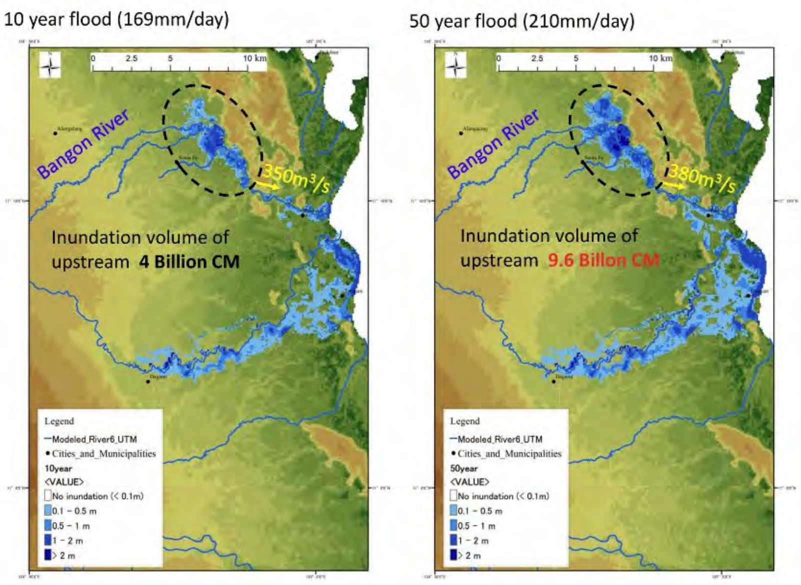
In this paragraph, effect of the flood protection dike designed by DPWH was evaluated by means of non-uniform flow. Comparing the water levels calculated based on 50 years return period flood and level of designed dike level, the designed flood dike has a discharge capacity only for 10 years return period (as shown in Figure 4.3-40).



*For given condition of non-uniform flow calculation, input discharges are 350m^3/s for 10years return period and 380 m^3/s for 50years, respectivly. These values were obtained in the flood inundation analysis conducted in 2015 JICA Study, which inundation volume at upstream are removed (refer to Figure 4.3-41).

Source: JICA Study Team

Figure 4.3-40 Comparison between Calculated Water Level for the Return Periods and Dike Level



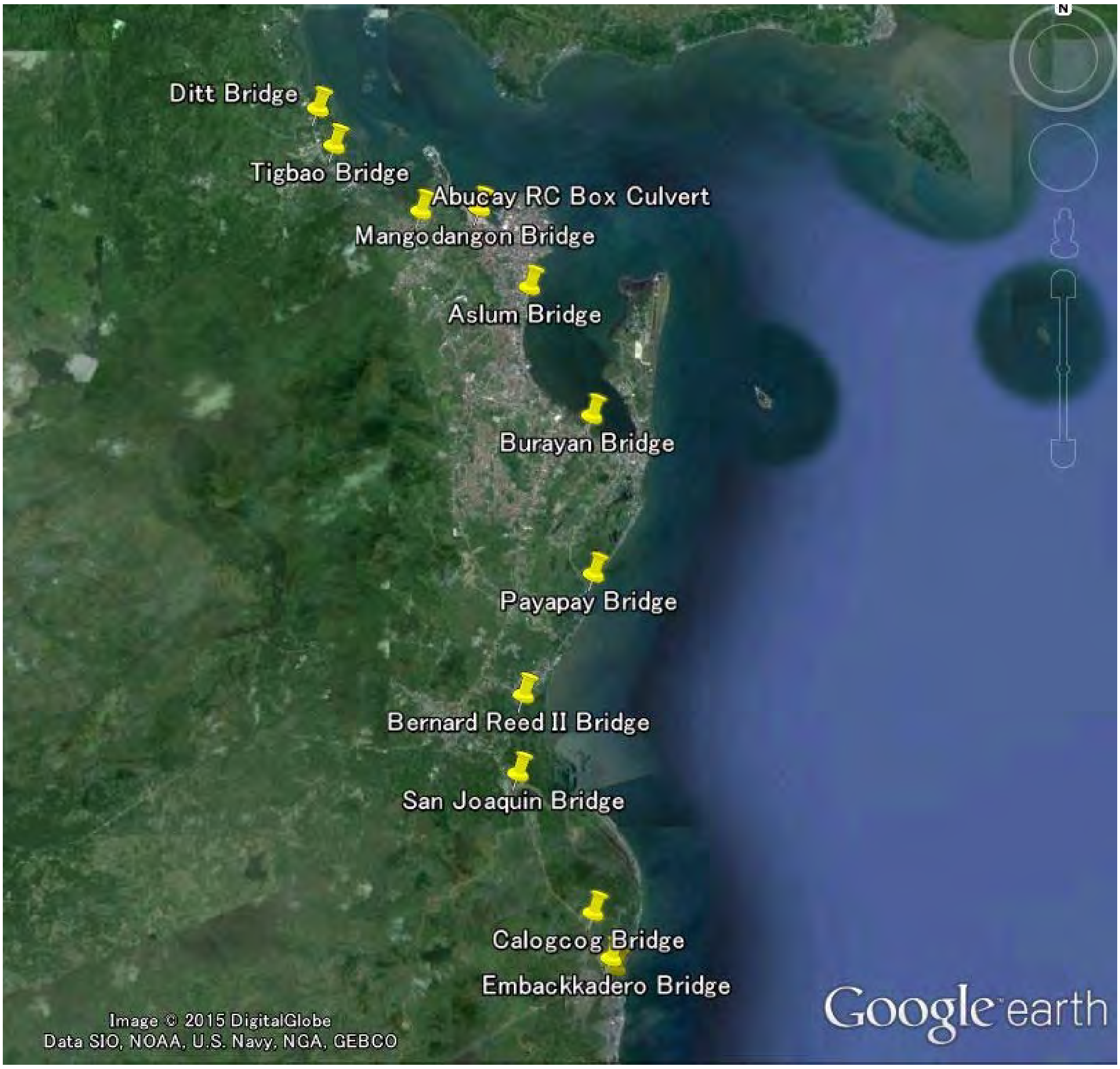
*Sourse: JICA Study Team

Figure 4.3-41 Inundation Volume and Calculated Discharge Reached to Downstream of Bangon River

4.3.5 Review of Existing Bridge and Introduction of Effective Use Method

(1) List of Existing Bridge

The aliment of tide protecting structure along national road and/or seashore is studied. The location of bridges on the tide protecting structure is shown in Figure 4.3-42 and the basic information of bridges is shown in Table 4.3-34.



Source: JICA Study Team

Figure 4.3-42 Location of bridges

There is no information about ages and detail (e.g. as built drawing) of bridges without Burayan Bridge that was re-constructed in 2014.

Table 4.3-34 Basic Information of Bridges

Section	Bridge	River	Surface Height *1 (m)	Elevation (m)		Length (m)
				Surface	Bottom of girder	
1	Diit	Ditt	5.8	-	-	22.00
	Tigbao	Tigbao	5.2	-	-	45.65
2	Abucay RC Box Culvert *2	Abucay Creek	1.8	-	-	
	Mangonbangon (1)	Mangonbangon	3.6	-	-	24.85
3	Aslum	Tanghas Lirang Creek	1.5	1.101	0.702	4.28
	Burayan (New)	Burayan	3.2	2.711	1.692	25.45
4	Payapay	Payapay	4.2	3.779	2.900	26.30
	Bernard Reed II	Bangon	4.9	4.819	3.200	58.85
5	San Joaquin	Binahan	3.3	-	-	99.10
6	Calogcog	Calogcog	3.7	-	-	68.02
	Embarkadero	Embarkadero	3.3	-	-	49.80
	Cambatista	Cambatista	3.6	-	-	38.69
Reference	Solano	Cambatista	3.3	-	-	31.80
	Bernard Reed	Bangon	-	4.215	2.873	54.64

*1: Height from the tide level (at field survey) to the bridge surface.

*2: Structure type is same as Aslum Bridge.

Source: JICA Study Team

(2) Review of Existing Bridges Condition

From the section 3 to the section 4 is the target area of basic plan. Bridges in this area are reviewed.

Bridges (without Old Burayan Bridge) didn't get serious damages from Yolanda (over 50 years return period). It is considered that they were protected from Yolanda storm surge as follow reasons,

- Elevation of girder bottom was higher than tide elevation by Yolanda storm surge,
- Bridge that has small river cross section area, behaved like a box culvert, and/or
- Slope and river bed around substructures were protected from scouring.

According to the above consideration, the tide elevation of Yolanda storm surge was lower than the girder of existing bridges.

(3) Introduction of Efficient Use Method

1) **In case of the elevation of bottom of girder is higher than the design tide level**

The storm surge runs to inland through the bridge, and the heightened river dike is able to protect the river side area. The other hand, the slope and river bed around the existing bridge have to protect from the scouring due to go and return flow of storm surge.

2) **In case of the elevation of bottom of girder is lower than the design tide level**

There are two (2) methods.

(a) Re-construction to new bridge

- ➢ The existing bridge is re-constructed.
- ➢ The elevation of new girder bottom is higher than the design tide level.
- ➢ The river dike is heightened.
- ➢ Slope and river bed protecting structures are installed.

(b) Installation of river crossing gate

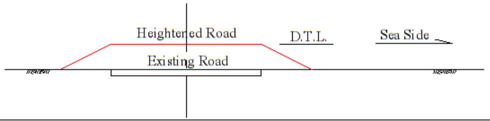
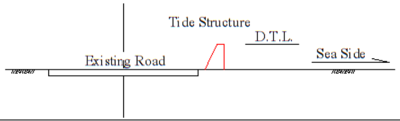
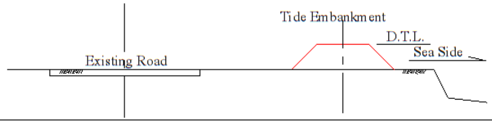
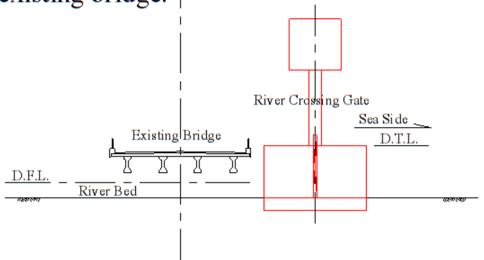
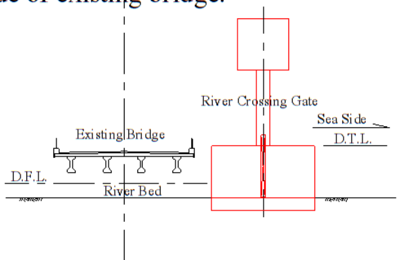
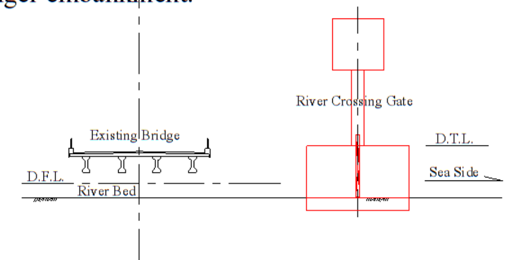
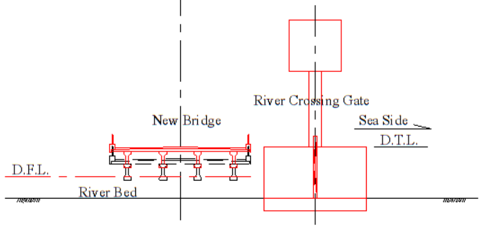
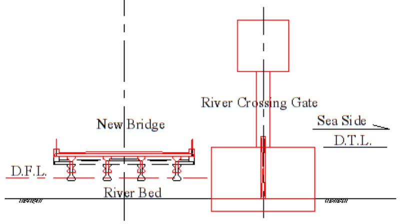
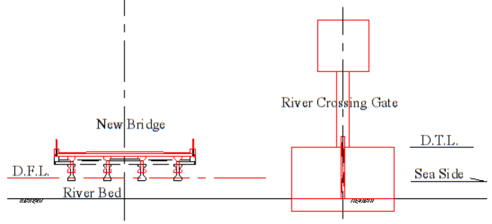
- ➢ The river crossing gate is installed at sea side from the existing bridge.

Based on the above consideration, some examples of effective use method are shown in Figure 4.3-43 and Figure 4.3-44.

By the way, the elevation of girder bottom should be higher than flood level, and there are some countermeasures as follows,

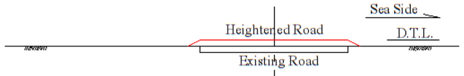
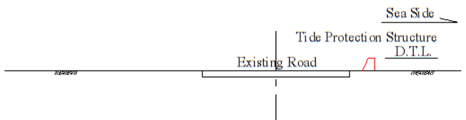
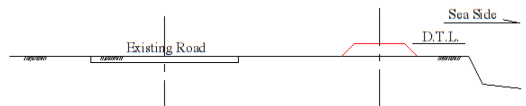
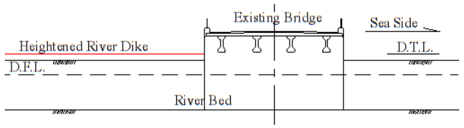
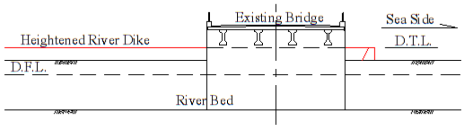
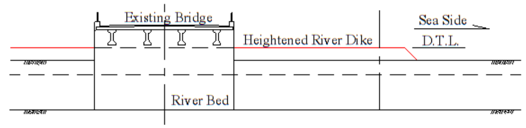
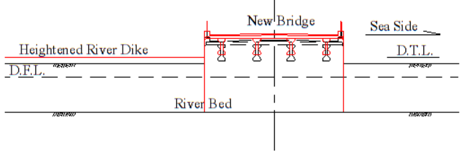
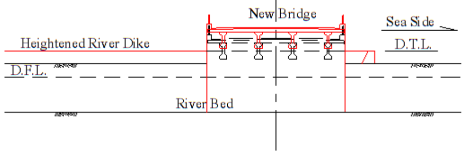
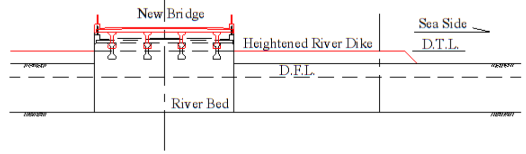
- Heightening of bridge/road; or
- Widening of river/bridge.

In case of the heightening of bridge is shown in Figure 4.3-43 and Figure 4.3-44.

Activation Method			Case A	Case-B	Case- C
Installation of river crossing gate	Land		- Existing road is heightened. 	- Tide structure is installed along existing road. 	- Tide embankment is constructed along seashore. 
	River (Flood level)	(Low)	- River crossing gate is installed at sea side of existing bridge. 	- River crossing gate is installed at sea side of existing bridge. 	- River crossing gate is installed on aliment of tiger embankment. 
		(high) Refer only	- Existing bridge is heightened against river flood. 	- Existing bridge is heightened against river flood. 	- Existing bridge is heightened against river flood. 

Source: JICA Study Team

Figure 4.3-43 Example of Effective Use Method (1)

Activation Method			Case A	Case-B	Case- C
Heightening of river dike against storm surge	Land		- Existing road is heightened.  	- Tide structure is installed along existing road.  	- Tide embankment is constructed along seashore. 
	River (Flood level)	(Low)	- Design tide level is lower than elevation of girder bottom. - River dike is heightened from existing bridge to upstream side.  	- Design tide level is lower than elevation of girder bottom. - River dike is heightened from tiger protecting structure to upstream side.  	- Design tide level is lower than elevation of girder bottom. - River dike is heightened from tiger embankment to upstream side. 
		(high) Refer only	- Existing bridge is heightened against river flood.  	- Existing bridge is heightened against river flood.  	- Existing bridge is heightened against river flood. 

Source: JICA Study Team

Figure 4.3-44 Example of Effective Use Method (2)

(4) Bridge re-construction

In case of the elevation of girder bottom is higher than the design tide level and the design flood level and added the freeboard, the existing bridge will be used with the installation of slope and riverbed protection against the storm surge flow.

In case of the bridge re-construction, the new bridge will have to keep as follows functions,

- The elevation of girder bottom will be higher than the design tide level;
- The elevation of girder bottom will be higher than the design flood level and added the freeboard as shown in **Table 4.3-35**;
- The approach road will be heightened;
- The river dike around the bridge will be heightened; and/or
- The traffic will be detoured or stopped during the re-construction.

Table 4.3-35 Freeboard Allowance

Design Discharge Q (m^3/s)	Value to be added to design water-level (m)
Less than 200	0.6
200 to less than 500	0.8
500 to less than 2000	1.0
2000 to less than 5000	1.2
5000 to less than 10000	1.5
More than 10000	2.0

Source: Design Guidelines Criteria and Standards Volume II DPWH

In case of the Installation of the river crossing gate, if the elevation of girder bottom is higher than the design flood level and added the freeboard, the existing bridge will be able to use. However, the elevation of girder bottom isn't higher than the design flood level and added the freeboard, the existing bridge have to be re-constructed.

4.4 Storm Water Drainage

Based on the current condition of drainage outlets in the Tacloban City and the Palo Municipality obtained through the field survey and data collection from those two Local Government Units and DPWH Region VIII, storm water drainage along the newly developed tide embankment is planned.

4.4.1 Study of Existing Conditions

Existing conditions of drainage outlet points along the newly developed tide embankment in the section 3 and 4 in Tacloban City and the Palo Municipality are studied through the field survey, data collection from those two LGUs.

Results of these studies are as follows;

(1) Existing Drainage Outlet Points

1) Field Survey of the Section 3 by JICA Study Team

Field survey of section 3 was conducted from May 12 to 16, 2015 by the JICA Study Team.

2) Field Survey of Section Four (4) by DPWH and JICA Study Team

Joint field survey between DPWH and JICA Study Team on drainage current condition in the section 4 was conducted on 27th and 28th May 2015 in order to confirm the location, shape and size of river mouth and drainage channel outlet.

3) Data Collection from the Tacloban City

Vicinity map and drawings of the Citywide Flood Control and Drainage System Project of the Tacloban City was provided by the Tacloban City Engineer’s Office. However, the Tacloban City explained that the planning and design documents related to the project were washed away by the Typhoon Yolanda. Therefore, the details of the project and the current drainage system in the Tacloban City were not confirmed and obtained by the DPWH TWG and the JICA Study Team.

Locations of 3 existing drainage outlets were provided by the Tacloban City on June 11, 2015.

4) Data Collection from the Palo Municipality

There is no drainage master plan in the Palo Municipality.

Data regarding existing drainage location was provided by the Palo Municipality on 16 June 2015.

5) Result of existing drainage outlet of Section 3 and 4

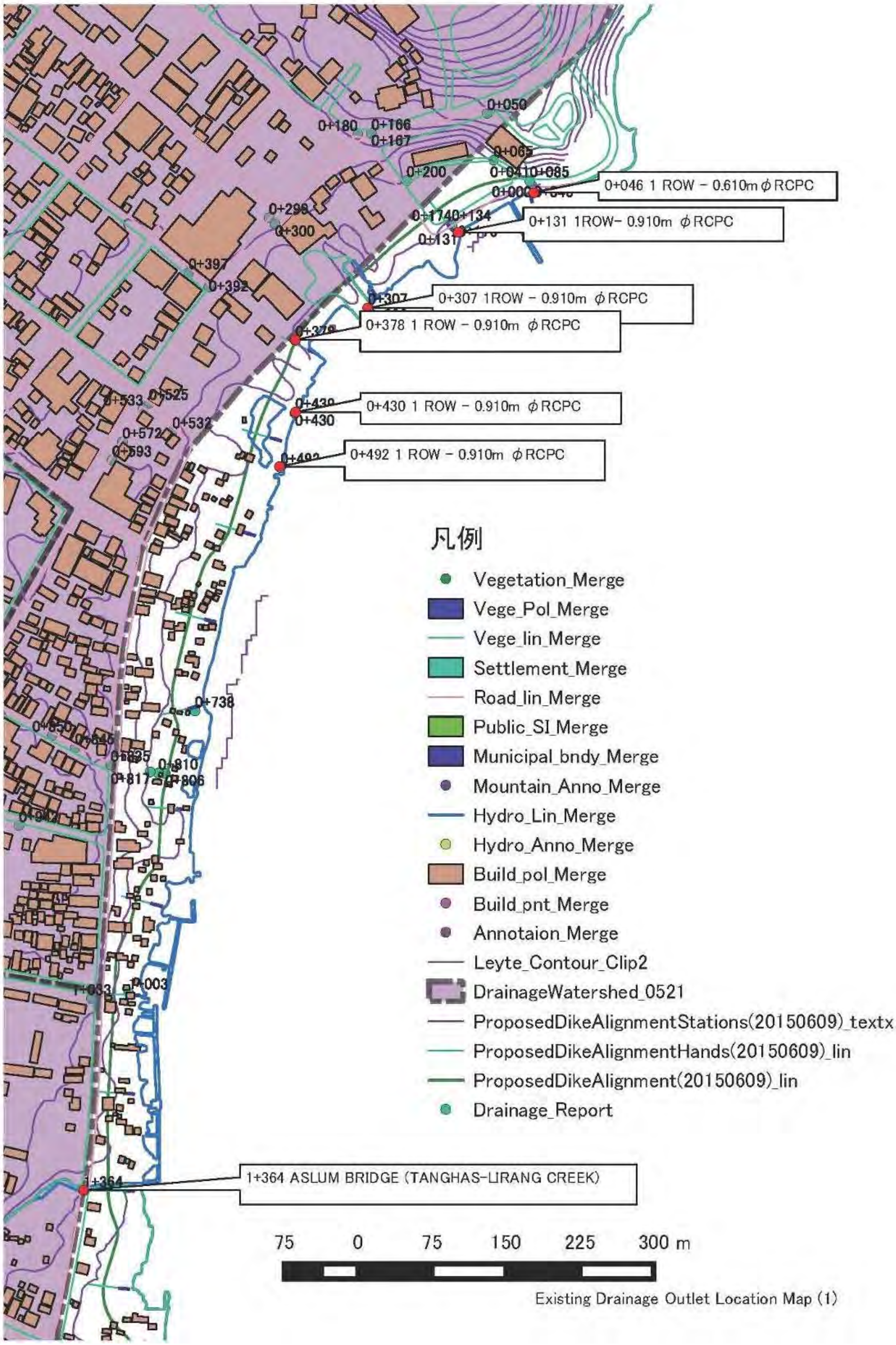
Based on the field survey conducted by DPWH and JICA Study Team on drainage current condition in the section 3 and 4, collected data from the Tacloban City and the Palo Municipality, list of existing drainage outlets and their locations are shown in the following table and figures.

Table 4.4-1 List of the existing drainage outlets of section 3 and 4

NO.	SEC.	LGU	STATION	STRUCTURE	STATUS	REMARKS
1	3-1	TC	0+046	1 ROW - 0.610 m Ø RCPC	ACTIVE	AT BALYUAN SITE
2	3-1	TC	0+131	1 ROW - 0.910 m Ø RCPC	ACTIVE	ALONG REAL STREET
3	3-1	TC	0+307	1 ROW - 0.910 m Ø RCPC	ACTIVE	ALONG REAL STREET
4	3-1	TC	0+378	1 ROW - 0.910 m Ø RCPC	ACTIVE	ALONG REAL STREET
5	3-1	TC	0+430	1 ROW - 0.910 m Ø RCPC	ACTIVE	
6	3-1	TC	0+492	1 ROW - 0.910 m Ø RCPC	ACTIVE	OUTLET IN PAMPANGO
7	3-1	TC	1+364	**ASLUM BRIDGE**	ACTIVE	TANGHAS LIRANG CREEK
8	3-1	TC	1+930	1 ROW - 0.910 m Ø RCPC	ACTIVE	EXISTING WITH HEADWALLS
9	3-1	TC	2+123	1 ROW - 0.910 m Ø RCPC	ACTIVE	
10	3-1	TC	2+393	BOX CULVERT	ACTIVE	SAGKAHAN CREEK
11	3-1	TC	2+441	1 ROW - 0.910 m Ø RCPC	ACTIVE	
12	3-1	TC	2+562	1 ROW - 0.910 m Ø RCPC	ACTIVE	BACK OF ASTRODOME
13	3-1	TC	2+681	1 ROW - 0.910 m Ø RCPC	ACTIVE	AT ASTRODOME - ALONG REAL STREET
14	3-2	TC	2+877	1 ROW - 0.910 m Ø RCPC	ACTIVE	ALONG REAL STREET
15	3-2	TC	2+922	1 ROW - 0.910 m Ø RCPC	ACTIVE	ALONG REAL STREET
16	3-2	TC	3+100	BOX CULVERT	ACTIVE	MAHAYAHAY CREEK
17	3-3	TC	4+382	**BURAYAN BRIDGE**	ACTIVE	BURAYAN RIVER
18	3-3	TC	5+493	OPEN LINED CANAL 0.9m(W)x0.8m(H)	ACTIVE	
19	3-3	TC	5+422	1 ROW - 0.910 m Ø RCPC	ACTIVE	
20	3-3	TC	9+702	1 ROW - 0.910 m Ø RCPC	ACTIVE	
21	3-3	TC	9+748	1 ROW - 0.910 m Ø RCPC	ACTIVE	
22	4-1	TC		BOX CULVERT 2m(W)x0.5m(H)	CLOGGED	
23	4-1	TC		Swamp (1)		
24	4-2	TC		Swamp (2)		
25	4-2	TC		Swamp (3)		
26	4-2	PL		Swamp (4)		
27	4-2	PL		Swamp (5)		
28	4-2	PL		Swamp (6)		
29	4-3	PL	8+850	**PAYAPAY BRIDGE**	ACTIVE	KILOT CREEK
30	4-3	PL	9+222	2 BARREL - 2.5m(W)x1.8m(H)x15m(L) BOX CULVERT	ACTIVE	BINOG CREEK
31	4-4	PL	10+150	1 BARREL - 2.7m(W)x1.7m(H)x12.80m(L) BOX CULVERT	ACTIVE	
32	4-5	PL		RCPC	ACTIVE	
33	4-5	PL		RCPC	ACTIVE	
34	4-5	PL	10+893	Swamp (BOX CULVERT in the upstream)	ACTIVE	
35	4-7	PL		CHANNEL FROM A FISH POND	ACTIVE	LEFT BANK OF BANGON RIVER
36	NA	PL		CONJUNCTION OF A TRIBUTARY	ACTIVE	LEFT BANK OF BANGON RIVER
37	NA	PL		NATURAL CHANNEL FROM A SWAMP		RIGHT BANK OF BANGON RIVER
38	NA	PL		1 ROW -0.6m φRCPC	ACTIVE	RIGHT BANK OF BANGON RIVER
39	NA	PL		1 ROW - 0.910 m Ø RCPC	ACTIVE	RIGHT BANK OF BANGON RIVER
40	NA	PL		1 ROW - 0.910 m Ø RCPC	ACTIVE	RIGHT BANK OF BANGON RIVER UNDER BRIDGE
41	NA	PL		1 ROW - 0.910 m Ø RCPC	ACTIVE	RIGHT BANK OF BANGON RIVER
42	NA	PL		1 ROW - 0.910 m Ø RCPC	ACTIVE	LEFT BANK OF BANGON RIVER

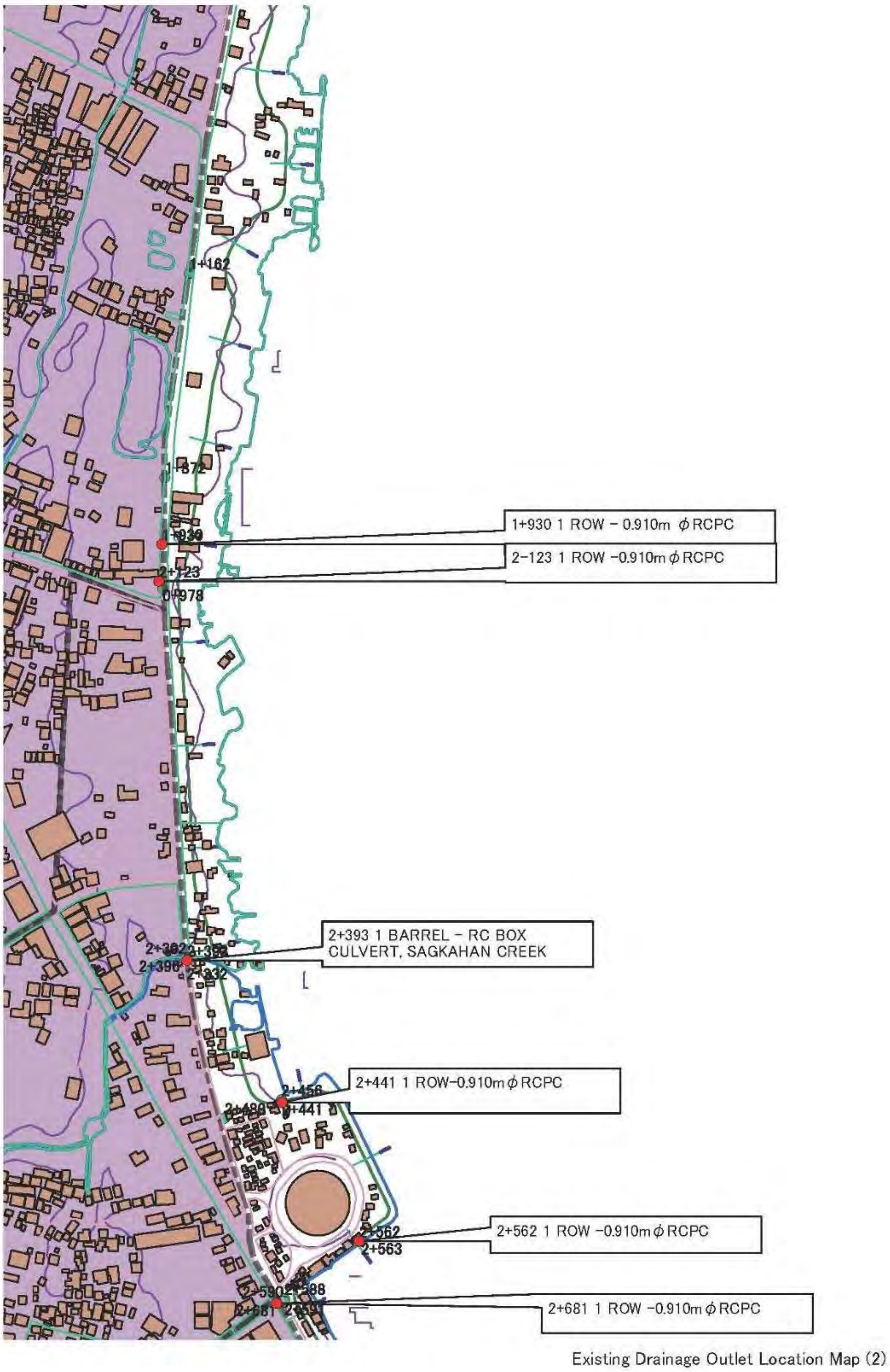
* TC: Tacloban, PL: Palo

Source: JICA Study Team



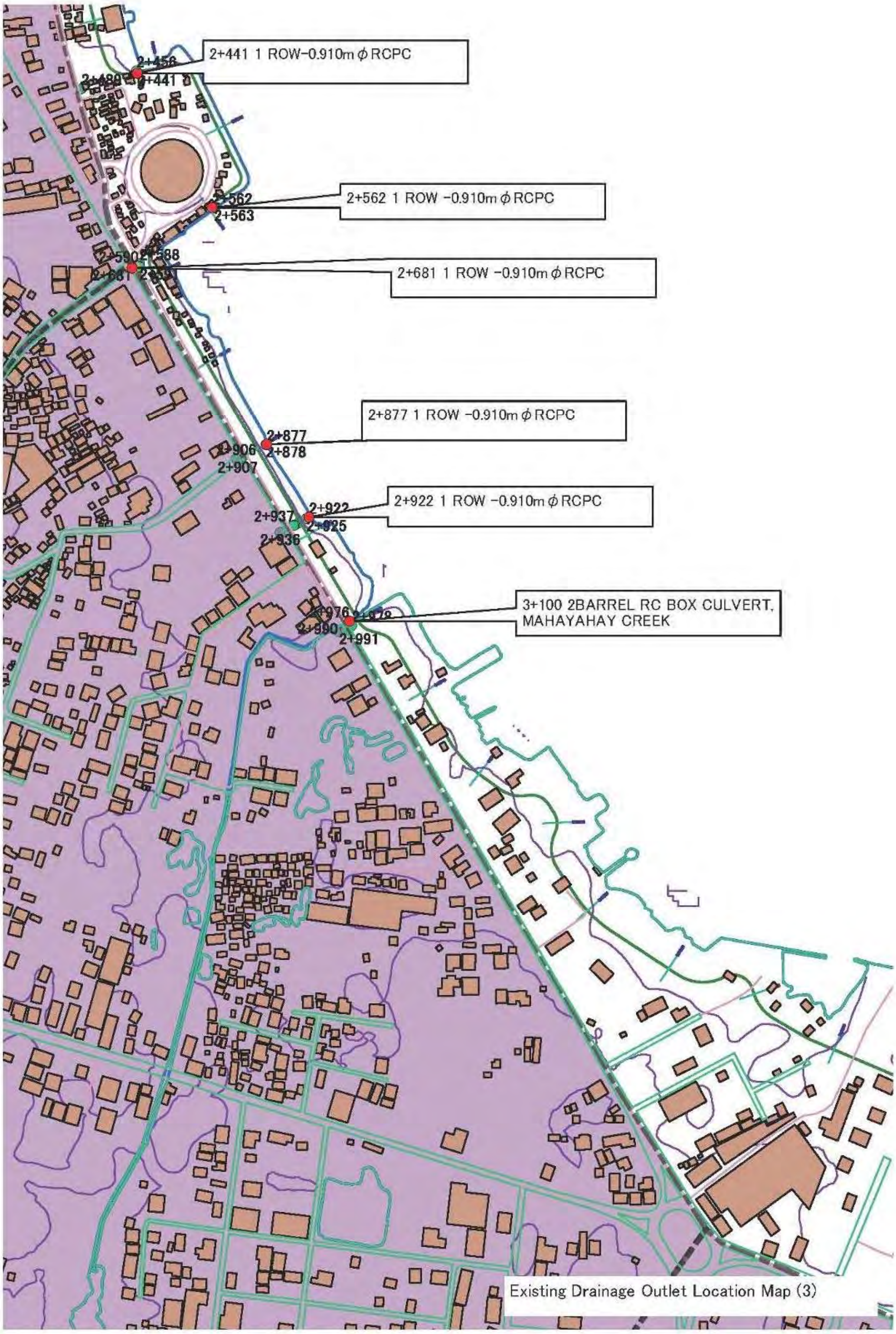
Source: JICA Study Team

Figure 4.4-1 Location of existing drainage outlets (1)



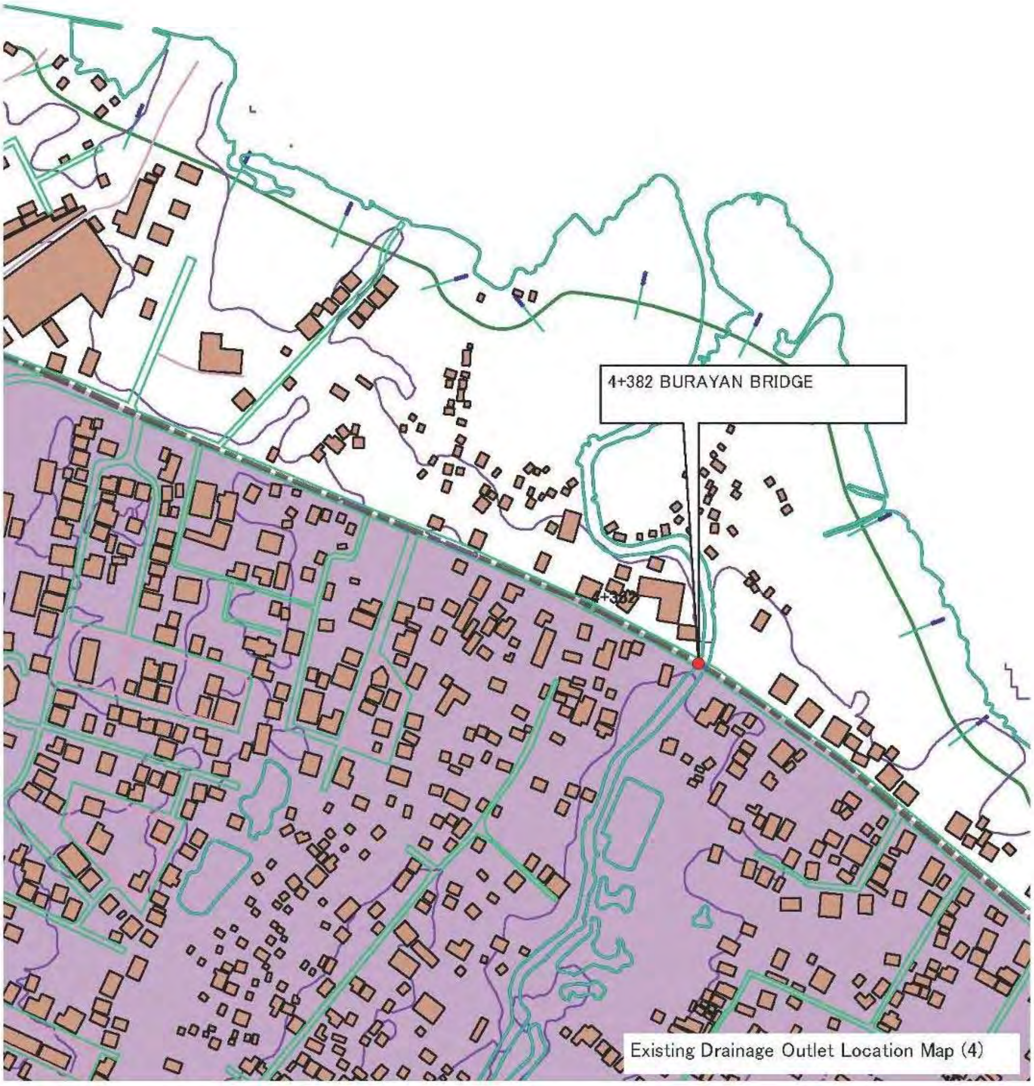
Source: JICA Study Team

Figure 4.4-2 Location of existing drainage outlets (2)



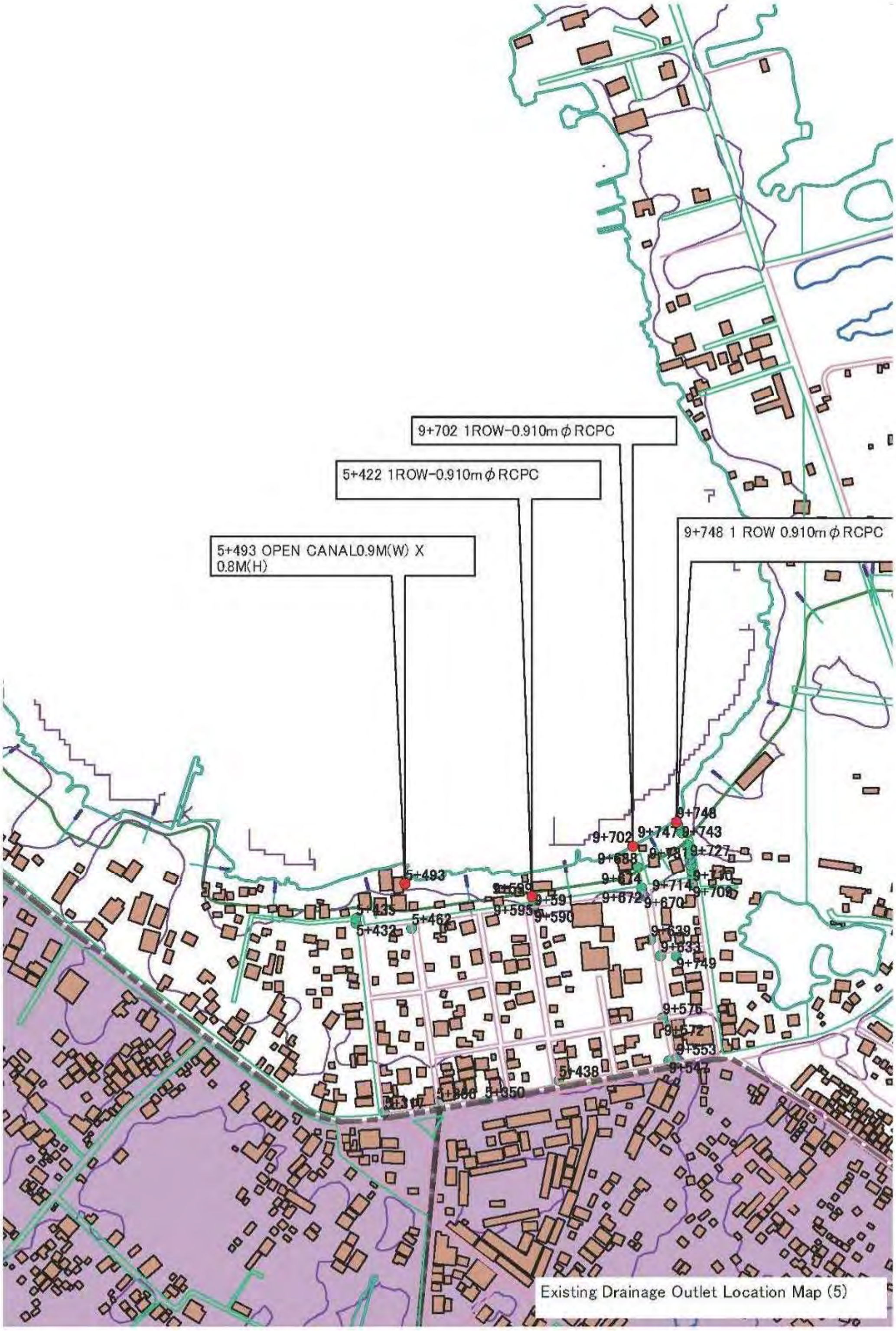
Source: JICA Study Team

Figure 4.4-3 Location of existing drainage outlets (3)



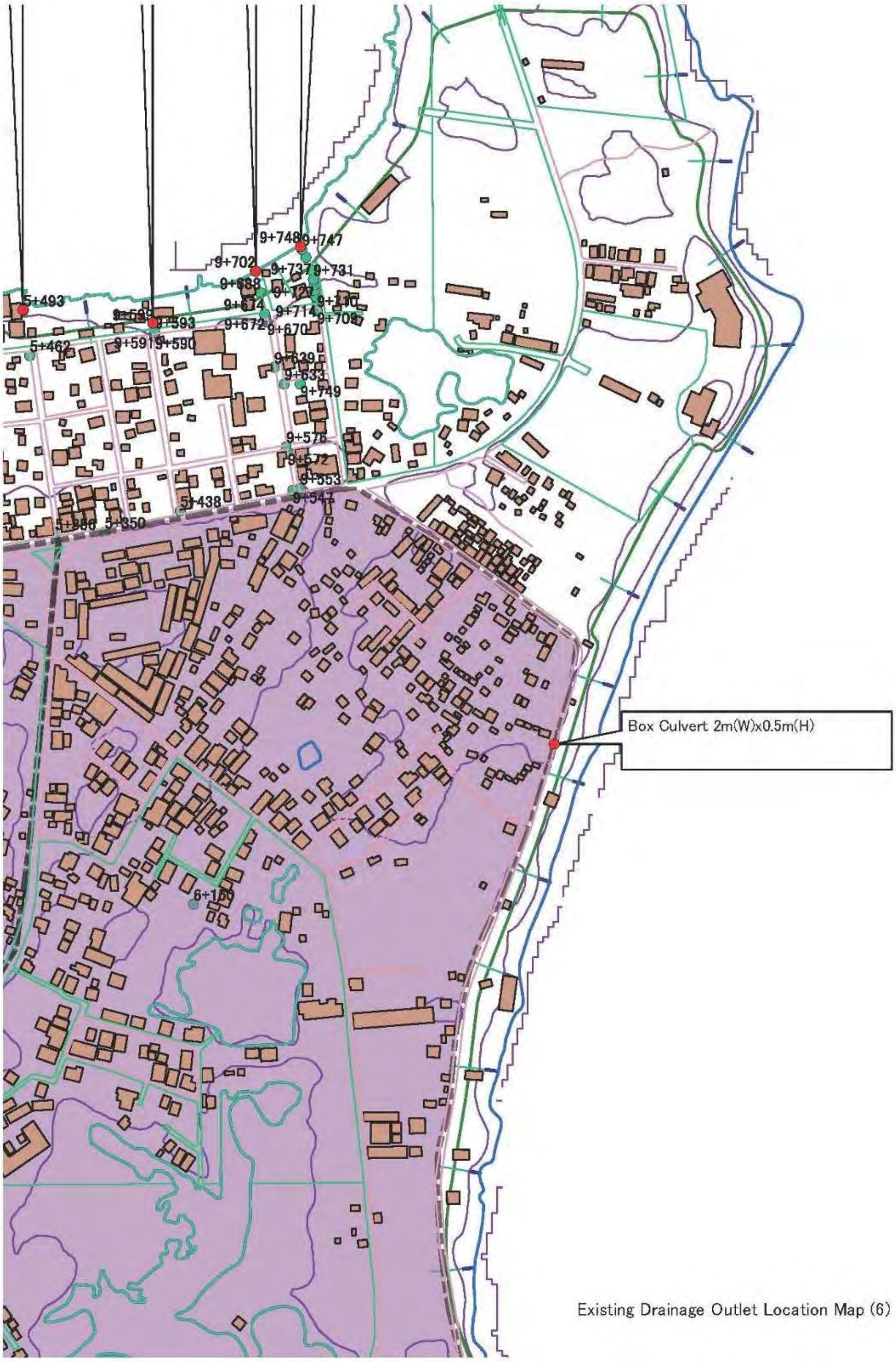
Source: JICA Study Team

Figure 4.4-4　Location of existing drainage outlets (4)



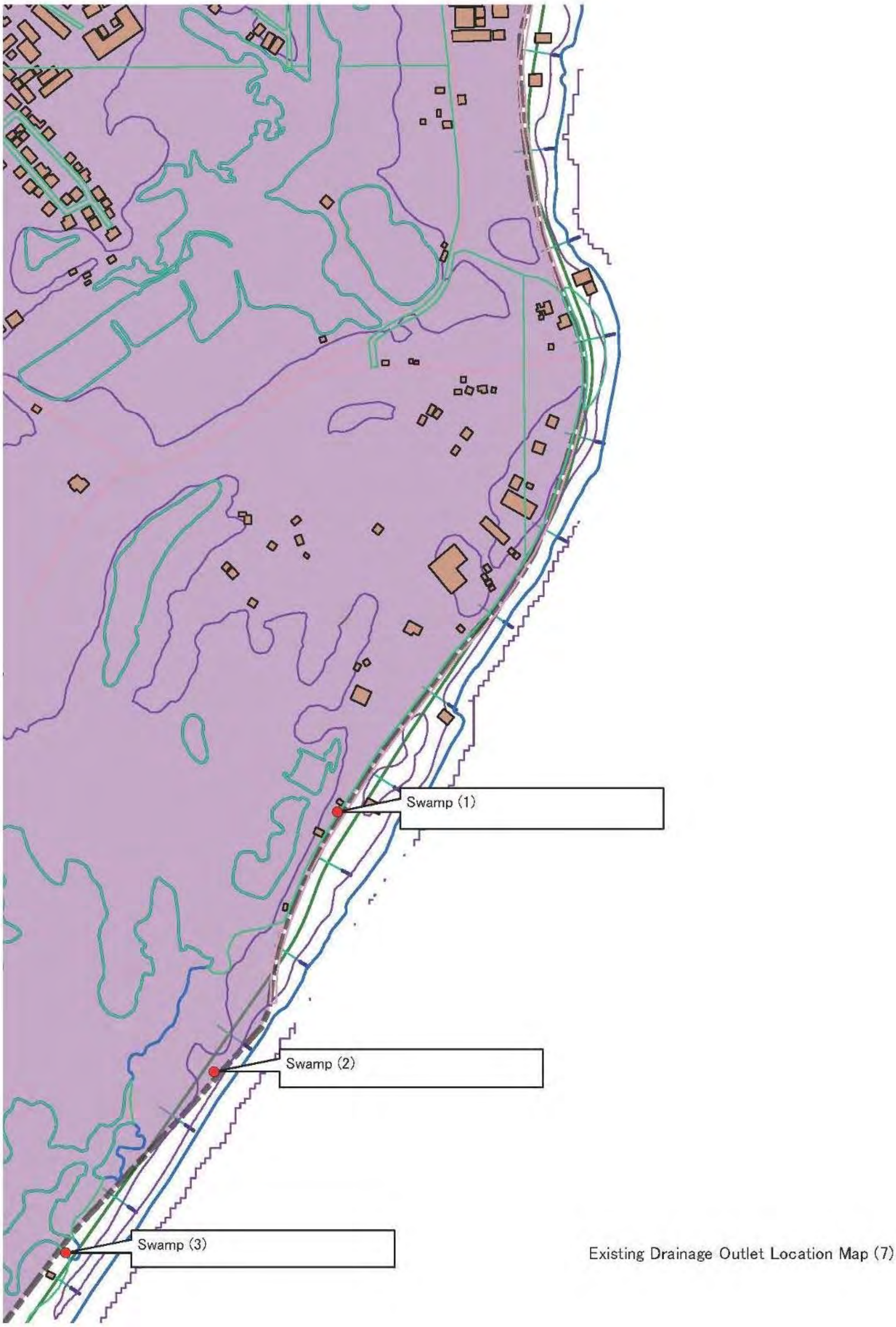
Source: JICA Study Team

Figure 4.4-5 Location of existing drainage outlets (5)



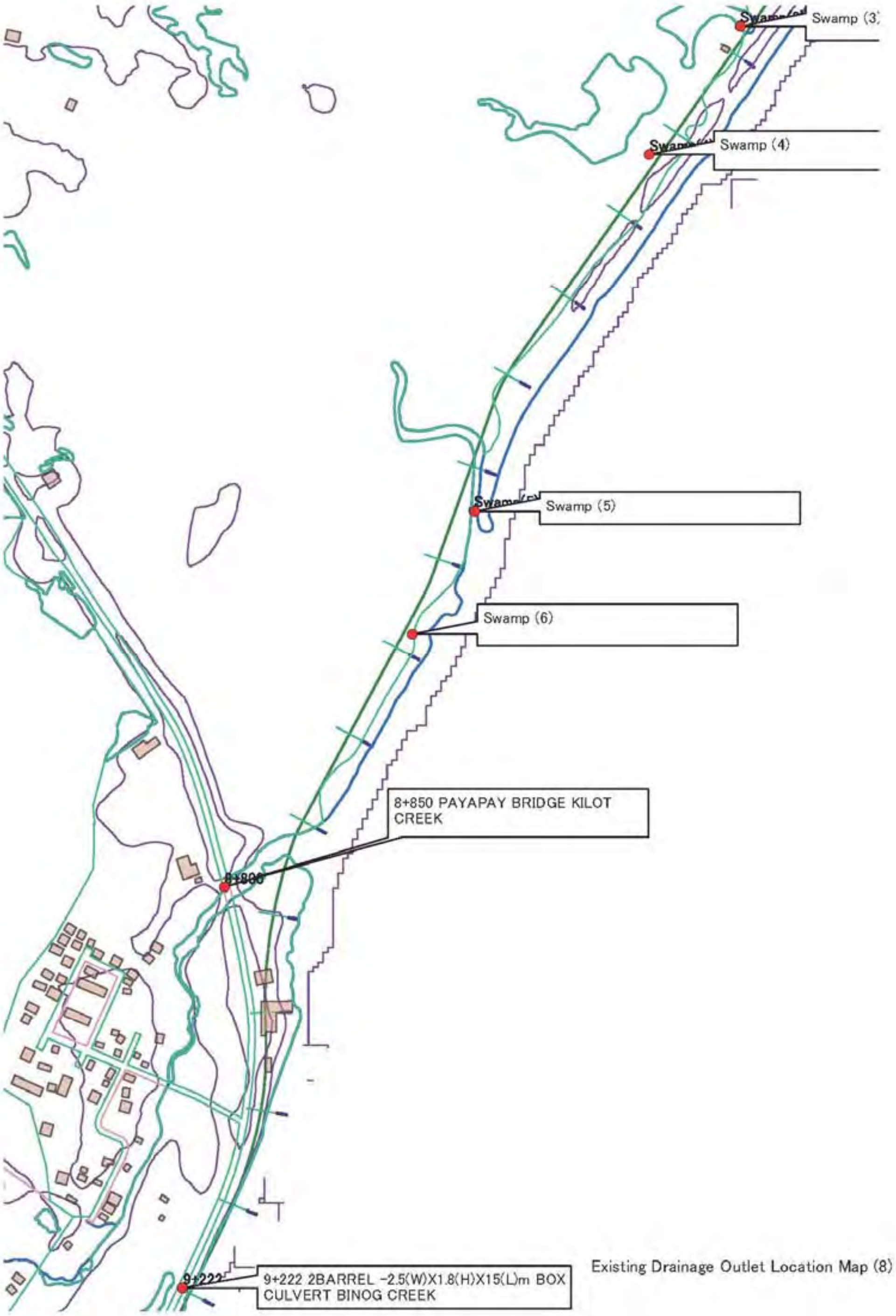
Source: JICA Study Team

Figure 4.4-6 Location of existing drainage outlets (6)



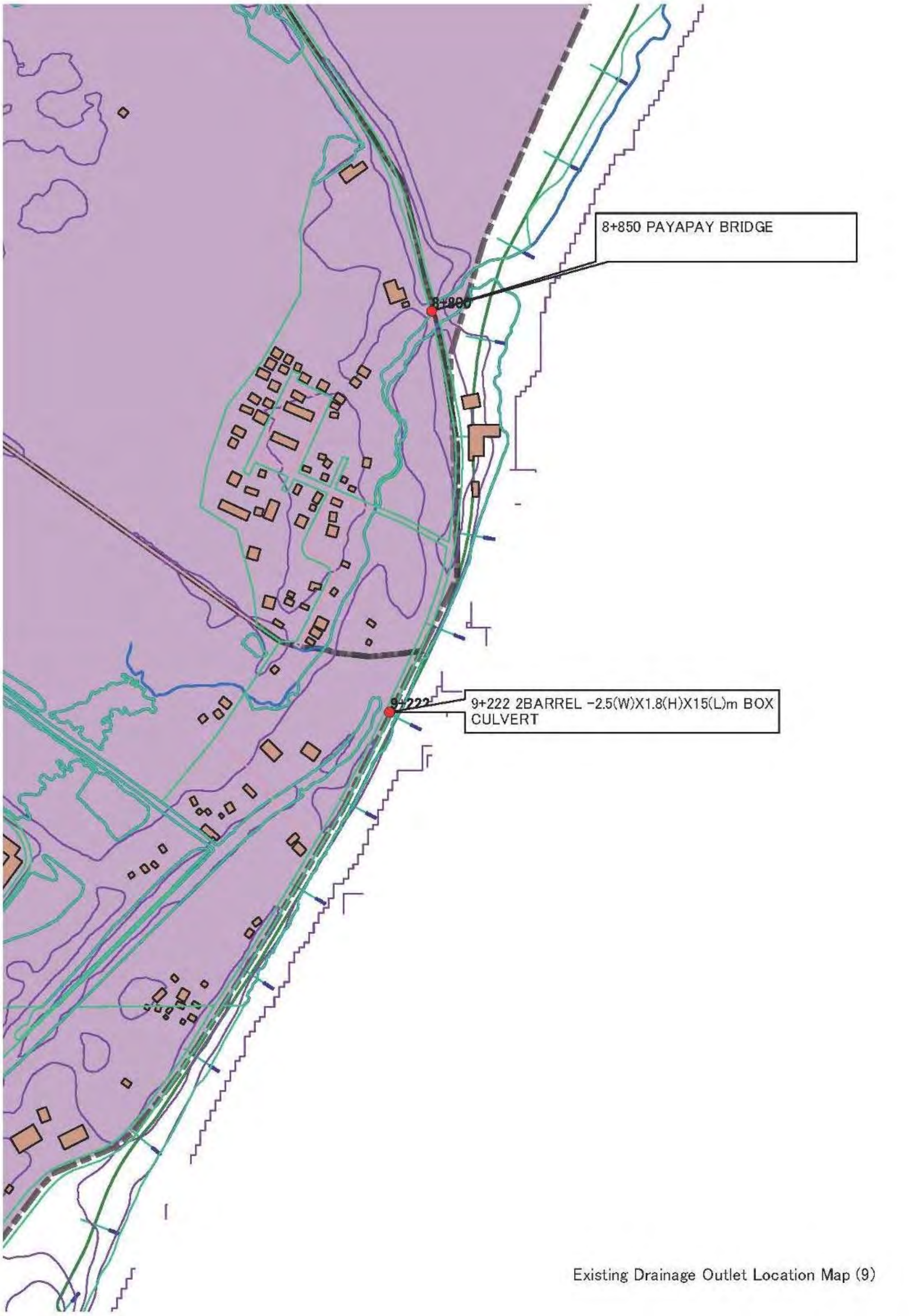
Source: JICA Study Team

Figure 4.4-7 Location of existing drainage outlets (7)



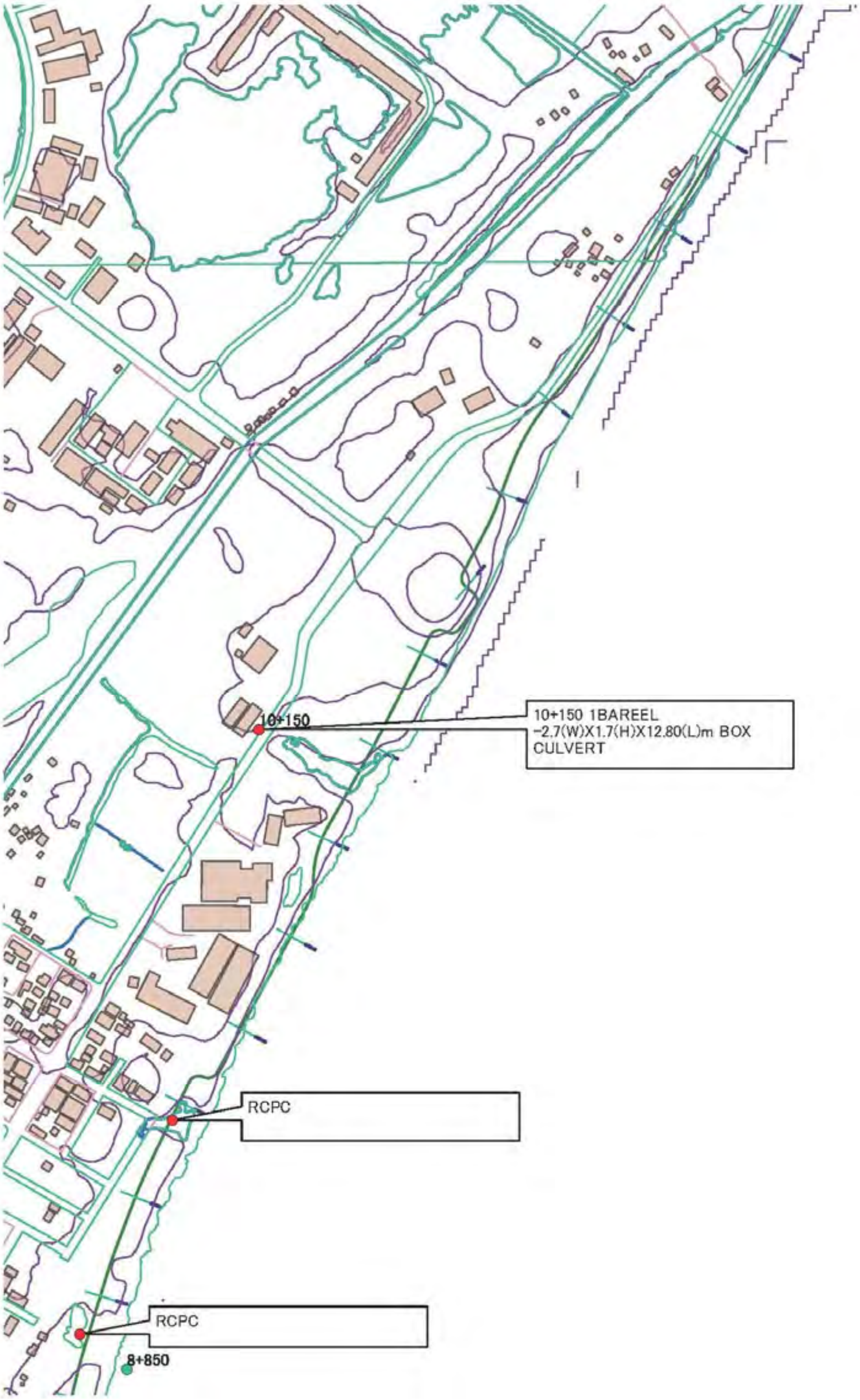
Source: JICA Study Team

Figure 4.4-8 Location of existing drainage outlets (8)



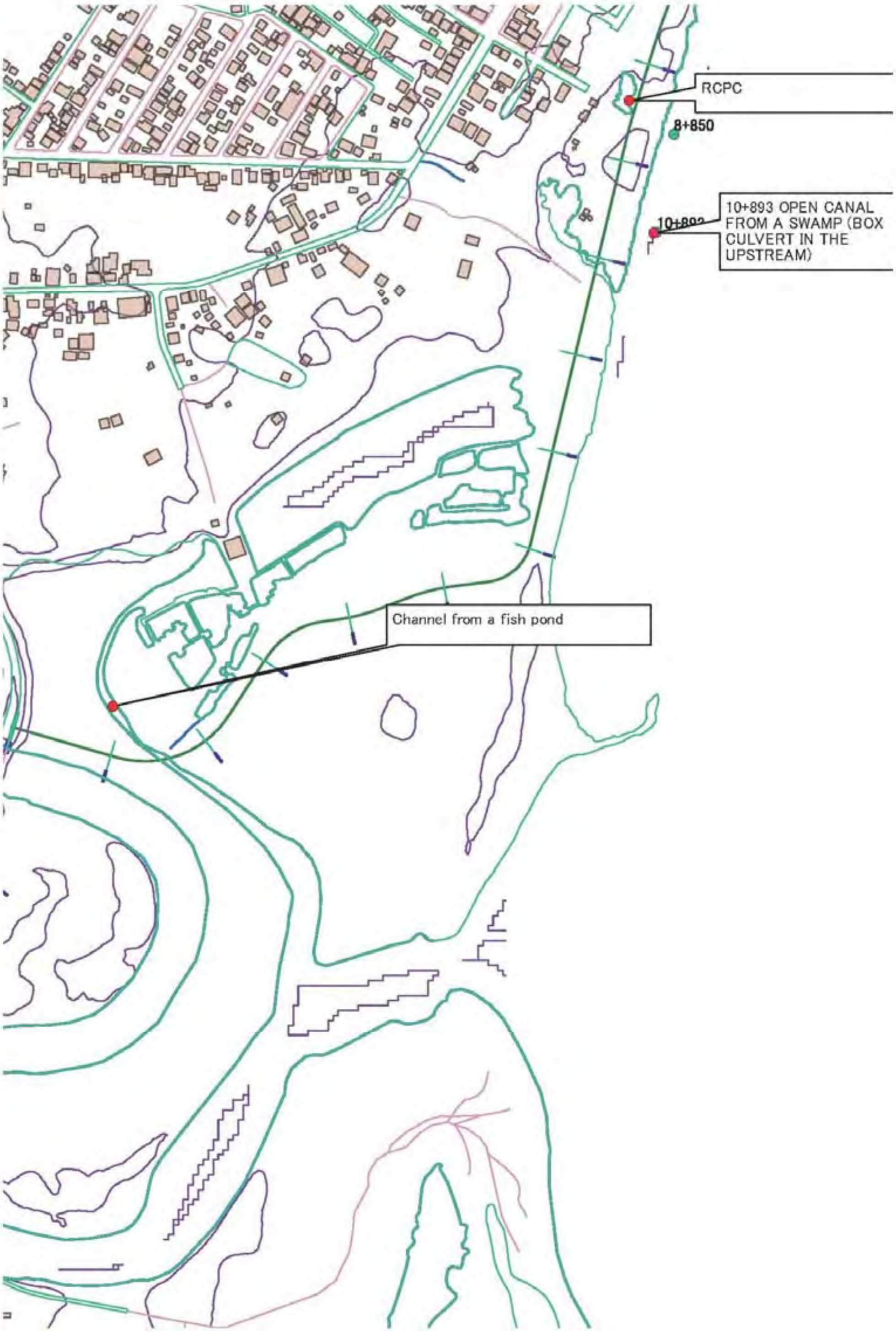
Source: JICA Study Team

Figure 4.4-9 Location of existing drainage outlets (9)



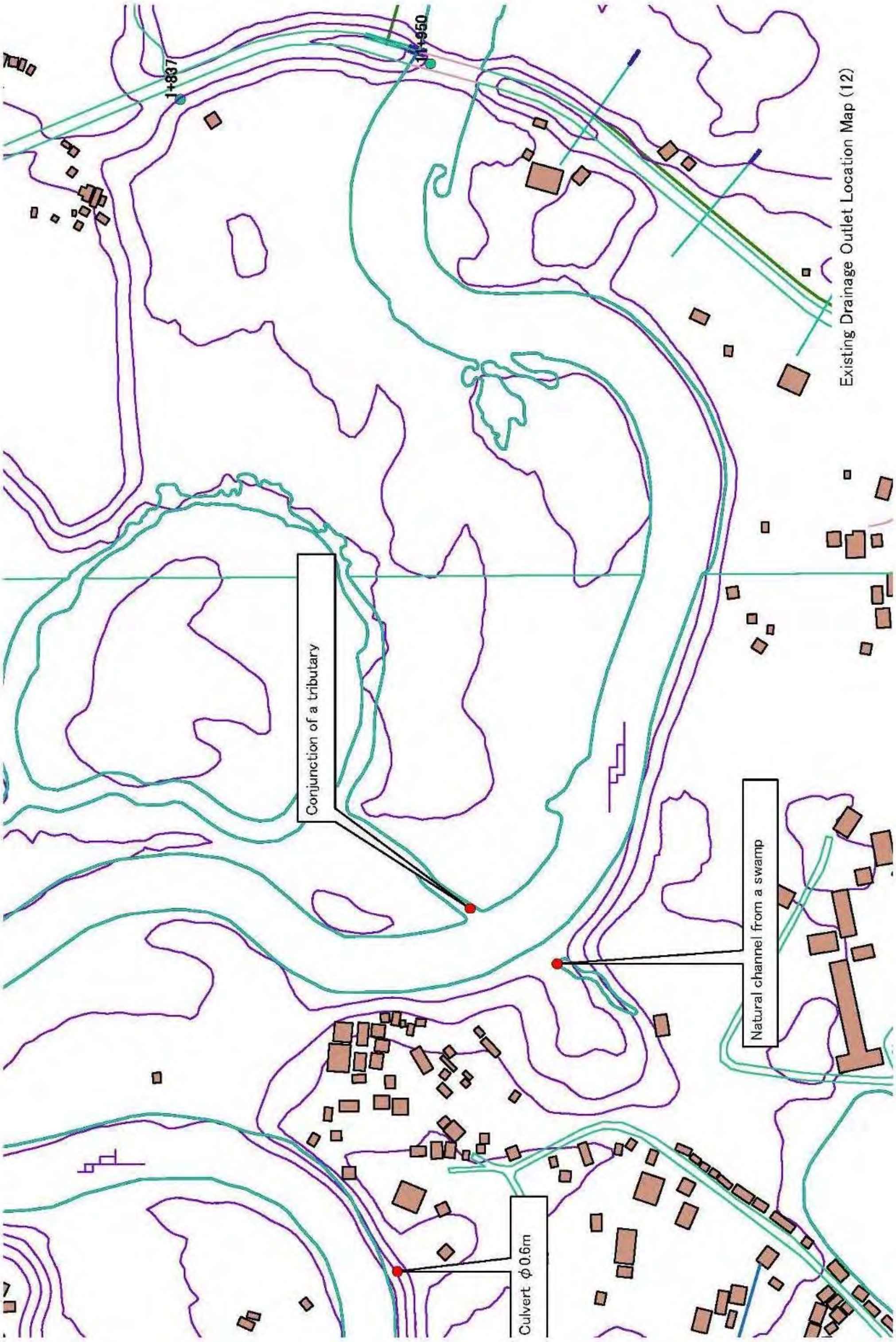
Source: JICA Study Team

Figure 4.4-10 Location of existing drainage outlets (10)



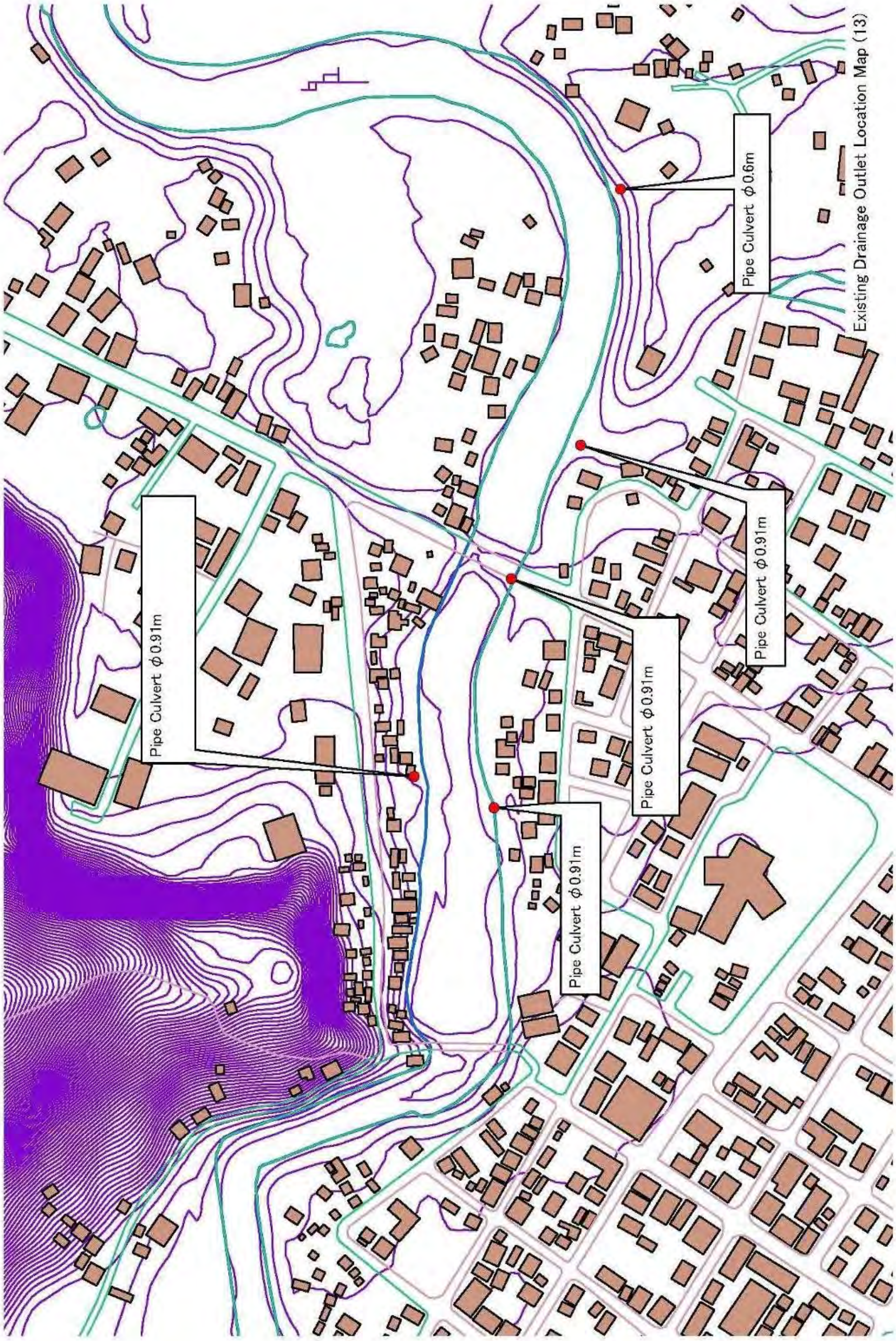
Source: JICA Study Team

Figure 4.4-11 Location of existing drainage outlets (11)



Source: JICA Study Team

Figure 4.4-12   Location of Existing Drainage Outlets (12)



Source: JICA Study Team

Figure 4.4-13 Location of Existing Drainage Outlets (13)

(2) Evaluation of the Existing Drainage Capacity

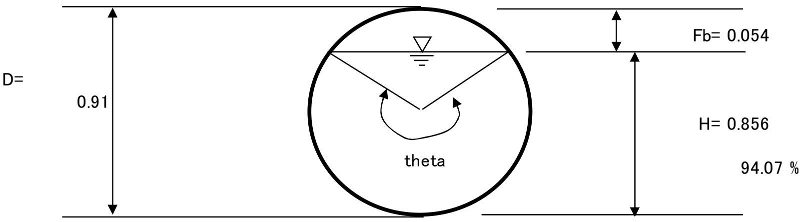
Uniform flow calculation method was applied for evaluation of the existing drainage flow capacity.

管渠断面流下能力水理計算　常流
Maximum flow calculation in pipe ordinary flow
（マニング平均流速公式による）
(using Manning's formula)

(1) 計算条件	Condition for calculation				
水路勾配	Channel slope	I	=	1/ 300.000	
管径	Pipe diameter	D	=	0.910	m
粗度係数	Coefficient of roughness	n	=	0.013	

(2) 計算結果	Result of calculation				
最大流量	Maximum flow	Qmax	=	1.161	cum
水深	Water depth	H	=	0.856	m
流積	Cross-section area of stream	A	=	0.635	sqm
潤辺	Cross-section length of strea	P	=	2.409	m
径深	Hydraulic radius	R	=	0.264	m
流速	Flow velocity	V	=	1.828	m/s
余裕高	Allowance height	Fb	=	0.054	m
フルード数	Froude number	Fr	=	0.631	< 1 (常流) (ordinary flow)
余裕部分の流積に対する割合	Rate of allowance area to cross-section area of stream			2.371	%

(3) 断面形状　Cross-section shape



(4) 計算
流量が最大となるthetaを試算により求め、各数値を計算する。

水深	Water depth			
		theta =	5.29553 rad	303.4115 degrees
		H =	D/2x(1-cos(theta/2))	
		=	0.910 /2x(1-cos(5.29553 /2))	
		=	0.856 m	
流積	Cross-section area of stream			
		A =	1/8x(theta-sin(theta))xD^2	
		=	1/8x(5.29553007 -sin 5.296)x 0.910 ^2	
		=	0.635 sqm	
潤辺	Cross-section length of stream			
		P =	D/2xtheta	
		=	0.910 /2x 5.29553	
		=	2.409 m	
径深	Hydraulic radius			
		R =	A/P	
		=	0.635 / 2.409	
		=	0.264 m	
流速	Flow velocity			
		V =	1/n x I^(1/2) x R^(2/3)	
		=	1/ 0.013 x(1/ 300)^(1/2)x 0.264 ^(2/3)	
		=	1.828 m/s	
最大流量	Maximum flow			
		Qmax =	A x V	
		=	0.635 x 1.828	
		=	1.161 cum/s	
余裕高	Allowance height			
		Fb =	D - H	
		=	0.910 - 0.856	
		=	0.054 m	
フルード数	Froude number			
		Fr =	V/(g x H)^(1/2)	
		=	1.828 /(9.8 x 0.856)^(1/2)	
		=	0.631	

(g m/sqs: 重力加速度)
(g m/sqs: gravitational acceleration)

管渠断面流下能力水理計算　　常流
Maximum flow calculation in pipe ordinary flow
（マニング平均流速公式による）
(using Manning's formula)

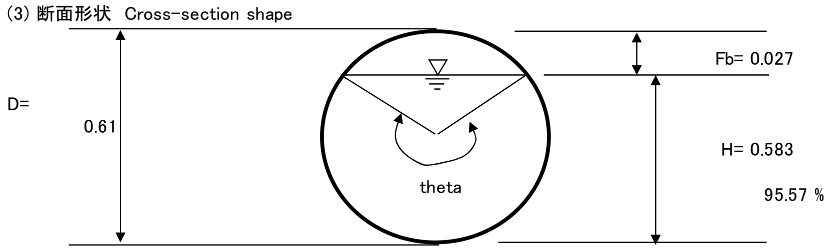
(1) 計算条件　Condition for calculation

水路勾配	Channel slope	I	=	1/	300.000
管径	Pipe diameter	D	=		0.610 m
粗度係数	Coefficient of roughness	n	=		0.013

(2) 計算結果　Result of calculation

最大流量	Maximum flow	Qmax	=	0.399 cum		
水深	Water depth	H	=	0.583 m		
流積	Cross-section area of stream	A	=	0.288 sqm		
潤辺	Cross-section length of strea	P	=	1.659 m		
径深	Hydraulic radius	R	=	0.174 m		
流速	Flow velocity	V	=	1.384 m/s		
余裕高	Allowance height	Fb	=	0.027 m		
フルード数	Froude number	Fr	=	0.579	<	1 (常流) (ordinary flow)
余裕部分の流積に対する割合	Rate of allowance area to cross-section area of stream			1.423 %		

(3) 断面形状　Cross-section shape



(4) 計算
流量が最大となるthetaを試算により求め、各数値を計算する。

水深　Water depth

theta = 5.440893 rad　　311.7402 degrees
H = D/2x(1-cos(theta/2))
= 0.610 /2x(1-cos(5.440893 /2))
= 0.583 m

流積　Cross-section area of stream

A = 1/8x(theta-sin(theta))xD^2
= 1/8x(5.44089302 -sin 5.441)x 0.610 ^2
= 0.288 sqm

潤辺　Cross-section length of stream

P = D/2xtheta
= 0.610 /2x 5.440893
= 1.659 m

径深　Hydraulic radius

R = A/P
= 0.288 / 1.659
= 0.174 m

流速　Flow velocity

V = 1/n x I^(1/2) x R^(2/3)
= 1/ 0.013 x(1/ 300)^(1/2)x 0.174 ^(2/3)
= 1.384 m/s

最大流量　Maximum flow

Qmax = A x V
= 0.288 x 1.384
= 0.399 cum/s

余裕高　Allowance height

Fb = D - H
= 0.610 - 0.583
= 0.027 m

フルード数　Froude number

Fr = V/(g x H)^(1/2)
= 1.384 /(9.8 x 0.583)^(1/2)
= 0.579

(g m/sqs: 重力加速度)
(g m/sqs: gravitational acceleration)

4.4.2 Analysis of Discharge Flow and Allocation of Drainage Outlet

(1) Policy of Allocation of Drainage Outlets

Basic policy of drainage outlet allocation is to compensate the current function of drainage outlet. And there are four options for allocation of drainage outlets as follows.

- **Option 1: To keep the location and design discharge of existing outlets for the newly installed ones**
- Option 2: To keep the location of the existing outlets for the new ones, but to increase the design discharge for the newly installed ones
- Option 3: To combine some existing outlets into new one
- Option 4: To add an outlet for each existing swamp area/ pond along the shoreline and the new tide embankment

Considering the basic policy, the option 1 shall be considered at the first.

And the option 2 and 3 can be considered if concerned Local Government Unit has a concrete future drainage improvement plan. However, when some existing outlets will be combined into one, the design discharge might be increased and the dimension of the outlet will be larger than as it is. In this case, maximum flap gate size procured in the Philippines local market might be a restriction.

(2) Necessary Function

The planned drainage outlet should have a function to prevent backwater into landside when the storm surge in addition to drain the rainwater from landside to sea side.

(3) Design Discharge for the Drainage Pipe Outlets

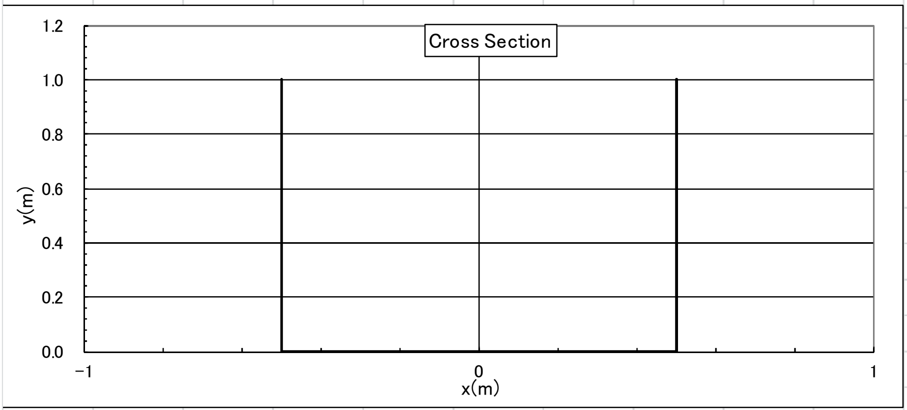
The design discharge for the planned drainage pipe outlet shall be equal to or more than the existing one based on the basic policy and the option 1 mentioned above.

The flow capacity for the existing largest drainage RCPCs was calculated using uniform flow calculation method and the result was 1.161 m3/s.

Rectangular Box Culvert with 1.0 meter width and 1.0 meter height can accommodate this flow discharge by using a uniform flow calculation method. The calculation sheet is shown below.

Uniform Flow Calculation Sheet

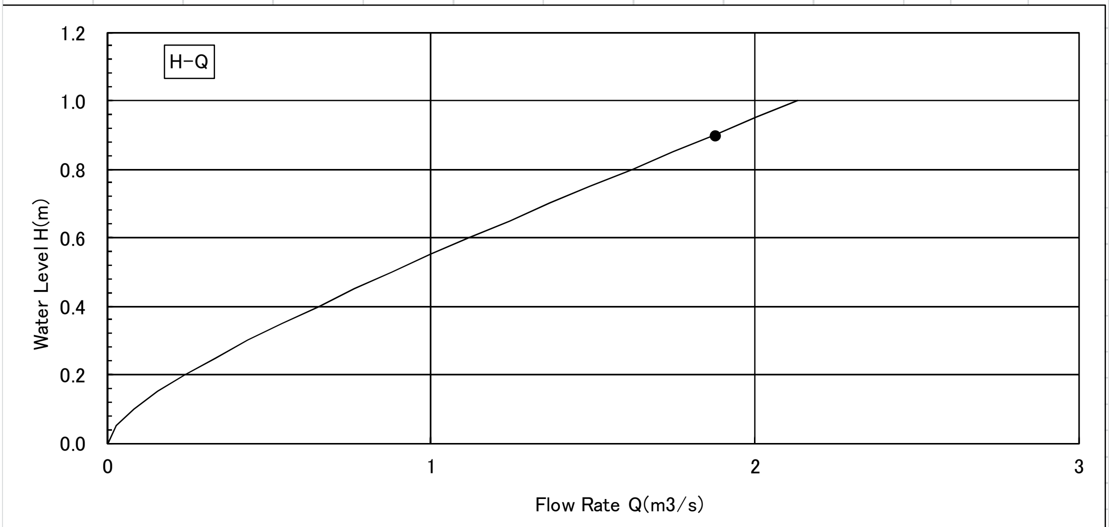
Section	Slope	Roughness	Lowest Bed EL.	No of stations
NO.1	1/ 300	0.013	0.000 m	1～ 5



No	X	Y
1	−0.50	1.00
2	−0.50	0.00
3	0.00	0.00
4	0.50	0.00
5	0.50	1.00
6		
7		
8		
9		
10		
11		
12		
13		
14		
15		
16		
17		
18		
19		
20		
21		
22		
23		
24		
25		
26		
27		
28		
29		
max	0.50	1.00

Hydraulic Conditions　Δh1= 0.05　Δh2= 0.05　Δh3= 0.05

W Level H(m)	W Depth h(m)	Area A(m2)	H Radius R(m)	Width B(m)	Velocity V(m/s)	Flow Rate Q(m3/s)	Wetted P S(m)	Froude Fr	Remarks
0.00	0.00	0.000	0.000	0.000	0.000	0.00	0.000	0.000	
0.05	0.05	0.050	0.045	1.000	0.566	0.03	1.100	0.808	
0.10	0.10	0.100	0.083	1.000	0.847	0.08	1.200	0.856	
0.15	0.15	0.150	0.115	1.000	1.053	0.16	1.300	0.868	
0.20	0.20	0.200	0.143	1.000	1.214	0.24	1.400	0.867	
0.25	0.25	0.250	0.167	1.000	1.345	0.34	1.500	0.859	
0.30	0.30	0.300	0.187	1.000	1.455	0.44	1.600	0.849	
0.35	0.35	0.350	0.206	1.000	1.548	0.54	1.700	0.836	
0.40	0.40	0.400	0.222	1.000	1.629	0.65	1.800	0.823	
0.45	0.45	0.450	0.237	1.000	1.700	0.77	1.900	0.810	
0.50	0.50	0.500	0.250	1.000	1.762	0.88	2.000	0.796	
0.55	0.55	0.550	0.262	1.000	1.818	1.00	2.100	0.783	
0.60	0.60	0.600	0.273	1.000	1.868	1.12	2.200	0.770	
0.65	0.65	0.650	0.283	1.000	1.913	1.24	2.300	0.758	
0.70	0.70	0.700	0.292	1.000	1.953	1.37	2.400	0.746	
0.75	0.75	0.750	0.300	1.000	1.990	1.49	2.500	0.734	
0.80	0.80	0.800	0.308	1.000	2.024	1.62	2.600	0.723	
0.85	0.85	0.850	0.315	1.000	2.055	1.75	2.700	0.712	
0.90	0.90	0.900	0.321	1.000	2.084	1.88	2.800	0.702	
0.95	0.95	0.950	0.328	1.000	2.110	2.00	2.900	0.692	
1.00	1.00	1.000	0.333	1.000	2.135	2.14	3.000	0.682	FULL
0.90	0.90	0.900	0.321	1.000	2.084	1.88	2.800	0.702	HWL



(4) Determination of Gate Type

1) **Flap Gate**

Considering the ease of operation of the gate, flap gate is the most appropriate gate type for prevention of entering sea water to the inland side. However, the maximum size of flap gate procured in the Philippines local market should be researched and considered.

2) **Lift Gate**

If the outlet dimension is larger than the maximum size of flap gate, lift gate should be considered.

Based on the analysis mentioned in “4.3.1 Selection for the River Crossing Structure (River Gate/ Backwater Dike)”, following 6 river/creek outlets along the tide embankment will be considered as lift gate (river gate);

・ Tanghas-Lirang Creek

・ Sagkahan Creek

・ Mahayahay Creek

・ Burayan River

・ Kilot Creek at Payapay Bridge

・ Binog Creek with existing box culvert located at the south of Payapay Bridge

Besides these six rivers/creeks, river gates at the following three rivers/creeks/channels shall be constructed.

・ Binog Creek with existing box culvert located near Mac Arthur Park

・ Channel from a fish pond on the left bank of Bangon River

・ Conjunction of a tributary on the left bank of Bangon River

a) Binog Creek with existing box culvert located near Mac Arthur Park

This box culvert was installed in case flood water of Binog Creek could not be accommodated through the existing 2 barrel box culvert. Therefore, existing opening size shall be kept as it is.

b) Channel from a fish pond on the left bank of Bangon River

Purpose of this channel is to exchange water for fish pond from the river mouth of Bangon River. And this channel outlet is located along the planned tide embankment. Therefore, the width of it shall be kept as it is. However, the exact width of this channel was not confirmed yet because of a thick/ dense forest. Based on the satellite image, the width of it was roughly estimated around 6 meters. Therefore, river gate should be constructed on this channel.

c) Conjunction of a tributary on the left bank of Bangon River

This conjunction of a tributary is located on the left bank of Bangon River which is now under construction of flood protection dike by DPWH. Therefore, a river gate is recommended to construct by DPWH.

(5) Basic Plan of Drainage Outlet (Number, Location, Dimension and Gate Type)

List of the planned drainage outlets in Section 3 and 4 and flood protection dike section of Bangon River is shown in the Table 4.4-2. 42 drainage outlets including rivers, creeks and swamps were listed.

In Tacloban City, from No. 1 to 25 in the list, 4 river gates and 21 Reinforced Concrete Box Culverts (RCBCs) will be constructed. The size of RCBCs is designed as 1.0m width and 1.0m height with a flap gate. Four river gates will be constructed at No.7 of Tanghas-Lirang Creek, No. 10 of Sagkahan Creek, No. 16 of Mahayahay Creek and No. 17 of Burayan Creek.

In Palo Municipality, from No.16 to 35 in the list, 4 river gates and 5 Reinforced Concrete Box Culverts (RCBCs) will be constructed along the proposed tide embankment. The size of RCBC is designed as 1.0m width and 1.0m height with a flap gate. 4 flood gates will be constructed at No. 29 of Payapay Bridge along Kilot Creek, No. 30 of the existing 2 barrel box culvert along Binog Creek, No. 31 of 1 barrel box culvert along Binog Creek near Mac Arthur Park and No. 35 of a channel from a fish pond on the left bank of Bangon River at the river mouth.

Structures from No. 36 to 41 will be located along the flood protection dike of Bangon River which is being constructed by DPWH. No. 36 is the confluence of a tributary of Bangon River on the left bank, and JICA Study Team suggests to DPWH constructing a flood gate.

Table 4.4-2 List of the planned drainage outlets of section 3 and 4

NO.	SEC.	LGU	STATION	PRESENT STRUCTURE	PLANNED STRUCTUURE	REMARKS
1	3-1	TC	0+046	1 ROW – 0.610 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
2	3-1	TC	0+131	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
3	3-1	TC	0+307	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
4	3-1	TC	0+378	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
5	3-1	TC	0+430	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
6	3-1	TC	0+492	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
7	3-1	TC	1+364	Aslum Bridge	River Gate	Tanghas-Lirang Creek
8	3-1	TC	1+930	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
9	3-1	TC	2+123	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
10	3-1	TC	2+393	Box Culvert	River Gate	Sagkahan Creek
11	3-1	TC	2+441	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
12	3-1	TC	2+562	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
13	3-1	TC	2+681	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
14	3-2	TC	2+877	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
15	3-2	TC	2+922	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
16	3-2	TC	3+100	Box Culvert	River Gate	Mahayahay Creek
17	3-3	TC	4+382	Burayan Bridge	River Gate	Burayan River
18	3-3	TC	5+493	Open Lined Canal 0.9m(W)x0.8m(H)	1.0m(W)X1.0m(H) RCBC + Flap Gate	
19	3-3	TC	5+422	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
20	3-3	TC	9+702	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
21	3-3	TC	9+748	1 ROW – 0.910 m Ø RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
22	4-1	TC		Box Culvert 2m(W)x0.5m(H)	1.0m(W)X1.0m(H) RCBC + Flap Gate	
23	4-1	TC		Swamp (1)	1.0m(W)X1.0m(H) RCBC + Flap Gate	
24	4-2	TC		Swamp (2)	1.0m(W)X1.0m(H) RCBC + Flap Gate	
25	4-2	TC		Swamp (3)	1.0m(W)X1.0m(H) RCBC + Flap Gate	
26	4-2	PL		Swamp (4)	1.0m(W)X1.0m(H) RCBC + Flap Gate	
27	4-2	PL		Swamp (5)	1.0m(W)X1.0m(H) RCBC + Flap Gate	
28	4-2	PL		Swamp (6)	1.0m(W)X1.0m(H) RCBC + Flap Gate	
29	4-3	PL	8+850	Payapay Bridge	River Gate	Kilot Creek
30	4-3	PL	9+222	2 Barrel - 2.5m(W)x1.8m(H)x15m(L) RCBC	River Gate	Binog Creek
31	4-4	PL	10+150	1 Barrel - 2.7m(W)x1.7m(H)x12.80m(L) RCBC	River Gate	Binog Creek near Mac Arthur Park
32	4-5	PL		RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
33	4-5	PL		RCPC	1.0m(W)X1.0m(H) RCBC + Flap Gate	
34	4-5	PL	10+893	Swamp (RCBC in the upstream)	1.0m(W)X1.0m(H) RCBC + Flap Gate	
35	4-7	PL		Channel from a Fish Pond	River Gate	Bangon R. left bank
36	NA	PL		Conjunction of a Tributary	River Gate	Bangon R. left bank
37	NA	PL		Natural Channel from a Swamp	1.0m(W)X1.0m(H) RCBC + Flap Gate	Bangon R. right bank
38	NA	PL		1 ROW -0.6m Ø RCPC	To Add Flap Gate	Bangon R. right bank
39	NA	PL		1 ROW - 0.910 m Ø RCPC	To Add Flap Gate	Bangon R. right bank
40	NA	PL		1 ROW - 0.910 m Ø RCPC	To Add Flap Gate	Bangon R. right bank
41	NA	PL		1 ROW - 0.910 m Ø RCPC	To Add Flap Gate	Bangon R. right bank
42	NA	PL		1 ROW - 0.910 m Ø RCPC	To Add Flap Gate	Bangon R. left bank

* TC: Tacloban, PL: Palo

Source: JICA Study Team