CHAPTER 16 BRIDGE REPLACEMENT OUTLINE DESIGN OF SELECTED BRIDGES

16.1 Design Criteria and Conditions for Bridge Replacement

16.1.1 Design Criteria and Conditions for Bridge Replacement

The following items show design criteria and conditions utilized for outline design of new bridges.

(1) Design Standards utilized for Outline Design of New Bridges

The Design standards utilized for outline design of new bridges shall be given as follows:

Table 16.1.1-1 Design Standards Utilized for Outline Design of New Bridges

Item	Design Condition	Specification
1) General		
Design Load Combination	LV2 Seismic Design: Extreme Event I	LRFD (2012)
Seismic Design	Design Spectrum (1,000year)	JICA Study Team
	Response Spectrum Analysis	JICA Study Team
2) Superstructure		
Design Lane Width	3350 mm (Pack and Guadalupe) 3000 mm (Lambingan)	DPWH, AASHTO
Dead Load	LRFD (2012)	
Live Load	HL-93 and Lane Loads	LRFD (2012)
3) Substructure		
Seismic Earth Pressure	LRFD(2012)	
Column Section Design	R-factor method	LRFD(2012)
4) Foundation		
Pile Foundation Analysis	JICA Study Team (JRA)	
Soil Type	JICA Study Team (JRA)	
Liquefaction design	JICA Study Team (JRA)	
Pile Bearing	L1: FS=2, L2: FS=1	JICA Study Team (JRA)
Pile Section Design	M-N chart (=1.0)	LRFD(2012)

(2) Load Factors and Combination

The outline design calculation shall be carried out based on LRFD methodology given in AASHTO LRFD 2012 as follows:

1) Loads

Table 16.1.1-2 Permanent and Transient Loads

Permanent Loads	DD = Down drag DC = Dead load of structural components and nonstructural attachment DW = Dead load of wearing surfaces and utilities EH = Horizontal earth pressure load EL = Accumulated locked-in force effects resulting from the construction process, including the secondary forces from post-tensioning ES = Earth surcharge load EV = Vertical pressure from dead load of earth fill
Transient Loads	BR = Vehicular braking force CE = Vehicular centrifugal force CR = Creep CT = Vehicular collision force CV = Vessel collision force EQ = Earthquake FR = Friction IM = Vehicular dynamic load allowance LL = Vehicular live load LS = Live load surcharge PL = Pedestrian live load SE = Settlement SH = Shrinkage TG = Temperature gradient TL = Train Load TU = Uniform temperature WA = Water load and stream pressure WL = Wind on live load WS = Wind load on structure

Source: LRFD 2012

2) Load Factors and Combination

Table 16.1.1-3 Load Combinations and Factors

Load Combination / Limit State	DC DW EH EV ES	LL IM CE BR PL LS EL	TL	WA	WS	WL	FR	TU CR SH	TG	SE	Use One of These At a Time: EQ	CT	CV
Extreme Event I	γ_p	0.50	0.50	1.00	-	-	1.00	-	-	-	1.00	-	-

Source: LRFD 2012

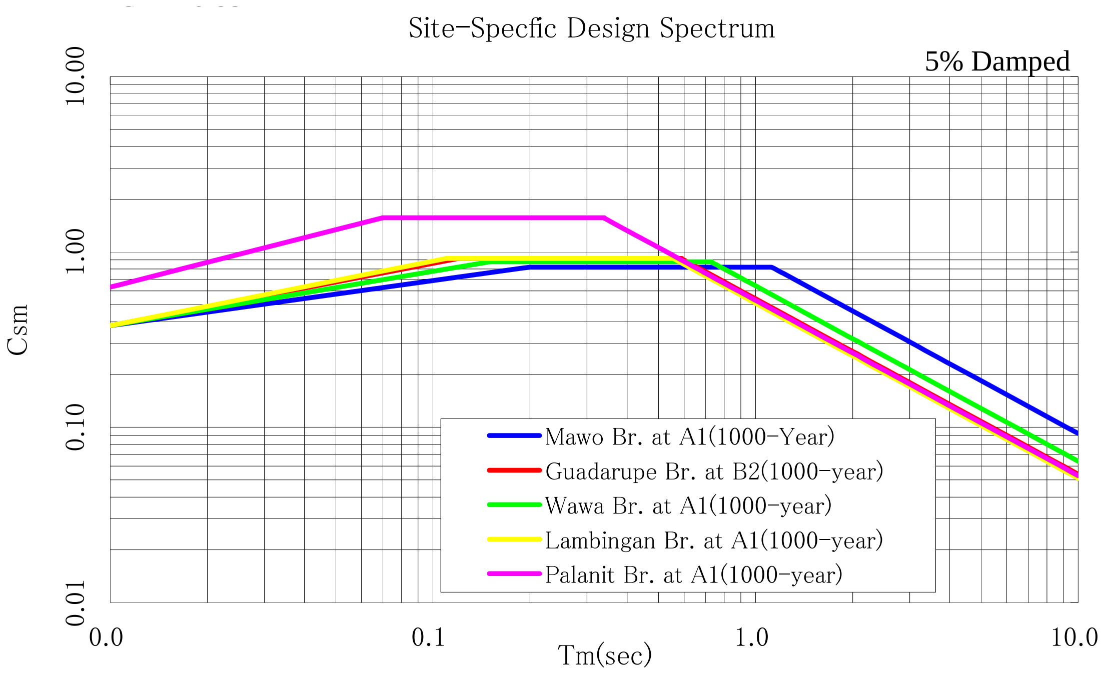
Table 16.1.1-4 Load Factors for Permanent Loads, γp

Type of Load			Load Factor	
			Maximum	Minimum
DC	:	Component and Attachments	1.25	0.90
DW	:	Wearing Surfaces and Utilities	1.50	0.65
EH	:	Horizontal Earth Pressure		
		Active	1.50	0.90
		At Rest	1.35	0.90
EL	:	Locked-in Erection Stress	1.00	1.00
EV	:	Vertical Earth Pressure		
		Overall Stability	1.00	N/A
		Retaining Structures	1.35	1.00
		Rigid Buried Structures	1.30	0.90
		Rigid Frames	1.35	0.90
ES	:	Earth Surcharge	1.50	0.75

Source: LRFD 2012

(3) Design Spectrum

The design spectrum utilized for modal analysis and response spectrum analysis shall be as following figure and table, evaluated in this project.



Soil Type & Response Coefficement

Soil Profile Type									
Guadarupe Br. at B2		Mawo Br. at A1 at A1		Wawa Br. at A1 at A1		Lambingan Br. at A1 & B1		Palanit Br. at A1	
T(sec)	Cs(g)	T(sec)	Cs(g)	T(sec)	Cs(g)	T(sec)	Cs(g)	T(sec)	Cs(g)
0.010	0.380	0.010	0.380	0.010	0.380	0.010	0.380	0.010	0.630
0.120	0.920	0.200	0.820	0.150	0.880	0.110	0.920	0.070	1.570
0.120	0.920	0.200	0.820	0.150	0.880	0.110	0.920	0.070	1.570
0.590	0.920	1.120	0.820	0.730	0.880	0.560	0.920	0.340	1.570
0.590	0.915	1.120	0.820	0.730	0.877	0.560	0.911	0.340	1.559
0.610	0.885	1.200	0.767	0.750	0.853	0.600	0.850	0.600	0.883
0.700	0.771	3.000	0.307	0.800	0.800	0.700	0.729	0.700	0.757
0.810	0.667	4.000	0.230	0.850	0.753	0.800	0.638	0.800	0.663
0.900	0.600	5.000	0.184	0.900	0.711	0.900	0.567	0.900	0.589
1.000	0.540	6.000	0.153	1.000	0.640	1.000	0.510	1.000	0.530
2.000	0.270	7.000	0.131	2.000	0.320	2.000	0.255	2.000	0.265
3.000	0.180	8.000	0.115	3.000	0.213	3.000	0.170	3.000	0.177
4.000	0.135	9.000	0.102	4.000	0.160	4.000	0.128	4.000	0.133
5.000	0.108	10.000	0.092	5.000	0.128	5.000	0.102	5.000	0.106
6.000	0.090	0.000	0.000	6.000	0.107	6.000	0.085	6.000	0.088
7.000	0.077	0.000	0.000	7.000	0.091	7.000	0.073	7.000	0.076
8.000	0.068	0.000	0.000	8.000	0.080	8.000	0.064	8.000	0.066
9.000	0.060	0.000	0.000	9.000	0.071	9.000	0.057	9.000	0.059
10.000	0.054	0.000	0.000	10.000	0.064	10.000	0.051	10.000	0.053

Figure 16.1.1-1 Design Spectrum for New Bridge Design

(4) Materials

The material properties for concrete, reinforcing bar, PC cable, piles and steel structure mainly utilized for steel deck superstructures shall be given as follows:

1) Concrete

Table 16.1.1-5 Concrete Strength by Structural Member

Compressive Strength at 28 days (MPa) (Cylinder Specimen)	Structural Member
40	Post-tensioned PC I-Girder Cast-in-situ PC Slab/Girder
35	Cast-in-situ PC Slab Cast-in-situ PC Crossbeam
28	Substructure (Pier, Abutment, Pile Caps, Wing wall) Retaining Wall, Box Culvert Precast Reinforced Concrete Plate Precast Parapet
21	Approach Slab
28	Cast-in-situ Bored Pile
18	Non-reinforced Concrete Structure Lean Concrete

Source: DPWH

2) Reinforcing Bar

Table 16.1.1-6 Properties and Stress Limit of Reinforcing Bars

Type	Yield Strength f y (MPa)	Tensile Strength f u (MPa)	Modulus of Elasticity (MPa)	Diameter of Bar (mm)
Grade 275	275	500	200,000	D10, D12, D16,D20
Grade 415	414	620	200,000	D25,D28,D32,D36

Source: DPWH

3) PC Cable

Table 16.1.1-7 Properties and Stress Limit of PC Cable for T girder bridge

	Min. Ultimate Strength (MPa)	Temporary Stress Before Loss due to Creep and Shrinkage = 0.8fs'	Stress at Service Load After Losses =0.7fs'
Grade 270	1862	1488	1300

Source: AASHTO

Table 16.1.1-8 Properties and Stress Limit of PC Cable for PC Box Girder bridge

	Diameter (mm)	Tensile Strength (kN)	Modulus of Elasticity (MPa)
12S15.2mm (SWPR7BL)	15.2mm	3130	200,000

Source: JIS

4) Steel Pipe Pile

Table 16.1.1-9 Properties and Stress Limit of Steel Pipe

Type	Yield Strength f y (MPa)	Tensile Strength f u (MPa)	Modulus of Elasticity (MPa)
Grade SKK 400	235	400	200,000
Grade SKK 490	315	490	200,000

Source: JIS

5) Steel Pipe Sheet Pile

Table 16.1.1-10 Properties and Stress Limit of Steel Pipe for Steel Pipe Sheet Pile

Type	Yield Strength f y (MPa)	Tensile Strength f u (MPa)	Modulus of Elasticity (Mpa)
Grade SKY 400	235	400	200,000
Grade SKY 490	315	490	200,000

Source: JIS

6) Steel members for superstructure

Table 16.1.1-11 Properties and Stress Limit of Steel Members

Type	Yield Strength f y (MPa)	Tensile Strength f u (MPa)	Modulus of Elasticity (MPa)
SM570	450 t < 40mm 430 40mm < t < 75mm 420 75mm < t < 100mm	570	200,000
SM490W	355 t < 40mm 335 40mm < t < 75mm 325 75mm < t < 100mm	490	200,000
SM400AW	235 t < 40mm 215 40mm < t < 100mm	400	200,000
SM490Y	355 t < 40mm 335 40mm < t < 75mm 325 75mm < t < 100mm	490	200,000
SS400	235 t < 40mm 215 40mm < t < 100mm	400	200,000

Source: JIS

16.1.2 Determination of New Bridge Types for Outline Design

Bridge types to be conducted in the outline design are determined based on comparison study considering multiple elements such as costs, structure advantage, constructability, environmental impact and maintenance ability. The following flowchart shows the basic procedure of the comparison study for selection of new bridge types.

STEP 1. Confirmation of ROAD CONDITION
- Cross Section of Bridge, Lane Arrangement
- Road Horizontal and Vertical Alignment

↓

STEP 2. Confirmation of HYDRAULIC CONDITION
- Confirmation of River Condition From Interim Report
- HWL/ HTW for Navigation Clearance

↓

STEP 3. Examination of SUBSTRUCTURE LOCATION
- Abutment Location
- Span Arrangement

↓

STEP 4. Confirmation of BASIC CONCEPT

↓

STEP 5. COMPARISON STUDY of New Bridge Types
- Extraction of Applicable Basic Types based on Actual Results
- Selection of Logically Suitable Types from above the Basic Types
- Applicability of Modern/ Rational/ Advanced Technological Types
- Comprehensive Comparison Study of New Bridge Types

Cost, Structure, Environment, Construction and Maintenance

Figure 16.1.2-1 Procedure of Comparison Study for Selection of New Bridge Types

For extraction of applicable basic types based on actual results shown at STEP 5 in the above procedure, the following table regarding the relationship between actual results of basic bridge types and span length is organized on the basis of '11 Design Data Book ('11 JBA Manual) and PC Bridge Planning Manual ('2007 JPPCA).

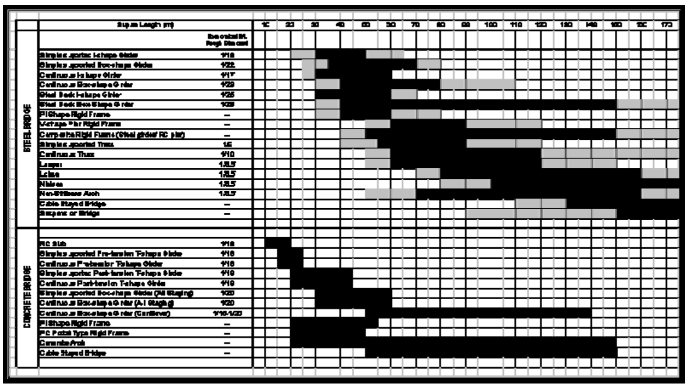


Figure 16.1.2-2 Relationships between Actual Results of Basic Bridge Types and Span Length

(1) Lambingan Bridge

STEP 1. Confirmation of ROAD CONDITION

i) Bridge Width

For the cross section and lane arrangement of new bridge, the examined results of road planning including approach roads shall be applied, shown as follows. For the current condition of Lambingan bridge, a water pipe bridge, which is a trussed arch bridge, is currently located at 2.0m separated from the existing road bridge for downstream side; thereby, adequate superstructure type without any influence against the water pipe bridge even during construction phase shall be selected.

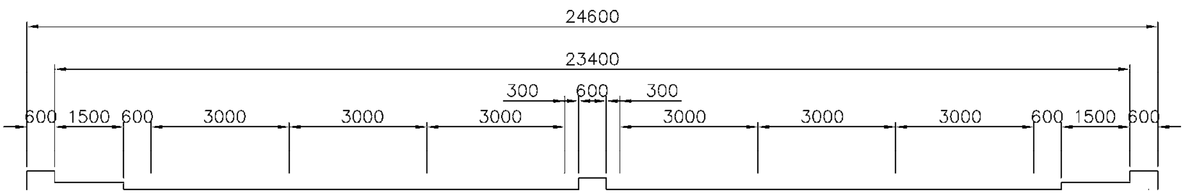


Figure 16.1.2-3 Cross Section/ Lane Arrangement of Lambingan Bridge

ii) Road Horizontal and Vertical Alignment

For horizontal and vertical alignment of new bridge, the examined results of road planning including approach roads shall be applied, shown as follows. In the road planning, multiple planning in consideration of river condition such as height of design flood level and required free board are examined, from the conclusion of which the girder height of Lambingan bridge is restricted within 2.0m.

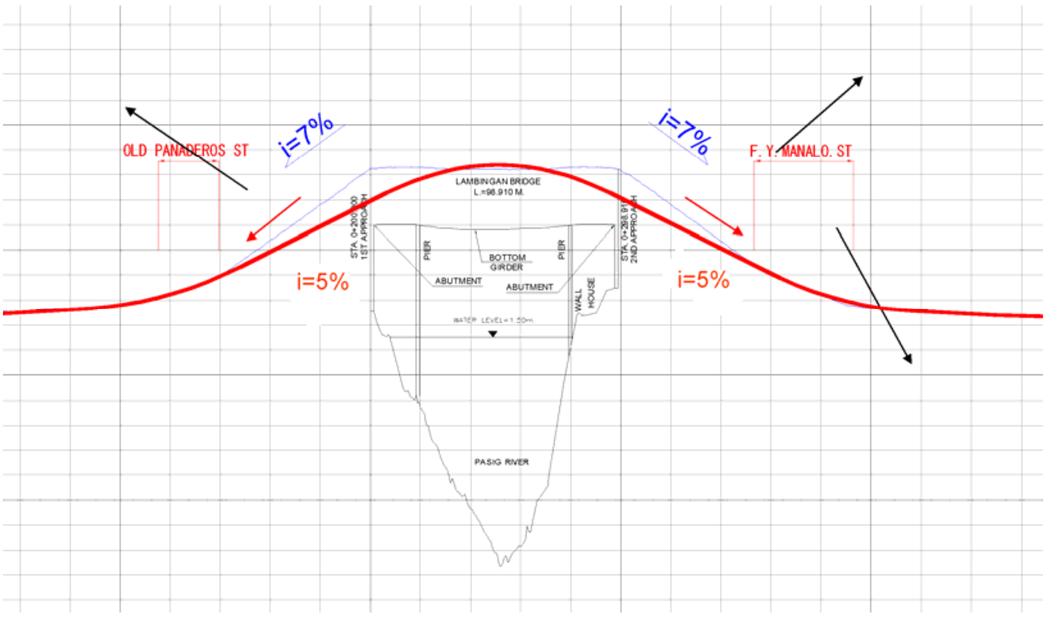


Figure 16.1.2-4 Vertical Alignment by Road Planning of New Lambingan Bridge

STEP 2. Confirmation of HYDRAULIC CONDITION

For hydraulic condition in the new bridge selection, the determined results in this project shall be applied. From the result of hydraulic examination, existing free board and navigation width shall be strongly secured from the aspect of safety of vessels.

STEP 3. Examination of SUBSTRUCTURE LOCATION

i) Location of Abutments for Simple Supported Bridge Type
For the location of abutments for simple supported bridge type, the abutment shall be planned at the section, at which the wing walls of new abutments and existing retaining walls can be continuously connected. Consequently, the bridge length for the simple supported bridge type can be resulted as 90m long.

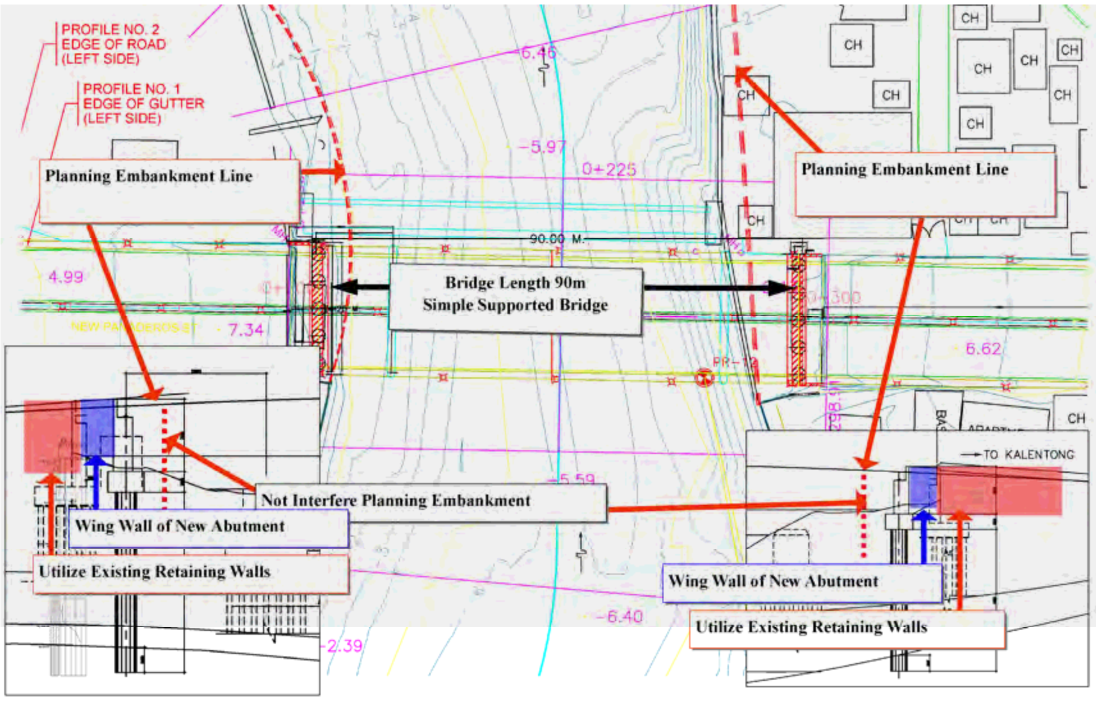


Figure 16.1.2-5 Determination of Abutment Location of Lambingan Bridge (Simple Supported Condition)

ii) Location of Abutments for 3-Span Continuous Bridge Type
2-Span and 4-Span bridges are negative to be applied in this location because the pier(s) would be naturally installed in the center of the river. Therefore, following the above mentioned simple supported bridge, applicability of 3-Span bridge type shall be considered in the multiple comparison study. In case that the new piers were planned at the same location of existing piers based on navigation condition, inadequate negative reactions would be caused at the abutments even in the dead load condition due to ambulanced span arrangement. Thereby, the location of new abutments shall be determined considering such the occasion.

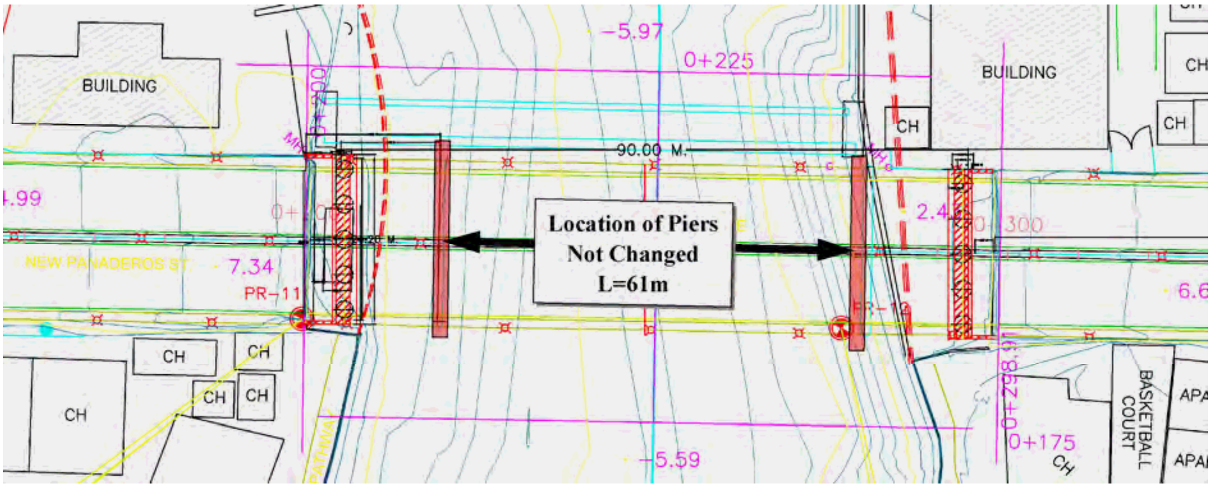


Figure 16.1.2-6 Determination of Abutment Location of Lambingan Bridge (3-Span Condition)

iii) Span Arrangement in Comparison Study

Following figure shows the span arrangements using in comparison study, resulted from above examination. 2-Span and 4-Span bridges are not included in the comparison study because they can not absolutely meet the navigation and river required conditions.

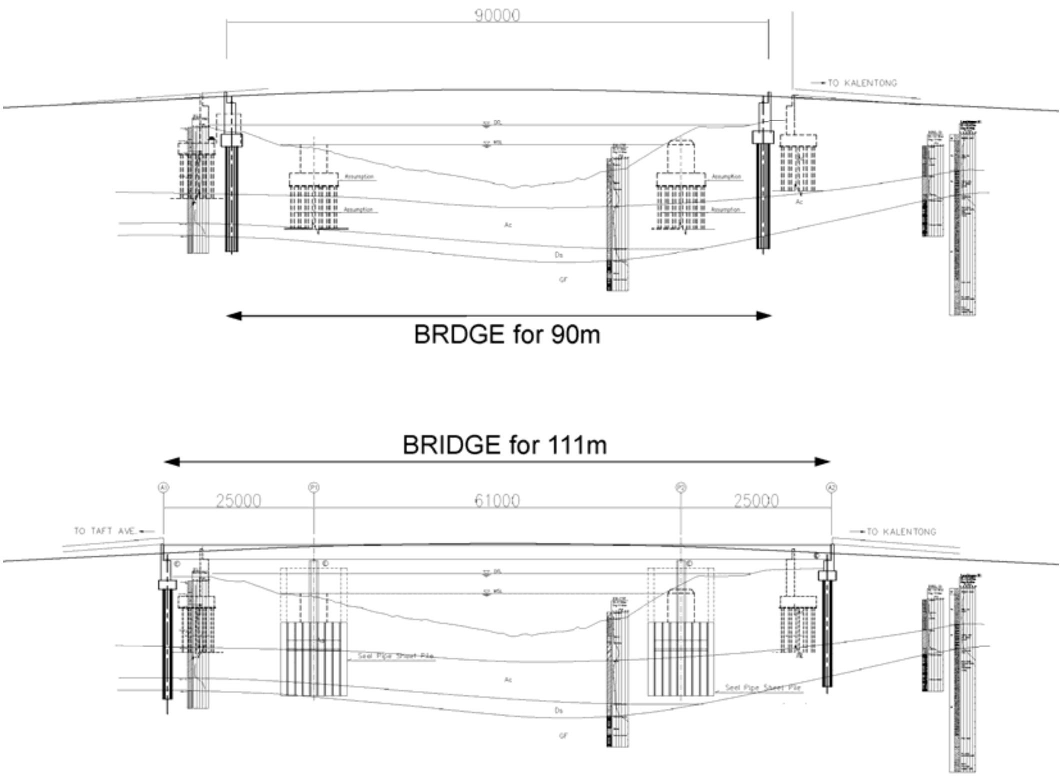


Figure 16.1.2-7 Span Arrangements of Lambingan Bridge in Comparison Study

iv) Examination of Applicable Bridge Types Considering Construction Condition

Lambingan bridge is of urban bridge, the both sides of approach roads of which have a lot of houses and buildings. Besides, other useful alternative bridges to be utilized as Detour bridge during construction phase do not exist near Lambingan bridge. Thereby, the adequate construction method is required not to affect existing traffic flow and residential removal. Based on such the background, following construction methodologies are compared in consideration of two of construction conditions such as Stage Construction and Total Construction.

< STAGE CONSTRUCTION>
The construction method to elect new bridge separately

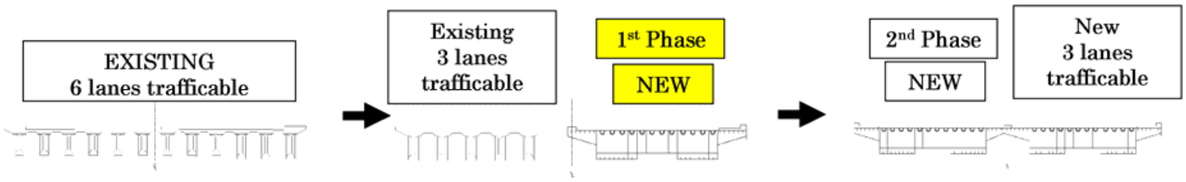


Figure 16.1.2-8 Construction Steps of Stage Construction

<Advantage>
- No need secure detour roads such as other alternative bridge or temporary bridge
- Minimum residential removal

<Disadvantage>
- Complicated construction steps and longer schedule, rather than Total Construction
- Limited superstructure type because even 1st phase structure should meet live loads influences.

< TOTAL CONSTRUCTION>
In order to secure existing traffic flow, detour road such like temporary bridge across the river is to be utilized and all of structure will be erected after demolishing the existing structure.

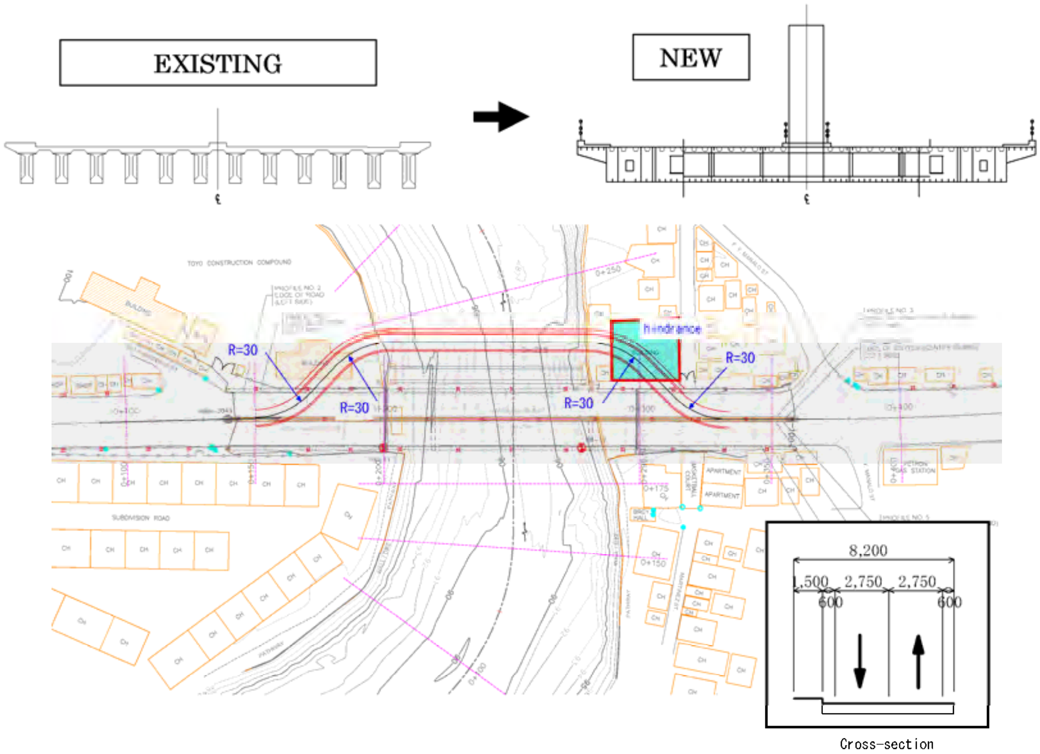


Figure 16.1.2-9 Detour Temporary Bridge under Total Construction Method

<Advantage>
- Erection schedule is to be shorter and familiar method is available to applied
- Structurally rational superstructure can be designed

<Disadvantage>
- A temporary detour bridge is required to secure existing wide navigation width (W=60m)
- The span length of the temporary detour bridge is needed over 61m, which may be extremely expensive because familiar structure can not apply.
- Influence of residential removal may be quite significant due to detour road and its approaches

Based on the above comparison, application of the total construction may not be realistic method; hence, the selection of bridge type of Lambingan bridge shall be examined based on the stage construction method.

STEP 4. Confirmation of BASIC CONCEPT

The basic concept resulted from above STEP 1 to STEP 3 is enumerated. The basic concept may be significantly important factor for bridge selection under the comparison study.

- New abutments locations are newly and carefully determined in consideration of existing and planning condition around the location
- Existing navigation width and navigation height shall be strictly secured.
- Based on road alignment examination, the girder height should be kept within 2.0m to secure the existing navigation clearance
- The longest span of New Bridge: 90m (Simple), 61m (3-Spans)
- 2-Span or 4 or more Span bridge not applicable
- In the Comparison Study of New Bridge Type, the concept of bridge construction should be reflected into the evaluation. Stage construction method is applicable to this bridge

STEP 5. COMPARISON STUDY of New Bridge Types

i) Extraction of Applicable Basic Types based on Actual Results
Based on the basic concept and several conditions, applicable bridge types are extracted from the table of Relationships between Actual Results of Basic Bridge Types and Span Length, shown as follows.

Table 16.1.2-1 Extraction of Applicable Basic Types based on Actual Results

STEEL	PC
Simple Supported	
Steel Truss Steel Langer Steel Lohse Arch Steel Lohse Arch Stiffened Steel Deck Box (Rational Structure)	PC Cable Stayed Concrete Lohse Arch
2-Span	
---	---
3-Span	
Continuous RC Slab Steel Box Continuous Steel Deck Box	Continuous PC Box

ii) Selection of Logically Suitable Types from above the Basic Types
Steel Truss bridge shall be included in one of the candidates. 2.0m of girder height is realizable. Because Floor system and trusses are separated structurally, connection between 1st and 2nd phase structures is smoothly executable using simple counter weights. Structurally 3-face truss type should be applied due to stage construction method.
3-span continuous steel deck box girder bridge shall be included in one of the candidates. 2.0m of girder height is realizable.
Simple steel Langer and Lohse arch bridges shall not be included in the detail comparison study. General Langer and Lohse arch bridges consist of 2 of arch ribs and floor system that is structurally separated and that is installed between the two arch ribs. Therefore, to realize stage construction, 3-arch-rib structure system should be applied. Consequently, in case of application of such the 3-arch-rib system, absolutely it will be much more expensive than the above truss bridge.
As structurally rational bridge type, other rational arch type bridge that arch ribs stiffen girder type bridge may be applicable. Thus, simple supported steel Lohse arch stiffening steel deck box girder bridge would be efficiently applicable to be included as a candidate, which is obviously erectable in this site. 90 m of span length of the steel deck box girder can not be applicable in this site because the girder height will be over 3.0m high, otherwise, application of steel Langer or Lohse arch bridges are as mentioned above concern about expensive costs. Therefore, the effectiveness of such the rational structure may be absolutely confirmed. Structurally separated Lohse stiffened box girders are needed because of application of stage construction; however, by applying same structures to 1st and 2nd phase superstructure types, connection between them is smoothly executable using simple counter weight.
PC cable stayed bridge and simple supported concrete Lohse arch bridge will be naturally the most expensive bridge type in this site, and there are no land spaces where stiffening concrete arch or towers can be constructed. Therefore, such the bridge types are not realistic bridge type to be included into the detail comparison study. Additionally, 3 span RC slab steel box girder and PC box girder can not be applicable because the girder height will be beyond 3.0m which can not meet the road and river required condition.

Based on the above evaluation, multiple comparison study is conducted considering cost, structure, environmental impact, constructability and maintenance ability.

Table 16.1.2-2 Candidates of comparison study

SEEL	PC
Simple Supported	
Steel Truss Steel Lohse Arch Stiffening Steel Deck Box (Rational Structure)	—
3-Span	
Continuous Steel Deck Box	—

Based on the evaluation, shown in the comparison table, the recommendable bridge type for outline design is **Simple Supported Lohse Arch Stiffening Steel Deck Girder Bridge**.

iii) Selection of Logically Suitable Types of Bridge Foundations
In the study on selection of bridge foundation, study of site conditions for selecting an appropriate foundation type is a key discussion.

The site conditions are shown in below table. In the abutment, particularly it was located very close to existing abutment, meanwhile, the Pier foundation is located very close to navigation channel.

Table 16.1.2-3 Site Condition for Study of Type-1

Study Type	Abutment Foundation	Pier Foundation
Foundation location	On the ground	In the River
proximity structure	Closed to existing abutment	Closed to narrow navigation channel
Navigation condition	-	in a curve with close to existing piers
Max. Water depth (m)	-	Around 10.5 m
Depth of bearing layer (m)	Around GL-17.5 to 19.0m	Around GL-15.0m
Type of bearing layer	Rock	Rock
Liquefaction	liquefiable site	liquefiable site
Lateral spreading	-	-

Based on the above site conditions, applicable foundation types are extracted, shown as follows.

Table 16.1.2-4 Extraction of Applicable Basic Types based on Actual Results

Abutment Foundation	Pier Foundation
Large Diameter Bored Pile (Rotary all casing boring method) Commonly used Cast-in-place Concrete Pile (Reverse circulation drill method)	Steel Pipe Sheet Pile Foundation (Press-in method) Commonly used Cast-in-place Concrete Pile (Reverse circulation drill method)

According to the above evaluation, multiple comparison study is conducted considering cost, construction period, neighboring construction, constructability, and environmental impact.

The result of comparative study of abutment foundation are shown in the next tables, the recommendable abutment foundation type for outline design is Large Diameter Bored Pile, because of its advantages in low construction cost and shortest construction period on account of consider neighboring Constructability.

The recommendable pier foundation type for outline design is Steel Pipe Sheet Pile (SPSP) Foundation, because of its advantages in minimum term of construction period & traffic control with efficient workability in the river. (refer to Chapter 16.1.2. (2) Guadarupe bridge)

Table 16.1.2-5 Site Candidates of Comparison Study

Abutment Foundation	Pier Foundation
Simple Supported	
Large Diameter Bored Pile (Rotary all casing boring method)	-
3-Span	
Large Diameter Bored Pile (Rotary all casing boring method)	Steel Pipe Sheet Pile Foundation (Press-in method)

Table 16.1.2-6 Comparison on Foundation Type of Lambingan Bridge Abutment（A2)

OUTLINE DESIGN

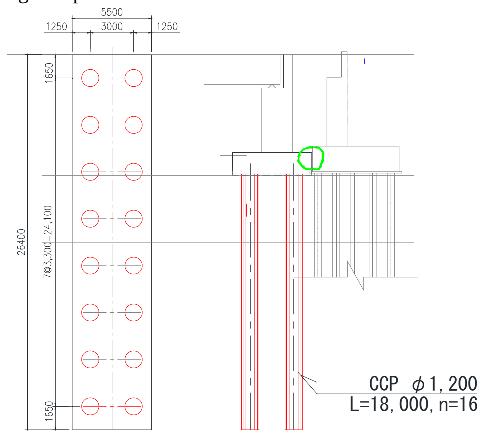
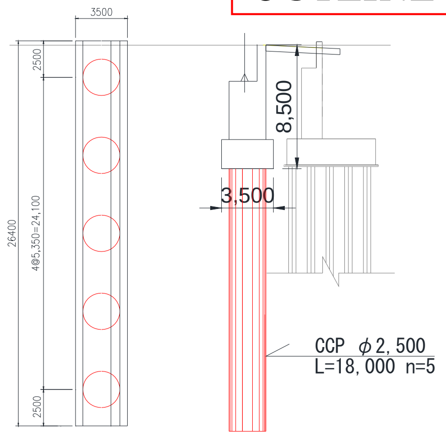
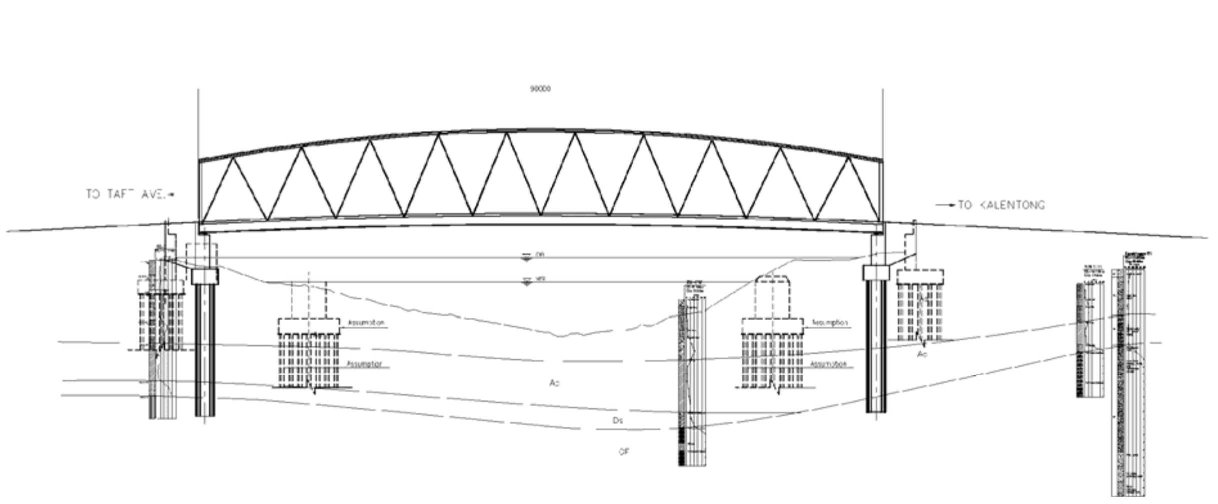
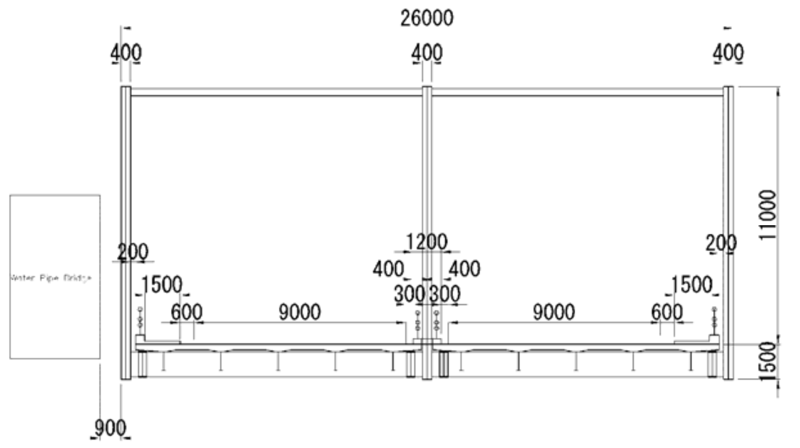
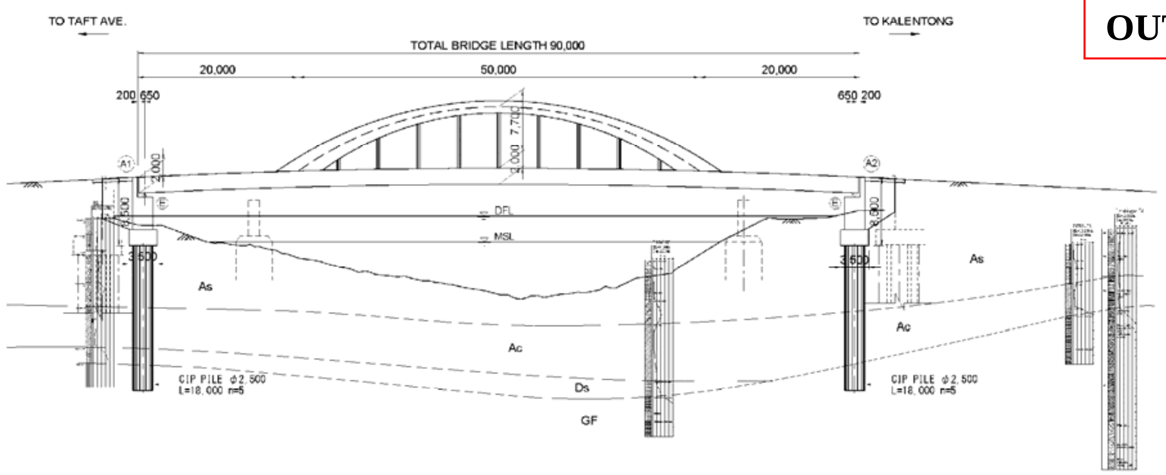
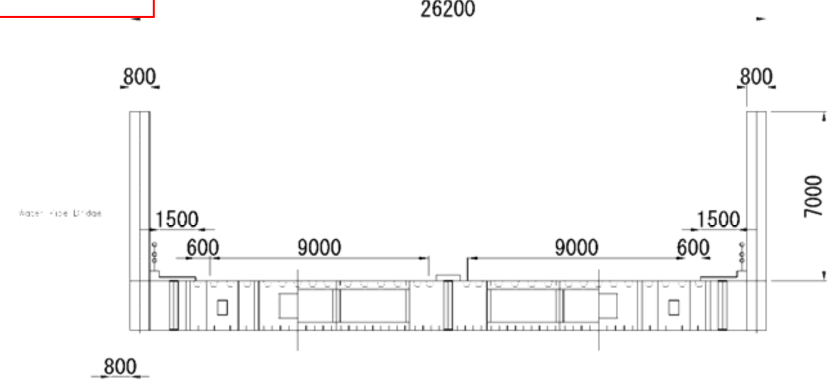
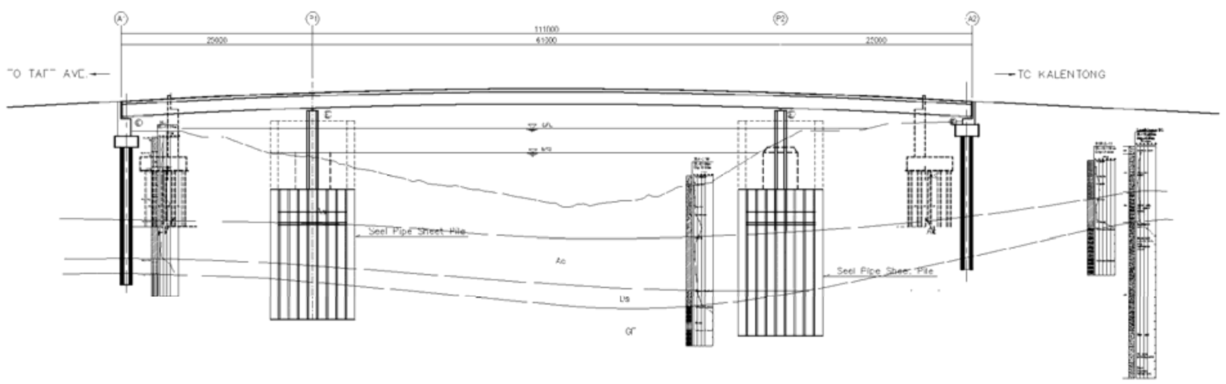
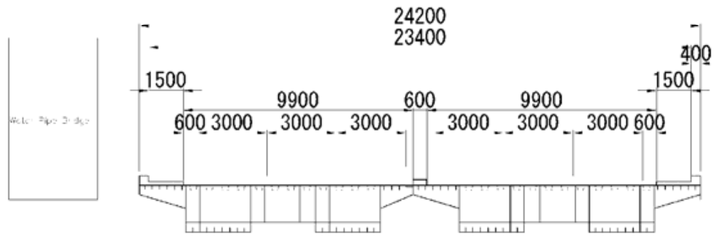
Evaluation Items	Alternative-1 (reverse circulation drill method) Commonly used Cast-in-place Concrete Pile D= 1.2m		Alternative-2 (rotary all casing boring method) Large Diameter Bored Pile(Cas-in-place Pile D= 2.5m)	
Side View Pile arrangement	Diameter of pile : 1200 mm Total number of pile : 16 Pile length : 18.0 m Total length of pile : 288.0 m 		Diameter of pile : 2500 mm Total number of pile : 5 Pile length : 18.0 m Total length of pile : 90.0 m 	
Structural Aspect and Stability	- Pile Displacement Ratio (Pile dosplacement/displacement limit) is 0.872 - Need to the large number of Cast-in-Place Concrete	A	- Pile Displacement Ratio (Pile dosplacement/displacement limit) is 0.980	B
Construction Cost (for Foundation)	| | Quantity | Unit Cost (Php) | Total (1,000Php) | | Pile Cap Concrete | 290m3 | 7,559.8 | 2,195 | | Reinforcement steel | 58ton | 52,600.0 | 3,055 | | Pile | 288m | 45,898.5 | 13,219 | | Cofferdam | 1077m2 | 21,181.9 | 22,813 | Total 41,282 Ratio **1.166**	C	| | Quantity | Unit Cost (Php) | Total (1,000Php) | | Pile Cap Concrete | 185m3 | 7,559.8 | 1,397 | | Reinforcement steel | 37ton | 52,600.0 | 1,944 | | Pile | 90m | 116,987.0 | 10,529 | | Cofferdam | 1017m2 | 21,181.9 | 21,542 | Total 35,412 Ratio **1.000**	A
Construction Plan and Period	- Working in Temporary cofferdam & low spaces of under the temporary stage. Cofferdam Work 40 days Pile work (1.5pile/day) 24 days Pile Cap 29 days Total **93 days** Ratio **1.405**	C	- Working in Temporary cofferdam & low spaces of under the temporary stage. Cofferdam Work 38 days Pile work (2pile/day) 10 days Pile Cap 18 days Total **66 days** Ratio **1.000**	A
Neighboring Construction	- Not keep out of exsisting abutment and new abutment foundation.	C	- Keep out of exsisting abutment and new abutment foundation by rotary all casing boring method.	A
Constructability	- Constructability is inferior due to large number of Cast-in-place concrete Pile work.	B	-Constructability is superior with small number of foundation work.	A
Environmental Aspect	- Inferior in Environmental aspect due to large number of excavated soil.	B	- Superior in Environmental aspect with small number of excavated soil.	A
Evaluation	- Construction cost is highest with long construction period. Not Recommended	**B**	- Construction cost is lowest with minimum Construction period with efficient workability. Most Recommended	**A**

Table 16.1.2-7 Comparison of New Bridge Types for Lambingan bridge

	Side View	Sectional View	Evaluation: Good-A-Fair-B-Poor-C
Alternative 1S	Simple Supported Truss Bridge 		COST: Superstructure 0.75; Substructure 0.06; Foundation 0.08; Others 0.14; Total 1.03 — B STRUCTURAL FEATURE — C CONSTRUCTABILITY — B ENVIRONMENTAL IMPACT — B MAINTENANCE — C
Alternative 2S	Simple Supported Steel Deck Stiffened Lohse Bridge 	**OUTLINE DESIGN** 	COST: Superstructure 0.72; Substructure 0.06; Foundation 0.08; Others 0.14; Total 1.00 — A STRUCTURAL FEATURE — A CONSTRUCTABILITY — A ENVIRONMENTAL IMPACT — A MAINTENANCE — A
Alternative 3S	3-Span Continuous Steel Deck Box Girder Bridge 		COST: Superstructure 0.62; Substructure 0.13; Foundation 0.53; Others 0.13; Total 1.42 — C STRUCTURAL FEATURE — A CONSTRUCTABILITY — C ENVIRONMENTAL IMPACT — C MAINTENANCE — B

(2) Guadalupe Bridge

STEP 1. Confirmation of ROAD CONDITION

i) Bridge Width
The superstructure of new bridge shall be the outer bridge, which is currently PC girder bridge.

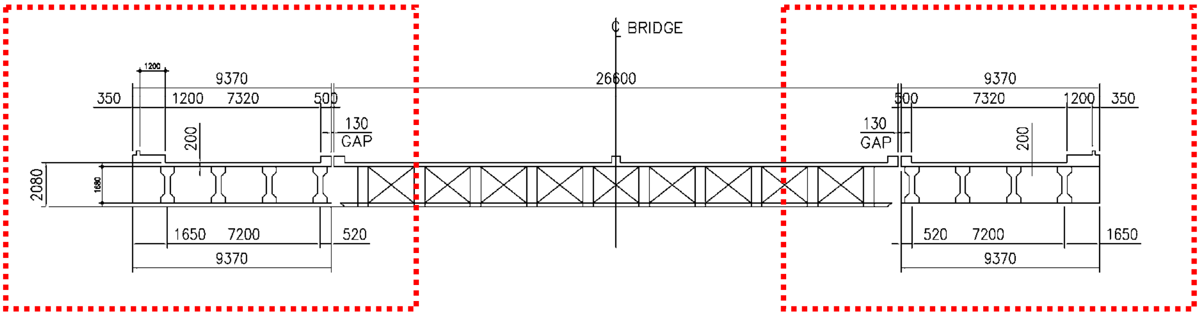


Figure 16.1.2-10 Cross Section/ Lane Arrangement of Existing Guadalupe Bridge

For the cross section and lane arrangement of new bridge, the examined results of road planning including approach roads shall be applied, shown as follows.

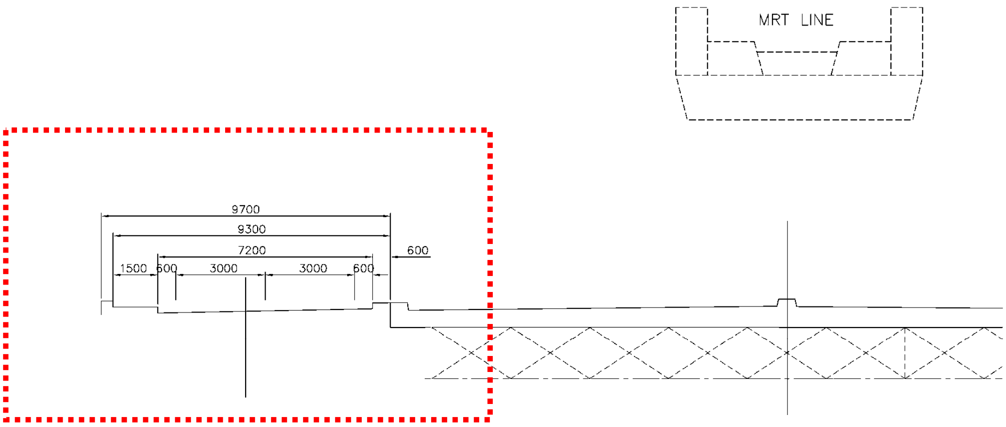


Figure 16.1.2-11 Cross Section/ Lane Arrangement of New Guadalupe Bridge

ii) Road Horizontal and Vertical Alignment
Horizontal and vertical alignment of the new bridge shall be adjusted to existing center bridge.

STEP 2. Confirmation of HYDRAULIC CONDITION

For hydraulic condition in the new bridge selection, the determined results in this project shall be applied. From the result of hydraulic examination, existing free board and navigation width shall be strongly secured from the aspect of safety of vessels.

STEP 3. Examination of SUBSTRUCTURE LOCATION

i) Location of Abutments for Simple Supported Bridge Type
Existing center bridge does not be replaced therefore automatically the span arrangement of side bridge can be determined as 3-span bridge. For determination of the location of abutments, the following items can be carefully evaluated.

- For the abutment A1 at left side bank, excavation during construction phase shall not affect existing roads.
- For the abutment A2 at right side bank, existing bank protection in front of new abutment A2 shall not be affected by the new abutment during completion as well as construction phase.

Based on above consideration, the locations of both abutments are appropriate to be planned in front of existing abutments. The new bridge length is 125m (41.1m+42.8m+41.1m).

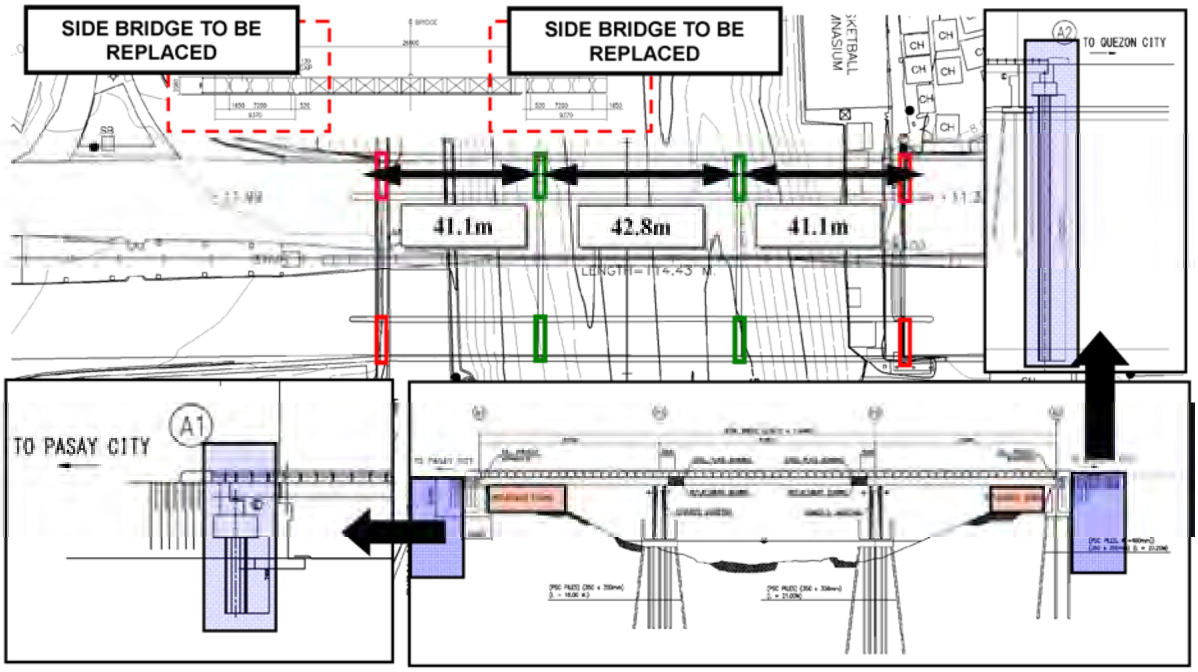


Figure 16.1.2-12 Determination of Abutment Location of Guadalupe Bridge

ii) Span Arrangement in Comparison Study
Following figure shows the span arrangement for comparison study of bridge types.

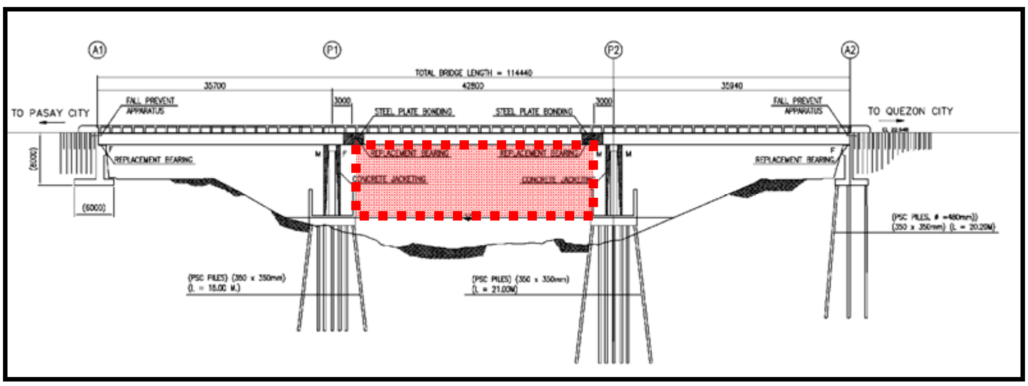


Figure 16.1.2-13 Span Arrangement for Comparison Study

STEP 4. Confirmation of BASIC CONCEPT

The basic concept resulted from above STEP 1 to STEP 3 is enumerated. The basic concept may be significantly important factor for bridge selection under the comparison study.

- New abutments locations are newly determined by the condition of existing structure based on constructability and cost efficiency.
- The location of the piers are not changed >>> 3 Span Bridge Only
- Bridge Length of New Bridge (Side): 125m
- The Span arrangement is 41.1m + 42.8m + 41.1m
- Same navigation clearance and width as those of existing center bridge shall be secured for the new side bridges, the girder height shall be within 2.1m
- To minimize influences of current traffic even during bridge construction stage
- To minimize land acquisition and resettlement of inhabitants even during bridge construction stage

STEP 5. COMPARISON STUDY of New Bridge Types

i) Extraction of Applicable Basic Types based on Actual Results
Based on the basic concept and several conditions, applicable bridge types are extracted from the table of Relationships between Actual Results of Basic Bridge Types and Span Length, shown as follows.

Table 16.1.2-8 Extraction of Applicable Basic Types based on Actual Results

STEL	PC
3-Span	
Continuous RC Slab Steel I-Shape Girder Continuous RC Slab Steel Box Girder Continuous Steel Deck I-Shape Girder Continuous Steel Deck Box Girder Continuous Steel Truss (Tubular) Continuous Steel Truss (Deck Truss)	Continuous PC-I Girder

ii) Selection of Logically Suitable Types from above the Basic Types
Continuous RC slab steel I-shape girder bridge is not included in the comparison study. The girder height including RC slab thickness will be approx. 2.5m, which can not secure existing free board. Besides, slab concrete will be constructed by cast-in-place method, therefore, the construction duration of superstructure will be longer and will affect significant traffic flow.
Continuous steel truss bridge is not included in the comparison. The truss height is to be approx. 4.5m or higher. Logically and structurally this superstructure type can be applied, but the bridge with heavy traffic like Guadalupe bridge, the bridge type that an important structural member exists on the bridge surface and on the same traffic lane affects significantly traffic function and may not be a realistic planning from the point of view of traffic safety and performance of accident processing on the bridge.
Continuous steel truss bridge (deck type truss) is not included in the comparison study. The girder height including RC slab thickness will be approx. 2.5m, which can not secure existing free board. Besides, slab concrete will be constructed by cast-in-place method, therefore, the construction duration of superstructure will be longer and will affect significant traffic flow.
For PC-I shape girder bridge, the girder height will be 2.3m which can not secure existing free board.
Otherwise, both of steel deck box and I-shape girder bridges are included into the comparison study. Both bridge type can be meet the requirement of free board of 2.1m. Generally, steel box girders are more expensive than steel I-shape girders. However because the bridge width of Guadalupe bridge is comparatively narrow, one-box type can be applied, which may reduce its cost efficiently.

Based on the above evaluation, multiple comparison study is conducted considering cost, structure, environmental impact, constructability and maintenance ability.

Table 16.1.2-9 Candidates of Comparison Study

SEEL	PC
3 Span	
Continuous Steel Deck I-Shape Girder Continuous Steel Deck Box Girder	—

Based on the study shown in the comparison table, the recommendable bridge type for outline design is **3-Span Continuous Steel Deck Box Girder Bridge.**

iii) Selection of Logically Suitable Types of Bridge Foundations

In the study on selection of bridge foundation, study of site conditions for selecting an appropriate foundation type is a key discussion. In the study on selection of Guadarupe bridge foundation, study of an abutment foundation type, pier foundation type and comparison of structure (pile type) are the key discussions.

The site conditions are shown in below table. In the abutment, particularly it was located very close to existing abutment, meanwhile, the Pier foundation is located very close to navigation channel.

Table 16.1.2-10 Site Candidates of Comparison Study

Study Type	Abutment Foundation	Pier Foundation
Foundation location	On the ground	In the River
proximity structure	Closed to existing abutment	Closed to narrow navigation channel
Navigation condition	-	Very close to existing piers
Max. Water depth (m)	-	Around 10.5 m
Depth of bearing layer (m)	Around GL- 1.0m (A1) Around GL-9.0m(A2)	Around GL-15.0m
Type of bearing layer	Rock/Sand	Sand
Liquefaction	liquefiable site	liquefiable site
Lateral spreading	-	-

Based on the above site conditions, applicable foundation types are extracted, shown as follows.

Table 16.1.2-11 Extraction of Applicable Basic Types based on Actual Results

Abutment Foundation	Pier Foundation
Large Diameter Bored Pile (Rotary all casing boring method) Commonly used Cast-in-place Concrete Pile (Reverse circulation drill method)	Steel Pipe Sheet Pile Foundation (Press-in method) Commonly used Cast-in-place Concrete Pile (Reverse circulation drill method)

According to the above evaluation, multiple comparison study is conducted considering cost, construction period, neighboring construction, constructability, and environmental impact.

The result of comparative study of abutment foundation are shown in the next tables, the recommendable abutment foundation type for outline design is Large Diameter Bored Pile, because of its advantages in low construction cost and shortest construction period on account of consider neighboring Constructability.

The recommendable pier foundation type for outline design is Steel Pipe Sheet Pile (SPSP) Foundation, because of its advantages in minimum term of construction period & traffic control with efficient workability in narrow navigation.

Table 16.1.2-12 Candidates of Comparison Study

Abutment Foundation	Pier Foundation
3-Span	
Large Diameter Bored Pile (Rotary all casing boring method)	Steel Pipe Sheet Pile Foundation (Press-in method)

Table 16.1.2-14 Comparison on Abutment Foundation Type of Guadarupe Bridge

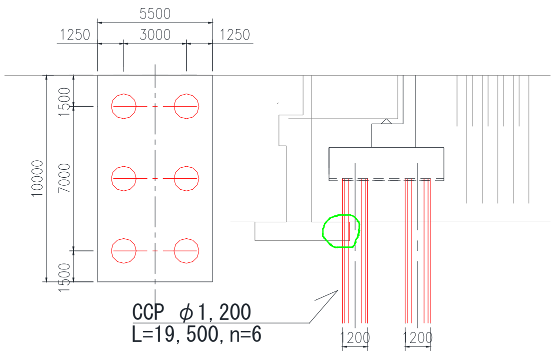
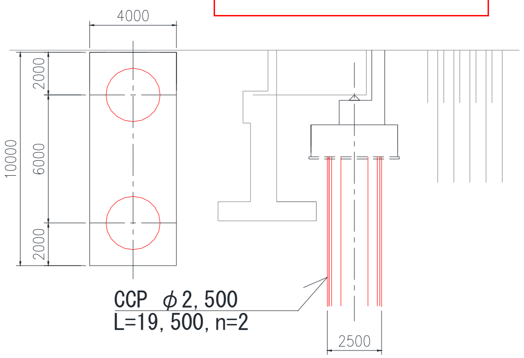
Evaluation Items	Alternative-1 (reverse circulation drill method) Cas-in-place Concrete Pile D= 1.2m		Alternative-2 (rotary all casing boring method) Cas-in-place Concrete Pile D= 2.5m	
Side View Pile arrangement	Diameter of pile : 1200 mm Total number of pile : 6 Pile length : 19.0 m Total length of pile : 114.0 m  		Diameter of pile : 2500 mm Total number of pile : 2 Pile length : 19.0 m Total length of pile : 38.0 m OUTLINE DESIGN  	
Structural Aspect and Stability	- Pile Displacement Ratio (Pile dosplacement/displacement limit) is 0.767 - Need to the large number of Cast-in-Place Concrete	A	- Pile Displacement Ratio (Pile dosplacement/displacement limit) is 0.768	A
Construction Cost (for Foundation)	| Item | Quantity | Unit Cost (Php) | Total (1,000Php) | Pile Cap Concrete | 212m3 | 7,559.8 | 1,603 Reinforcement steel | 42ton | 52,600.0 | 2,230 Pile | 114m | 45,898.5 | 5,232 Cofferdam | 390m2 | 21,181.9 | 8,261 Total | 17,326 Ratio | **1.128**	C	| Item | Quantity | Unit Cost (Php) | Total (1,000Php) | Pile Cap Concrete | 182m3 | 7,559.8 | 1,376 Reinforcement steel | 36ton | 52,600.0 | 1,915 Pile | 38m | 116,987.0 | 4,446 Cofferdam | 360m2 | 21,181.9 | 7,625 Total | 15,362 Ratio | **1.000**	A
Construction Plan and Period	- Working in Temporary cofferdam & low spaces of under the temporary stage. Cofferdam Work 14 days Pile work (1.5pile/day) 9 days Pile Cap 21 days Total **45 days** Ratio **1.256**	C	- Working in Temporary cofferdam & low spaces of under the temporary stage. Cofferdam Work 13 days Pile work (2pile/day) 4 days Pile Cap 18 days Total **36 days** Ratio **1.000**	A
Neighboring Construction	- Not keep out of existing abutment and new abutment foundation.	C	- Keep out of existing abutment and new abutment foundation by Press-in Pile Driving Method	A
Constructability	- Constructability is inferior due to large number of Cast-in-place concrete Pile work.	B	- Constructability is superior with small number of foundation work.	A
Environmental Aspect	- Inferior in Environmental aspect due to large number of excavated soil.	B	- Superior in Environmental aspect with small number of excavated soil.	A
Evaluation	- Construction cost is highest with long construction period. -	**B**	- Construction cost is lowest with minimum Construction period with efficient workability. Recommended	**A**

Table 16.1.2-15 Comparison on Pier Foundation（P2）Type of Guadarupe Bridge

OUTLINE DESIGN

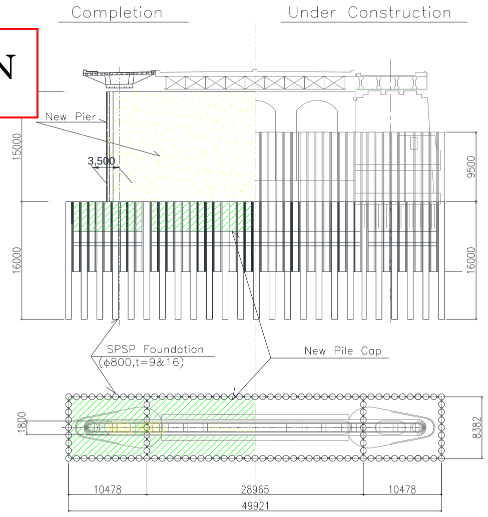
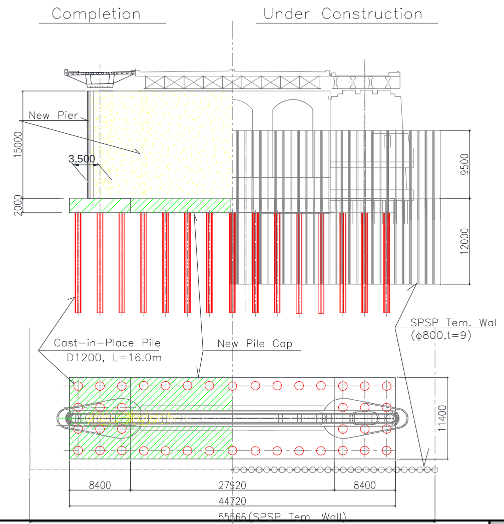
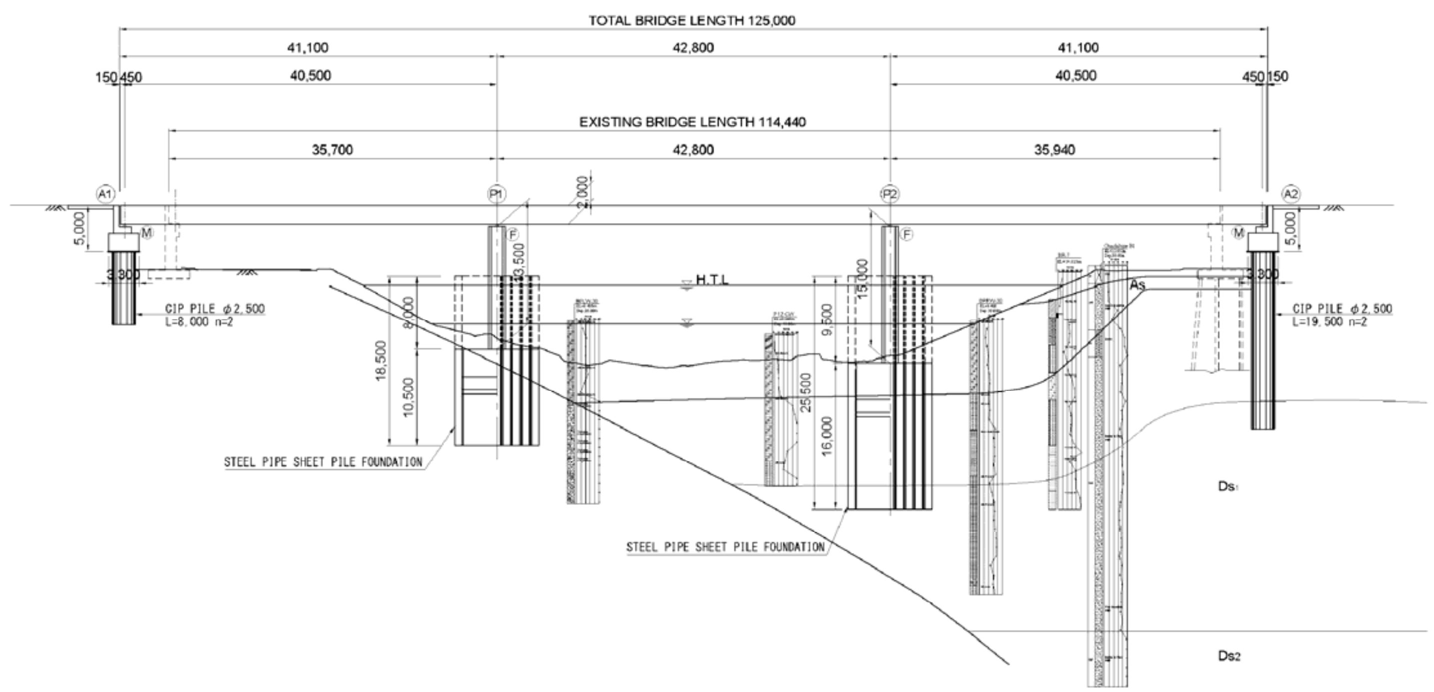
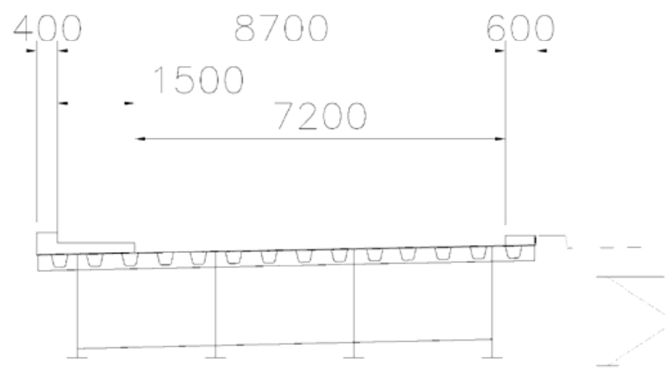
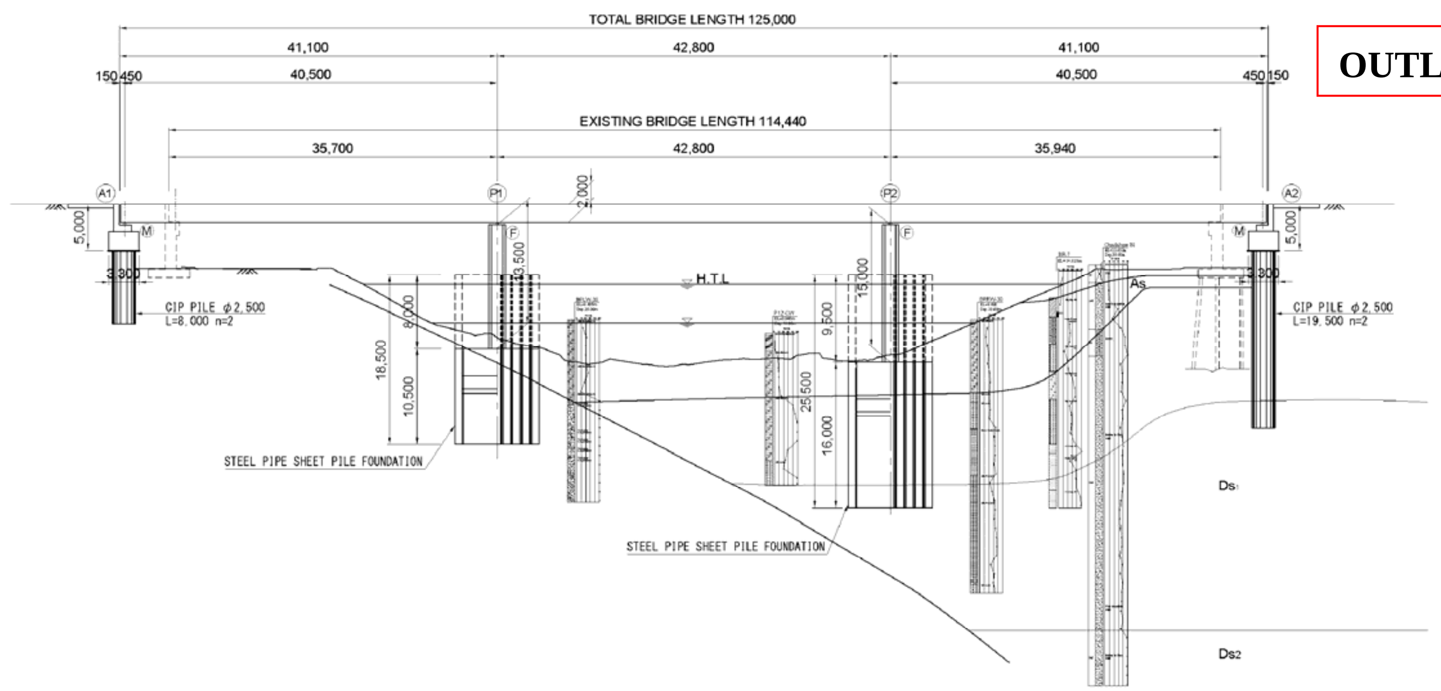
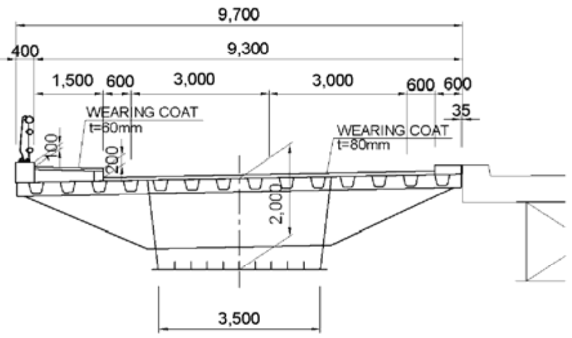
	Alternative-1 SPSP Foundation (D=0.8m, L=16m)		Alternative-2 Cas-in-place Concrete Pile Foundation（D=1.2m, L=14m） with Temporary SPSP (self-support type)	
Side View				
Structural Aspect	- Small number of Pile Cap concrete.	A	- Large number of Pile Cap concrete and C.I.P. Piles	B
Navigation Width at during construction)	- Wilder (Over 25.0m)	A	- Narrow (Under 25.0m) due to temporary SPSP cofferdam	C
Neighboring Construction	- Superior in neightborning construction with Press-in Pile Driving Method.	A	- Inferior in neightborning construction with the reverse circulation drill method.	B
Constructability of under girder	- Constructability in superior with small number of foundation work.	A	- Constructability is inferior due to large number of foundation work.	B
Term of Foundation Construction	- Shorter (48 months)	A	- Longer (61months)	C
Term of Traffic Control	- Shorter (20 months)	A	- Longer (26months)	B
Total Cost (Ratio)	- A Little Expensive (1.111)	B	- A Little Inexpensive (1.000)	A
Environmental Aspect	- Superior in Environmental aspect with small number of excavated soil.	A	- Inferior in Environmental aspect due to large number of excavated soil.	B
Evaluation	- Minimum term of construction period & traffic control with efficient workability. - Superior in Environmental aspect with small number of excavated		- Maximum term of construction period & traffic control. - Workability is inferior due to large number of foundation work in narrow river.	
	Recommended		-	

Table 16.1.2-16 Comparison of New Bridge Types for Guadalupe Side bridge

	Side View	Sectional View	Evaluation: Good·A·Fair·B·Poor·C			
Alternative 1	3·Span Continuous Steel Deck I·Shape Girder 		COST	Superstructure	0.44	B
				Substructure	0.13	
				Foundation	0.35	
				Others:	0.10	
				Total	1.02	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			B
			ENVIRONMENTAL IMPACT			B
			MAINTENANCE			B
Alternative 2	3·Span Continuous Steel Deck Box Girder 	OUTLINE DESIGN 	COST	Superstructure	0.42	B
				Substructure	0.13	
				Foundation	0.35	
				Others	0.10	
				Total	1.00	
			STRUCTURAL FEATURE			A
			CONSTRUCTABILITY			A
			ENVIRONMENTAL IMPACT			A
			MAINTENANCE			B

(3) Palanit Bridge

STEP 1. Confirmation of ROAD CONDITION

i) Bridge Width

For the cross section and lane arrangement of new bridge, the examined results of road planning including approach roads shall be applied, shown as follows.

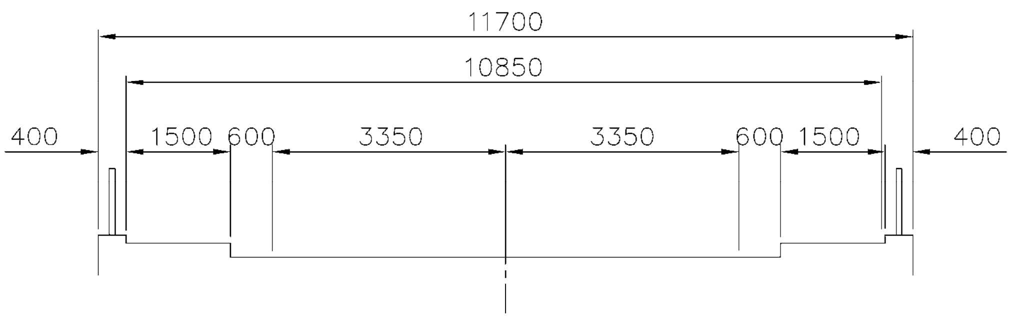


Figure 16.1.2-14 Cross Section/ Lane Arrangement of Palanit Bridge

ii) Rising of Vertical Alignment

Existing bridge is steel truss type bridge with 1.3m of substance girder height, which can secure 1.5m of free board. However, in this project, applicability of other bridge types except steel truss tubular bridge should be examined based on multiple new bridge comparison study. Therefore, road condition around the approach bridge is verified in case of rising of vertical alignment for the purpose to evaluate the applicability of other bridge type, the substance girder height of which is over 1.3m.

<1.5m rising of vertical alignment >

Impact:

- 20 - 25cm rising caused in approach road
- Impact against settlements beside the approach roads may be slight because the influence of rising can be stay around 20 or 25cm
- In case that amount of rising can be keep under 1.0m, the rising caused around approach roads can be absorbed by vertical slope on the bridge
- Therefore, as above, for Palanit bridge, 1.5m rising of vertical alignment can be acceptable, and the additional costs due to the rising shall be partly included to the relevant structure

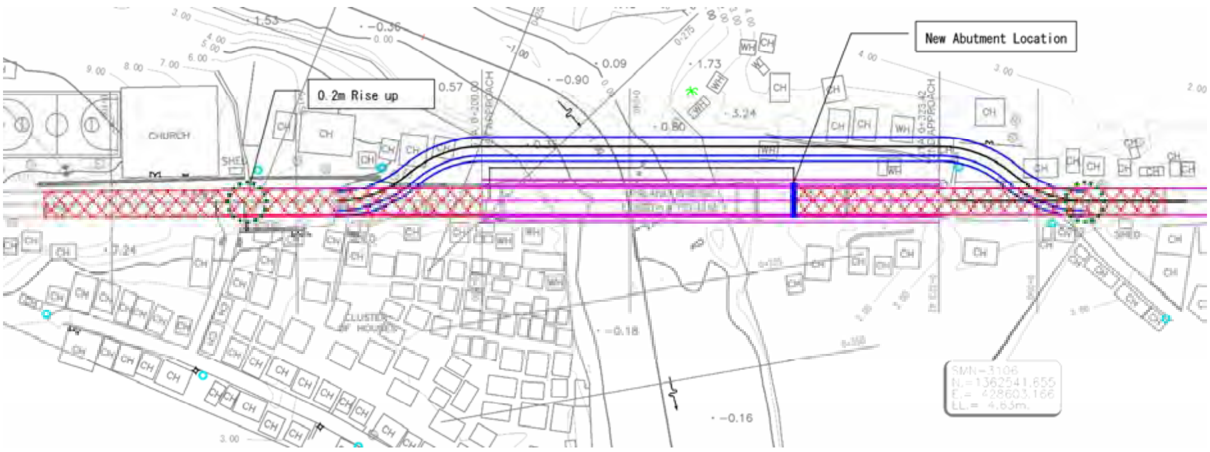


Figure 16.1.2-15 Rising of Vertical Alignment

STEP 2. Confirmation of HYDRAULIC CONDITION

For hydraulic condition in the new bridge selection, the determined results in this project shall be applied shown as follows. For the design flood level, the level of 1.9m (197m3/s) which is a design high water level may be suitable for the design. The free board is determined as 1.5m or more.

This water level is determined by simple hydraulic analysis and interview. Therefore, in the detail design stage, detail hydraulic analysis should be carried out to verify the level of high water level.

Table 16.1.2-17 DHW of Palanit Bridge

Location	Palanit Bridge		Mawo Bridge	
Return Period	50-year	100-year	50-year	100-year
Design Flood Discharge (m^3/s) (calculated by Specific Discharge Method)	164	197	1,035	1,245
Flow Velocity (m/s)	1.79	1.92	1.47	1.75
Design Flood Level (m)	1.72	1.90	1.31	1.35

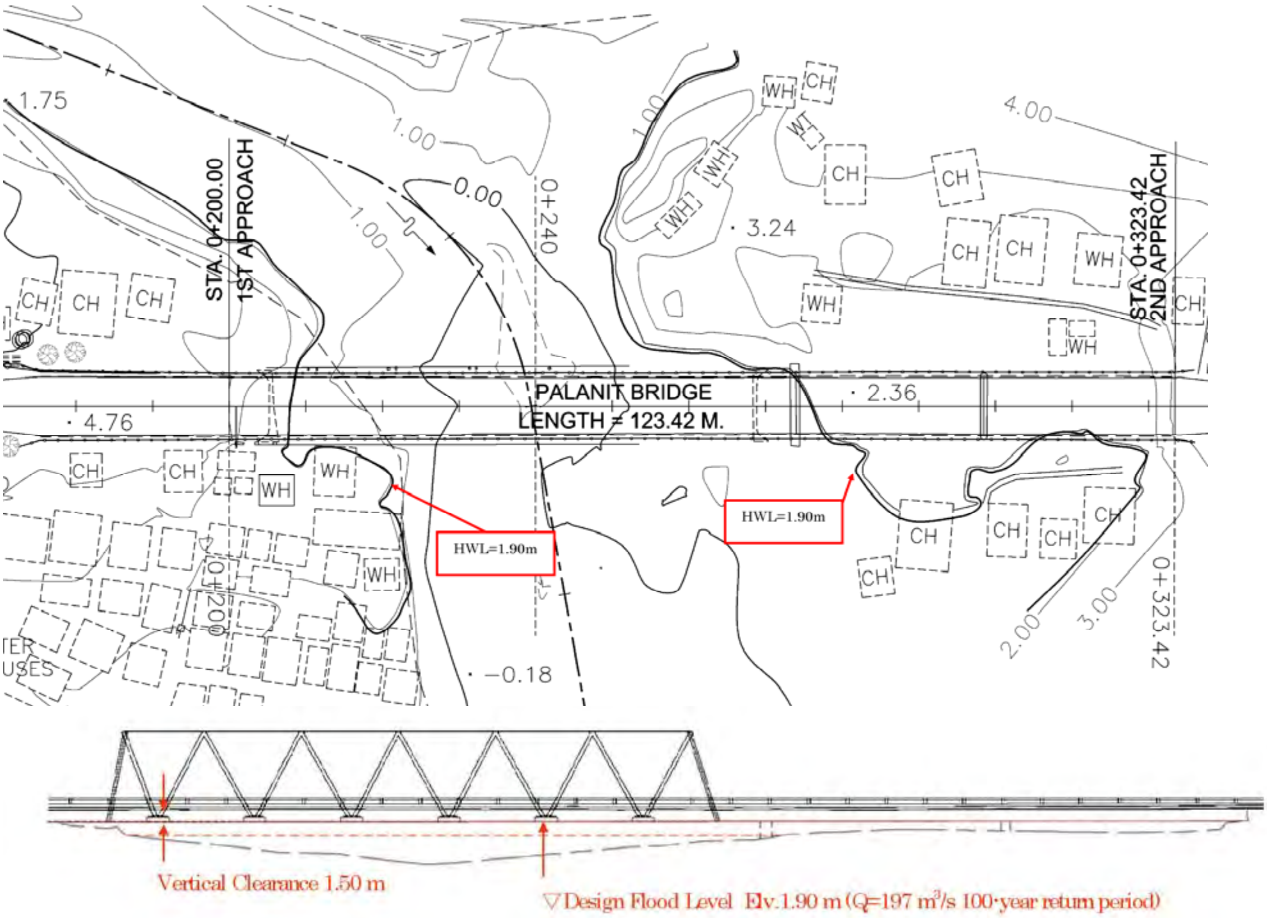


Figure 16.1.2-16 DHW and Free Board of Palanit Bridge

STEP 3. Examination of SUBSTRUCTURE LOCATION

i) Location of Abutments

The locations of abutments are determined based on high water level, determined by hydraulic analysis and site interview. The abutment shall not be affected by the boundary lines of high water level. The new bridge length considering such the condition is 82m.

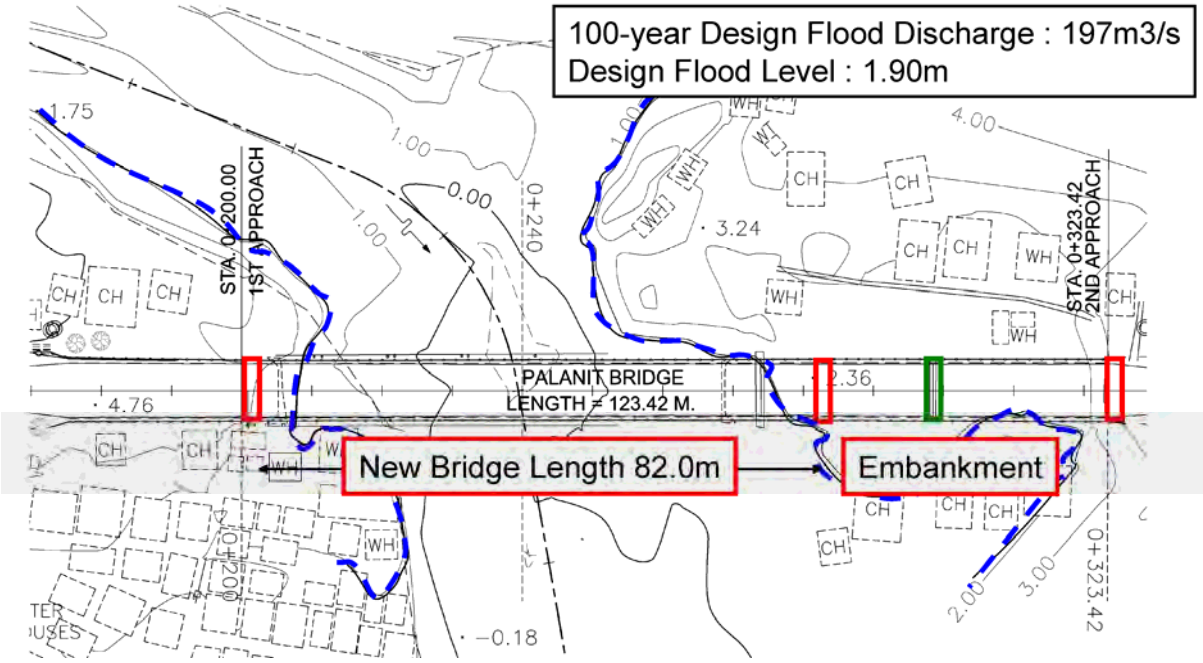


Figure 16.1.2-17 Location of Abutments

ii) Span Arrangement in Comparison Study

The water depth under the bridge is very shallow such as 50cm to 100cm and hard rocks are exposed. Therefore, even middle size of barges is not passable under the bridge; inhabitants bring small boats directly into the river. Therefore, following four of points shall be considered:

- Piers should not interrupt inhabitants' small boat
- Centerline of stream should be opened not to interrupt navigating boats
- As reference, understand the value of water rising due to inhibition ration by gross hydraulic analysis

Therefore, piers can basically be installed outside the area by red dashed lines in the following figure.

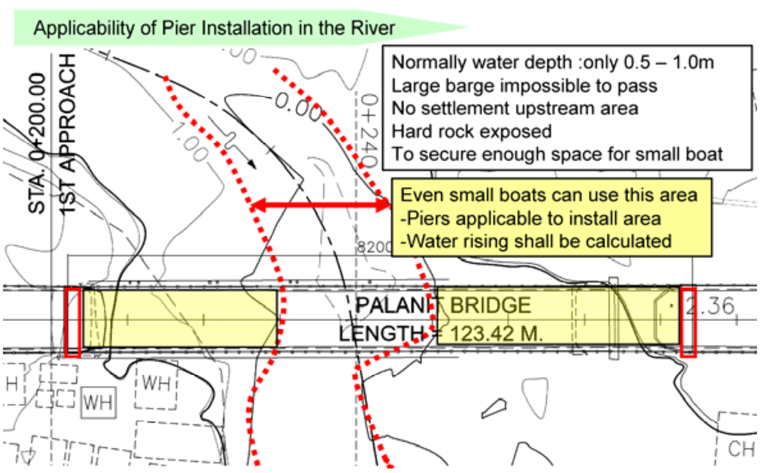


Figure 16.1.2-18 Installable Area of Piers

<2-Span Bridge>

A pier should be installed at the center of the river, which affect navigating boats. Otherwise, water rising is not confirmed by gross hydraulic analysis. Therefore, this span arrangement is not the most appropriate span arrangement but is included into comparison study.

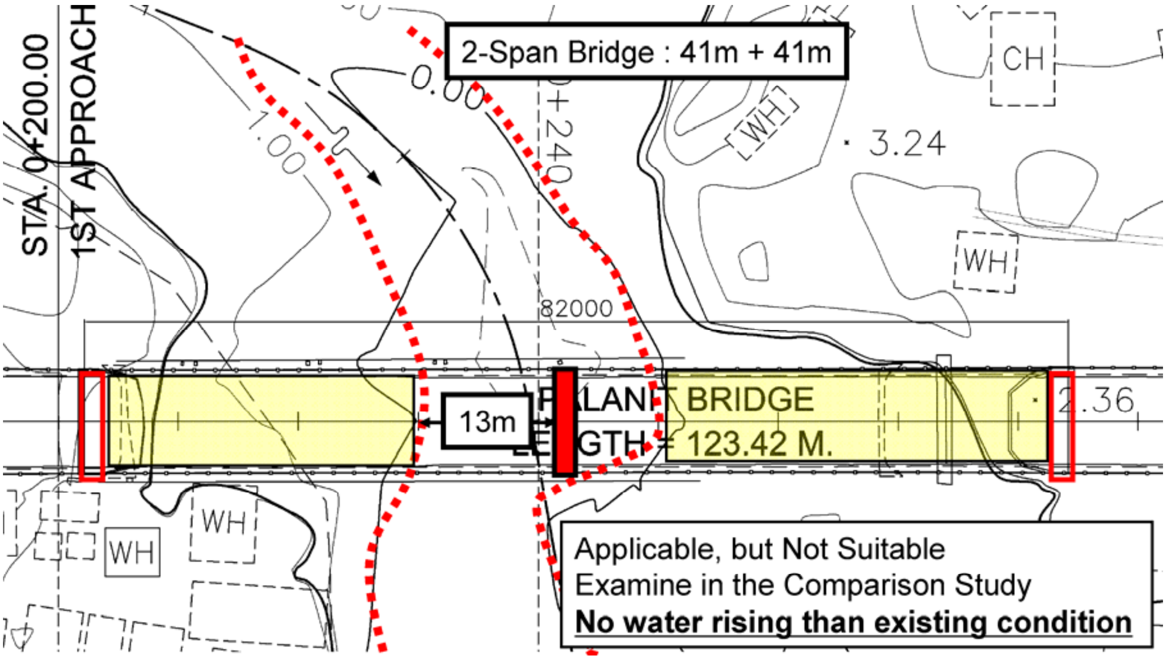


Figure 16.1.2-19 2 Span Bridge

<3-Span Bridge>

For 3-span bridge, the center span can be a role of opening section for the navigating boats, which may be very advantage. Otherwise, the inhibition ratio due to the substructures is approx 4.9%, however the water rising calculated by this condition is very slight.

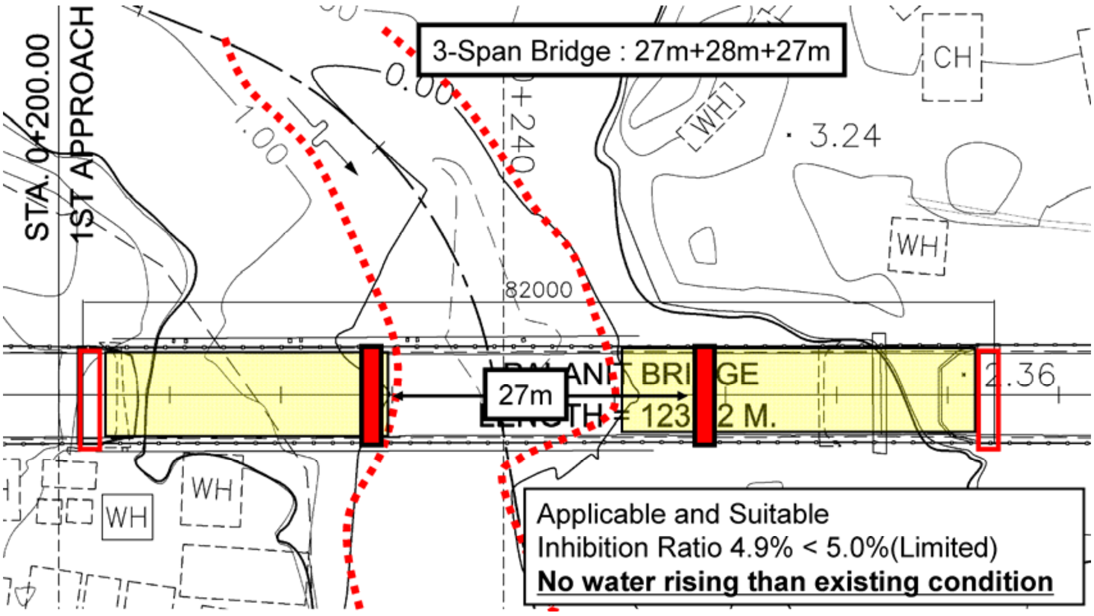


Figure 16.1.2-20 3 Span Bridge

<4-Span Bridge>

For 4-span bridge, piers in the river must interrupt navigating boats. Besides, the inhibition ratio due to substructures is approx. 6.8% that causes about 15cm water rising from the result of gross calculation. Therefore, to apply this number or more spans cause hydraulic problems absolutely; hence this span arrangement is not included into the comparison study.

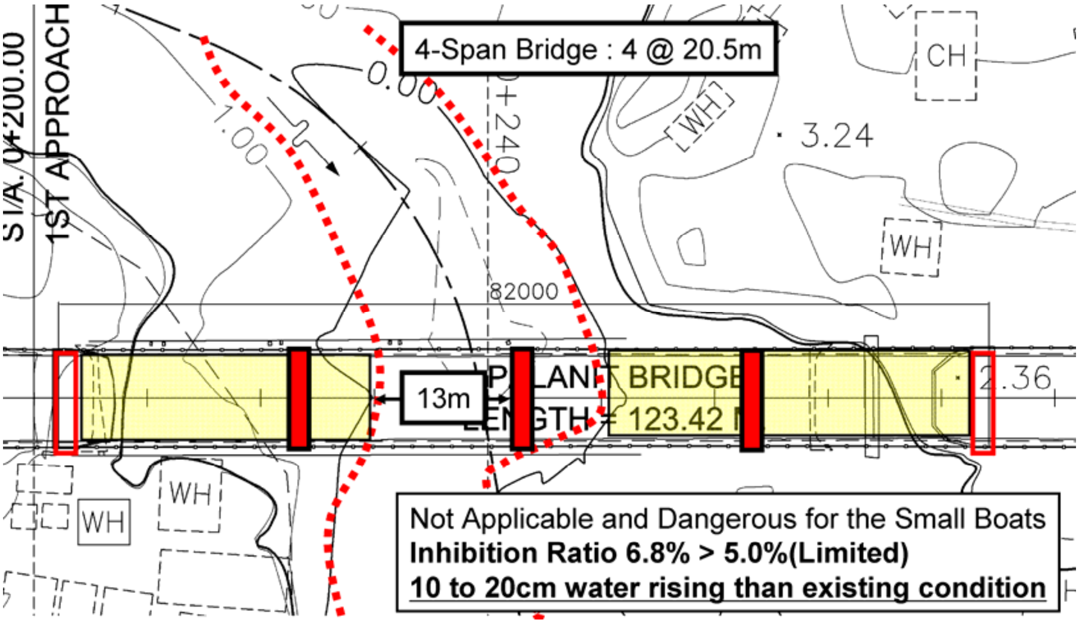


Figure 16.1.2-21 4 Span Bridge

STEP 4. Confirmation of BASIC CONCEPT

The basic concept resulted from above STEP 1 to STEP 3 is enumerated. The basic concept may be significantly important factor for bridge selection under the comparison study.

- New abutments locations are determined by Hydraulic analysis
- Such the new abutments are not affected by the Flood and High Tide
- Bridge Length of New Bridge: 82m, Other section is embankment of 123m

- Oceanfront/ Shelving bottom/ No utilization as a Port
- No settlement at the upstream area of the river
- Only small boats passing , normal barge or ships can not pass due to shallow water and No need to use such the barges because no settlements at the upstream area
- 2-Span (Max 41m) or 3-Span (Max 28m) is available span arrangement
- Minimal maintenance bridge will be recommended, reflecting on-site request.
- Girder height can be allowed until 2.8m

STEP 5. COMPARISON STUDY of New Bridge Types

i) Extraction of Applicable Basic Types based on Actual Results
Based on the basic concept and several conditions, applicable bridge types are extracted from the table of Relationships between Actual Results of Basic Bridge Types and Span Length, shown as follows.

Table 16.1.2-18 Extraction of Applicable Basic Types based on Actual Results

STEEL	PC
Simple Supported	
Steel Deck Box Steel Truss (Tubular) Steel Lohse Arch	—
2-Span	
Continuous RC Slab Steel I-Shape Girder	Continuous PC Box Continuous PC-I Girder
3-Span	
Continuous RC Slab Steel I-Shape Girder Continuous RC Slab Steel Box Girder	Continuous PC-I Girder

ii) Selection of Logically Suitable Types from above the Basic Types
Suitable bridge types are selected logically among above extracted bridges, to be utilized for final comparison study as follows:

Table 16.1.2-19 Extraction of Basic Types for Final Comparison Study (Steel)

STEEL	Inclusion of Final Comparison Study
Simple supported Steel Deck Box	- Not Included in Final Comparison Study - Girder height 3.0m, Not accepted - Over Specification - Disadvantage for Maintenance ability
Simple Supported Steel Truss (Tubular)	- Included in Final Comparison Study - Classic Truss - Disadvantage for Maintenance ability
Simple Supported Steel Lohse Arch	- Not Included in Final Comparison Study - Over Specification - Disadvantage for Maintenance ability
2-Span RC Slab Steel I-Shape Girder	- Included in Final Comparison Study - Classic Type Steel Girder Bridge - Girder height 2.4m, 1.1m road rising - Disadvantage for Maintenance ability - Disadvantage for navigating small boat
3-Span RC Slab Steel I-Shape Girder	- Included in Final Comparison Study - Classic Type Steel Girder Bridge - Girder height 1.8m, approx. 0.5m road rising - Disadvantage for Maintenance ability - Disadvantage for navigating small boat

Table 16.1.2-20 Extraction of Basic Types for Final Comparison Study (PC)

PC	Inclusion of Final Comparison Study
2-Span PC Box Girder	**- Included in Final Comparison Study** - Girder height 2.1m, approx 0.8m road rising - Disadvantage for navigating small boat
2-Span PC-I Girder	**- Included in Final Comparison Study** - Girder height 2.3m, approx 1.1m road rising - Disadvantage for navigating small boat
3-Span PC-I Girder	**- Included in Final Comparison Study** - Girder height 1.7m, approx 0.5m road rising

Based on the above evaluation, multiple comparison study is conducted considering cost, structure, environmental impact, constructability and maintenance ability.

Table 16.1.2-21 Candidates of Final Comparison Study

STEEL	PC
Simple Supported	
Simple Supported Steel Truss (Tubular)	**—**
2 Span	
2-Span RC Slab Steel I-Shape Girder	**2-Span PC Box Girder** **2-Span PC-I Girder**
3 Span	
3-Span RC Slab Steel I-Shape Girder	**3-Span PC-I Girder**

Based on the evaluation, shown in the comparison table, the recommendable bridge type for outline design is **3-Span Connected PC-I Girder bridge**.

iii) Selection of Logically Suitable Types of Bridge Foundations
In the study on selection of bridge foundation, study of site conditions for selecting an appropriate foundation type is a key discussion.

The site conditions are shown in below table. In the abutments & Piers were located on the rock with shallow water.

Table 16.1.2-22 Site Candidates of Comparison Study

Study Type	Abutment Foundation	Pier Foundation
Foundation location	On the ground	In the River
Proximity structure	-	-
Navigation condition	-	No navigation
Max. Water depth (m)	-	Around 3.0 m
Depth of bearing layer (m)	Around GL- 1.0m (A1) Around GL-9.0m(A2)	Around GL-3. m
Type of bearing layer	Rock	Rock
Liquefaction	-	-
Lateral spreading	-	-

Based on the above site conditions, applicable foundation types are recommended spread footing type.

Table 16.1.2-23 Comparison of New Bridge Types for Palanit bridge (STEEL)

STEEL BRIDGES

COMPARISON of NEW BRIDGE TYPES for PALANIT BRIDGE

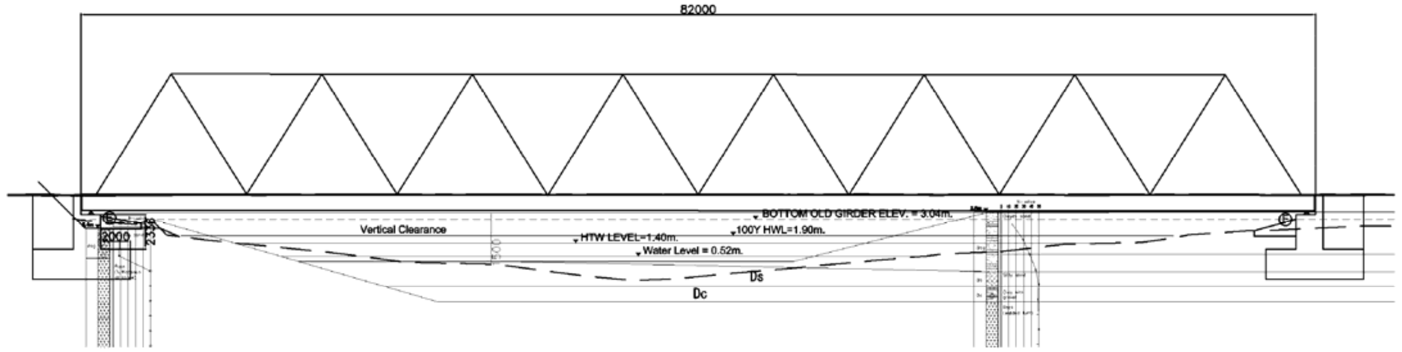
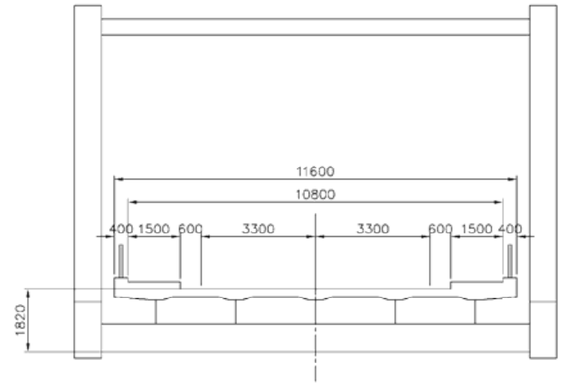
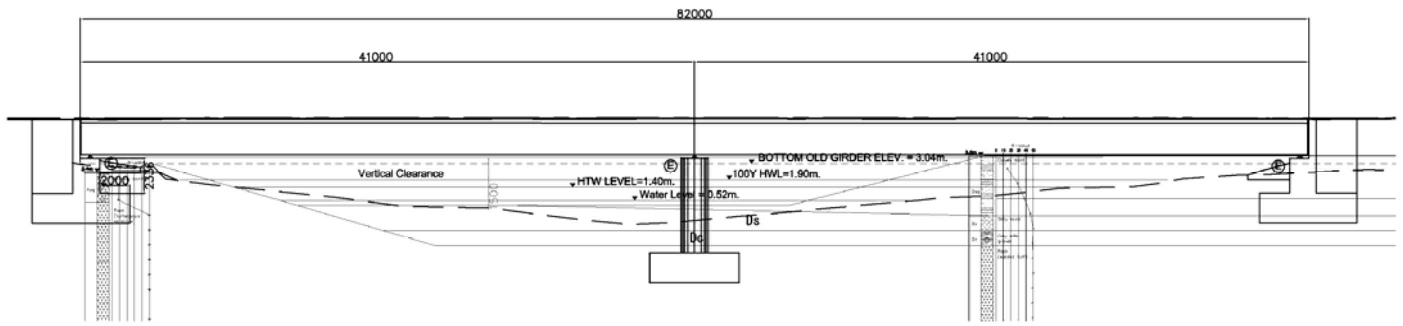
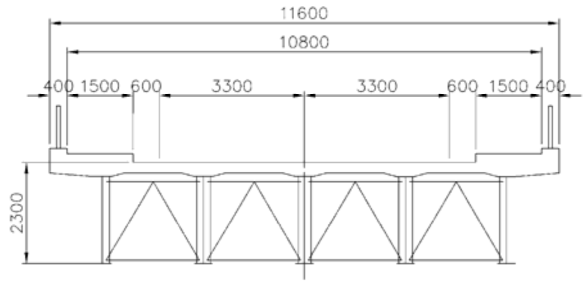
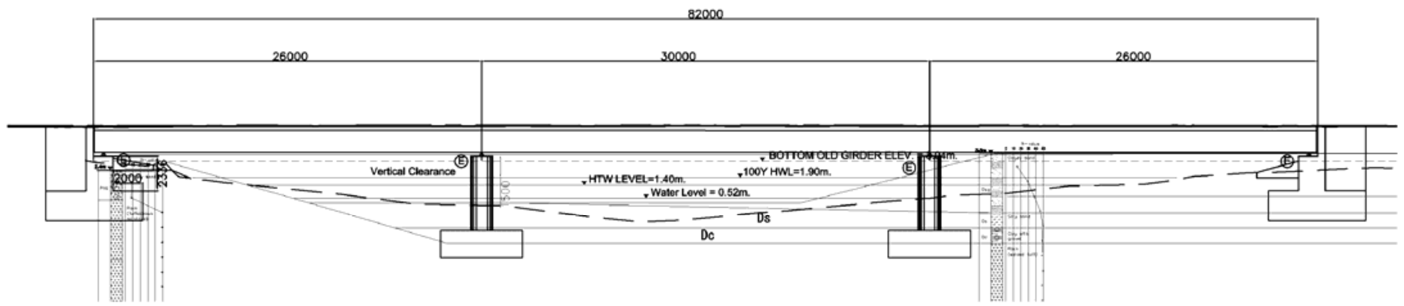
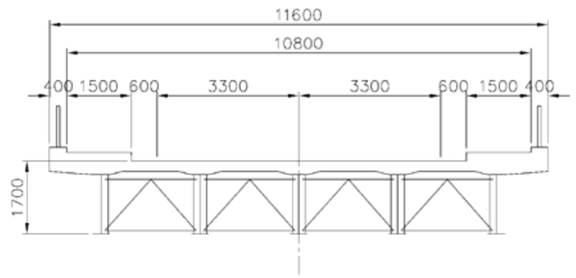
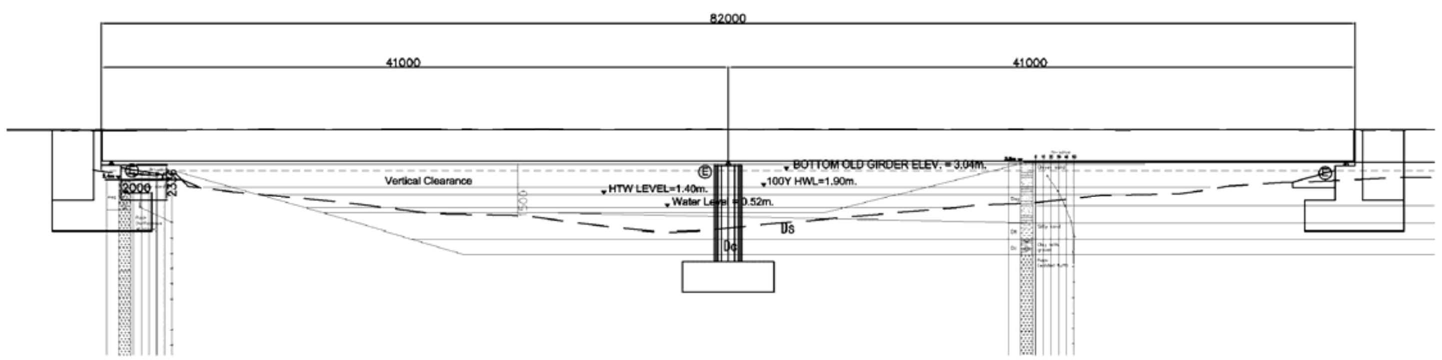
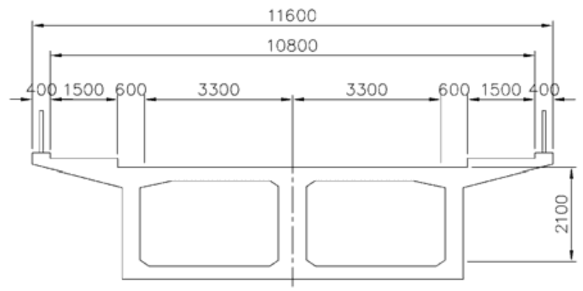
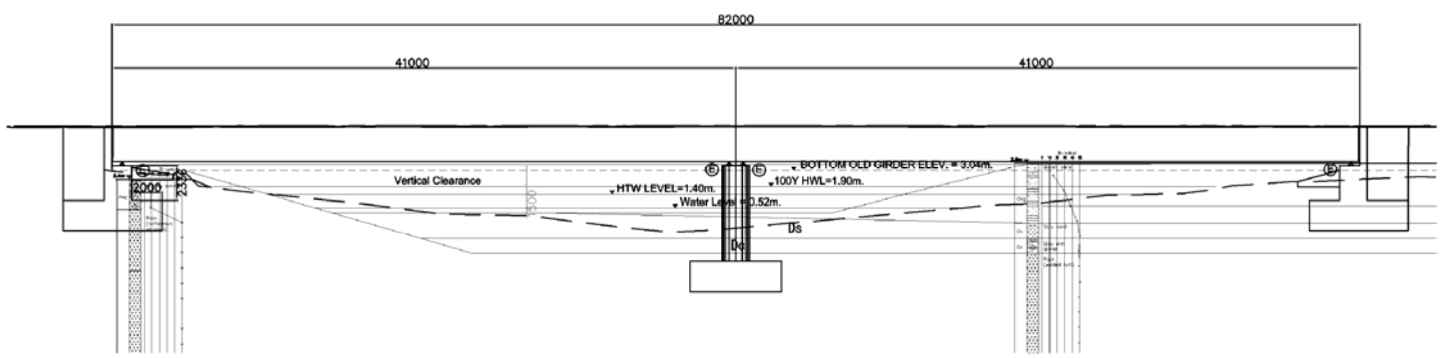
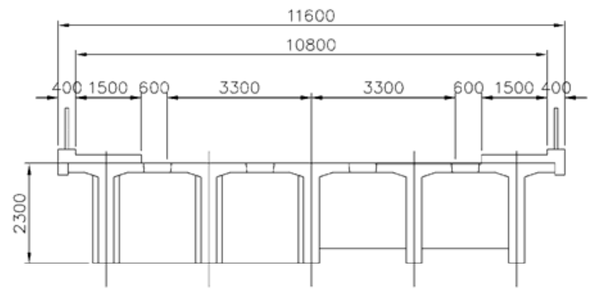
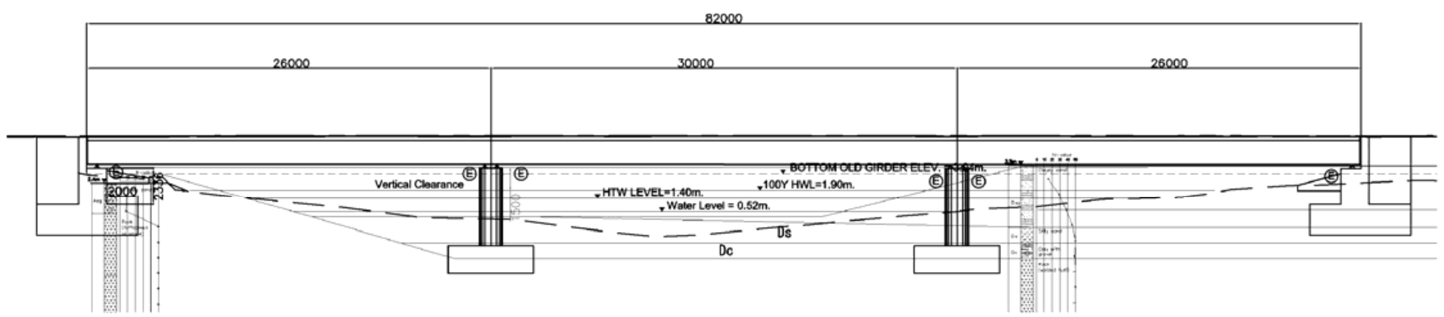
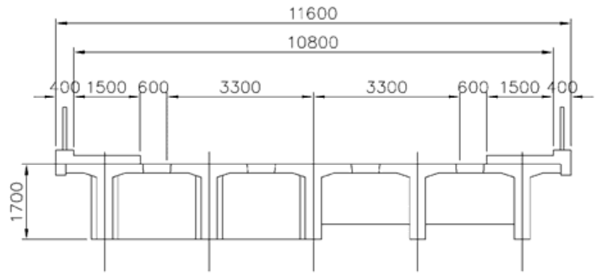
	Side View	Sectional View	Evaluation: Good-A-Fair-B-Poor-C			
Alternative 1S	Simple Supported Steel Truss 		COST	Superstructure	0.95	C
				Substructure	0.06	
				Road	0.00	
				Others	0.33	
				Total	1.34	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			B
			ENVIRONMENTAL IMPACT			A
			MAINTENANCE			C
Alternative 2S	2-Span Continuous Steel I-Shpae Girder 		COST	Superstructure	0.58	B
				Substructure	0.11	
				Road	0.04	
				Others	0.33	
				Total	1.06	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			A
			ENVIRONMENTAL IMPACT			C
			MAINTENANCE			B
Alternative 3S	3-Span Continuous Steel I-Shpae Girder 		COST	Superstructure	0.54	B
				Substructure	0.13	
				Foundation	0.02	
				Others	0.33	
				Total	1.01	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			A
			ENVIRONMENTAL IMPACT			A
			MAINTENANCE			B

Table 16.1.2-24 Comparison of New Bridge Types for Palanit bridge (PC)

PC BBRIDGE

COMPARISON of NEW BRIDGE TYPES for PALANIT BRIDGE

	Side View	Sectional View	Evaluation: Good-A-Fair-B-Poor-C			
Alternative 1P	2-Span Continuous PC Box Girder 		COST	Superstructure	0.62	B
				Substructure	0.06	
				Road	0.03	
				Others	0.33	
				Total	1.05	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			C
			ENVIRONMENTAL IMPACT			B
			MAINTENANCE			A
Alternative 2P	2-Span Connected PC Post-Tension Girder 		COST	Superstructure	0.56	B
				Substructure	0.11	
				Road	0.04	
				Others	0.33	
				Total	1.04	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			B
			ENVIRONMENTAL IMPACT			B
			MAINTENANCE			A
Alternative 3P	3-Span Connected PC Post-Tension Girder OUTLINE DESIGN 		COST	Superstructure	0.52	A
				Substructure	0.13	
				Road	0.02	
				Others	0.33	
				Total	1.00	
			STRUCTURAL FEATURE			A
			CONSTRUCTABILITY			B
			ENVIRONMENTAL IMPACT			A
			MAINTENANCE			A

(4) Mawo Bridge

<u>STEP 1. Confirmation of ROAD CONDITION</u>

i) Bridge Width

For the cross section and lane arrangement of new bridge, the examined results of road planning including approach roads shall be applied, shown as follows.

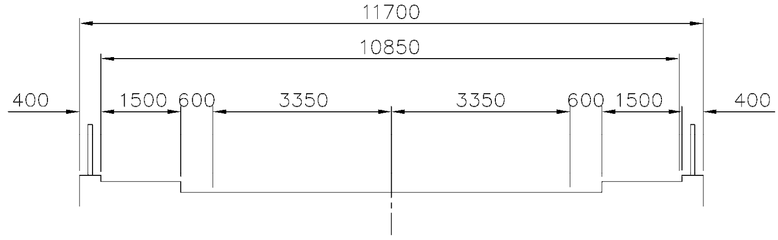


Figure 16.1.2-22 Cross Section/ Lane Arrangement of Mawo Bridge

ii) Rising of Vertical Alignment

Existing bridge consists of two of simple supported steel Langer bridges with 1.6m of substance girder height, which can secure 1.5m of free board. However, in this project, applicability of other bridge types except steel arch bridge should be examined based on multiple new bridge comparison study. Therefore, road condition around the approach bridge is verified in case of rising of vertical alignment for the purpose to evaluate the applicability of other bridge type, the substance girder height of which is over 1.6m.

<1.5m rising of vertical alignment>

Impact:

- 45cm road rising at left side approach road and 1.6m road rising at right side approach road
- The gradient of sub approach road joining to main road will be approx. 8% over
- In case of 1.6m road rising at right side bank, inhabitants can not utilize the main road as residential road. They need new detour long sub approach road
- Therefore, as above, for Mawo bridge, 1.5m rising of vertical alignment, what is called as large scale rising, can not acceptable.
- Otherwise, in case that amount of rising can be keep under 0.5m, the rising caused around approach roads can be absorbed by vertical slope on the bridge, which should be included as additional costs due to the rising shall be partly included to the relevant structure

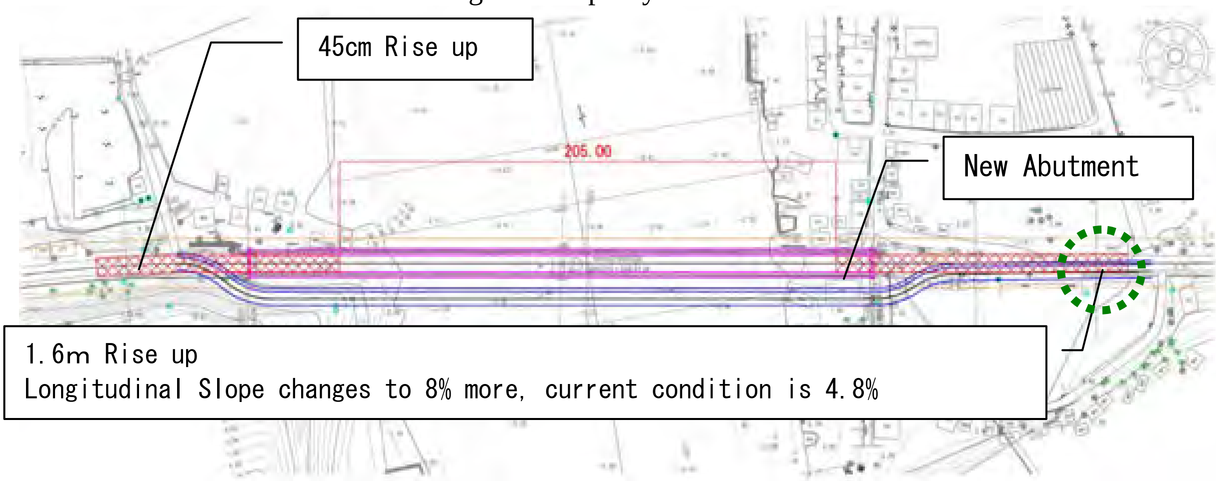


Figure 16.1.2-23 Rising of Vertical Alignment

STEP 2. Confirmation of HYDRAULIC CONDITION

For hydraulic condition in the new bridge selection, the determined results in this project shall be applied shown as follows. The design flood level is 1.35m (1245m3/s, 100yrs). However, the High Tide Water Level is observed as 1.40m. Therefore, as the design water level the High Tide Water Level should be utilized. The free board is determined as 1.5m.

However, around this area, the water flows is comparatively complicated condition hydraulically. And the results of hydraulic analysis will affect critically the results of bridge planning. Therefore, in detail design stage, detail hydraulic analysis based upon further investigation shall be implemented by river/ hydraulic engineers.

Table 16.1.2-25 DHW of Mawo Bridge

Location	Palanit Bridge		Mawo Bridge	
Return Period	50-year	100-year	50-year	100-year
Design Flood Discharge (m^3/s) (calculated by Specific Discharge Method)	164	197	1,035	1,245
Flow Velocity (m/s)	1.79	1.92	1.47	1.75
Design Flood Level (m)	1.72	1.90	1.31	1.35

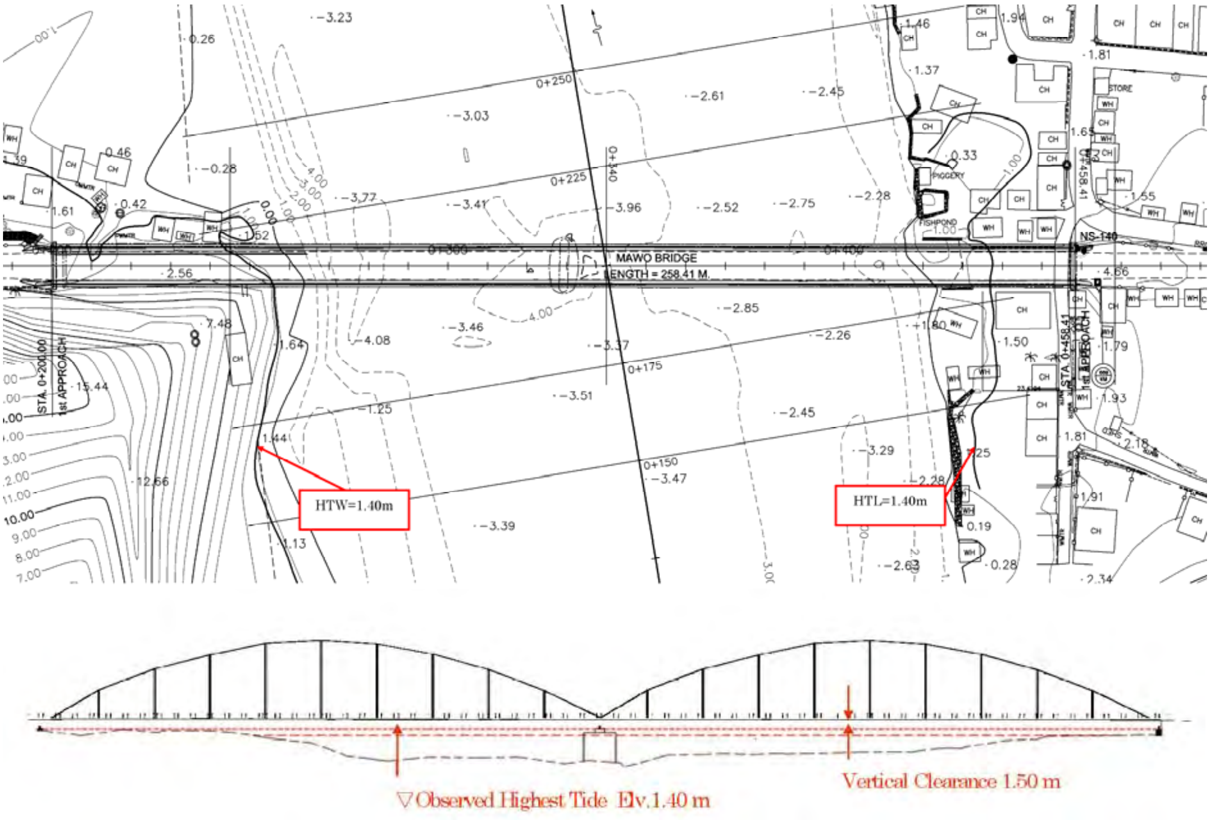


Figure 16.1.2-24 DHW and Free Board of Mawo Bridge

STEP 3. Examination of SUBSTRUCTURE LOCATION

i) Location of Abutments

The locations of abutments are determined based on high water level, determined by hydraulic analysis and site interview. The abutment shall not be affected by the boundary lines of high water level. The new bridge length considering such the condition is 205m.

However, around this area, the water flows is comparatively complicated condition hydraulically. And the results of hydraulic analysis will affect critically the results of bridge planning. Therefore, in detail design stage, detail hydraulic analysis based upon further investigation shall be implemented by river/ hydraulic engineers.

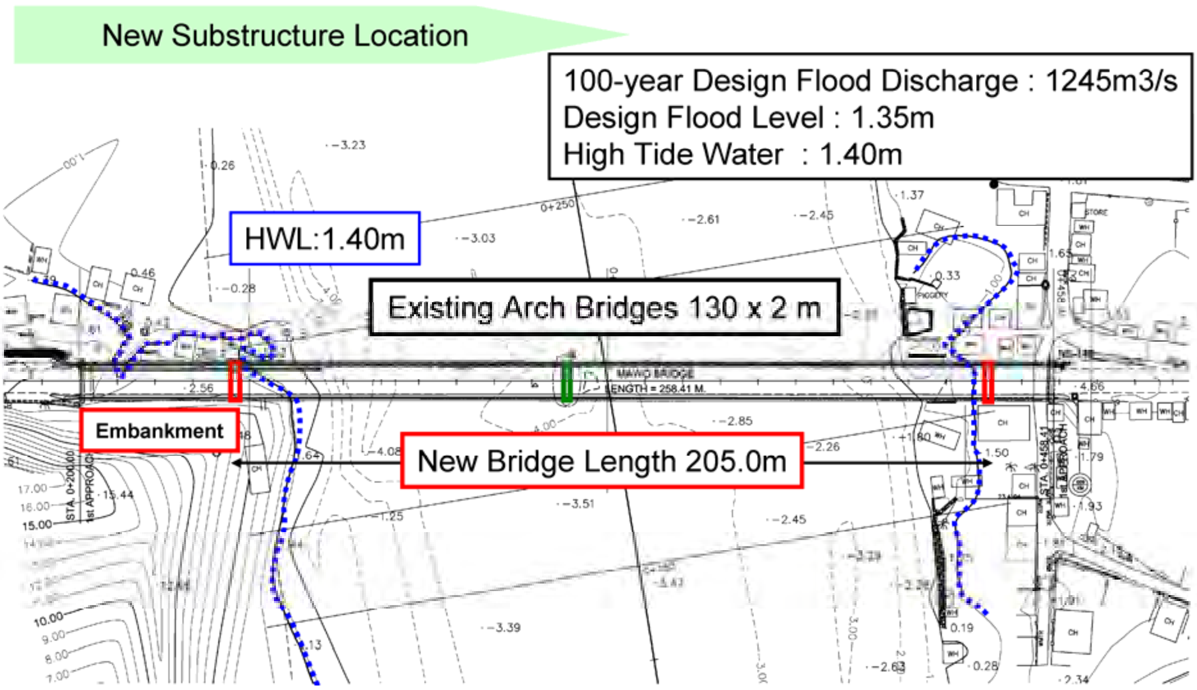


Figure 16.1.2-25 Location of Abutments

ii) Span Arrangement in Comparison Study

The location of piers, which is important factor for study of span arrangement, is determined in consideration of bridge structure and environmental conditions. In contrast to the condition of Palanit bridge, there are some settlements around the area of upstream side of Mawo bridge. And the water depth of the river is enough condition that middle size barge can pass under the bridge. Therefore, same as to the existing bridge, the new bridge planning may be implemented considering the possibility of water logistics for development of upstream side of the river. Thus, minimum size of barge passable under the river should be estimated to realize such the future situation.

Firstly, in order to study applicable span arrangement to be included in comparison study, classes of barges and required width from the classes should be estimated on the basis of existing condition. In this project, the estimation is implemented based on a specification of "Guide Specifications and Commentary for Vessel Collision Design of Highway Bridges, Second Edition 2009".

<Assumed barge>

The classes consist of various classes corresponding to their purposes such like open hopper and tank barge, etc. According to the river condition, the water depth from standard water level (EL=0.5m) to river bed is approximately 3.3m, in which normal small barge, full loaded draft 12.5ft/3.8m, may be passable in 53m width at left side bank and in 67m width at right side bank, shown as following figure. Therefore, existing two of 130m class Langer bridges have not their major purpose to secure horizontal clearance for navigating barge; the new bridge type can be planned in the scope that span length does not interrupt navigating barges.

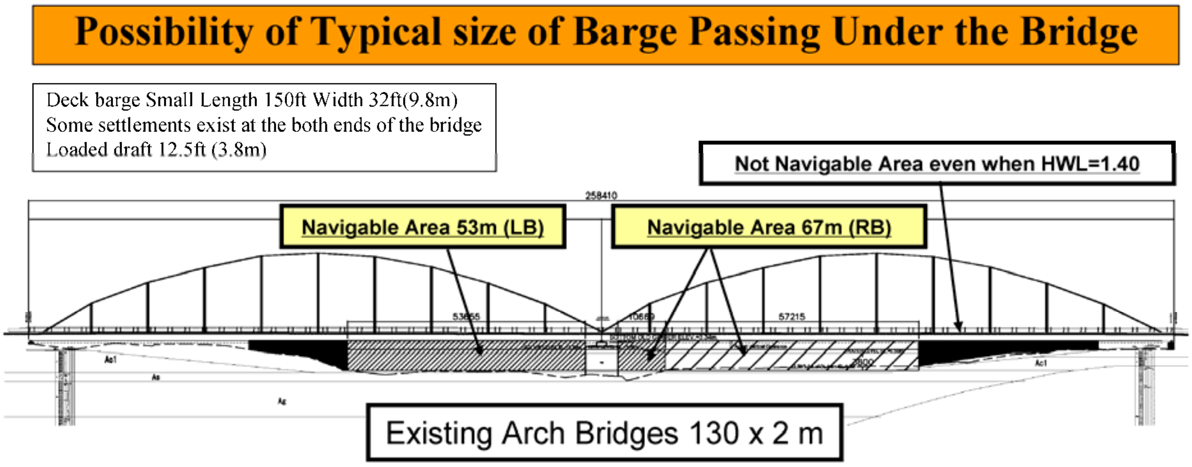


Figure 16.1.2-26 Study of Navigation Width

<The navigation width to be applied in new bridge planning>

As shown in the above figure, the width in which the assumed barge is passable are 53m at left side bank and 67m at right side bank. These navigation widths are respectively 0.86LOA and 1.1LOA for the LOA that is overall length of the assumed barge. According to the relationship between ship collision and span length specified in "Guide Specifications and Commentary for Vessel Collision Design of highway Bridges, Second Edition 2009, AASHTO", the values of 0.86 and 1.1 may be close to the limited value of span length, which ship collision is incident. Consequently, the number of span must not reduce such the limited value that can secure opining of 53m width.

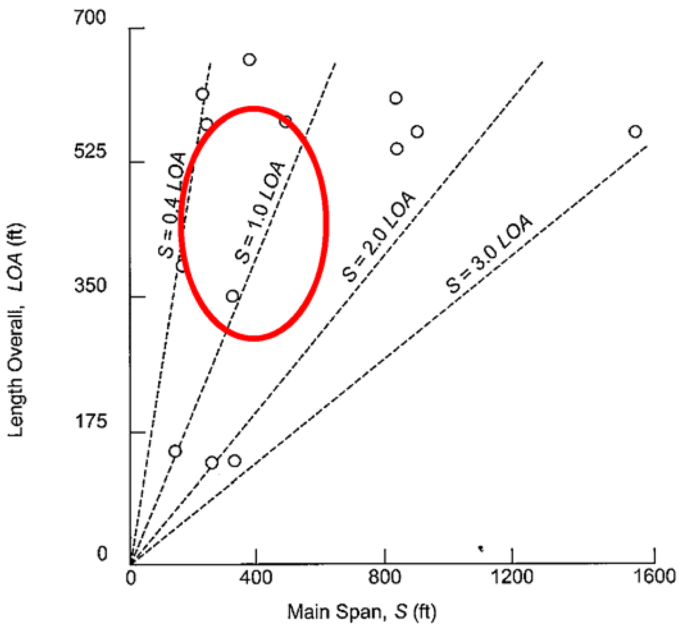
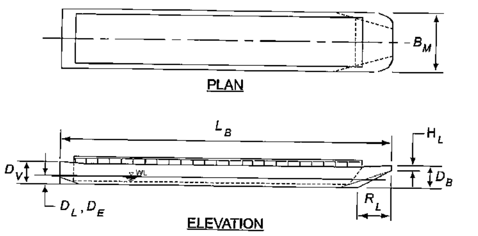


Figure 16.1.2-27 Relationship between ship collision and span length specified

"Guide Specifications and Commentary for Vessel Collision Design of highway Bridges, Second Edition 2009, AASHT"

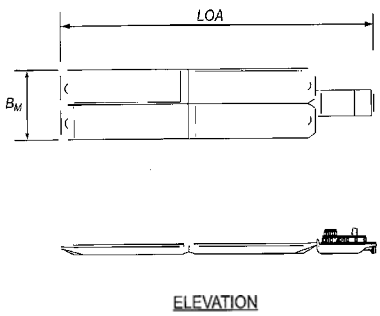


			Jumbo Hopper	Oversize Tank	Special Deck
B_M	=	width (ft)	35	53	72
L_B	=	length (ft)	195	290	250
H_L	=	head log height (ft)	2–3	2–3	3–5
D_V	=	depth of vessel (ft)	12	12	17
D_B	=	depth of bow (ft)	13	13	18
R_L	=	bow rake length (ft)	20	25	30
D_L	=	loaded draft (ft)	8.7	8.7	12.5
D_E	=	empty [light] draft (ft)	1.7	1.7	2.5
C_C	=	cargo capacity (tons)	1700	3700	5000
W_E	=	empty displacement (tons)	200	600	1300
W_L	=	loaded displacement (tons)	1900	4300	6300
WL	=	waterline			

Figure 3.5.1-1—Typical Barge Tow Characteristics

Table 3.5.1-1—Typical Characteristics of Barges on the Inland Waterways System

Barge Type	Size	Length (ft)	Width (ft)	Draft (ft)	Capacity (tons)
Open Hopper	Small	120	30	7	630
Open Hopper	Standard	175	26	9	1,060
Open Hopper	Jumbo	195	35	9	1,700
Open Hopper	Oversize	245	35	10	2,400
Covered Hopper	Jumbo	195	35	9	1,700
Deck Barge	Small	100/150	26/32	6	350/600
Deck Barge	Jumbo	195	35	9	1,700
Deck Barge	Oversize	200	50	9	2,050
Tank Barge	Small	135	40	9	1,300
Tank Barge	Jumbo	195	35	9	1,700
Tank Barge	Oversize	185/290	53	9	2,530/3,740



Barge Type	Average Size of Barge in Tow	Average Number of Barges Along the Length of Tow
Jumbo Hopper	35 × 195	3
Oversize Tank	53 × 290	2
Special Deck	72 × 250	1

Figure 3.5.1-2—Typical Barge Configurations

LOA=150+50=200ft
=200*0.3048
=61m

Table 3.5.1-2—Typical Characteristics of Towboats on the Inland Waterways System

Towboat Type	Horsepower	Typical Length (ft)	Typical Width (ft)	Typical Draft (ft)
Harbor Boat	<600	50	16	4 1/2
Line Haul	600–1200	78	23	7
Line Haul	1200–2500	120	30	9
Line Haul	2500–4300	146	35	9
Line Haul	4300–8400	160	45	9
Line Haul	>8,400	185	55	9

Figure 16.1.2-28 Assumed barges

"Guide Specifications and Commentary for Vessel Collision Design of highway Bridges, Second Edition 2009, AASHT"

<2-Span Bridge>
The navigation width is almost same to existing condition. The span arrangement is adequate structurally and hydraulically. However, the costs of the bridges may be more expensive than any other case.

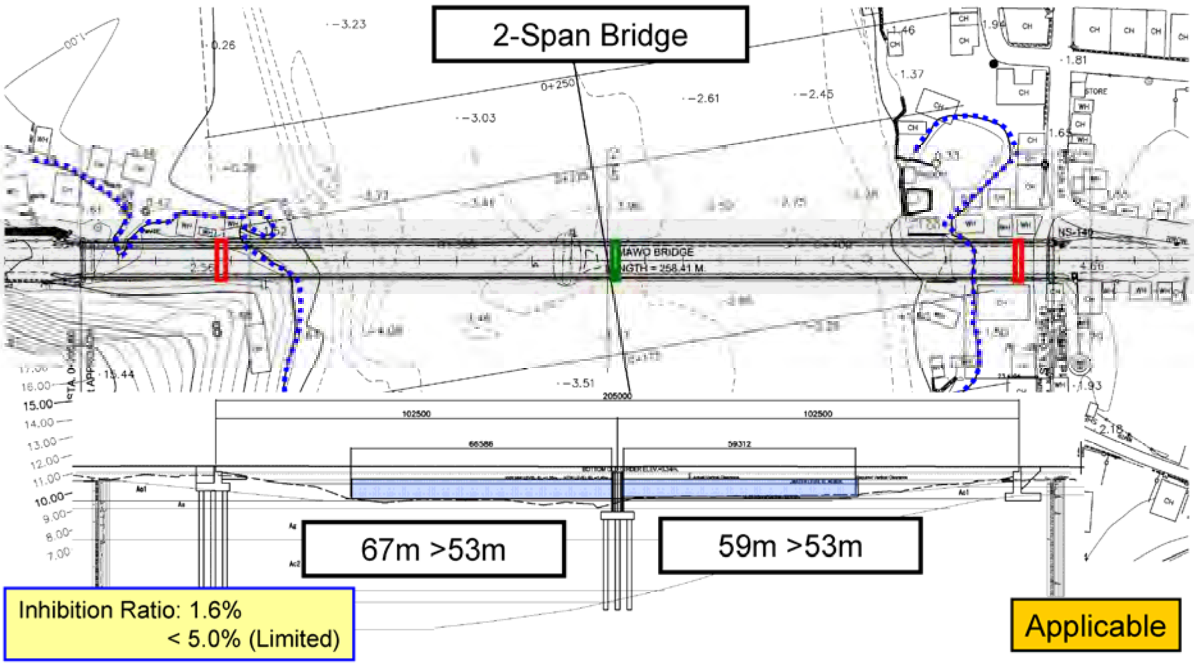


Figure 16.1.2-29 2 Span Bridge

<3-Span Bridge>
For 3-span bridge, the center span length should be keep 80m in consideration structural balance. The center span can secure wider navigation clearance than existing condition.

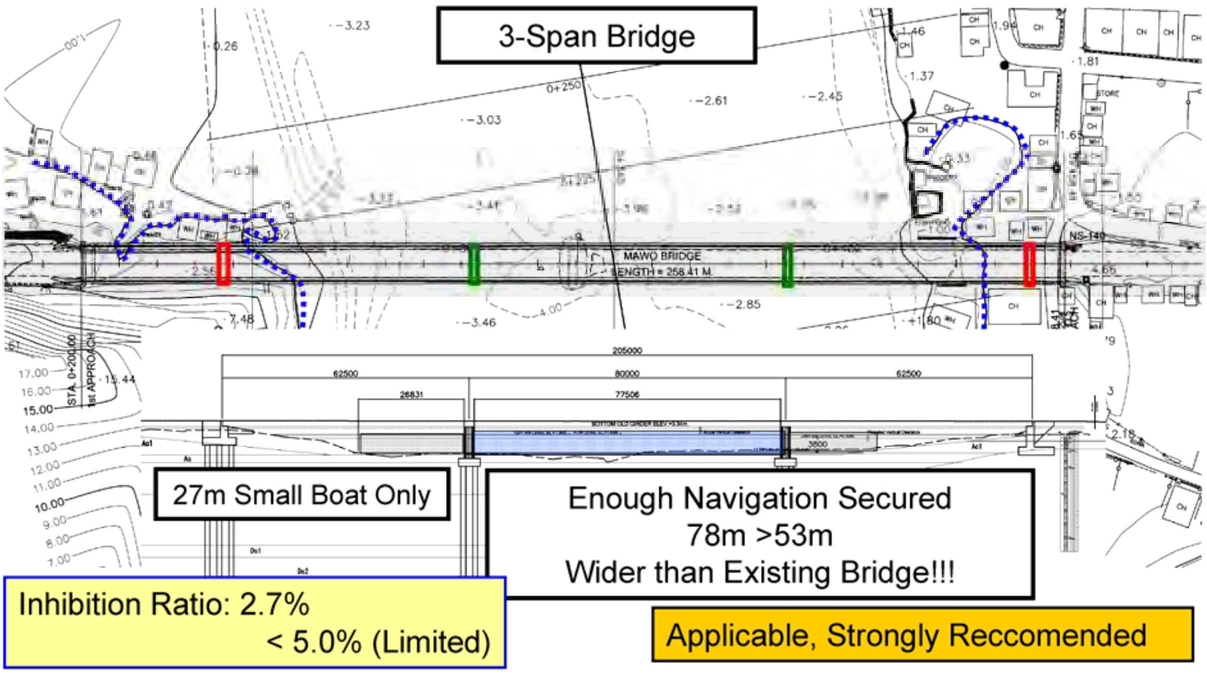


Figure 16.1.2-30 3 Span Bridge

<4-Span Bridge>
For 4-span bridge, the center span length should be keep 52m in consideration structural balance. Therefore, the navigation width is 49m that is narrower than that of existing bridge. Additionally the width is 0.8 LOA, dangerousness of ship collision would be significantly increased. This span arrangement is not recommendable but as reference, final cost comparison is examined between the finally recommended bridge type and the suitable type of 4-span bridge.

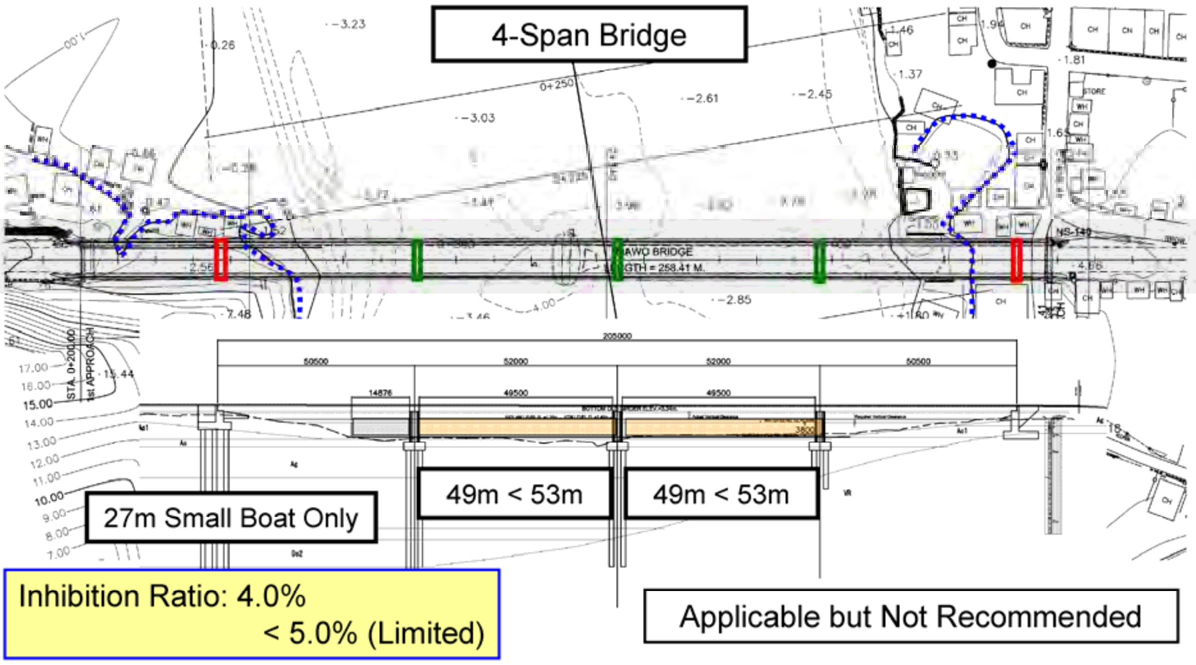


Figure 16.1.2-31 4 Span Bridge

STEP 4. Confirmation of BASIC CONCEPT

The basic concept resulted from above STEP 1 to STEP 3 is enumerated. The basic concept may be significantly important factor for bridge selection under the comparison study.

- New abutments locations are newly determined by Hydraulic analysis
- Such the new abutments are not affected by the Flood and High Tide
- Bridge Length of New Bridge: 205m

- Enough water depth
- Possibility of development plan for the settlements of the upstream area in the future
- Existing maritime transportation capacity under the bridge shall be secured for development of the upstream area
- 2-Span (Max 102.5m), 3-Span (Max 80m) is available span arrangement
- Minimal maintenance bridge will be recommended, reflecting on-site request.

STEP 5. COMPARISON STUDY of New Bridge Types

i) Extraction of Applicable Basic Types based on Actual Results
Based on the basic concept and several conditions, applicable bridge types are extracted from the table of Relationships between Actual Results of Basic Bridge Types and Span Length, shown as follows.

Table 16.1.2-26 Extraction of Applicable Basic Types based on Actual Results

STEEL	PC
Simple Supported	
Self-Anchored Suspension Bridge Nielsen Lohse Arch	—
2-Span	
Continuous Lohse Arch Continuous Langer Continuous Truss (Tubular) Continuous Steel Deck Box Girder	PC Cable Stayed Bridge
3-Span	
Continuous Truss (Tubular) Continuous Steel Deck Box Girder	PC Cable Stayed Bridge PC Extradosed Bridge (Rational Structure) PC Panel Stayed Bridge (Rational Structure)e Continuous PC Fin Back Girder (Rational Structure)

ii) Selection of Logically Suitable Types from above the Basic Types
Suitable bridge types are selected logically among above extracted bridges, to be utilized for final comparison study as follows:

Table 16.1.2-27 Extraction of Basic Types for Final Comparison Study (Steel)

STEEL	Inclusion of Final Comparison Study
Self-Anchored Suspension Bridge	- Not Included in Final Comparison Study - Clearly expensive - Over Specification
Nielsen Lohse Arch	- Not Included in Final Comparison Study - Clearly expensive - Over Specification
2-Span Continuous Steel Lohse Arch	**- Included in Final Comparison Study** - Low girder height - No Road Rising
2 of Steel Langer	- Not Included in Final Comparison Study - Not continuous bridge - Disadvantage seismically
2- Span Continuous Steel Truss (Tubular)	**- Included in Final Comparison Study** - Low girder height - No Road Rising
2- Span Continuous Steel Deck Box	**- Included in Final Comparison Study** - Girder height 3.6m - 1.5m of Road Rising
3-Span Continuous Steel Lohse Arch	**- Included in Final Comparison Study**
3- Span Continuous Steel Truss (Tubular)	**- Included in Final Comparison Study**
3- Span Continuous Steel Deck Box	**- Included in Final Comparison Study** - Girder height 2.9m - 0.8m of Road Rising

Table 16.1.2-28 Extraction of Basic Types for Final Comparison Study (PC)

PC	Inclusion of Final Comparison Study
2-Span Continuous PC Cable-Stayed Bridge	- Not Included in Final Comparison Study - Clearly expensive - Over Specification
3-Span Continuous PC Cable-Stayed Bridge	- Not Included in Final Comparison Study - Clearly expensive - Over Specification
3-Span Continuous PC Box Girder	- Cost is comparatively high because this new bridge will be over span length of 77m that is the actual results of general PC erection method - Girder height 5.3m - 3.2m of road rising necessary - Not Acceptable the amount of road rising **- Propose Rational Structure of this Type** **- Included the Rational Structure**

For PC bridges, basic PC bridges can not adequately meet the fundamental requirements of Mawo bridge, therefore, in addition to the above basic bridge types, following rational structures that are lately constructed in Japan are proposed and included in the final comparison study.

Table 16.1.2-29 Bridge Types for Final Comparison Study, including Rational Structures (PC)

PC	Inclusion of Final Comparison Study
2-Span Continuous PC Extradosed Bridge	- Not Included in Final Comparison Study - Intermediate structure between Cable-stayed bridge and girder bridge - Girder height 3.4m at Towers - 1.4m of road rising necessary - Clearly expensive
2-Span Continuous PC Panel-Stayed Bridge	- Not Included in Final Comparison Study - Intermediate structure between Cable-stayed bridge and girder bridge - The cables of extradosed bridge were covered by concrete. - Anticorrosion property of the cables covered by concrete is positive but replacements are not easy to repairing work - Girder height 3.4m at Towers - 1.4m of road rising necessary - Clearly expensive
3-Span Continuous PC Extradosed Bridge	- Not Included in Final Comparison Study - Applicable adequately but maintenance ability beside the coast is negative
3-Span Continuous PC Panel Stayed Bridge	**- Included in Final Comparison Study** - Girder height 2.0m - No road rising
3-Span Continuous PC Box Girder	**Explained Above**
3-Span Continuous PC Fin Back Box Girder	**- Included in Final Comparison Study** - PC Half-Through bridge - Intermediate structure between PC extradosed bridge and PC girder bridge - PC cables are installed in the wing walls decent ring prestressing forces - Rational structure - Girder height 2.5m - 50cm road rising but can be absorbed in vertical alignment of the bridge itself

Based on the above evaluation, multiple comparison study is conducted considering cost, structure, environmental impact, constructability and maintenance ability.

Table 16.1.2-30 Candidates of Final Comparison Study

SEEL	PC
Simple Supported	
—	—
2-Span	
2-Span Continuous Steel Lohse Arch **2- Span Continuous Steel Truss (Tubular)** **2- Span Continuous Steel Deck Box**	—
3-Span	
3-Span Continuous Steel Lohse Arch **3- Span Continuous Steel Truss (Tubular)** **3- Span Continuous Steel Deck Box**	**3-Span Continuous PC Panel Stayed Bridge** **3-Span Continuous PC Box Girder** **3-Span Continuous PC Fin Back Box Girder**

Based on the evaluation, shown in the comparison table, the recommendable bridge type for outline design is **3-Span Continuous PC Fin Back Box Girder Bridge**.

iii) Selection of Logically Suitable Types of Bridge Foundations

In the study on selection of bridge foundation, study of site conditions for selecting an appropriate foundation type and comparison of structure (pile diameter) are the key discussions.

The site conditions are shown in below table. In the abutment A2 and Piers, there were located deep bearing layer with liquefiable soil.

Table 16.1.2-31 Site Candidates of Comparison Study

Study Type	Abutment Foundation	Pier Foundation
Foundation location	On the ground	In the River
proximity structure	Small houses	-
Navigation condition	-	-
Max. Water depth (m)	-	Around 6.5 m
Depth of bearing layer (m)	Around GL- 6.0m (A1) Around GL-38.0m(A2)	Around GL-16.0m (P1) Around GL-34.0m(P2)
Liquefaction	liquefiable site (for A2)	liquefiable site
Lateral spreading	-	-

Based on the above site conditions, applicable type of Abutment A1 foundation is recommended spread footing type, Abutment A2 & Piers are recommended cast-in-site pile foundation.

According to the above evaluation, the pile diameter comparison study is conducted considering cost, construction period, constructability, and environmental impact.

The result of comparative study of pier foundation are shown in the next tables, the recommendable pile diameter of pile foundation for outline design is 1.5m Bored Pile, because of its advantages in low construction cost, minimum construction period and efficient constructability.

Table 16.1.2-32 Comparison on Pile Diameter of Mawo Bridge at P1 Pier

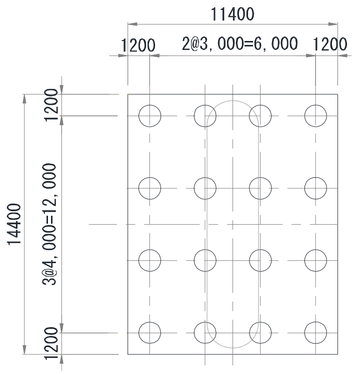
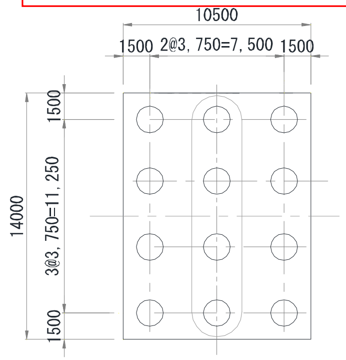
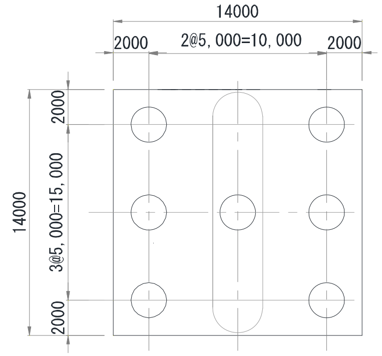
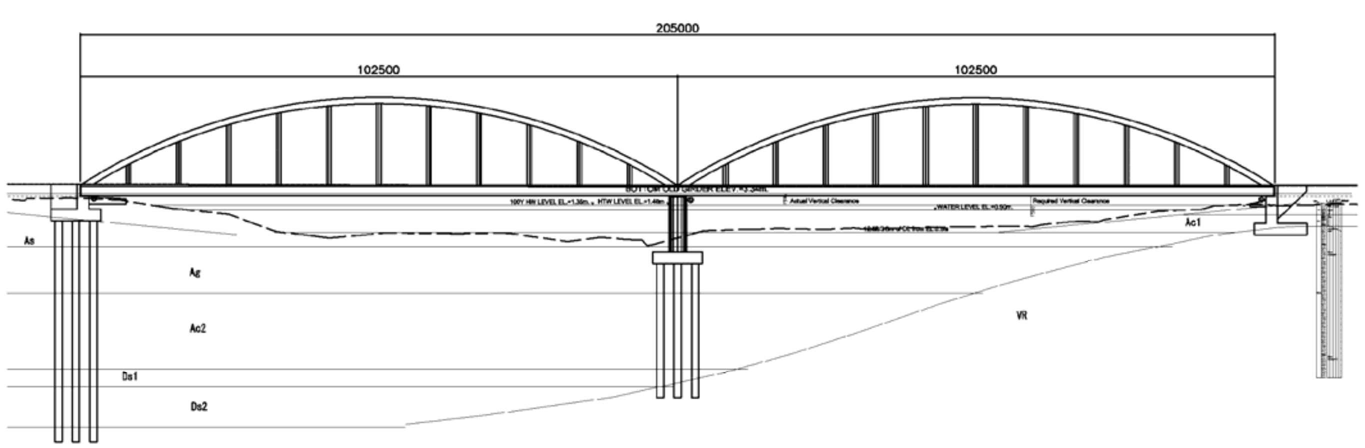
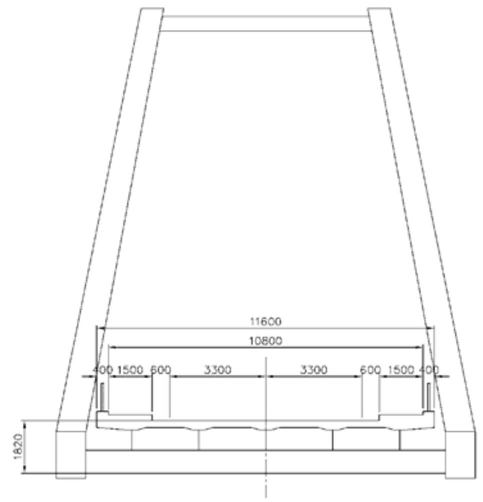
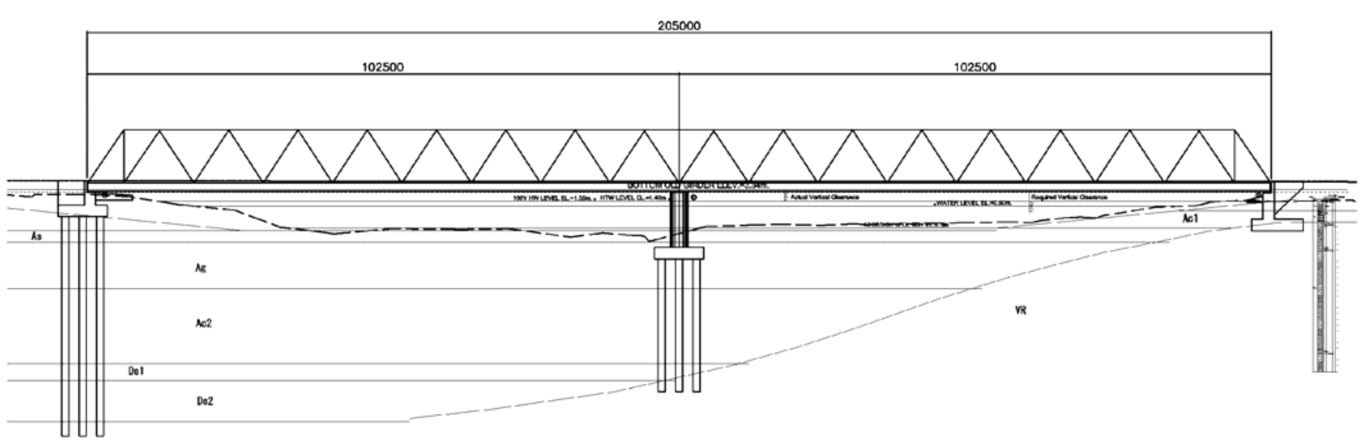
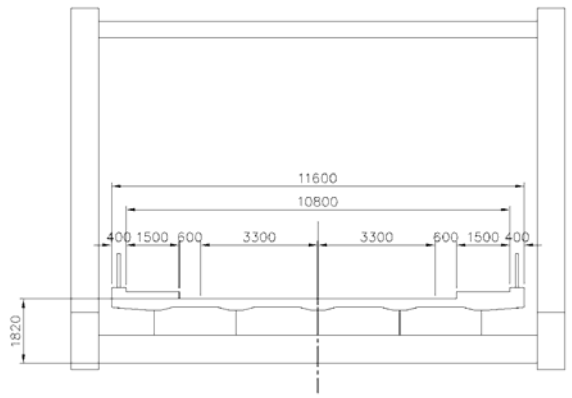
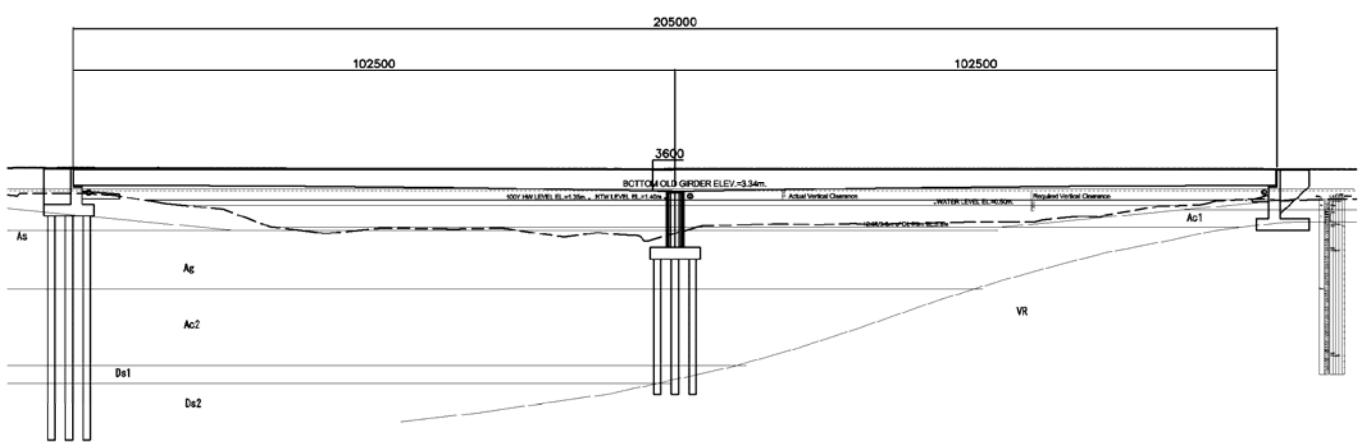
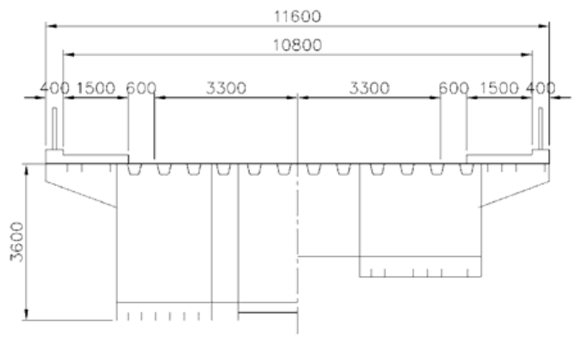
Evaluation Items	Alternative-1 Cas-in-place Concrete Pile D= 1.2m		Alternative-2 Cas-in-place Concrete Pile D= 1.5m		Alternative-3 Cas-in-place Concrete Pile D= 2.0m	
Side View Pile arrangement	Diameter of pile : 1200 mm Total number of pile : 16 Pile length : 14.0 m Total length of pile : 224.0 m 		Diameter of pile : 1500 mm Total number of pile : 12 Pile length : 14.0 m Total length of pile : 168.0 m OUTLINE DESIGN 		Diameter of pile : 2000 mm Total number of pile : 7 Pile length : 14.5 m Total length of pile : 101.5 m 	
Structural Aspect and Stability	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.733 - Large number of Steel Sheet Piles and steel .	B	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.707	A	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.734.	B
Construction Cost (for Foundation)	Item / Quantity / Unit Cost (Php) / Total (1,000Php) Pile Cap Concrete / 328m3 / 7,559.8 / 2,482 Reinforcement steel / 66ton / 52,600.0 / 3,454 Pile / 224m / 45,898.5 / 10,281 Cofferdam / 894m2 / 21,181.9 / 18,937 Total 35,154 Ratio **1.092**	B	Item / Quantity / Unit Cost (Php) / Total (1,000Php) Pile Cap Concrete / 294m3 / 7,559.8 / 2,223 Reinforcement steel / 59ton / 52,600.0 / 3,093 Pile / 168m / 52,238.6 / 8,776 Cofferdam / 855m2 / 21,181.9 / 18,111 Total 32,202 Ratio **1.000**	A	Item / Quantity / Unit Cost (Php) / Total (1,000Php) Pile Cap Concrete / 392m3 / 7,559.8 / 2,963 Reinforcement steel / 78ton / 52,600.0 / 4,124 Pile / 102m / 88,169.5 / 8,949 Cofferdam / 960m2 / 21,181.9 / 20,335 Total 36,371 Ratio **1.129**	B
Construction Plan and Period	- Working in temporary cofferdam at the river. Cofferdam Work 33 days Pile work (1.5pile/day) 24 days Pile Cap 33 days Total **90 days** Ratio **1.138**	C	- Working in temporary cofferdam at the river. Cofferdam Work 32 days Pile work (1.5pile/day) 18 days Pile Cap 29 days Total **79 days** Ratio **1.000**	A	- Working in temporary cofferdam at the river. Cofferdam Work 36 days Pile work (2pile/day) 14 days Pile Cap 39 days Total **89 days** Ratio **1.123**	C
Constructability	- Constructability is inferior due to large number of Cast-in-place concrete Pile work.	C	- Constructability is superior with small number of foundation work.	A	- Need to large number of Steel Sheet Pile & Pile Cap cconcrete works.	B
Environmental Aspect	- Inferior in Environmental aspect due to large number of excavated soil.	C	- Better in Environmental aspect with small number of excavated soil & bentonite water.	B	- Superior in Environmental aspect with small number of excavated soil & bentonite water.	A
Evaluation	- Construction cost is highest with long construction period. Not Recommended	**C**	- Construction cost is lowest with minimum Construction period with efficient workability. Most Recommended	**A**	- Construction cost is highest with long construction period. Not Recommended	**B**

Table 16.1.2-33 Comparison of New Bridge Types for Mawo bridge (STEEL 1/2)

STEEL BRIDGES 1/2

COMPARISON of NEW BRIDGE TYPES for MAWO BRIDGE

	Side View	Sectional View	Evaluation: Good-A-Fair-B-Poor-C
Alternative 1S	2-Span Continuous Steel Lohse 		see evaluation below
Alternative 2S	2-Span Continuous Steel Truss 		see evaluation below
Alternative 3S	2-Span Continuous Steel Deck Box Girder 		see evaluation below

Evaluation: Good-A-Fair-B-Poor-C

			Alternative 1S		Alternative 2S		Alternative 3S	
COST	Superstructure		0.83	C	0.78	C	0.72	B
	Substructure		0.10		0.10		0.10	
	Road		0.00		0.00		0.02	
	Others		0.22		0.22		0.22	
		Total	1.14		1.10		1.05	
STRUCTURAL FEATURE				B		B		A
CONSTRUCTABILITY				B		B		A
ENVIRONMENTAL IMPACT				A		A		C
MAINTENANCE				C		C		B

Table 16.1.2-34 Comparison of New Bridge Types for Mawo bridge (STEEL 2/2)

STEEL BRIDGES 2/2

COMPARISON of NEW BRIDGE TYPES for MAWO BRIDGE

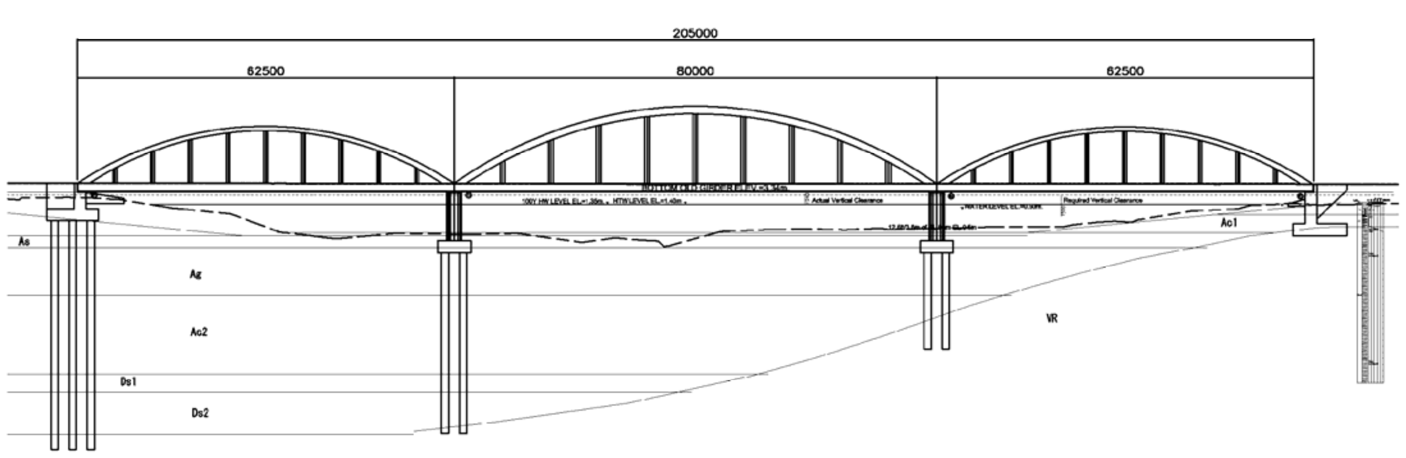
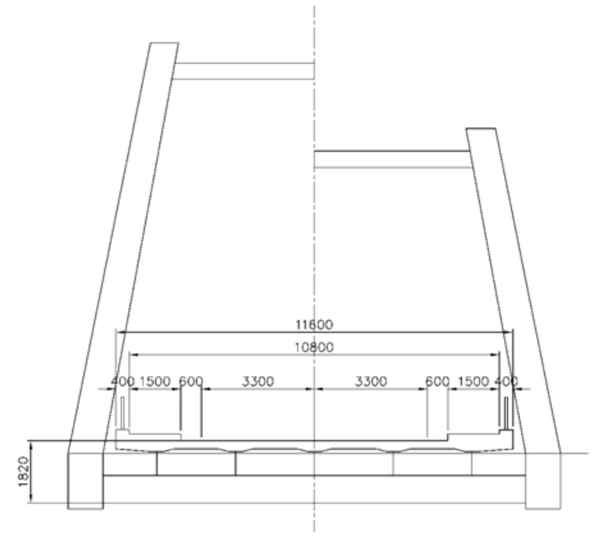
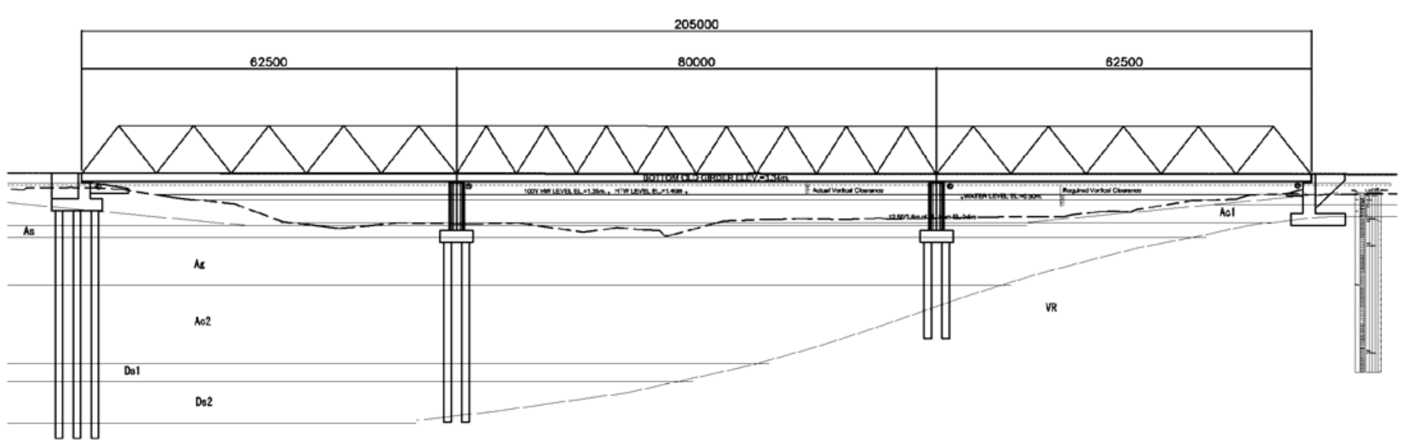
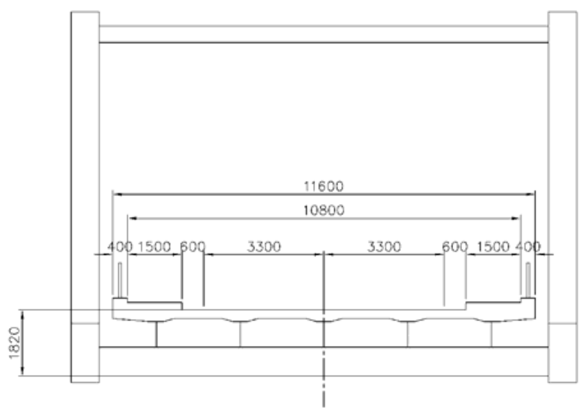
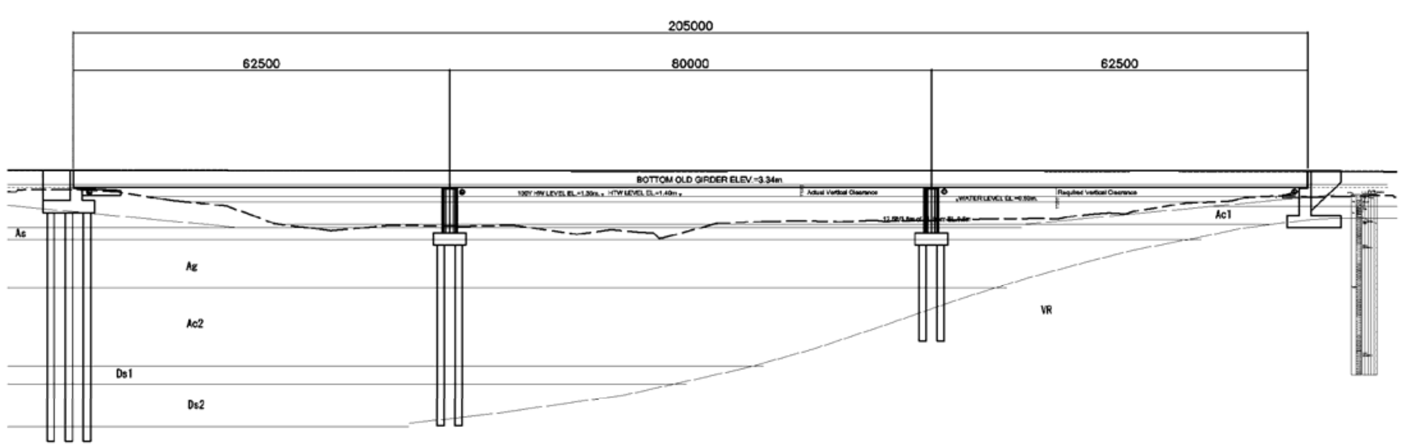
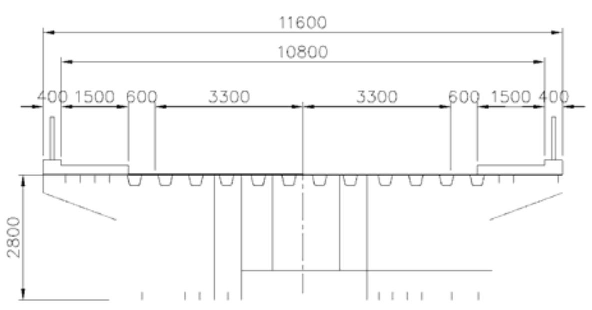
	Side View	Sectional View	Evaluation: Good-A-Fair-B-Poor-C			
Alternative 4S	3-Span Continuous Steel Lohse 		COST	Superstructure	0.81	C
				Substructure	0.12	
				Road	0.00	
				Others	0.22	
				Total	1.15	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			B
			ENVIRONMENTAL IMPACT			A
			MAINTENANCE			C
Alternative 5S	3-Span Continuous Steel Truss 		COST	Superstructure	0.69	B
				Substructure	0.12	
				Road	0.00	
				Others	0.22	
				Total	1.03	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			A
			ENVIRONMENTAL IMPACT			B
			MAINTENANCE			C
Alternative 6S	3-Span Continuous Steel Deck Box Girder 		COST	Superstructure	0.70	B
				Substructure	0.12	
				Road	0.00	
				Others	0.22	
				Total	1.04	
			STRUCTURAL FEATURE			A
			CONSTRUCTABILITY			A
			ENVIRONMENTAL IMPACT			B
			MAINTENANCE			B

Table 16.1.2-35 Comparison of New Bridge Types for Mawo bridge (PC)

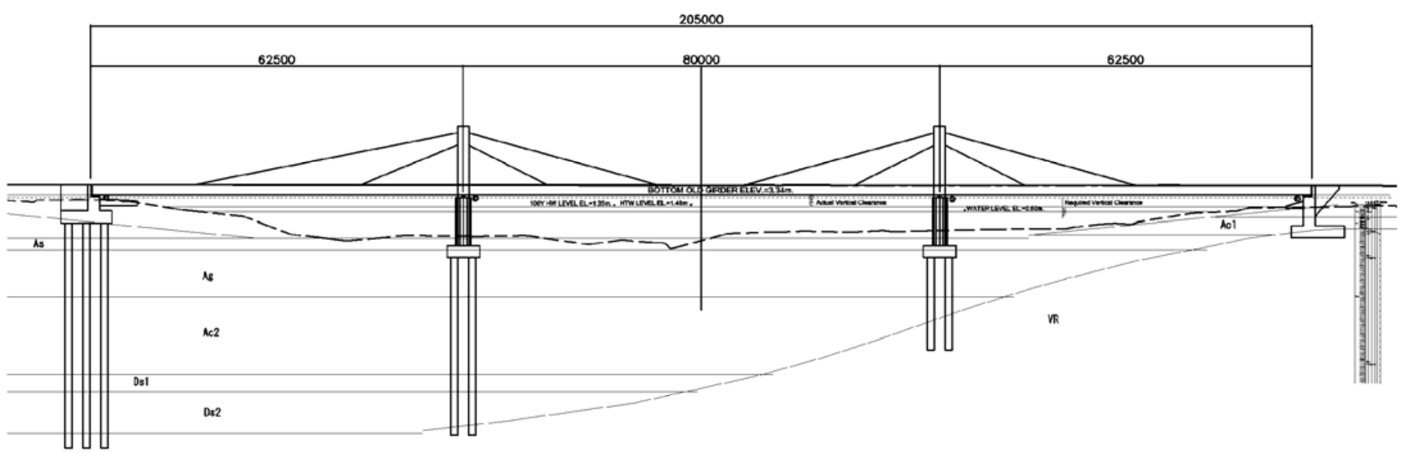
PC BRIDGE

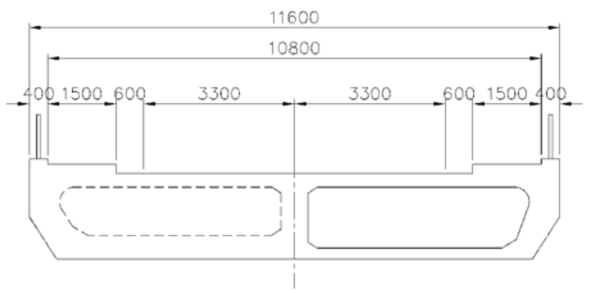
COMPARISON of NEW BRIDGE TYPES for MAWO BRIDGE

Side View | Sectional View | Evaluation: Good-A-Fair-B-Poor-C

Alternative 1P

3-Span Continuous Panel-Stayed Bridge

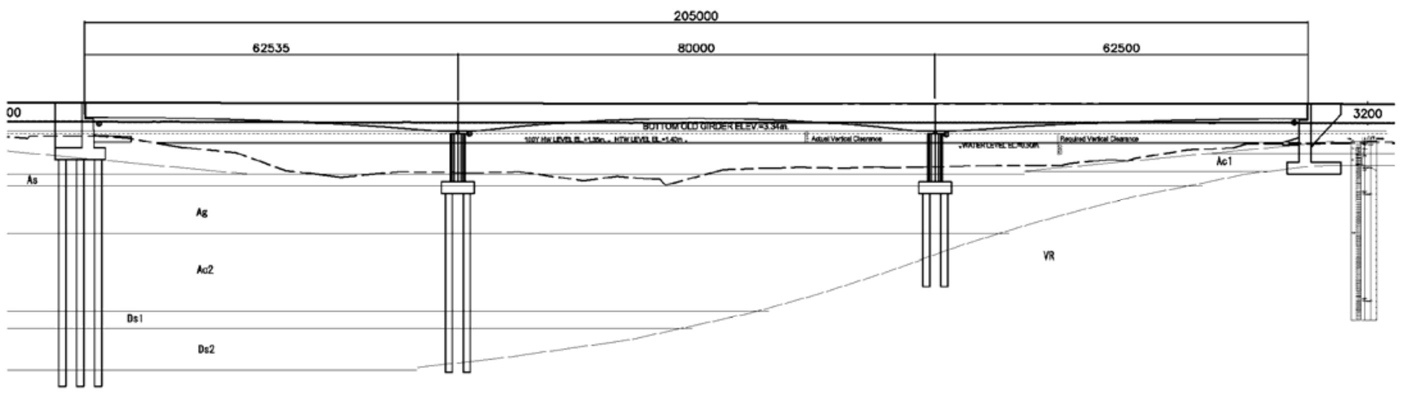


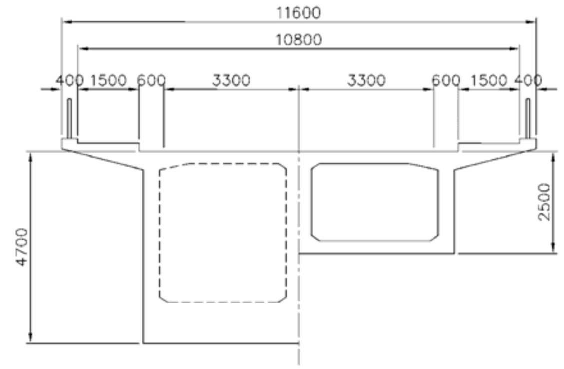


COST	Superstructure	0.77	C
	Substructure	0.12	
	Road	0.00	
	Others	0.22	
	Total	1.11	
STRUCTURAL FEATURE			B
CONSTRUCTABILITY			C
ENVIRONMENTAL IMPACT			A
MAINTENANCE			A

Alternative 2P

3-Span Continuous PC Box

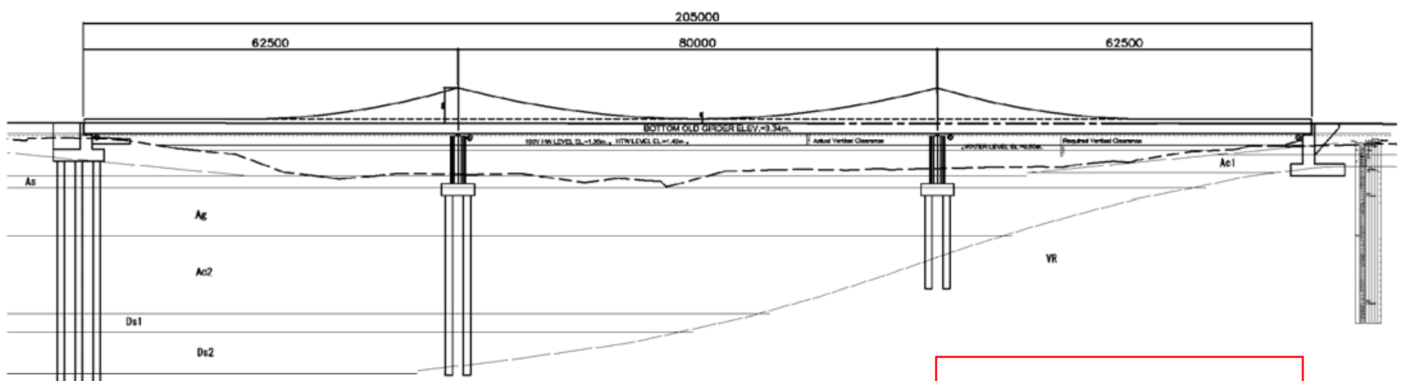


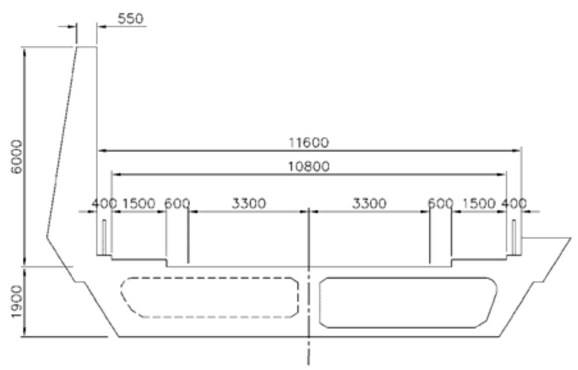


COST	Superstructure	0.64	B
	Substructure	0.13	
	Road	0.03	
	Others	0.22	
	Total	1.02	
STRUCTURAL FEATURE			C
CONSTRUCTABILITY			B
ENVIRONMENTAL IMPACT			C
MAINTENANCE			A

Alternative 3P

3-Span Continuous Fin-Back PC Box Girder





COST	Superstructure	0.66	A
	Substructure	0.12	
	Road	0.00	
	Others	0.22	
	Total	1.00	
STRUCTURAL FEATURE			B
CONSTRUCTABILITY			B
ENVIRONMENTAL IMPACT			A
MAINTENANCE			A

OUTLINE DESIGN

(5) Wawa Bridge

STEP 1. Confirmation of ROAD CONDITION

i) Bridge Width

For the cross section and lane arrangement of new bridge, the examined results of road planning including approach roads shall be applied, shown as follows.

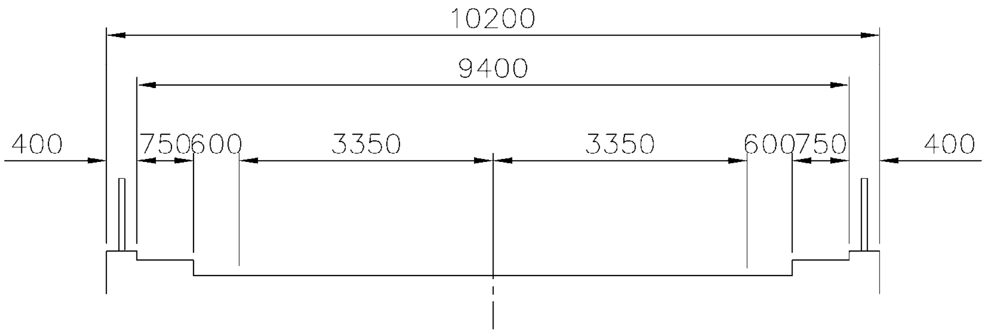


Figure 16.1.2-32 Cross Section/ Lane Arrangement of Wawa Bridge

ii) Horizontal Alignment

For Wawa bridge, horizontal alignment of new bridge structure may be shiftable comparing to other bridges because no houses and buildings besides approach roads exist and because there are no other sub approach roads entering to the main road. Therefore, the horizontal alignment of new bridge shall be shifted to 20m down stream side from the following advantage points, and the existing bridge can be utilized as detour road during construction stage.

- In case of upstream side shifting, significant amount of rock cutting may be caused
- In case of downstream side shifting, existing small road descending to the site is already exist; hence, mobilization of heavy equipment is quite facility
- The specific location shall be determined in the area of down stream side based on:
    - Smoothly linkable to main roads
    - No impact to settlements on the right side bank
    - Boundary lines of ROW shall be strictly secured
- The amount of shifting is to be 15m

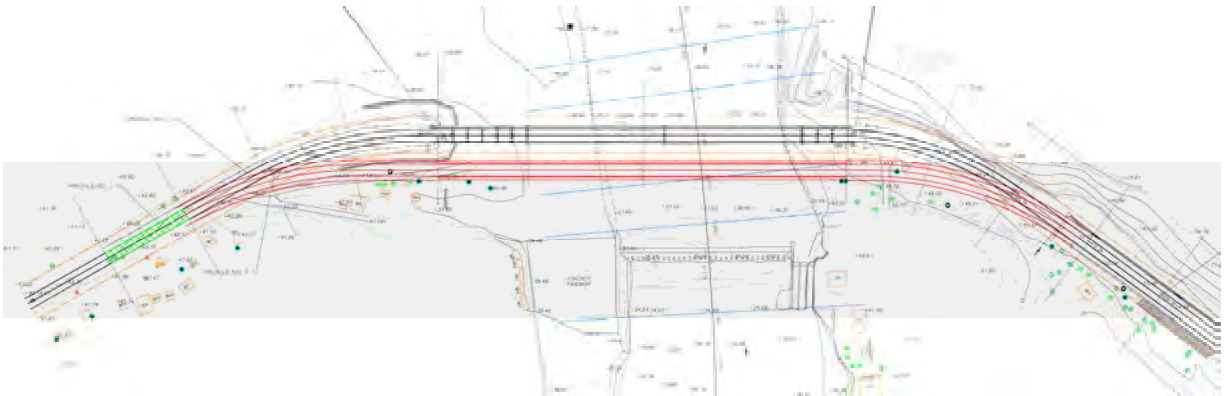


Figure 16.1.2-33 Horizontal Alighment

iii) Rising of Vertical Alignment

For Wawa bridge, rising of vertical alignment of bridge and approach roads may be acceptable partly comparing to other bridge site because no houses and buildings besides approach roads exist and because there are no other sub approach roads entering to the main road. However, the influences for crossing conditions between existing approach road and newly installed approach road to be installed 20m or downstream side should be confirmed.

Existing superstructure is 2 of steel truss bridge, the free board of which is approximately 3.8m against observed high water level. That is too enough allowance. Therefore, not only same type of existing structure but also applicability of deck type steel composite bridge may be available to be examined based on multiple comparison study. Beside, this site is located in mountainous area, application of rational truss structure using weathering steel may be acceptable. Thereby, in order to include such the bridge type, the girder height of which will be higher than existing bridge, into comparison study, the crossing condition between new and old approach bridges is examined in case of rising of vertical alignment.

< 2.0m rising of vertical alignment>

Impact:

- Need 50m of longitudinal execution right side bank
- However, inadequate influences against existing houses and buildings of settlements will not be caused.
- Naturally additional cost needed
- As above, for Wawa bridge, 2.0m rising of vertical alignment can be acceptable, and the additional costs due to the rising shall be partly included to the relevant structure

STEP 2. Confirmation of HYDRAULIC CONDITION

For hydraulic condition in the new bridge selection, the determined results in this project shall be applied shown as follows. For the design flood level, the water level of 41.65m (2159m3/s) which is a observed water level may be suitable for the design. The free board is determined as 1.5m.

However, around this area, the water flows is comparatively complicated condition hydraulically. And the results of hydraulic analysis will affect critically the results of bridge planning. Therefore, in detail design stage, detail hydraulic analysis based upon further investigation shall be implemented by river/ hydraulic engineers. Based on the detail examination, the free board to be secured, level of high water level, abutment location and reevaluation of comparison study may be preferable to be re-implemented in the detail design stage.

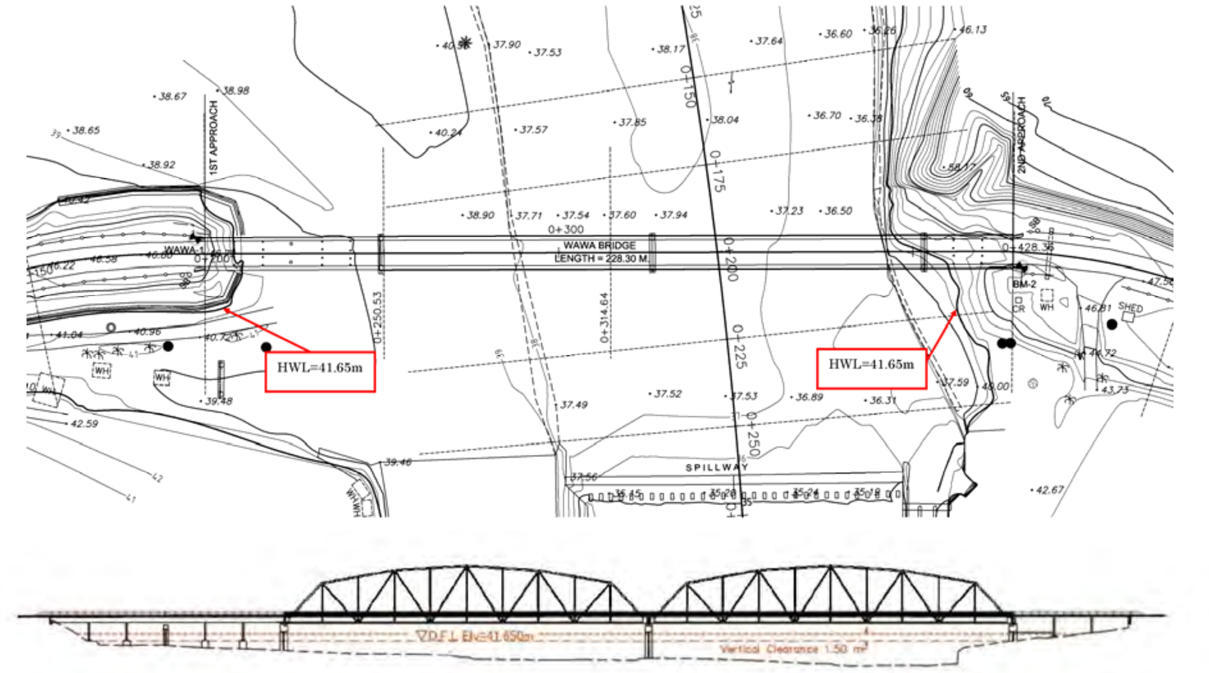


Figure 16.1.2-34 DHW and Free Board of Wawa Bridge

STEP 3. Examination of SUBSTRUCTURE LOCATION

i) Location of Abutments

The locations of abutments are determined based on high water level, determined by hydraulic analysis and site interview. The abutment shall not be affected by the boundary lines of high water level. The new bridge length considering such the condition is 230m.

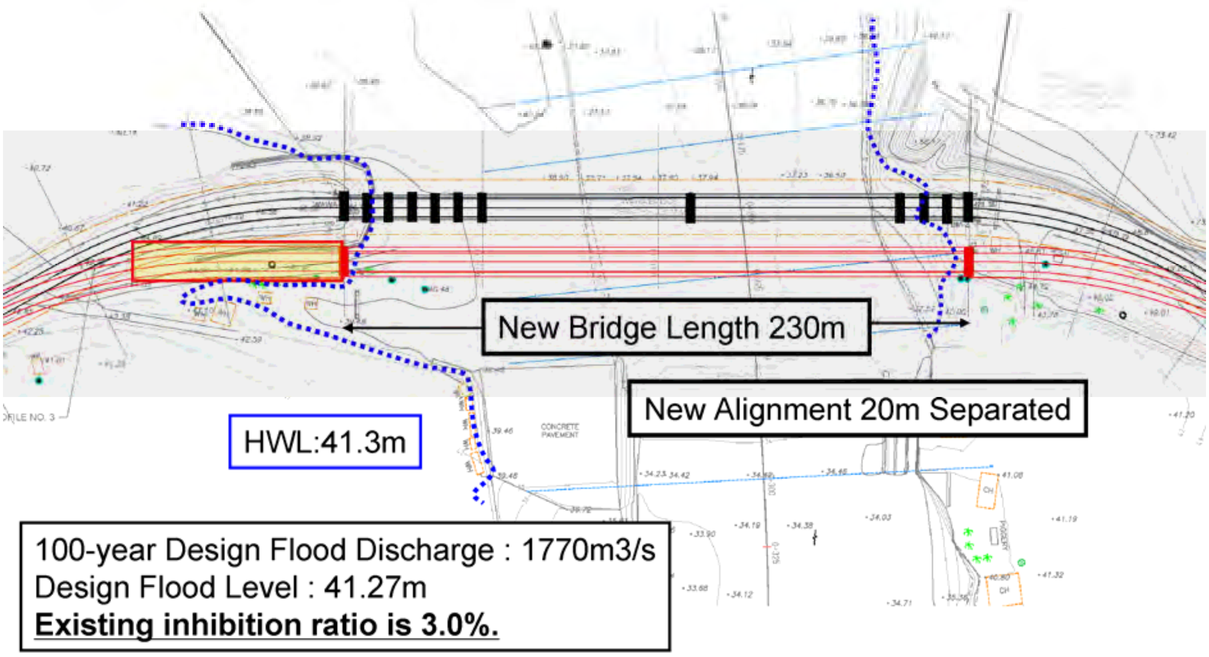


Figure 16.1.2-35 Determination of Abutment Location of Wawa Bridge

ii) Span Arrangement in Comparison Study

Span arrangement including determination of pier location shall be executed based on above mentioned bridge length and river condition. Therefore, as basic concept to determine suitable span arrangements to be included comparison study, following attentions should be considered.

- This site is filled with nature beauty such as mountain and clean rivers. The possibility of future large-scale development may be low but certain level of aesthetic elements may be preferable to be included.
- Adequate spam length shall be determined considering influences of debris or flood wood from upstream.
- Existing bridge is desirable to be removed after new bridge completion from the aspect of river hydraulics.
- Applicability of Steel bridges consisting of weathering steel members may be acceptable.
- Past flood level against the settlement at the right side bank shall be carefully verified.
- Condition of Inhibition ratio due to piers, which becomes the major cause of flood water rising, shall be carefully verified.
- Therefore, existing inhibition ratio, new inhibition ratio and outline estimation water rising shall be conducted for each candidate of span arrangement.

The following figure shows the boundary lines of high water level and its influence area. As shown in the figure, the line of the high water level is just near the area of the settlement at the right side bank even under 3.0% of existing inhibition ratio by existing piers. Therefore, careful verification regarding inhibition ratio and water rising by outline hydraulic calculation is required for examination of span arrangement.

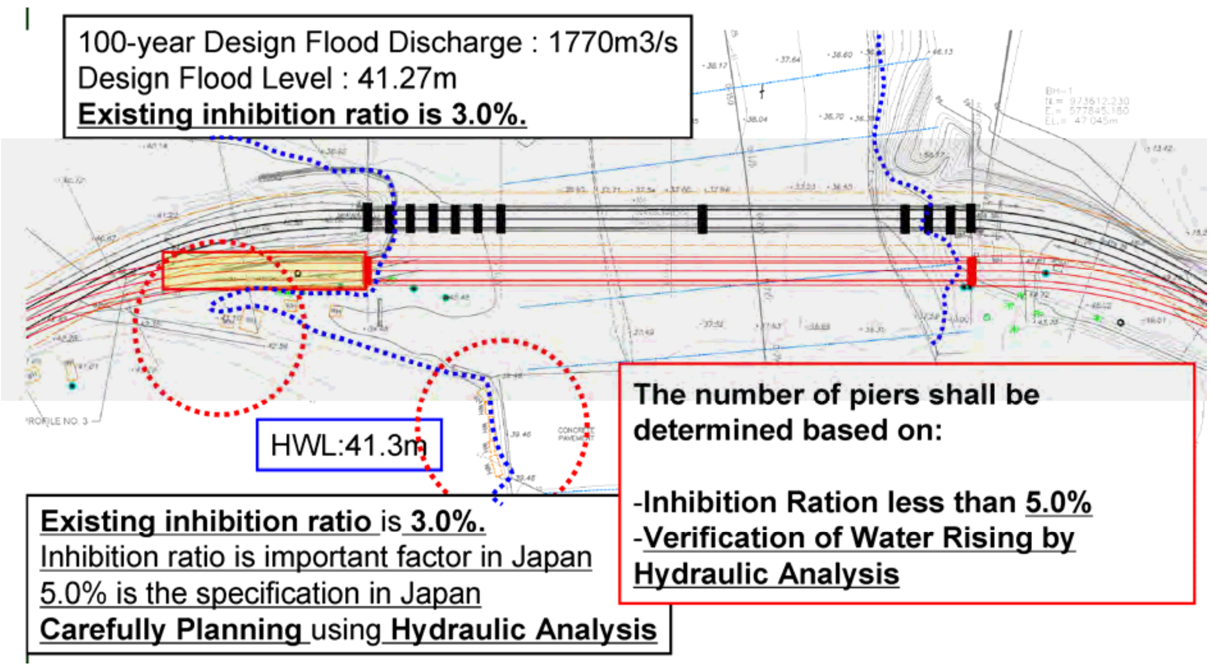


Figure 16.1.2-36 Boundary Lines of HWL and Influence Area

<2-Span Bridge>

The pier location of 2-span bridge may be adequate point, the separation from existing pier is 13.1m (<15m). Also, the new inhibition ratio is 1.5% that is less than 3.0% of existing inhibition ratio. Therefore, the influences of impact to river condition are not confirmed; this span arrangement is applicable to be included in comparison study

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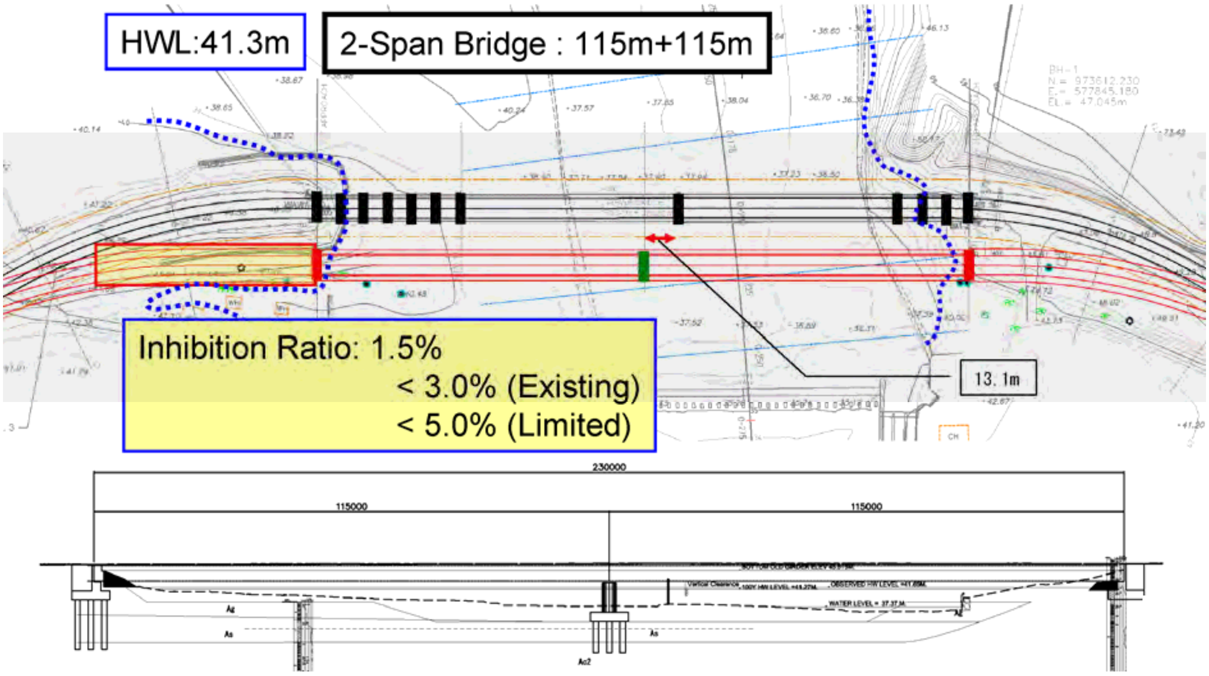


Figure 16.1.2-37 2 Span Bridge

<3-Span Bridge>

The pier location of 3-span bridge may also be adequate points, the separation from existing pier is 26.0m (>15m). Also, the new inhibition ratio is 2.6% that is less than 3.0% of existing inhibition ratio. Therefore, the influences of impact to river condition are not confirmed; this span arrangement is applicable to be included in comparison study

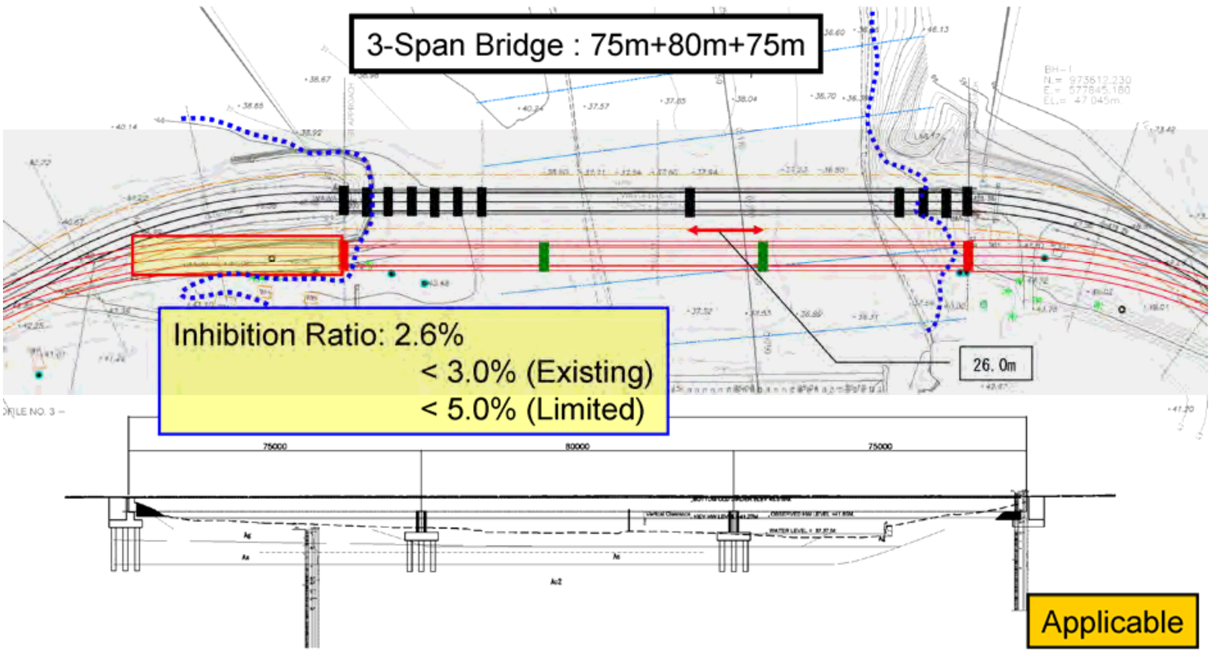


Figure 16.1.2-38 3 Span Bridge

<4-Span Bridge>

4-span bridge may not be recommendable structure from the reasons of new inhibition ratio and separation between new and existing piers. The new inhibition ratio is 3.8% that has become over 3.0% of existing inhibition ratio. However, because that may not be critical impact to river condition including water rising, this 4-span bridge is included into comparison study.

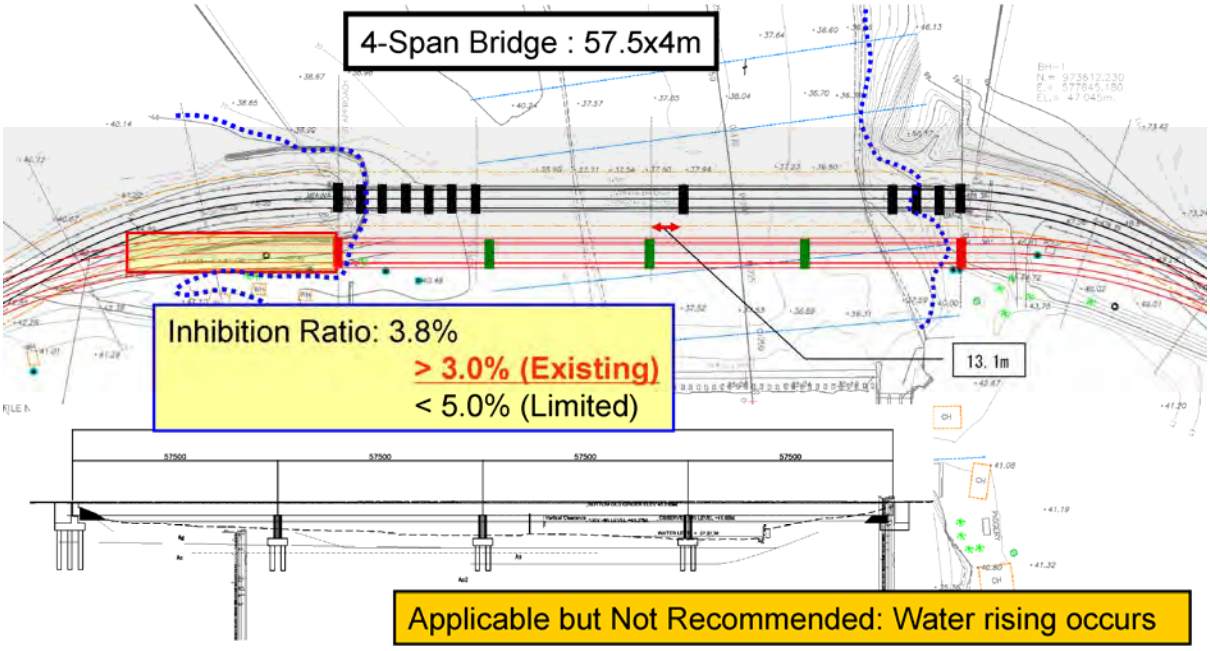


Figure 16.1.2-39 4 Span Bridge

<5-Span Bridge>

The new inhibition ratio due to 4 piers in 5-span bridge is over 5.2% that is significantly larger than 3.0% of existing inhibition ratio. And 20cm of water rising resulted by outline hydraulic calculation is confirmed, which would affect the area of settlement at right side bank critically. Therefore, the span arrangements of 5-span or more shall not absolutely be included into the bridge comparison study.

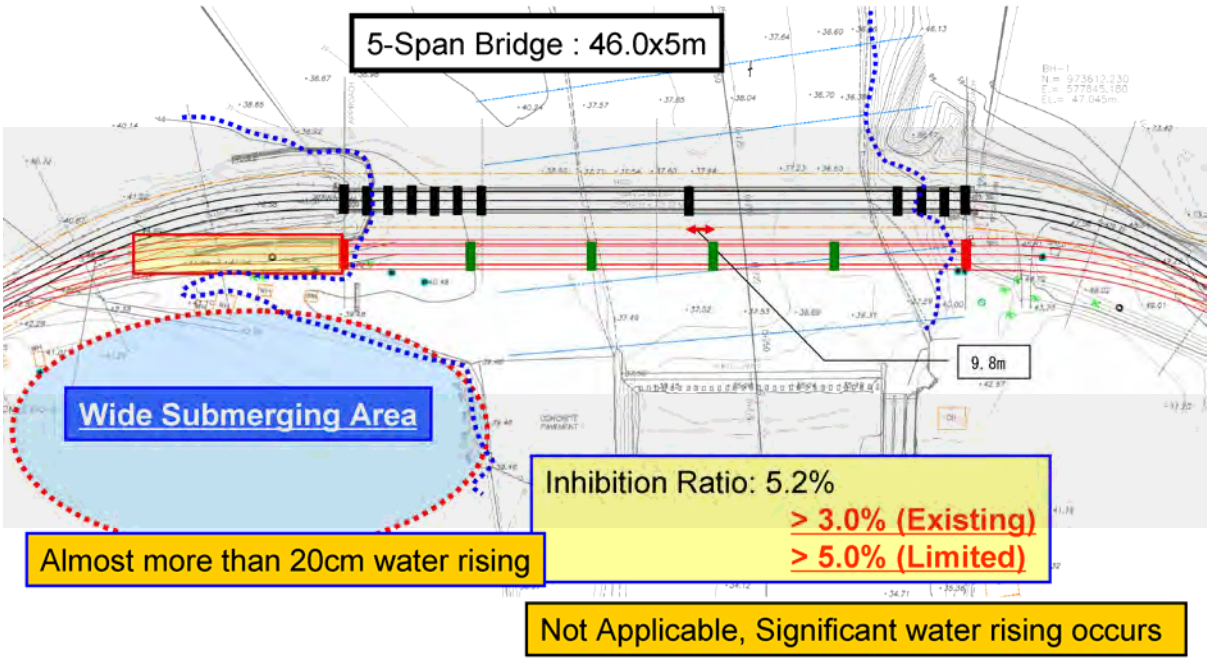


Figure 16.1.2-40 5 Span Bridge

STEP 4. Confirmation of BASIC CONCEPT

The basic concept resulted from above STEP 1 to STEP 3 is enumerated. The basic concept may be significantly important factor for bridge selection under the comparison study.

- New horizontal alignment is newly determined based on cost efficiency and surrounding conditions of the bridge. 20m shifted to downstream side.
- New abutments locations are newly determined by Hydraulic analysis
- The effect of the right side abutment and new embankment to the flood shall be carefully evaluated based on Hydraulic analysis. The left side abutment has no problems.
- Bridge Length of New Bridge: 230m
- Pier location is determined based on multiple verification of hydraulic analysis consisting of existing and planning bridge/river condition
- 2-Span (Max 115.0m), 3-Span (Max 80m), 4-Span (Max 57m) is available span arrangement
- Minimal maintenance bridge will be recommended, reflecting on-site request.

STEP 5. COMPARISON STUDY of New Bridge Types

i) Extraction of Applicable Basic Types based on Actual Results

Based on the basic concept and several conditions, applicable bridge types are extracted from the table of Relationships between Actual Results of Basic Bridge Types and Span Length, shown as follows.

Table 16.1.2-36 Extraction of Applicable Basic Types based on Actual Results

STEEL	PC
Simple Supported	
Suspension Bridge	—
2-Span	
Steel Lohse Arch Steel Truss Steel Composite Deck Truss (Rational Structure) Steel Deck Box	PC Cable Stayed Bridge
3-Span	
Steel Lohse Arch Steel Truss Steel Composite Deck Truss (Rational Structure) Steel Deck Box	PC Cable Stayed Bridge PC Extradosed Bridge PC Panel Stayed Bridge PC Hybrid Box (Rational Structure)

ii) Selection of Logically Suitable Types from above the Basic Types

Suitable bridge types are selected logically among above extracted bridges, to be utilized for final comparison study as follows:

Table 16.1.2-37 Extraction of Basic Types for Final Comparison Study (Steel)

STEEL	Inclusion of Final Comparison Study
Suspension Bridge	- Not Included in Final Comparison Study - Clearly expensive - Over Specification
2-Span Continuous Steel Lohse Arch	- Not Included in Final Comparison Study - Much expensive than steel truss (existing type) - Over Specification
2-Span Continous Steel Truss (Tubular)	**- Included in Final Comparison Study** - Same type to existing main bridge - As a candidate of basic tubular steel truss
2-Span Continuous Steel Deck Box	**- Included in Final Comparison Study** - Girder height 4.0m, 1.0m road rising necessary
3-Span Continuous Steel Lohse Arch	- Not Included in Final Comparison Study - Clearly expensive - Over Specification
3-Span Continous Steel Truss (Tubular)	**- Included in Final Comparison Study** - Same type to existing main bridge - As a candidate of basic tubular steel truss
3-Span Continuous Steel Deck Box	**- Included in Final Comparison Study** - Girder height 3.2m - No influence to vertical alignment
4-Span Continuous Steel Deck Box	**- Included in Final Comparison Study** - Girder height 2.0m - No influence to vertical alignment
4-Span Continuous RC Slab Steel Box	**- Included in Final Comparison Study** - Girder height 2.5m - No influence to vertical alignment。

Table 16.1.2-38 Extraction of Basic Types for Final Comparison Study (PC)

PC	Inclusion of Final Comparison Study
2-Span Continuous PC Cable Stayed Bridge	- Not Included in Final Comparison Study - Clearly expensive - Over Specification
3-Span Continuous PC Panel Stayed Bridge	**- Included in Final Comparison Study**
3-Span Continuous PC Box	**- Included in Final Comparison Study** - Cost is comparatively high because this new bridge will be over span length of 77m that is the actual results of general PC erection method - Girder height 5.3m - 1.3m of road rising necessary
4-Span Continuous PC Box	**- Included in Final Comparison Study** - Girder height 3.5m - No influence to vertical alignment

In addition to the above basic bridge types, following rational structures that are lately constructed in Japan are proposed and included in the final comparison study.

Table 16.1.2-39 Bridge Types for Final Comparison Study, including Rational Structures (Steel)

STEEL	Inclusion of Final Comparison Study
2-Span Continuous Steel Truss (Tubular)	**Explained Above**
2-Span Steel Composite Deck Truss	**- Included in Final Comparison Study** - A lot of overloaded lorries passed - Gate member of existing truss bridge damaged - Deck type bridge advantageous from visibility of driver - Application of PC Slab - Minimize steel members by composite structure - Truss height 7.0m - 3.2m road rising necessary
2-Span Continuous Steel Deck Box	**Explained Above**
3-Span Continous Steel Truss (Tubular)	**Explained Above**
3-Span Steel Composite Deck Truss	**- Included in Final Comparison Study** - A lot of overloaded lorries passed - Gate member of existing truss bridge damaged - Deck type bridge advantageous from visibility of driver - Application of PC Slab - Minimize steel members by composite structure - Truss height 4.5m - 1.0m road rising necessary
3-Span Continuous Steel Deck Box	**Explained Above**
4-Span Continuous Steel Deck Box	**Explained Above**
4-Span Continuous RC Slab Steel Box	**Explained Above**

Table 16.1.2-40 Bridge Types for Final Comparison Study, including Rational Structures (Steel)

PC	Inclusion of Final Comparison Study
3-Span Continuous PC Panel Stayed Bridge	**Explained Above**
3-Span Continuous PC Box	**Explained Above**
3-Span Continuous PC Hybrid Box	**- Included in Final Comparison Study** - Web: wave shape steel plate - Reduction of dead weight - Girder height is same to PC box girder - Complicated connection work
4-Span Continuous PC Box	**Explained Above**
4-Span Continuous PC Hybrid Box	**- Included in Final Comparison Study** - Web: wave shape steel plate - Reduction of dead weight - Girder height is same to PC box girder - Complicated connection work

Based on the above evaluation, multiple comparison study is conducted considering cost, structure, environmental impact, constructability and maintenance ability.

Table 16.1.2-41 Candidates of Final Comparison Study

SEEL	PC
2 Span	
2-Span Continuous Steel Truss (Tubular) **2-Span Steel Composite Deck Truss** **2-Span Continuous Steel Deck Box**	—
3 Span	
3-Span Continuous Steel Truss (Tubular) **3-Span Steel Composite Deck Truss** **3-Span Continuous Steel Deck Box**	**3-Span Continuous PC Panel Stayed Bridge** **3-Span Continuous PC Box** **3-Span Continuous PC Hybrid Box**
4 Span	
4-Span Continuous Steel Deck Box **4-Span Continuous RC Slab Steel Box**	**4-Span Continuous PC Box** **4-Span Continuous PC Hybrid Box**

Based on the evaluation, shown in the comparison table, the recommendable bridge type for outline design is **3-Span Continuous Steel Composite Deck Truss bridge**.

However, around this area, the water flows is comparatively complicated condition hydraulically. And the results of hydraulic analysis will affect critically the results of bridge planning. Therefore, in detail design stage, detail hydraulic analysis based upon further investigation shall be implemented by river/ hydraulic engineers. Based on the detail examination, the free board to be secured, level of high water level, abutment location and reevaluation of comparison study may be preferable to be re-implemented in the detail design stage.

iii) Selection of Logically Suitable Types of Bridge Foundations
In the study on selection of bridge foundation, study of site conditions for selecting an appropriate foundation type and comparison of structure (pile diameter) are the key discussions.

The site conditions are shown in below table. In the abutment A2 and Piers, there were located deep bearing layer with liquefiable soil.

Table 16.1.2-42 Site Candidates of Comparison Study

Study Type	Abutment Foundation	Pier Foundation
Foundation location	On the ground	In the River
proximity structure	-	-
Navigation condition	-	-
Water depth (m)	-	Around 6.0 to 8.5 m
Depth of bearing layer (m)	Around GL- 6.0m (A1) Around GL-38.0m(A2)	Around GL-16.0m (P1) Around GL-34.0m(P2)
Liquefaction	-	-
Lateral spreading	-	-

Based on the above site conditions, applicable type of Abutment A2 foundation is recommended spread footing type, Abutment A1 & Piers are recommended cast-in-site pile foundation.

According to the above evaluation, the pile diameter comparison study is conducted considering cost, construction period, constructability, and environmental impact.

The result of comparative study of pier foundation are shown in the next tables, the recommendable pile diameter of pile foundation for outline design is 1.2m Bored Pile, because of its advantages in low construction cost, minimum construction period and efficient constructability.

Table 16.1.2-43 Comparison on Pile Diameter of Wawa Bridge at P1 Pier

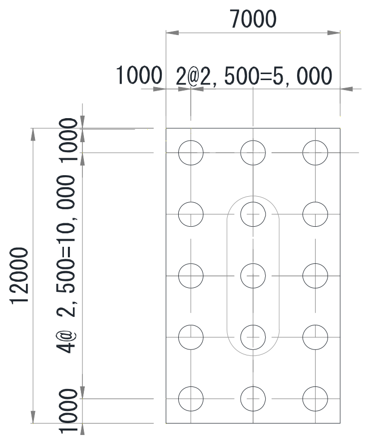
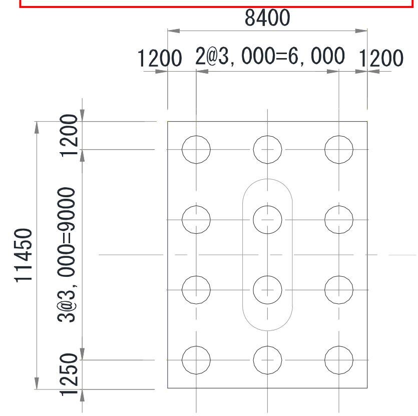
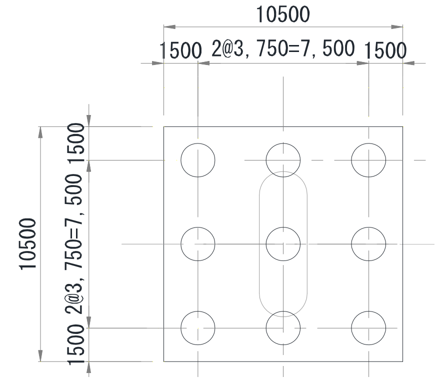
Evaluation Items	Alternative-1 Cas-in-place Concrete Pile D= 1.0m		Alternative-2 Cas-in-place Concrete Pile D= 1.2m		Alternative-3 Cas-in-place Concrete Pile D= 1.5m	
Side View Pile arrangement	Diameter of pile : 1000 mm Total number of pile : 16 Total length of pile : 112.0 m  		Diameter of pile : 1200 mm Total number of pile : 12 Total length of pile : 84.0 m OUTLINE DESIGN  		Diameter of pile : 1500 mm Total number of pile : 9 Total length of pile : 63.0 m  	
Structural Aspect and Stability	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.598 - Need to the large number of Cast-in-Place Concrete Piles.	A	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.615	B	- Pile Bearing Ratio (Pile Reaction/Pile Bearing) is 0.562.	A
Construction Cost (for Foundation)	Item / Quantity / Unit Cost (Php) / Total (1,000Php) Pile Cap Concrete / 168m3 / 7,559.8 / 1,270 Reinforcement steel / 34ton / 52,600.0 / 1,767 Pile / 105m / 45,898.5 / 4,819 Cofferdam / 552m2 / 21,181.9 / 11,692 Total 19,549 Ratio **1.007**	B	Item / Quantity / Unit Cost (Php) / Total (1,000Php) Pile Cap Concrete / 192m3 / 7,559.8 / 1,448 Reinforcement steel / 38ton / 52,600.0 / 2,015 Pile / 84m / 45,898.5 / 3,855 Cofferdam / 571m2 / 21,181.9 / 12,099 Total 19,417 Ratio **1.000**	A	Item / Quantity / Unit Cost (Php) / Total (1,000Php) Pile Cap Concrete / 221m3 / 7,559.8 / 1,667 Reinforcement steel / 44ton / 52,600.0 / 2,320 Pile / 63m / 52,238.6 / 3,291 Cofferdam / 600m2 / 21,181.9 / 12,709 Total 19,987 Ratio **1.029**	B
Construction Plan and Period	- Working in temporary cofferdam at the river. Cofferdam Work 20 days Pile work (1.0pile/day) 16 days Pile Cap 17 days Total **53 days** Ratio **1.018**	B	- Working in temporary cofferdam at the river. Cofferdam Work 21 days Pile work (1.0pile/day) 12 days Pile Cap 19 days Total **52 days** Ratio **1.000**	A	- Working in temporary cofferdam at the river. Cofferdam Work 22 days Pile work (1.0pile/day) 9 days Pile Cap 22 days Total **53 days** Ratio **1.018**	B
Constructability	- Constructability is inferior due to large number of Cast-in-place concrete Pile work.	C	- Constructability is superior with small number of foundation work.	A	- Need to large number of Steel Sheet Pile & Pile Cap cconcrete works.	B
Environmental Aspect	- Inferior in Environmental aspect due to large number of excavated soil.	C	- Better in Environmental aspect with small number of excavated soil & bentonite water.	B	- Superior in Environmental aspect with small number of excavated soil & bentonite water.	A
Evaluation	- Construction cost is highest with long construction period. Not Recommended	**C**	- Construction cost is lowest with minimum Construction period with efficient workability. Most Recommended	**A**	- Construction cost is highest with long construction period. Not Recommended	**B**

Table 16.1.2-44 Comparison of New Bridge Types for Wawa bridge (STEEL 1/3)

STEEL BRIDGES 1/3

COMPARISON of NEW BRIDGE TYPES for WAWA BRIDGE

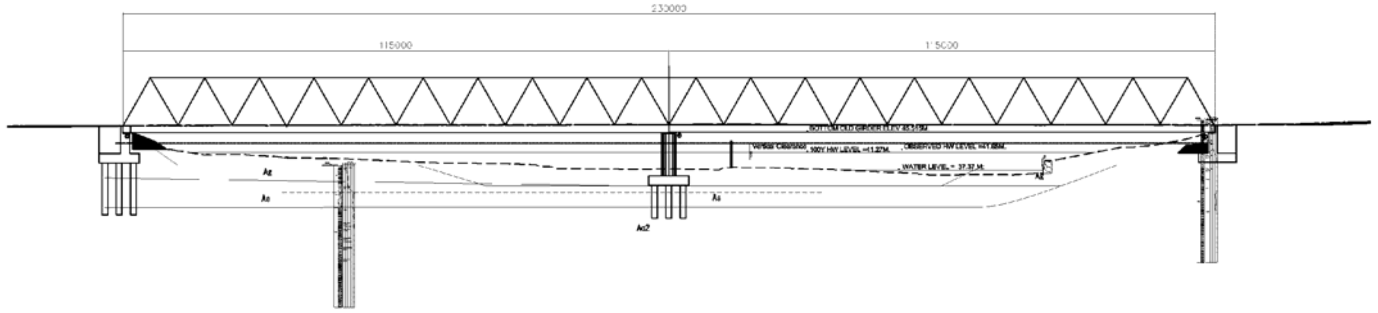
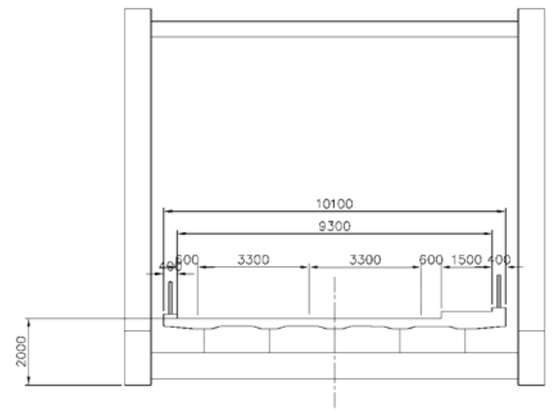
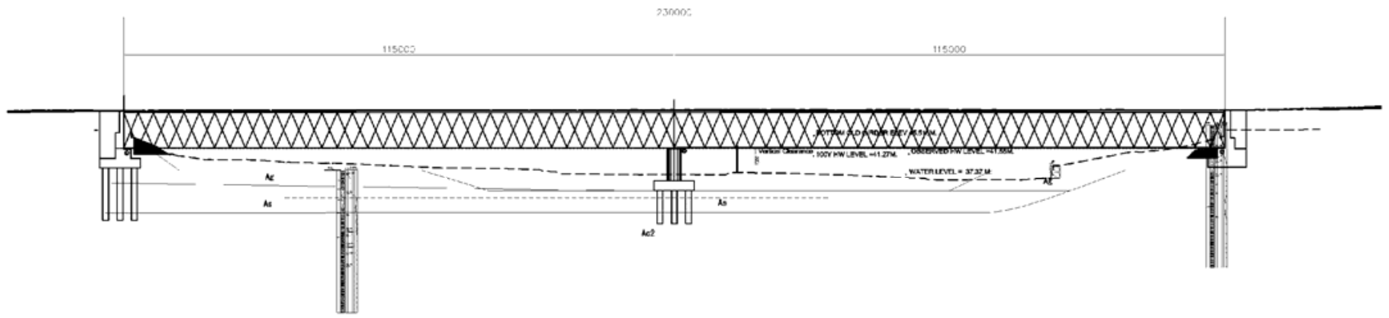
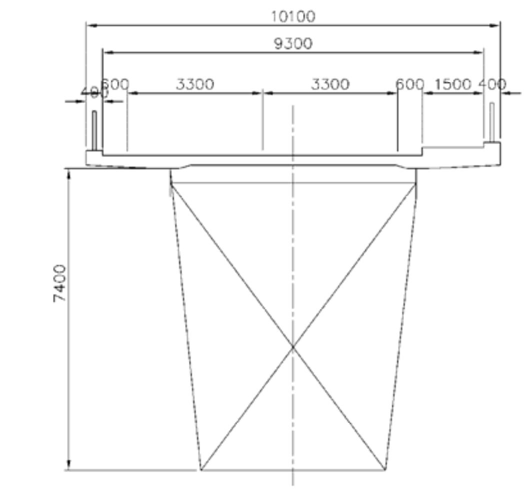
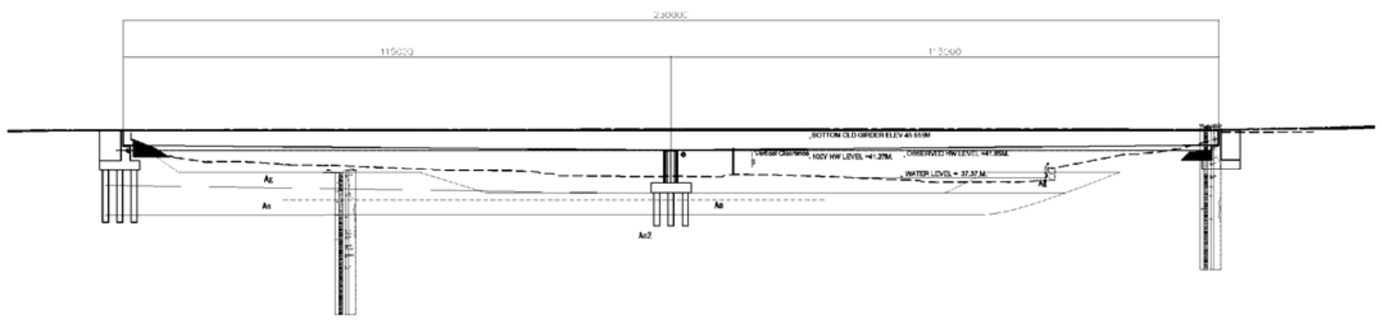
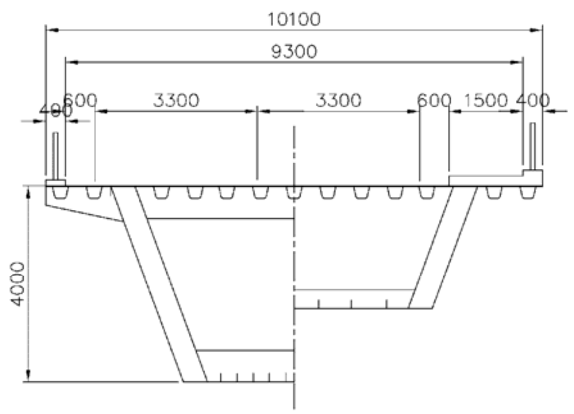
	Side View	Sectional View	Evaluation: Good-A-Fair-B-Poor-C			
Alternative 1S	2-Span Continuous Steel Truss 		COST	Superstructure	0.95	C
				Substructure	0.07	
				Road	0.02	
				Others	0.11	
				Total	1.15	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			B
			ENVIRONMENTAL IMPACT			C
			MAINTENANCE			C
Alternative 2S	2-Span Continuous Composite Lattice Truss 		COST	Superstructure	0.93	C
				Substructure	0.07	
				Road	0.04	
				Others	0.11	
				Total	1.15	
			STRUCTURAL FEATURE			A
			CONSTRUCTABILITY			A
			ENVIRONMENTAL IMPACT			C
			MAINTENANCE			A
Alternative 3S	2-Span Continuous Steel Deck Box Girder 		COST	Superstructure	0.96	C
				Substructure	0.07	
				Road	0.02	
				Others	0.11	
				Total	1.17	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			A
			ENVIRONMENTAL IMPACT			B
			MAINTENANCE			B

Table 16.1.2-45 Comparison of New Bridge Types for Wawa bridge (STEEL 2/3)

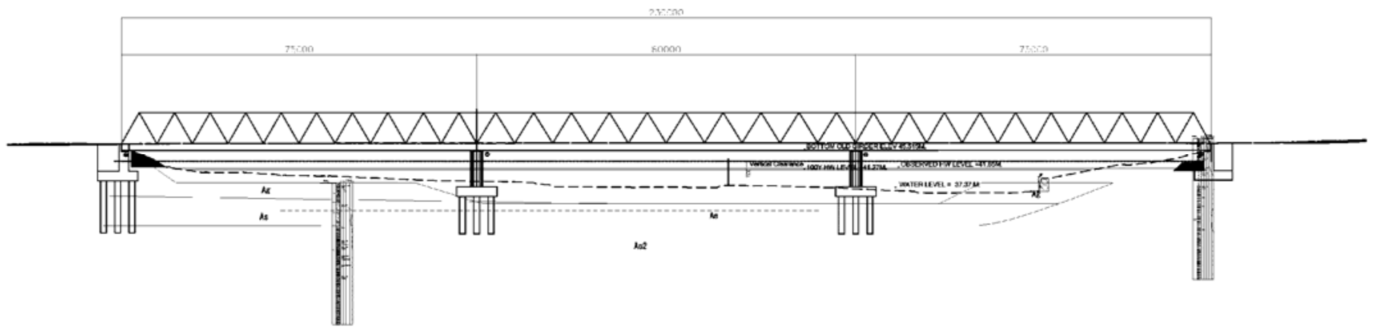
STEEL BRIDGES 2/3

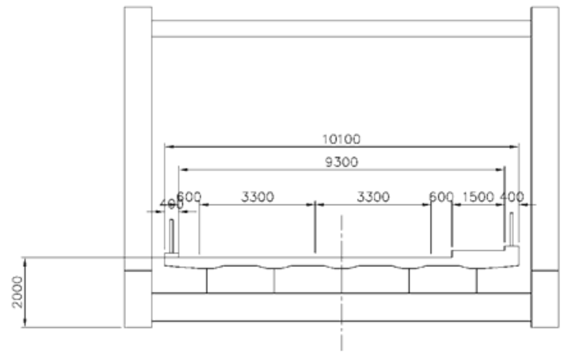
COMPARISON of NEW BRIDGE TYPES for WAWA BRIDGE

	Side View	Sectional View	Evaluation: Good-A-Fair-B-Poor-C

Alternative 4S

3-Span Continuous Steel Truss



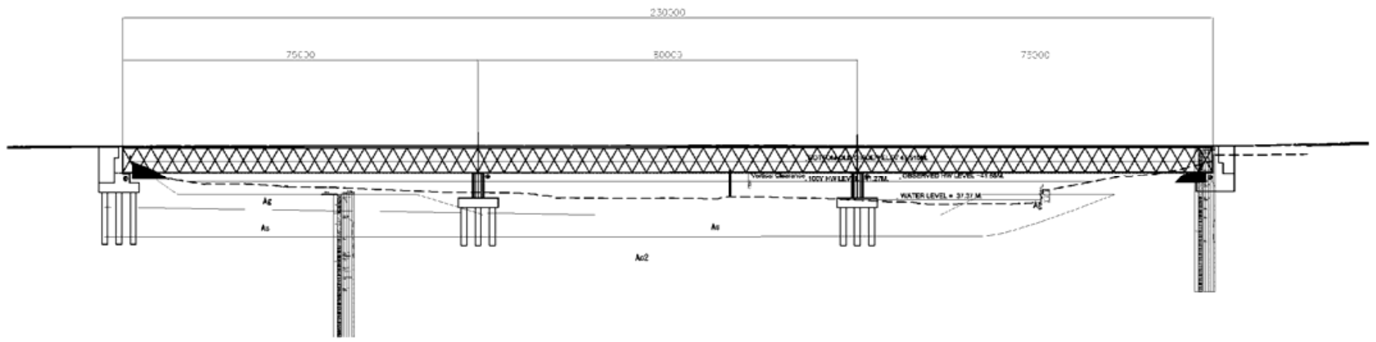


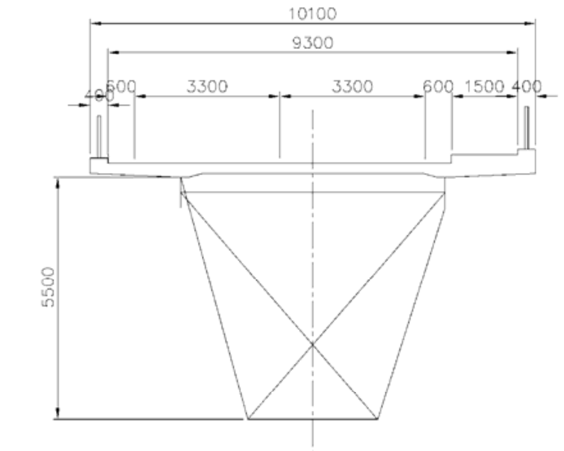
COST	Superstructure	0.81	B
	Substructure	0.09	
	Road	0.02	
	Others	0.11	
	Total	1.03	
STRUCTURAL FEATURE			B
CONSTRUCTABILITY			B
ENVIRONMENTAL IMPACT			C
MAINTENANCE			C

Alternative 5S

3-Span Continuous Composite Lattice Truss

OUTLINE DESIGN

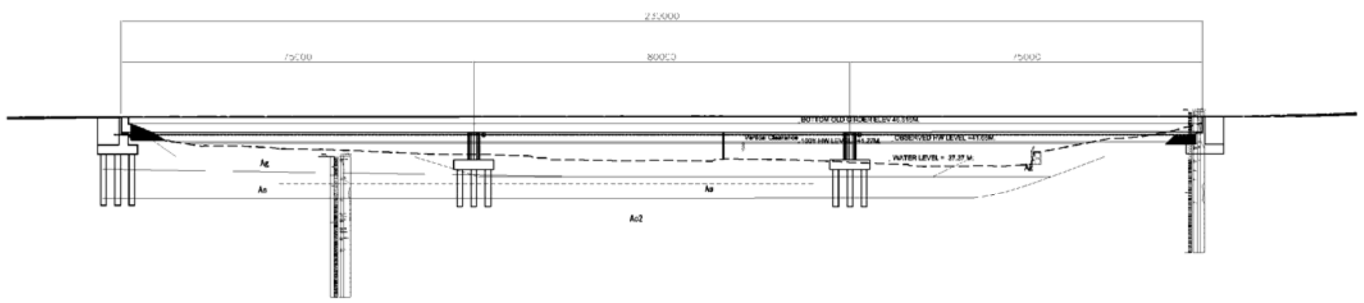


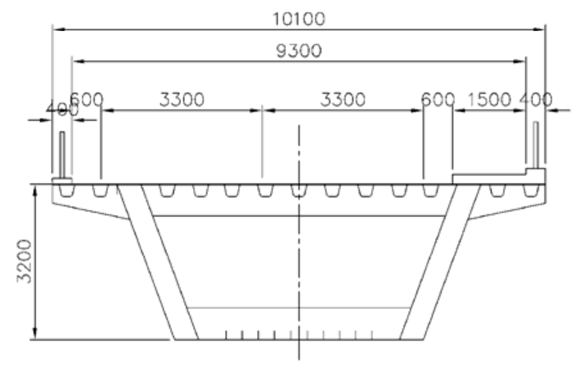


COST	Superstructure	0.78	A
	Substructure	0.09	
	Road	0.02	
	Others	0.11	
	Total	1.00	
STRUCTURAL FEATURE			A
CONSTRUCTABILITY			A
ENVIRONMENTAL IMPACT			B
MAINTENANCE			A

Alternative 6S

3-Span Continuous Steel Deck Box Girder





COST	Superstructure	0.84	B
	Substructure	0.09	
	Road	0.02	
	Others	0.11	
	Total	1.06	
STRUCTURAL FEATURE			B
CONSTRUCTABILITY			A
ENVIRONMENTAL IMPACT			B
MAINTENANCE			A

Table 16.1.2-46 Comparison of New Bridge Types for Wawa bridge (STEEL 3/3)

STEEL BRIDGES 3/3

COMPARISON of NEW BRIDGE TYPES for WAWA BRIDGE

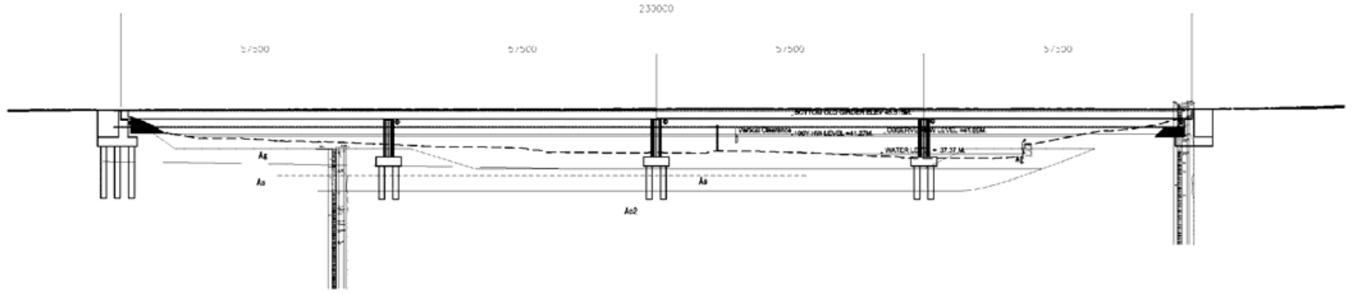
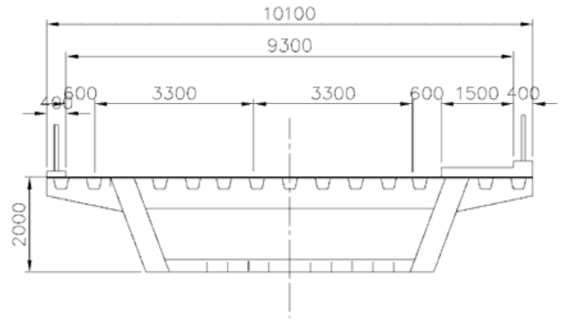
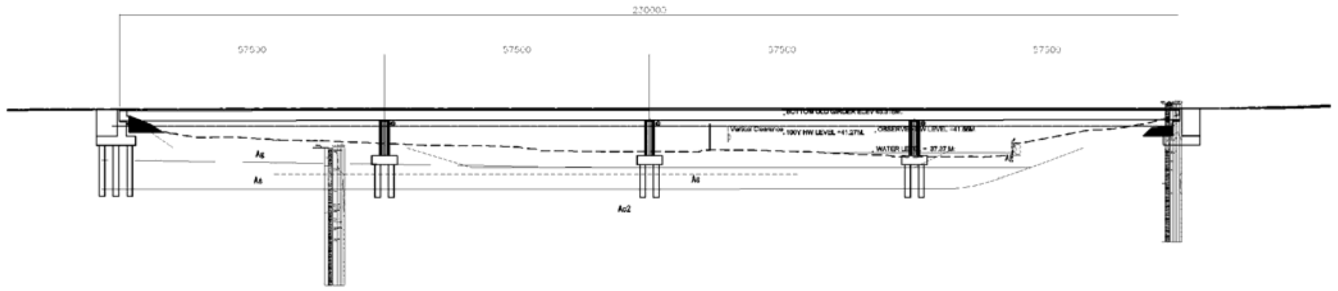
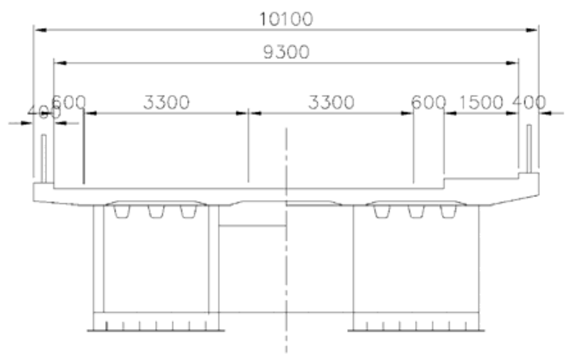
	Side View	Sectional View	Evaluation: Good-A-Fair-B-Poor-C			
Alternative 7S	4-Span Continuous Steel Deck Box Girder 		COST	Superstructure	0.80	B
				Substructure	0.10	
				Road	0.02	
				Others	0.11	
				Total	1.02	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			A
			ENVIRONMENTAL IMPACT			C
			MAINTENANCE			B
Alternative 8S	4-Span Continuous Steel I-shape Girder 		COST	Superstructure	0.78	B
				Substructure	0.10	
				Road	0.09	
				Others	0.02	
				Total	1.01	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			A
			ENVIRONMENTAL IMPACT			C
			MAINTENANCE			B

Table 16.1.2-47 Comparison of New Bridge Types for Wawa bridge (PC 1/2)

PC BBRIDGE 1/2

COMPARISON of NEW BRIDGE TYPES for WAWA BRIDGE

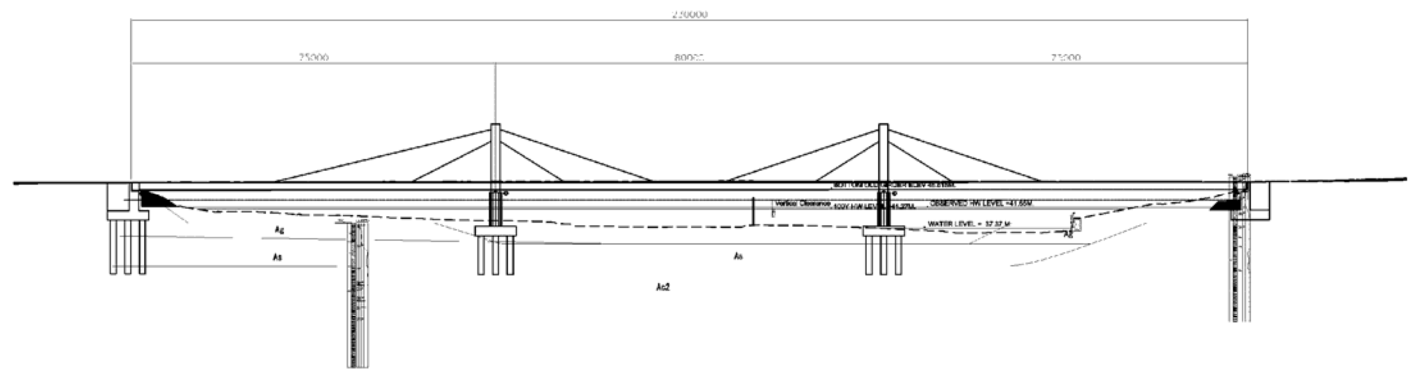
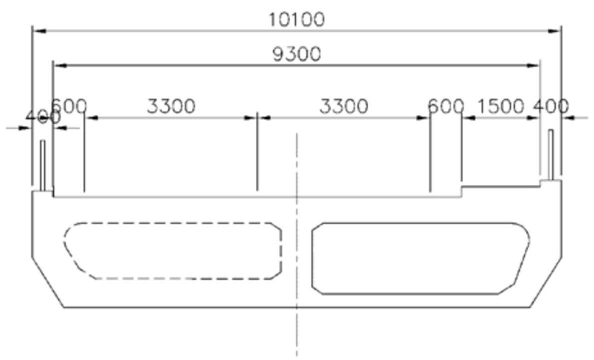
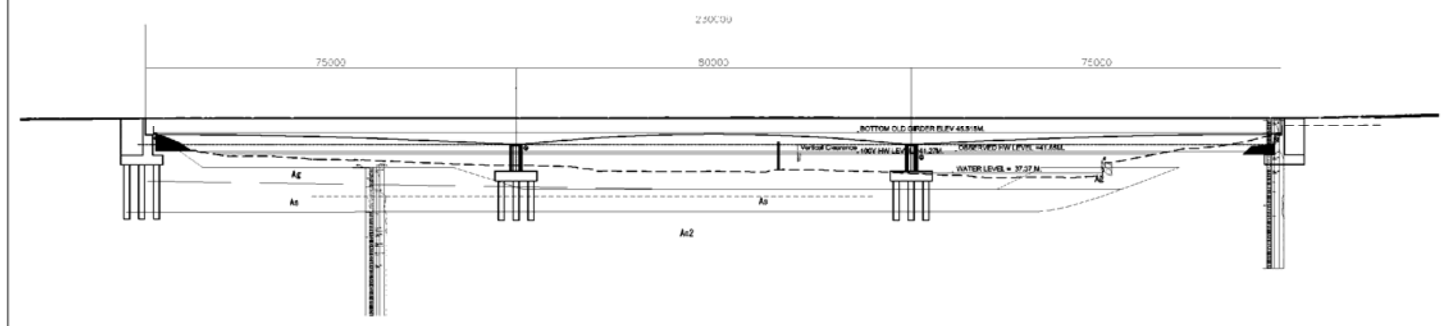
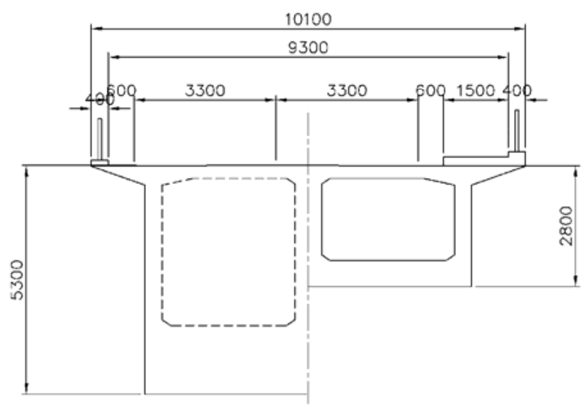
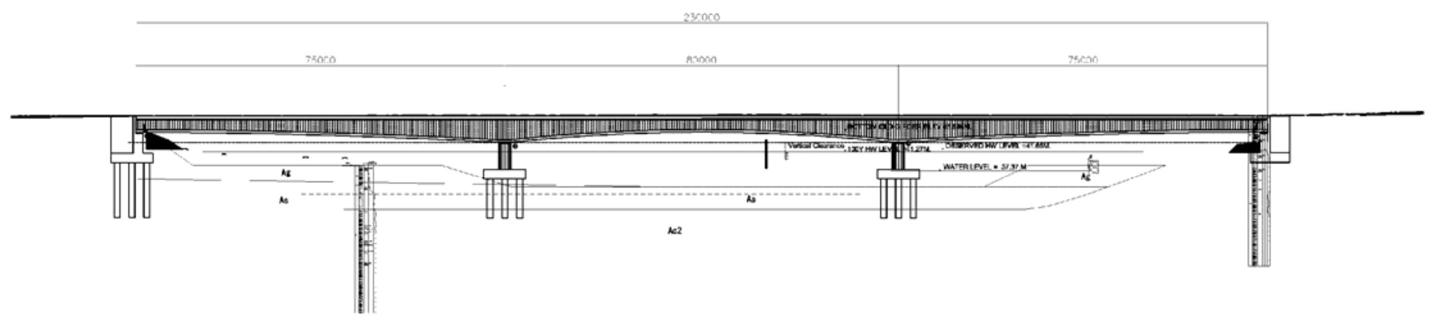
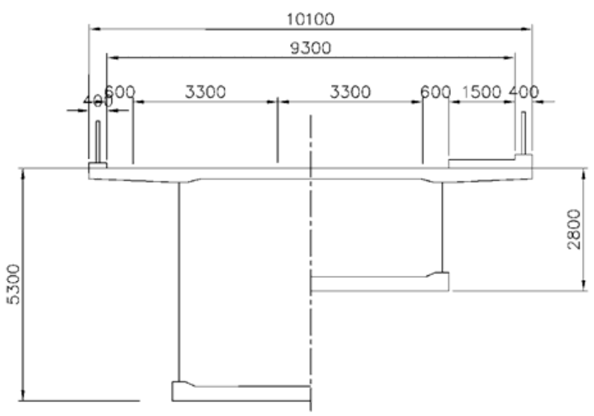
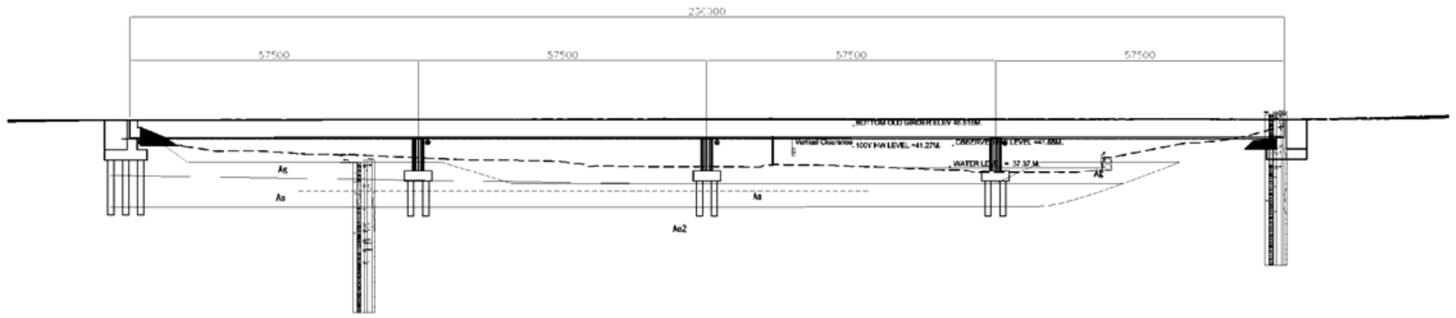
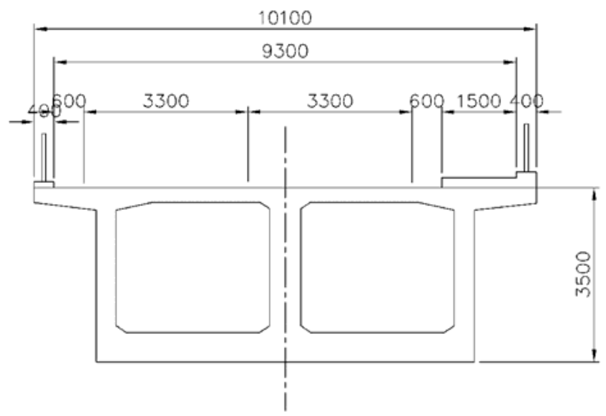
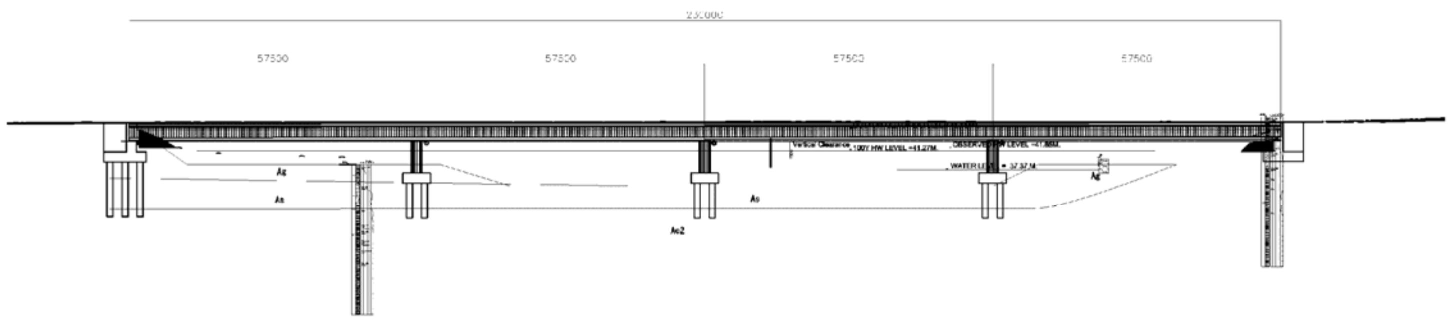
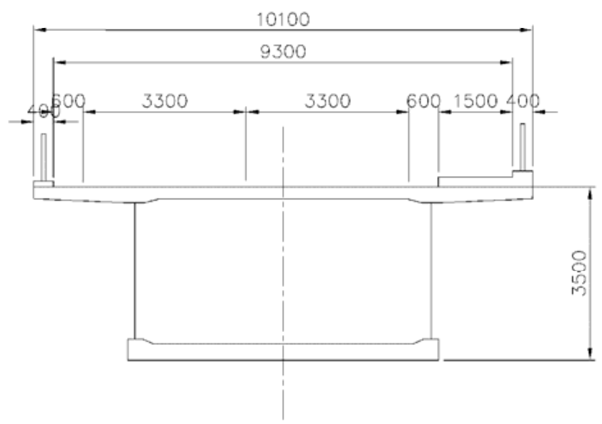
	Side View	Sectional View	Evaluation: Good-A·Fair-B·Poor-C			
Alternative 1P	3-Span Continuous Panel-Stayed Bridge 		COST	Superstructure	1.01	C
				Substructure	0.10	
				Road	0.02	
				Others	0.11	
				Total	1.23	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			C
			ENVIRONMENTAL IMPACT			B
			MAINTENANCE			A
Alternative 2P	3-Span Continuous PC Box Girder 		COST	Superstructure	0.85	B
				Substructure	0.10	
				Road	0.02	
				Others	0.11	
				Total	1.08	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			B
			ENVIRONMENTAL IMPACT			B
			MAINTENANCE			A
Alternative 3P	3-Span Continuous PC Hybrid Box Girder 		COST	Superstructure	0.89	C
				Substructure	0.09	
				Road	0.02	
				Others	0.11	
				Total	1.12	
			STRUCTURAL FEATURE			A
			CONSTRUCTABILITY			C
			ENVIRONMENTAL IMPACT			B
			MAINTENANCE			A

Table 16.1.2-48 Comparison of New Bridge Types for Wawa bridge (PC 2/2)

PC BBRIDGE 2/2

COMPARISON of NEW BRIDGE TYPES for WAWA BRIDGE

	Side View	Sectional View	Evaluation: Good-A-Fair-B-Poor-C			
Alternative 4P	4-Span Continuous PC Box Girder 		COST	Superstructure	0.75	B
				Substructure	0.13	
				Road	0.02	
				Others	0.11	
				Total	1.01	
			STRUCTURAL FEATURE			B
			CONSTRUCTABILITY			B
			ENVIRONMENTAL IMPACT			C
			MAINTENANCE			A
Alternative 5P	4-Span Continuous PC Hybrid Box Girder 		COST	Superstructure	0.79	B
				Substructure	0.10	
				Road	0.02	
				Others	0.11	
				Total	1.02	
			STRUCTURAL FEATURE			A
			CONSTRUCTABILITY			C
			ENVIRONMENTAL IMPACT			C
			MAINTENANCE			A

16.1.3 Methodology of Seismic Analysis of New Bridge

(1) Methodology of Seismic Analysis

For seismic design, responses of structure by assumed seismic forces must not be exceeded allowable limitation values. As the calculation methodologies to obtain such the responses of structure, various numerical computing analytical approaches are worldwidely utilized such as static analysis, dynamic analysis, liner analysis and non-linear analysis.
Currently, familiar analytical approaches utilized in earthquake countries including Japan is categorized into static analysis and dynamic analysis, furthermore dynamic analysis can be categorized into eigenvalue analysis, response spectrum analysis and time-history response analysis. In this sentence, the characteristic properties of such the various methodologies are organized and the seismic methodology utilized in outline design is explained.

(2) Static Analysis

In JRA, static analysis is utilized for the seismic design under LV 1 earthquake motion except seismically irregular bridges such as high influences of higher mode and laxness of the places where plastic hinges cause. Additionally it's utilized for the seismic design under LV2 earthquake motion on seismically regular bridges such as defined dominance of basic mode and basic bearing support system.
According to previously explained, verification approaches of seismic performance of bridges have two methodologies which are static method and dynamic method. The static analysis is the most simplified method because vibration characteristic has been transposed to static load system under the precondition that equal energy assumption is approval. However, the load system of static analysis is commonly based on a basic vibration mode vector, what it is a basic shape of mode vector that can be transposed to mono-mass system model so is not applicable to seismically irregular bridge. Furthermore, damping matrix as well as mass matrix does not exist naturally; because responses should be computed depending on only stiffness matrix and because structural damping and hysteresis damping of seismic countermeasure devices such as LRB and viscosity damper can not be considered in the methodology, the design freedom may be quite low. The concept of the static analysis is shown in the following equation.

< Static Analysis >

Internal Forces (Member Forces) = External Forces (Horizontal Loads):

$$K \times U = P \qquad \text{Eq.}$$

K: Stiffness matrix, U: Displacement of Nodes, P: Horizontal Forces

< Dynamic Analysis >

Internal Forces (Inertial Forces + Damping Forces + Member Forces) = Seismic Forces:

$$M \times \ddot{U} + C \times \dot{U} + K \times U = M \times \ddot{Z} \qquad \text{Eq.}$$

M: Mass matrix, C: Damping matrix, K: Stiffness Matrix, $\ddot{Z}$ *:Acceleration Vector,*
$\ddot{U}$: *Acceleration Vector of Nodes,* $\dot{U}$ *: Velocity of Nodes,* U *: Displacement of Nodes*

Therefore, modeling to the static analysis and estimation of seismic behavior must not be applicable to all of bridge types and structural conditions from the aspect of its property; firstly, based on eigenvalue analysis, basic vibration mode shown below should be confirmed whether the deformation shape obtained by static analysis are similar to the basic vibration mode, which can be defined as first mode, or not. In case of not synchronization, response spectrum analysis with eigenvalue modal analysis or time history response analysis should be applied.

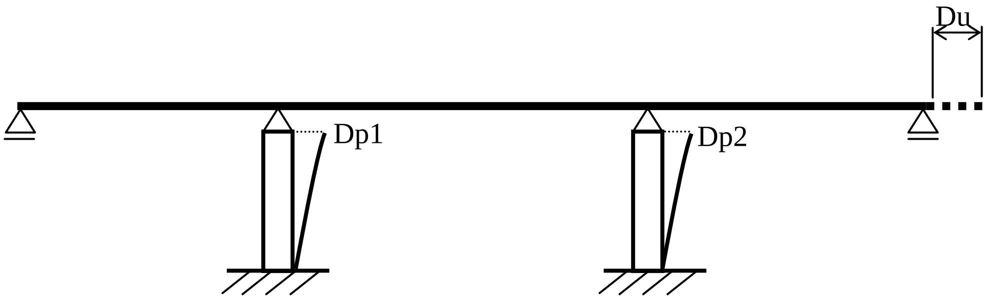


Figure 16.1.3-1 Basic Vibration Mode (Longitudinal Direction)

(3) Eigenvalue Analysis

Responses of bridges are calculated based on vibration property of the bridge and inputted seismic motion. Therefore, before calculating specific response values such as sectional forces and displacement against the inputted seismic motions, understanding the vibration property of the bridge must be extremely important phase because not only understanding dynamic behaviors but also previously mentioned dominant basic vibration mode can be understood to be utilized for static analysis. The most familiar methodology to clear this problem is eigenvalue analysis. Multi-Degree-of-Freedom and Multi-Mass-Vibration system such as bridge structure has same number of natural periods and vibration modes to number of mass. Such like that, eigenvalue analysis can be defined as calculating characteristic values of multi-mass-vibration system; the following values are commonly utilized.

(i) Natural Frequency and Natural Period

Natural frequency is defined as the vibration frequency (Hz), and Natural Period is the time (seconds) for a cycle, which indicates the period of well-vibrated vibration system. Eigenvalue analysis is to obtain characteristic values of vibration system, the principal is conformed to the above mentioned equation regarding dynamic analysis in which right side member is zero. Then, damping term should be separated from eigenvalue analysis but should be considered to determine mode damping based on various damping property when response spectrum analysis or time history response analysis. Therefore:

- No effects from inputted seismic motion and its direction
- Effects from mass and structural system
- Non-linear performance of structural members not considered
- Damping coefficient not considered, but later can be considered for response spectrum analysis or time history response analysis

In eigenvalue analysis, the natural frequency ω is obtained without consideration of damping factor, using the following equation. Where, the natural period T is the inverse number of the natural frequency.

$$\left|[K]-\omega^2[M]\right|=0 \qquad \text{Eq.}$$

$[K]$: *Stiffness matrix,* $[M]$*: Mass matrix*

(ii) Participation factor and Effective mass

The participation factor at "j" th mode can be obtained by following the equation. The standard coordination "q_j" that is the responses of the mode with larger participation factor become larger and commonly the participation factor have both positive and negative values.

$$\beta_j = \{\Phi_j\}^T [M]\{L\} / \overline{M}_j \qquad \text{Eq.}$$

β_j : Model participation factor, $\{\Phi_j\}$: Mode matrix, $[M]$: Mass matrix, $\{L\}$: Acceleration distribution vector: $\{\ddot{Z}\} = \ddot{z}\{L\}$: $\{\ddot{Z}\}$: Acceleration vector, $\ddot{z}$:Ground motion acceleration, $\overline{M}_j$: Equivalent mass

From the participation factor, the effective mass at "j" th mode can be obtained by the following equation and have always positive value and the summation of effective mass of all of the vibration modes must conform to total mass of the structure. This effective mass indicates "vibrating mass in all of mass". In case of modal analysis, accurate analytical results are generally obtained on the basis of adoption of the vibration modes including generally 90% of total mass. Thus, the participation factor and the effective mass can present useful indicator of dominant property regarding mass of each vibration mode such as which mass, which direction, how much amount.

$$m_j = (\{\Phi_j\}^T [M]\{L\})^2 / \overline{M}_j \qquad \text{Eq.}$$

m_j : Effective mass

(iii) Natural Vibration Mode (Mode Vector)

Natural vibration mode, what is called as mode vector, indicated the vibration shape at any mode based on dynamic equation of n-freedom system, which is very important factor because it is required in all the terms consisting of dynamic equation such as mass, damping and stiffness matrix. Generally, standard vibration mode vector $\{\Phi_j\}$ can be obtained by modal coordination which is transformed from displacement vector $[u]$ under ratio constant condition; then, coupling parameters are disappeared; n-freedom problem can be treated as "n" of mono-freedom systems. Such the analytical method is called and modal analysis method.

(4) Response Spectrum Analysis

Response spectrum analysis method can be defined as one of dynamic analytical approach under elastic conditions; maximum responses of structural members are easily confirmed for seismically irregular bridges. In JRA, this methodology can be utilized except the bridges the behavior of which is not complicated under seismic motion and except the seismic verification for the bridges with multiple plastic hinges under LV 1 seismic motion.
When standard vibration mode vector can be obtained based on previously explained eigenvalue analysis, the modal analysis for the mode vector corresponding to the natural period and damping factors can be easily implemented and can compute maximum response of structural members.
Dynamic analysis consists this response spectrum analysis and time history response analysis for which response can be computed historically by inputting wave shape historical seismic motion. However, it is not usually necessary to obtain complicated historical responses on seismic design but is frequently necessary to obtain only maximum responses of the structural members. Therefore, maximum responses for each vibration mode under a seismic motion are preliminarily prepared until a certain mode, and then the spectrum processed and organized by natural period and mode damping factor is absolutely response spectrum.
Natural modes can be called as 1st mode, 2nd mode and 3rd mode in the order corresponding to longer natural period or shorter natural vibration.
Where, the vibration modes that should be preliminarily prepared are to be adopted until the mode that over 90% of effective mass against total mass has been accumulated. For the bridges in this project, the bridge types such as Guadalupe, Lambingan, Palanit, Mawo and Wawa, are all categorized in girder type bridge not cable supported bridge; hence, 1st mode shape may be dominant mode. Therefore, it is not necessary to consider high modes like suspension bridges.
For superposition of maximum responses of multiple-mass system using response spectrum of each mode, SRSS, Square Rood of Sum of Square, and CQC method, Complete Quadratic Combination are worldwidely utilized.

(5) Damping

Structural damping usually strongly affects the results of dynamic analysis; appropriate examined damping coefficient must be incorporated into the model regardless linear, non-linear, modal analysis or time history response analysis.
For superstructures of general bridge types, viscous damping material internal damping, friction damping at bearing supports and aero dynamical damping can be considered. Also, for piers, material internal damping and friction damping as well as fugacity damping and friction damping between ground and footing can be considered.

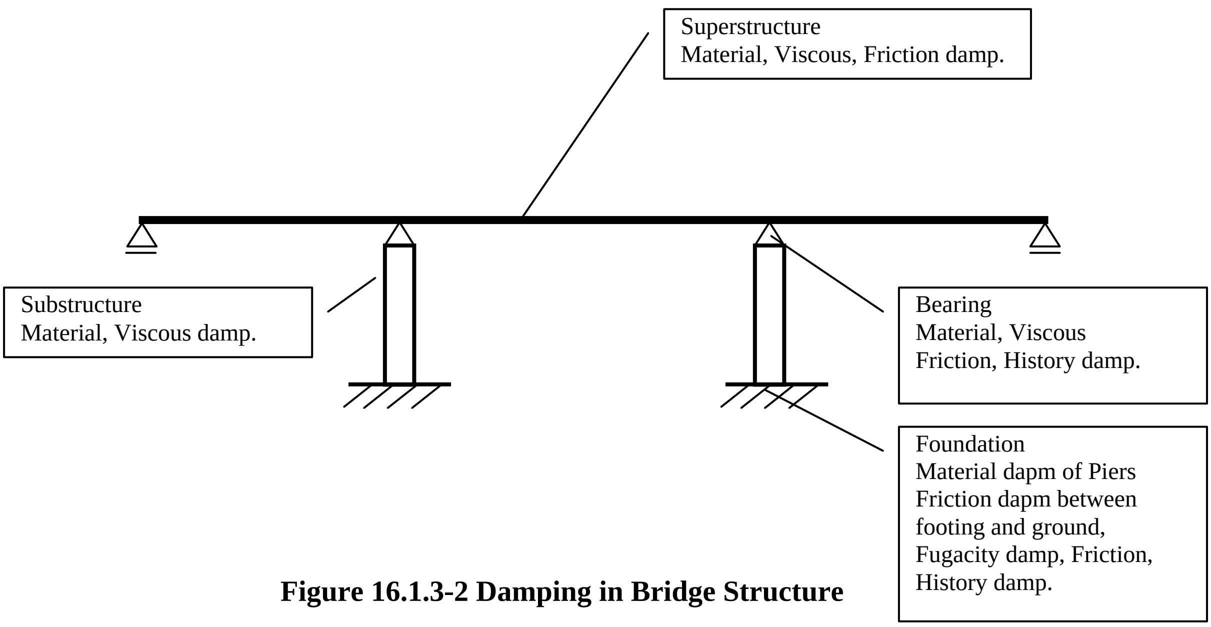


Figure 16.1.3-2 Damping in Bridge Structure

However, the specific mechanism of each damping factors are absolutely complicated, for execution of dynamic analysis, such the specific mechanism is not necessary to be understood. Generally damping forces are treated as equivalent damping forces in proportional to mass and strain energy.
Generally, because equivalent damping factor of each structural member can not directly be incorporated into dynamic equation, for response spectrum analysis, damping forces should be transformed to mode damping factors in order to be considered in the analysis.

<Dynamic Equation>

Damping

$$M \times \boxed{\ddot{U} + C} \times \dot{U} + K \times U = M \times \ddot{Z} \qquad \text{Eq.}$$

Where, generally for girder type bridge, strain energy proportional method, shown in the following equation, are utilized because this method can be incorporated into the dynamic in proportional to the amount of strain of the members and structural springs that do not have any mass.

<Mode damping hi: Strain Energy Proportional Method>

$$h_i = \frac{\sum_{j=1}^{n} c_j x_i^t k_j x_i}{x_i^t K x_i} \qquad \text{Eq.}$$

c_j : Structure damping factor of each element, x_i : Mode at i, k_j : Stiffness matrix of each element,
K : Stiffness matrix of all structure

For the bridges in this project, as the Cj in the above equation, following values are adopted.
- 0.01 for steel members
- 0.02 for concrete members
- 0.1 for foundation
- 0.03 for LRB under force distribution method

(6) Time History Analysis

Time history analysis is a dynamic analytical approach to obtain historical responses by inputting historical wave seismic motion. Generally, fiber elements are utilized for analytical model that may be complicated model because historical curves should be inputted into each element. However, in contrast to above mentioned response spectrum method, more advanced and high freedom dynamic behaviors can be obtained because the vibration system under material non-linearity as well as nonlinear historical properties of piers and rubber bearings can be accurately incorporated into the fiber elements.

(7) Applied Methodology of Seismic Analysis

Based on the new seismic specification prepared in this project, application of dynamic analysis to obtain definite solution of seismic behavior is highly recommended.
In this project, a lot of design spectrum are produced and proposed. These spectrums are all processed by equalization of various seismic forces. The analytical methodology that can highly and efficiently utilize the results may be preferable to be applied in the seismic analysis. Also, damping forces by LRB should be appropriately incorporated into the analysis and higher modes should be partially considered because the recommendable bridge type of Lambingan is arch type bridge that may have irregular behavior under seismic motion.
Otherwise, the philosophy of seismic analysis is based on linear analysis supplemented by R-factor, besides, time history analysis requiring validity of historical properties of each members between AASHTO and JRA may not be ready in that specification.
Consequently, the response spectrum analysis based on modal analysis may be the most efficient and most appropriate method to be applied to replace bridges in this project.

Table 16.1.3-1 Seismic Analysis

Analysis	Seismic Motion	Non-linear Member	Historical Properties	Damping Factor	Applicability for this Project
Static	Seismic Coefficient	Specific Point	Negative	Uniform	Positive
Response Spectrum "Dynamic Analysis"	Design Spectrum	Assumed Point	Negative	Damping Matrix	Positive Recommendable
Time History Response "Dynamic Analysis"	Historical Wave	Historical Property	Positive	Damping Matrix	Negative

16.2 Outline Design of Lambingan Bridge

16.2.1 Design Condition

The following items show design condition for the outline design of Lambingan Bridge.

(1) Road Conditions

- Road Design Standard : AASHTO STANDARD VALUE
- Design Speed : V = 50 kmph
- Live Loads : AASHTO Live Loads HL93 and Lane Loads
- Road Width : Shown as follows:

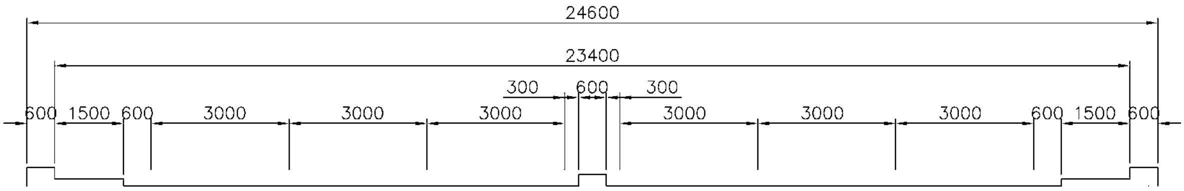


Figure 16.2.1-1 Cross Section/ Lane Arrangement of Lambingan Bridge

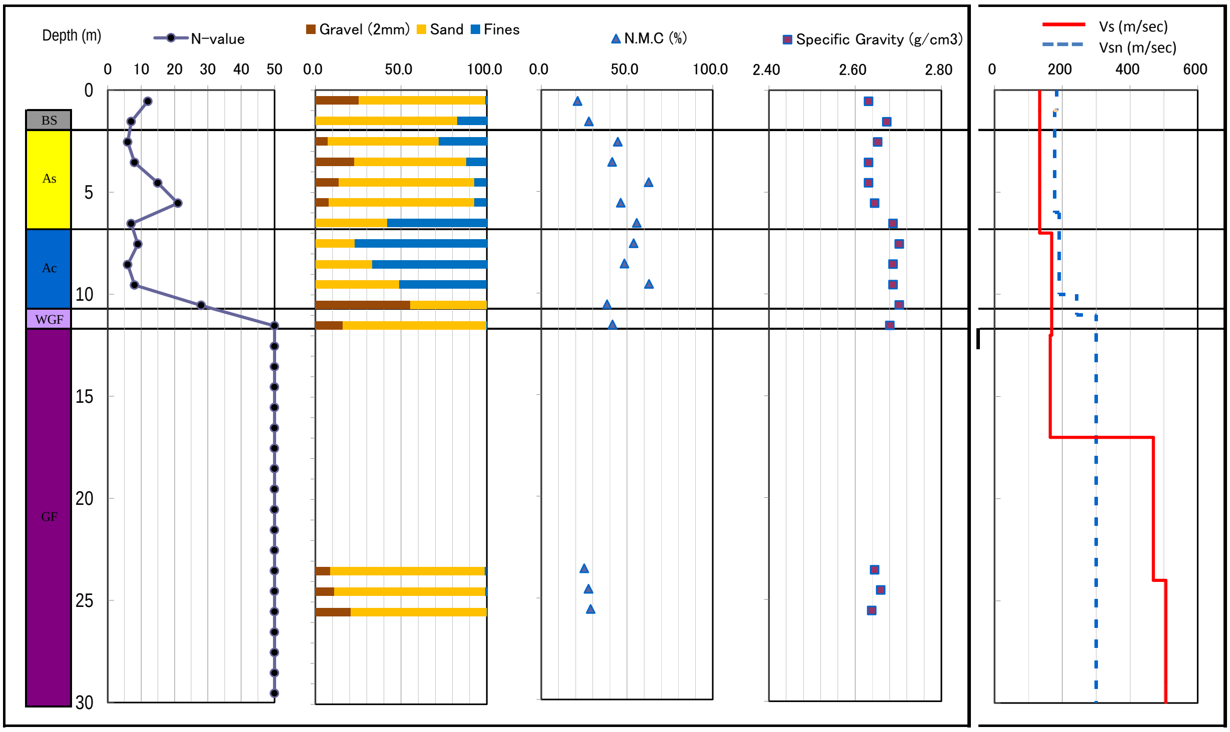
(2) Soil Conditions

The results of ground investigation are shown in below illustrations and following tables. The weathered rock layer that can be regarded as the bearing layer is distributed E.L. -40.0m to E.L.-50.0m depth, and has a thick surface layer predominant with clay on top. Specialty, liquefiable sand (AS) is thickly deposited from ground surface to GL-15m, of which N-value is 0 to 2, will be affected by liquefaction occurs with reduction of geotechnical parameter. .

Table 16.2.1-1 Summary for Soil Parameters (1)

Input by Tanaka - A2 Side - Lambingan B1 EL.3.0m									Input by EASCON						Soil Parameters					
Number	Depth		Nstd	S-wave	S-wave Ave.	Vsn	Soil Layer	Soil Classification	GSA-J			N.M.C (%)	PI	Specific Gravity	Nd	γt	C	ϕ	E0	Vsn
	Upper	Lower						Observation	Gravel (%)	Sand (%)	Fines (%)					(tf/m²)	(kN/m²)	(°)	(kN/m²)	(m/sec)
1	0.55	1.00	12	134	134	183	Bs	Medium sand	25.4	73.66	0.9	21.2	N/A	2.63	12	17	0	35	8,400	183
2	1.55	2.00	7	134		178		Sandy silt	0.0	82.70	17.3	27.8	N/A	2.67		17				
3	2.55	3.00	6	134		178		Silty fine sand	7.2	64.76	28.0	44.6	N/A	2.65		17				
4	3.55	4.00	8	134	134	178	As	Fine sand w/ silt	22.6	65.40	12.0	41.4	N/A	2.63	11	17	0	34	7,700	178
5	4.55	5.00	15	134		178		Silty fine sand	13.5	79.18	7.3	62.7	N/A	2.63		17				
6	5.55	6.00	21	134		178		Silty fine sand	7.8	84.87	7.3	46.4	N/A	2.65		17				
7	6.55	7.00	7	134		191		Clay w/ sand	0.0	41.90	58.1	55.7	12	2.69		15				
8	7.55	8.00	9	169	160	191	Ac	Clay w/ sand	0.0	22.93	77.1	53.9	33	2.70	7	15	44	0	4,900	191
9	8.55	9.00	6	169		191		Sandy clay	0.0	33.13	66.9	48.5	19	2.69		15				
10	9.55	10.00	8	169		191		Sandy clay	0.0	49.03	51.0	62.9	45	2.69		15				
11	10.55	11.00	28	169	169	243	WGF	Gravel/sand w/ fines	55.2	44.58	0.2	38.4	N/A	2.70	28	17	-	37	19,600	243
12	11.55	12.00	150	169		300		Sandy weathered rock	15.8	83.74	0.5	41.6	N/A	2.68		21				
13	12.55	13.00	150	165		300		Rock								21				
14	13.55	14.00	300	165		300		Rock								21				
15	14.55	15.00	300	165		300		Rock								21				
16	15.55	16.00	300	165		300		Rock								21				
17	16.55	17.00	300	165		300		Rock								21				
18	17.55	18.00	300	469		300		Rock								21				
19	18.55	19.00	300	469		300		Rock								21				
20	19.55	20.00	300	469		300		Rock								21				
21	20.55	21.00	300	469	385	300	GF	Rock							268	21	480	21	126,044	300
22	21.55	22.00	300	469		300		Rock								21				
23	22.55	23.00	300	469		300		Rock								21				
24	23.55	24.00	300	469		300		Rock	8.4	90.47	1.1	25.1	N/A	2.65		21				
25	24.55	25.00	300	506		300		Fine sand	10.9	88.08	1.0	27.6	N/A	2.66		21				
26	25.55	26.00	150	506		300		Fine sand	20.5	79.29	0.2	28.8	N/A	2.64		21				
27	26.55	27.00	150	506		300		(Fine sand)								21				
28	27.55	28.00	300	506		300		(Fine sand)								21				
29	28.55	29.00	300	506		300		Rock								21				
30	29.55	30.00	300	506		300		Rock								21				

Table 16.2.1-2 Summary for Soil Parameters (2)



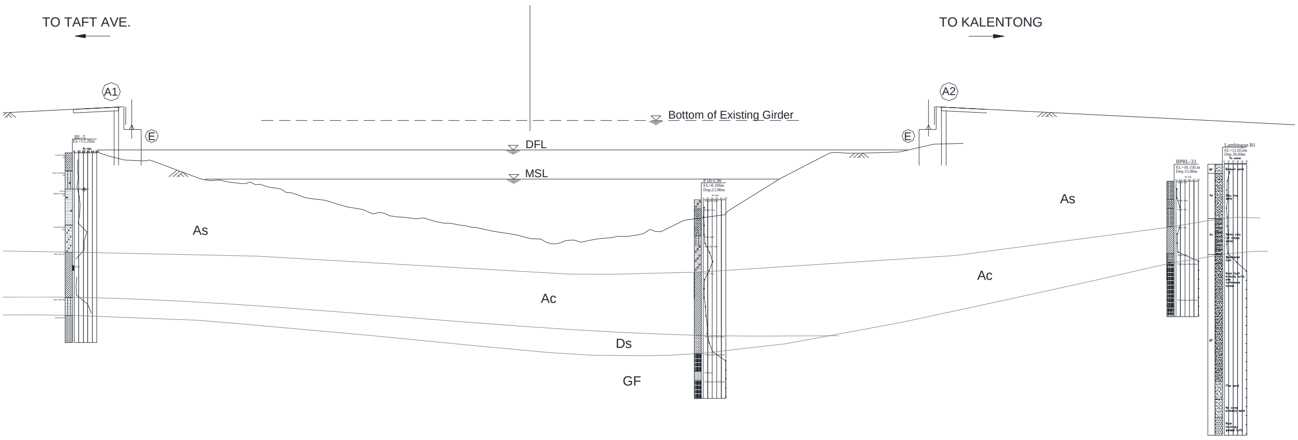


Figure 16.2.1-2 Soil Profile of Lambingan Bridge (Included previous SPT)

(3) Hydraulic Conditions

- Design Water Level : EL= 1.48m
- Freeboard from Design Flood Level : H = 3.75m (To secure existing freeboard)

Hydraulic condition shall be carefully verified and examined by detail hydraulic analysis in the detail design stage, based on which the road and bridge planning shall be reevaluated in such the phase including comparison study of bridge types.

(4) Bridge Type

- Superstructure Type : Steel Deck Lohse Arch Stiffening Box Girder
- Bridge Length : L=90m
- Transversal Slope : 2.0%
- Longitudinal Slope : 5.0%/ -5.0% (Crown at the center of the bridge)
- Horizontal Alignment : R=∞
- Angle of Alignment : 90 Degrees

  Wearing coat : Guss asphalt and Polymer Modified Asphalt
  t=80mm for Vehicle lane, t=30mm for walkway
- Railing : Steel railing for vehicle and pedestrian
- Bearing : NRB Rubber Bearings - Force Distribution Bearing
- Expansion Joint : Steel type
- Drainage Appliances : PVC pipe
- Bridge Falling Prevention Device: Cable type

- Substructure Type : RC wall type
- Foundation Type : Cast-in-place Pile (D=2.5m)
- Bearing Soil Condition : Clay with Gravel Layer (N>45)

(5) Design Cases of Outline Design

The outline design of superstructure shall be designed based on the above load condition, specified in AASHTO 2012. On the basis of various reactions and forces, substructures and foundation shall be designed throughout response spectrum analysis under the limit state of "Extreme Event I" specified in AASHTO 2012.

STEP 1. Outline Design of Superstructure
- Design Load, specified in AASHTO 2012
- Stress check

↓

STEP 2. Organization of Reaction Forces
- Organization of reaction forces for substructure design
- Organization of masses for response spectrum analysis

↓

STEP 3. Modal Analysis and Response Spectrum Analysis
- Eigenvalue analysis based on modal analysis
- Organization of response values by the seismic analysis

↓

STEP 4. Substructure and Foundation Design
- Extreme Event I

Figure 16.2.1-3 Flow of Outline Design

16.2.2 Outline Design of Superstructure

(1) Design Condition

- Superstructure Type : Steel Deck Lohse Arch Stiffening Box Girder
- Bridge Length : L=90m
- Angle of Alignment : 90 Degrees
- Wearing coat : Guss asphalt and Polymer Modified Asphalt
  t=80mm for Vehicle lane, t=30mm for walkway
- Railing : Steel railing for vehicle and pedestrian
- Bearing : NRB Rubber Bearings - Force Distribution Bearing
- Road Width : Shown as follows:

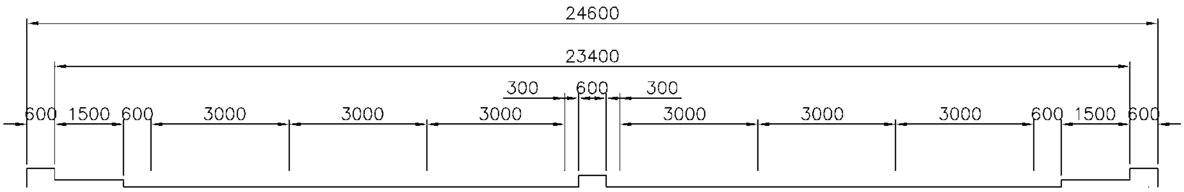


Figure 16.2.2-1 Cross Section/ Lane Arrangement of Lambingan Bridge

Based on stage construction, half of structure should be designed separately, shown as follows.

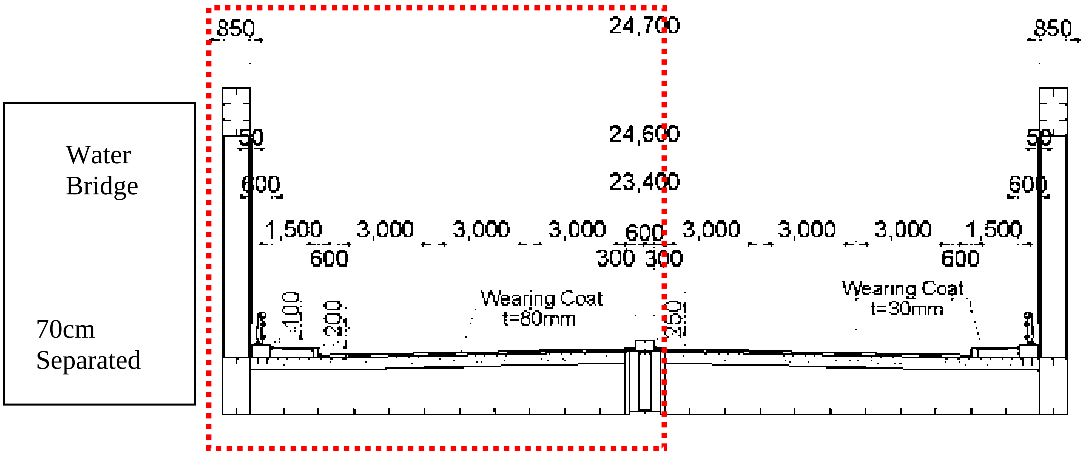


Figure 16.2.2-2 Design Section of Lambingan Bridge

(2) Design Loads

- Dead Loads : AASHTO 2012
- Live Loads : HL93 and Lane Loads in AASHTO 2012, utilized by influence line evaluation
- Limit State and Load Combination : Strength I in AASHTO 2012

Table 16.2.2-1 Load Combinations and Factors at Strength I in AASHTO 2012

Load Combination / Limit State	DC DD DW EH EV ES EL PS CR	LL IM CE BR PL LS	WA	WS	WL	FR	TU	TG	SE	Use One of These At a Time: EQ	CT	CV
Strength I	γ_p	1.75	1.00	-	-	1.00	0.5/1.2	γ_{tg}	γ_{se}	-	-	-

Source: LRFD 2012

(3) Analytical model

In the outline design, only the first stage structural system is conducted by using fish-born frame model based on stage construction. The following figure shows the analytical model for outline design of Lambingan bridge. All elements in the analysis are truss and beam model which have 6 of DOFs

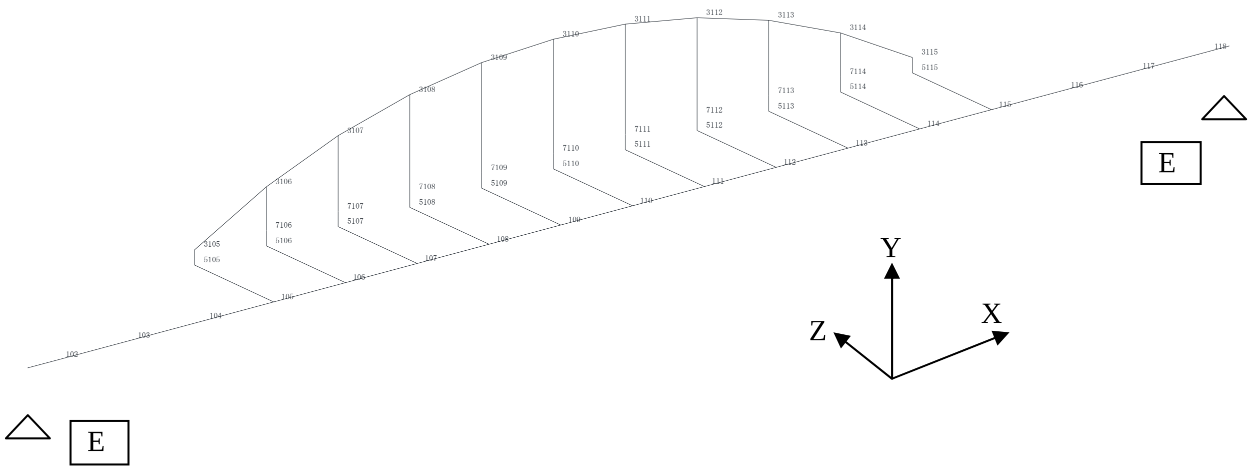
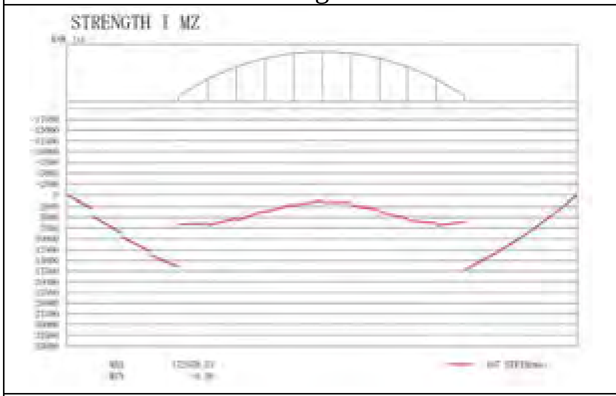
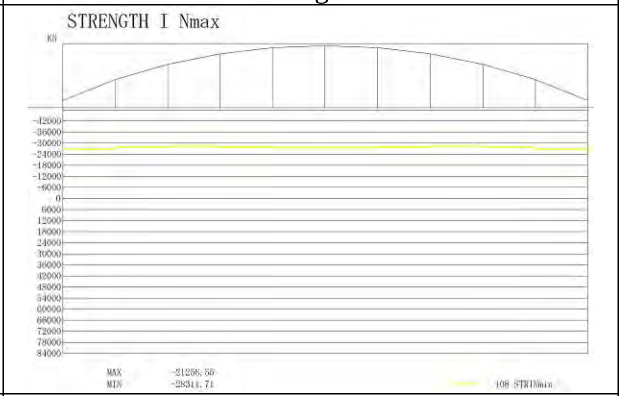


Figure 16.2.2-3 Analytical Model for Superstructure

(4) Sectional forces under Load Combination Strength I

Based on the analytical model, various sectional forces to be utilized for outline design can be obtained. In this report, two of figures regarding distribution of bending moments and axial forces under the combination sectional forces of "Strength I" are in the following figures.
The length of arch rib is determined based on constructability. And the height of arch rib is determined from the balances between cased sectional forces of arch rib and steel deck. The balances are well adjusted, in which the dimension of arch rib and steel deck can be worked efficiently

Table 16.2.2-2 Distribution of Sectional Forces under Combination of Strength I

Bending Moment in the Steel Deck "Strength I"	Axial Forces in the Arch Rib "Strength I"
	
Arch rib works efficiently and rationally to reduce huge bending moment in the steel deck. If the arch rib were not installed on the deck, the maximum bending moment would be beyond 200,000kNm, by which the thickness of the steel deck box would be approx. 100mm around.	The length of arch rib, which is 50m, is determined based on constructability and separation length of existing piers. Axial forces caused in the arch rib are less than 30,000kN. The horizontal size of arch rib can not be larger than about 900mm because of water bridge in the downstream side. Therefore, such the caused axial forces can be acceptable for such the arch rib size.

(5) Stress Check

Based on the sectional forces of the load combination Strength I, stress checks are conducted for the following sections of the superstructure.

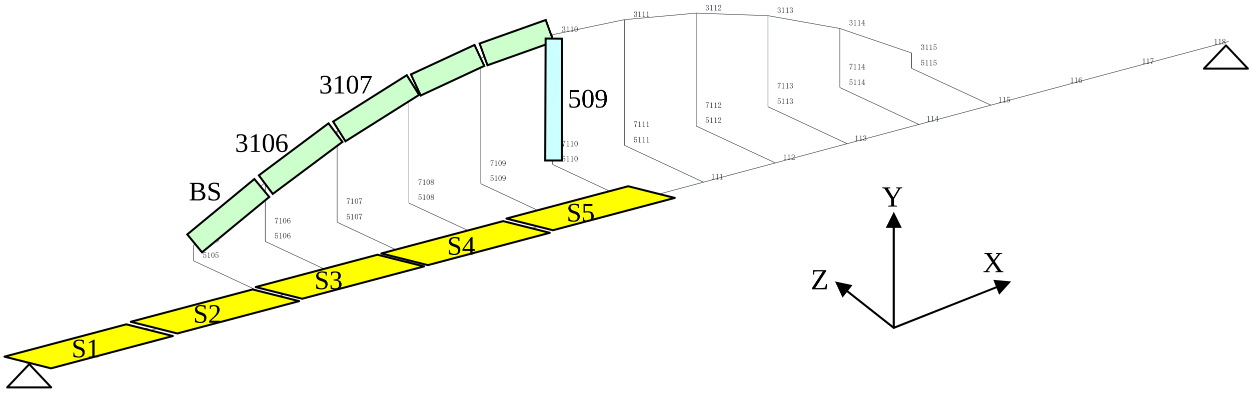
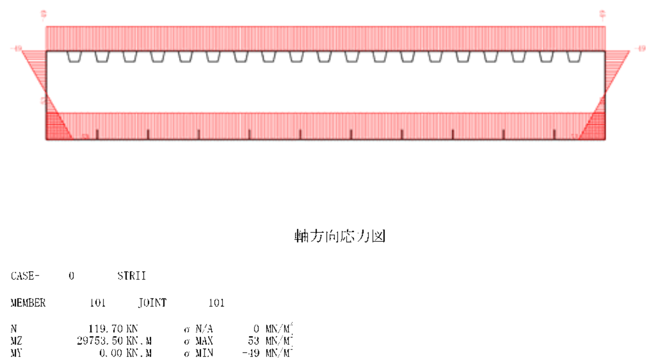


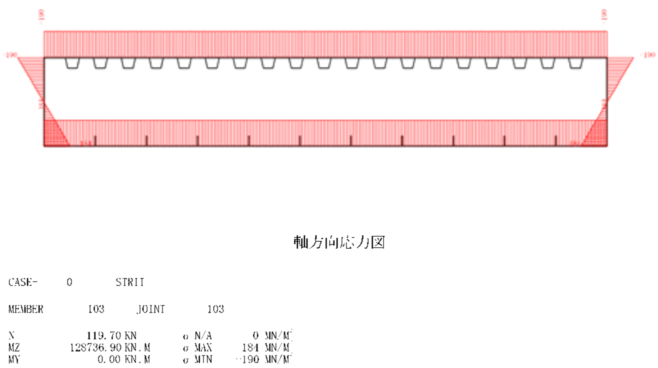
Figure 16.2.2-4 Sections for Stress Check

The results of stress checks are shown as follows.

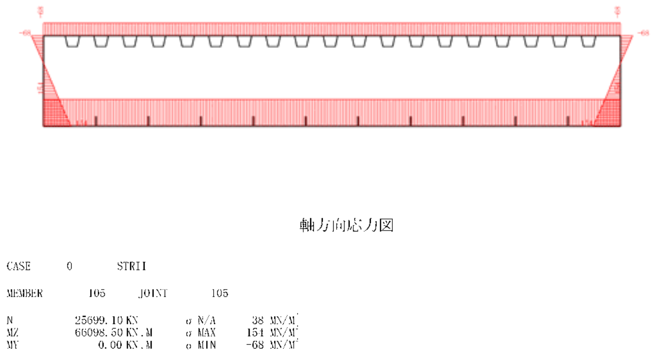
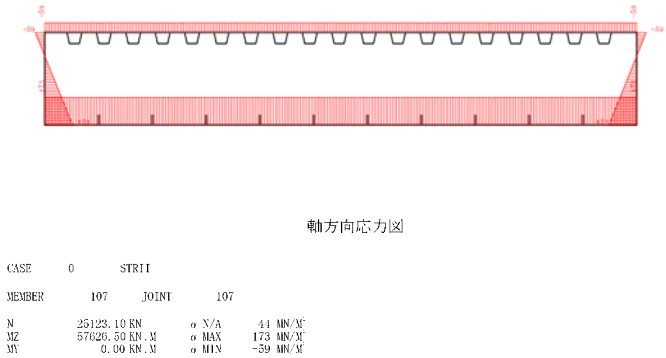
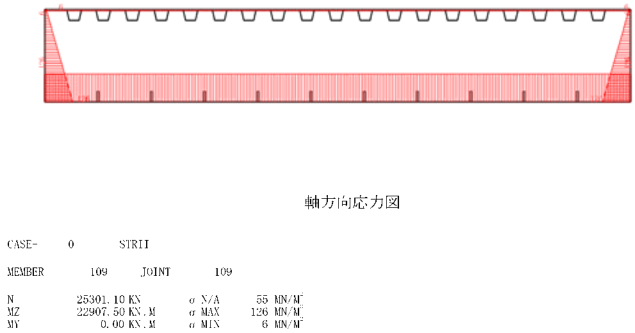
<Steel Deck>

Table 16.2.2-3 Stress Check of Steel Deck

<table>
<tr><th></th><th>Distribution of Normal Stresses</th><th>Results</th></tr>
<tr><td>S1</td><td></td><td>N = 120kN
Mz = 29800kNm
S = 6620 kN

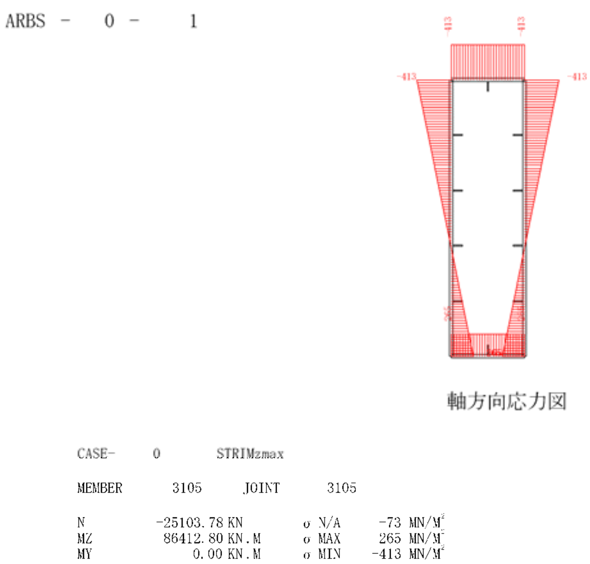
<SM490Y>
Upper flange : 18mm
Web : 20mm
Lower flange : 18mm
σ max = 53Mpa < 355 Mpa
τ max = 86 Mpa < 178 Mpa</td></tr>
<tr><td>S2</td><td></td><td>N = 120kN
Mz = 129000kNm
S = 6700 kN

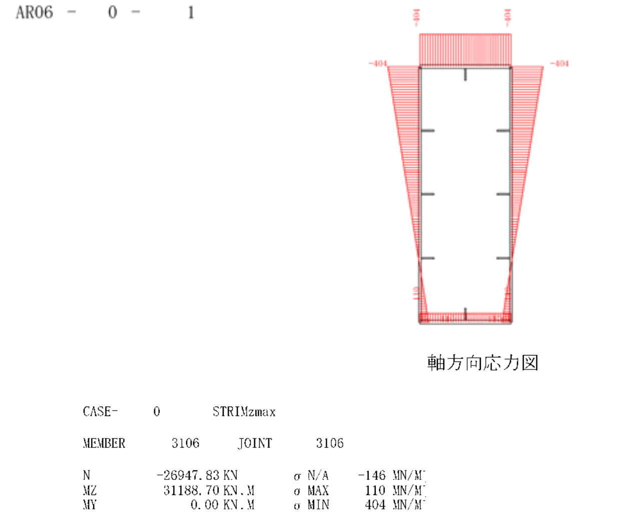
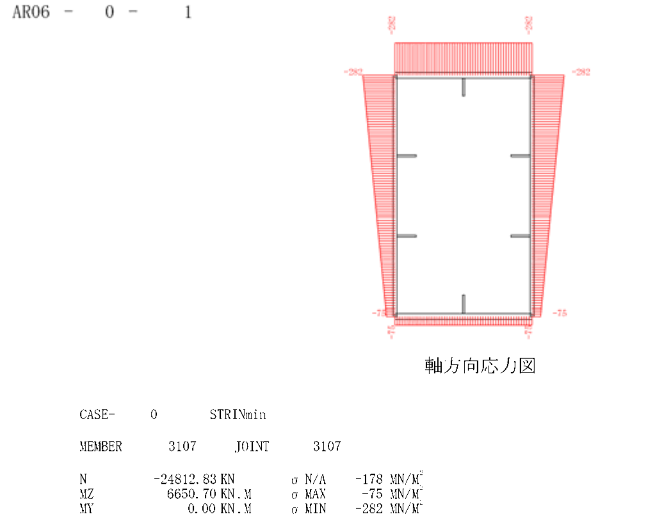
<SM490Y>
Upper flange : 21mm
Web : 17mm
Lower flange : 24mm
σ max = 190Mpa < 355 Mpa
τ max = 100 Mpa < 178 Mpa</td></tr>
</table>

	Distribution of Normal Stresses	Results
S3		N = 26000kN Mz = 66000kNm S = 350 kN <SM490Y> Upper flange : 19mm Web : 14mm Lower flange : 19mm σ max = 154Mpa < 355 Mpa τ max = 68 Mpa < 178 Mpa
S4		N = 25000kN Mz = 58000kNm S = 4500 kN <SM490Y> Upper flange : 17mm Web : 11mm Lower flange : 14mm σ max = 173Mpa < 355 Mpa τ max = 103 Mpa < 178 Mp
S5		N = 25000kN Mz = 23000kNm S = 2100 kN <SM490Y> Upper flange : 14mm Web : 10mm Lower flange : 9mm σ max = 126Mpa < 355 Mpa τ max = 48 Mpa < 178 Mp

<Arch Rib>

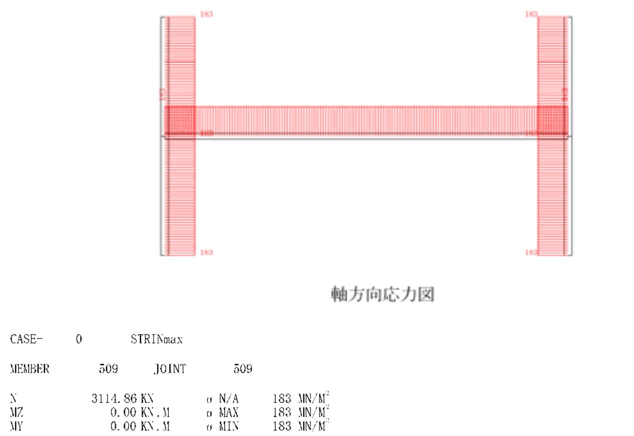
Table 16.2.2-4 Stress Check of Arch Rib

	Distribution of Normal Stresses	Results
BS		N = -25100 kN Mz = 86500 kNm <SM570> Upper flange : 40mm Web : 22mm Lower flange : 40mm σ min = -403Mpa < -450 Mpa When considering resistance factor σ min = -403Mpa < -413 Mpa τ max = 86 Mpa < 240 Mpa

	Distribution of Normal Stresses	Results
3106		N = -27000 kN Mz = 31200 kNm <SM490Y> Upper flange : 35mm Web : 23mm Lower flange : 35mm σ max = -146Mpa < 355 Mpa τ max = 103 Mpa < 178 Mpa
3107		N = -25000 kN Mz = 6650 kNm <SM490Y> Upper flange : 35mm Web : 23mm Lower flange : 35mm σ max = -282Mpa < 355 Mpa τ max = 17 Mpa < 178 Mpa

<Hangers>

Table 16.2.2-5 Stress Check of Hangers

	Distribution of Normal Stresses	Results
509		N = 3114 kN <SM490Y> Upper flange : 10mm Web : 13mm Lower flange : 10mm σ max = 183Mpa < 450 Mpa

(6) Summary

Based on that the following dimensions are obtained as the superstructure of Lambingan bridge

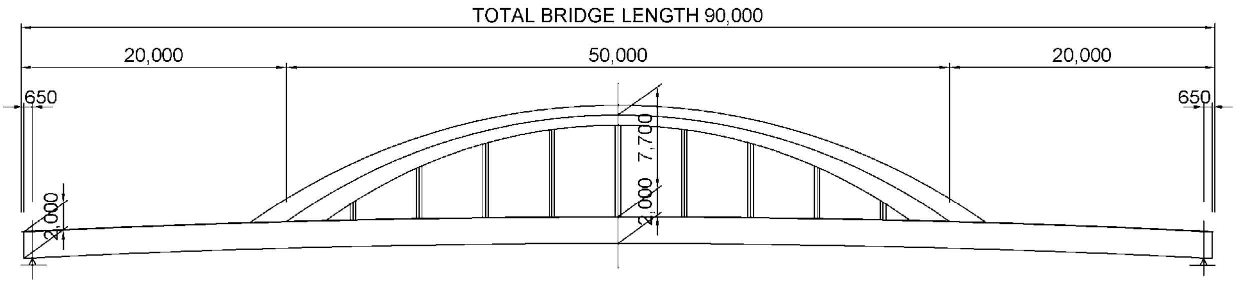


Figure 16.2.2-5 Side View of Superstructure of Lambingan Bridge

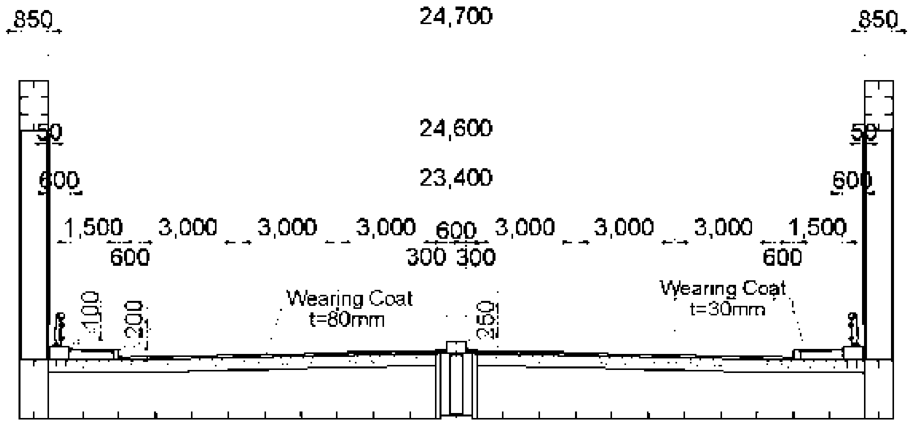


Figure 16.2.2-6 Sectional View of Superstructure of Lambingan Bridge

Table 16.2.2-6 Summary of Calculated Results

Steel Deck	Material	U-Flange (mm)	Web (mm) H=2000mm	L-Flange (mm)
Sec.1	BOX-SM490Y	18	20	18
Sec.2	BOX-SM490Y	21	17	24
Sec.3	BOX-SM490Y	25	14	30
Sec.4	BOX-SM490Y	29	11	33
Sec.5	BOX-SM490Y	29	10	33
Arch Rib	Material	U-Flange (mm)	Web (mm) H=2000mm	L-Flange (mm)
BS	BOX-SM570	40	22	40
106	BOX-SM490Y	35	23	35
107	BOX-SM490Y	35	23	35
108	BOX-SM490Y	35	23	35
109	BOX-SM490Y	35	23	35
110	BOX-SM490Y	35	23	35
Hanger	Material	U-Flange (mm)	Web (mm) H=2000mm	L-Flange (mm)
Min. Thick	I-SM490Y	10	13	10

16.2.3 Seismic Design

In this project, as seismic analysis, modal response spectrum analysis is conducted for seismic design. Based on the response results, various structural members can be determined such as piers, foundations, bearings and expansion joints. Analytical model to be utilized for modal analysis commonly utilizes truss and beam type elements in the world.
Based on the results of the outline design of superstructure such as member dimension and masses, analytical model and results of modal response spectrum analysis are explained in this item.

The analytical model for response spectrum analysis is not 1st-stage structure utilized in outline design of superstructure but final stage structure of the superstructure. The connection between 1st stage and 2nd stage may be joined with bolting connection defined as hinge -connection under live loads and seismic loads. Therefore, such the connection is accurately modeled in the analytical model.

Besides, in this design, abutments are not modeled in the seismic analysis because abutments may have enough strength and stiffness fixed by grounds for seismic vibration; if abutments are modeled in the analysis, excess damping efficiency would be expected to the whole of structural responses.

Additionally, as bearings, forces distribution bearings consisting of natural rubber bearing are applied in order to reduce actual seismic forces affecting structures. The stiffness of the bearing is determined based on cyclic evaluation of horizontal response displacements and period of eigenvalue analysis.

(1) Analytical Model

- Seismic Analysis: Response Spectrum Analysis based on Modal Eigenvalue Analysis
- Superstructure Type: Steel Deck Lohse Arch Stiffening Box Girder
- Bridge Length : L=90m
- Angle of Alignment: 90 Degrees
- Analytical Model :

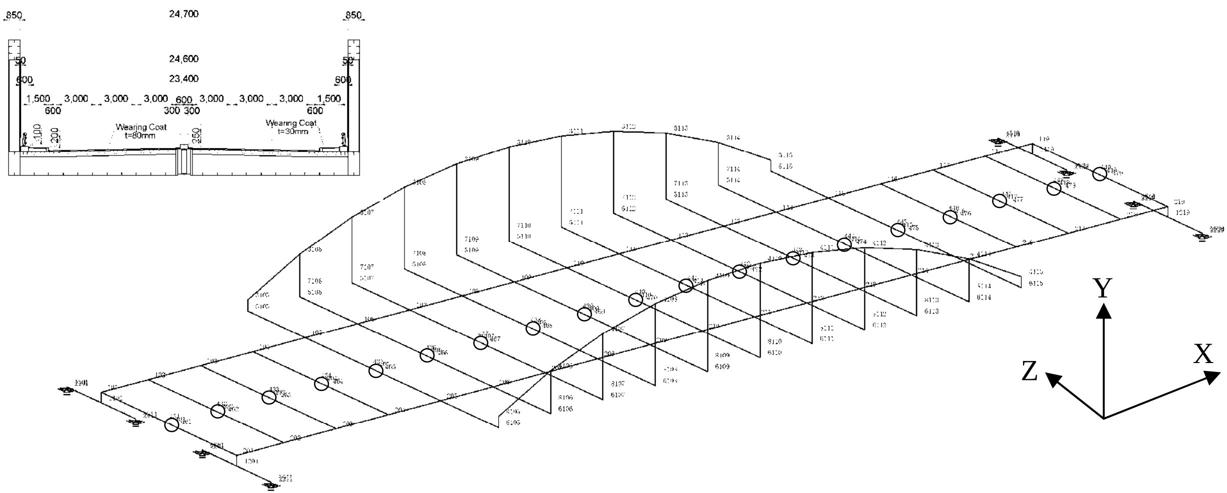


Figure 16.2.3-1 Analytical Mode of Seismic Analysis

Table 16.2.3-1 Support Condition

	X	Y	Z	RX	RY	RZ
Abutment 1	Elastic	Fix	Fix	Fix	Free	Free
Connection	Fix	Fix	Fix	Free	Free	Free
Abutment 2	Elastic	Fix	Fix	Fix	Free	Free

- Abutments : Not Modeled
- Piers : No piers
- Bearing : Following Force Distribution Bearing:

Table 16.2.3-2 Force Distribution Bearing

Supports	Nos.	Dimension	Thickness	G
Abutment 1	4	700mmx700mm	17mmx6layers	1.4 N/mm2
Abutment 2	4	700mmx700mm	17mmx6layers	1.4 N/mm2

- Foundation : Following springs shall be :

Table 16.2.3-3 Springs of Foundations

Foundations	X: Longitudinal kN/m	Z: Transversal kN/m	RX kNm/rad	RZ kNm/rad
Abutment 1	Fix	Fix	Fix	Fix
Abutment 2	Fix	Fix	Fix	Fix

- Damping coefficient :Following damping coefficients are applied:

Table 16.2.3-4 Damping Coefficient

Structural Element	Damping
Steel	0.01
Concrete	0.02
Force Distribution Bearing	0.03
Foundation	0.10

(2) Comparison Studies of Seismic Capacity Improvement Schemes

In order to improve seismic capacity of this bridge, the following methodologies are applied based on technical comparison studies.

< Adequate Bearing Type >

Force distribution method by laminated rubber bearings (LRB) shown in the following figure are commonly utilized in viaducts and bridges in Japan as efficient devices to achieve appropriate seismic design.

Fig. Laminated Rubber Bearing

Fig. Steel Bearing

This bearing consists of rubber and steel plate layers. By changing the stiffness of the laminated rubber, such for thickness, number of layers and sizes, seismic horizontal forces can be freely and evenly shared to substructures. Therefore, the boundary condition between superstructure and substructure is "E" that means "elastic".

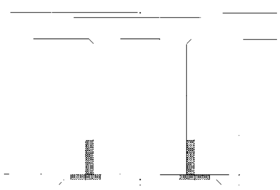
Otherwise, in Philippines, commonly thin-rubber bearing with anchor bars is utilized as bearing. By this bearing, only two ways of the boundary condition such as "Fix" or "Move" can be applied, which means that controlling of horizontal seismic forces or contribution forces to substructures depends on not horizontal stiffness of bearing but just only the period of its dynamic properties.

Otherwise, for steel deck girder bridge like Lambingan bridge, it is appropriate to apply steel type bearing instead of above bearings, which shall resist LV2 seismic forces. Naturally the boundary condition will be two ways such as "FIX" and "MOVE", which can not apply force distribution method. Generally, steel type bearings for viaducts are utilized under following conditions in Japan:

- Light weight superstructure
- No advantage to extend the period of the superstructure
- Slender piers can be maintained without high dumping

In this item, as an improvement scheme, technical comparison study between laminated rubber bearing, thin-rubber bearing with anchor bars and steel bearing is explained from the point of view of seismic behavior, shown as following table.

Table 16.2.3-5 Comparison Study of Bearing in Lambingan Bridge

<table>
<tr><td>Bearing</td><td>Results of Evaluation</td></tr>
<tr><td>Laminated Rubber Bearing
Under Force Distribution Method
</td><td>Boundary Condition:
LD: Elastic (A1), Elastic (A2), TD: Fix (A1), Fix (A2)
Time Period
LD: 1.2S, TD: 0.52s
Modal Dumping of 1st mode
LD: 3%. TD: 1%
Total Horizontal Forces of Superstructure using Modal Dumping
LD: 10000kN of 20800kN
Seismic Force Distribution
LD: A1:A2=1:1, TD: A1:A2=1:1</td></tr>
<tr><td>Pad Rubber Bearing with Dowel
Under Not Force Distribution
</td><td>Boundary Condition:
LD: Fix (A1), Move (A2), TD: Fix (A1), Fix (A2)
Time Period
LD: 0.66S, TD: 0.52s
Modal Dumping of 1st mode
LD: 1%. TD: 1%
Total Horizontal Forces of Superstructure using Modal Dumping
LD: 19800kN of 20800kN
Seismic Force Distribution
LD: A1:A2=1:0, TD: A1:A2=1:1</td></tr>
<tr><td>Steel Bearing
Under Not Force Distribution
</td><td>Boundary Condition:
LD: Fix (A1), Move (A2), TD: Fix (A1), Fix (A2)
Time Period
LD: 0.66S, TD: 0.52s
Modal Dumping of 1st mode
LD: 1%. TD: 1%
Total Horizontal Forces of Superstructure using Modal Dumping
LD: 19800kN of 20800kN
Seismic Force Distribution
LD: A1:A2=1:0, TD: A1:A2=1:1</td></tr>
<tr><td colspan="2">By using LRB, the period of longitudinal 1st mode achieves beyond 1.2s, which is much longer than the structure applying Pad Rubber bearing and Steel bearing.

Therefore, the seismic forces based on design spectrum are significantly reduced by extended period.

And also, dominant deformation of the 1st mode is obviously longitudinal deformation of superstructure, which caused by shearing deformation of the LRB; hence, the modal damping of 1st mode can achieve 3% despite just only 1%, modal damping, which is the damping factor of steel structure, of the structure using Pad Rubber bearing or Steel bearing.

Consequently the total horizontal forces of superstructure using LRB is greatly decreased comparing to the structure using common bearing due to extended period and higher structural mode dumping. Additionally, the seismic forces can be distributed evenly by LRB.
Therefore, structurally, superiority of application of LRB is extremely high.</td></tr>
</table>

* LD: Longitudinal Direction, TD: Transversal Direction

(3) Summary of Seismic Analysis

i) Results of Eigenvalue Analysis

The following figure and table shows the results of eigenvalue analysis.

Table 16.2.3-6 Results of Eigenvalue Analysis

Modes	Frequency (Hz)	Period (s)	Ratio of Effective Mass		Mode Damping
			Longitudinal	Transversal	
1	0.800	1.248	**1.000**	0.000	0.030
2	1.909	0.524	0.000	0.101	0.010
3	1.928	0.519	0.000	0.101	0.010
4	4.943	0.202	0.000	0.101	0.010
5	4.962	0.202	0.000	0.101	0.010
6	5.901	0.169	0.000	**0.675**	0.010
7	9.433	0.106	0.000	0.000	0.010
8	9.581	0.104	0.000	0.004	0.010
9	15.122	0.067	0.000	0.000	0.010
10	15.401	0.065	0.000	0.000	0.010

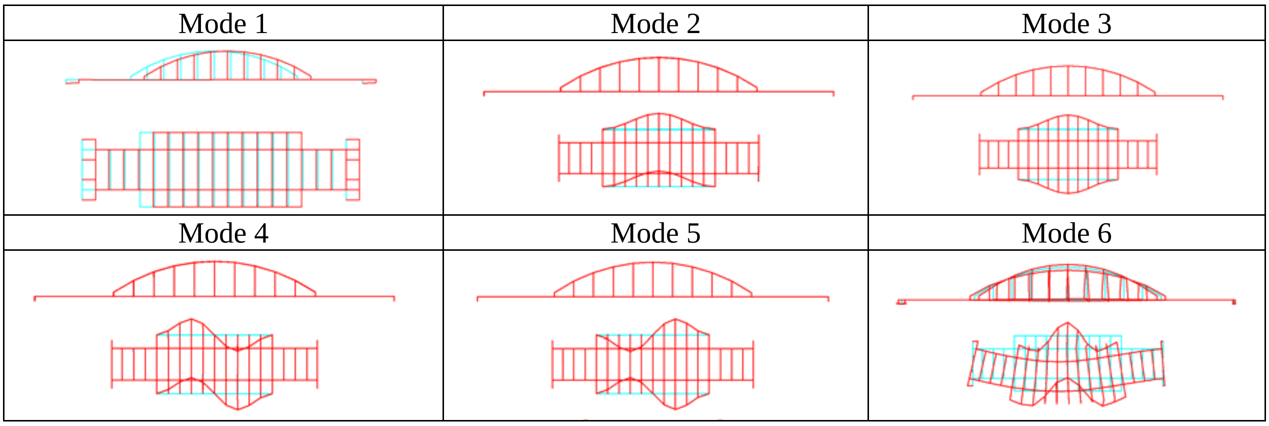


Figure 16.2.3-2 Results of Eigenvalue Analysis

According to the results, predominant mode for longitudinal direction is obviously obtained at 1st mode, in which its period is 1.2s and effective mass ration is 100% of modes for longitudinal direction. Therefore, the period of 1.2s is so important mode.

And the mode damping of the 1st mode is 0.03 that is same to damping coefficient of rubber bearing. The reasons would be definitely understood from the aspect that predominant mode of the 1st mode is caused by mainly the displacements of the rubber bearing. Therefore, in this modal analysis with strain energy proportional method, the mode damping of the 1st mode has been consonant with the damping coefficient of the rubber bearing.

ii) Response Displacement by Response Spectrum Analysis (EQ)

The following table shows the response displacement of relative displacements between substructure and superstructure.

Table 16.2.3-7 Relative Displacement between Substructure and Superstructure

Location	Longitudinal (mm)	Transversal (mm)
Abutment 1	185	0.00
Abutment 2	185	0.00

According to the results, the longitudinal displacements are well converged in realistic scale, for which common expansion joints can be applied.

(4) Seismic Design of Substructure and Foundation

i) Ground Surface in Seismic Design

The following figure shows the ground surface in seismic design.

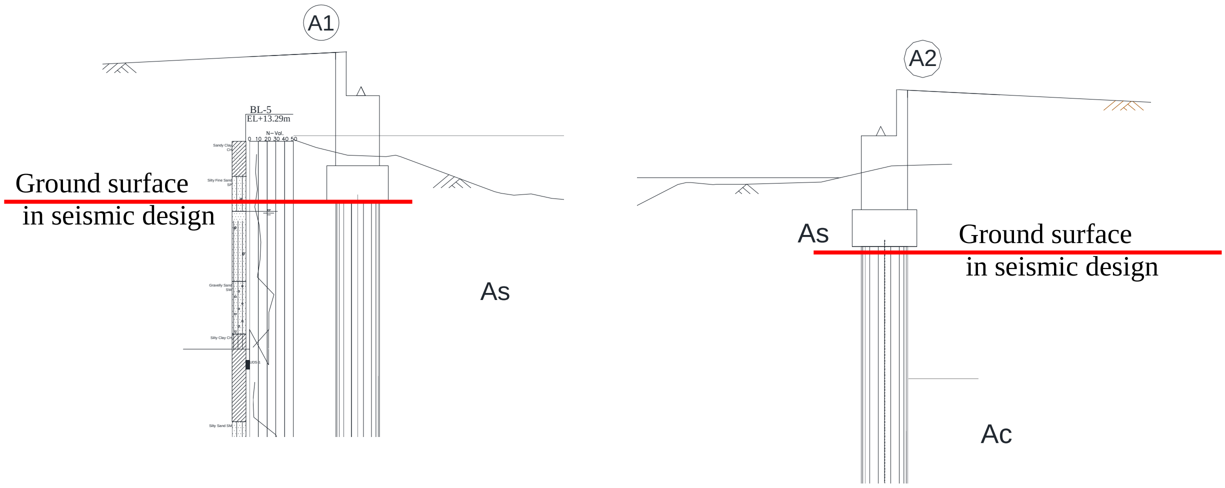


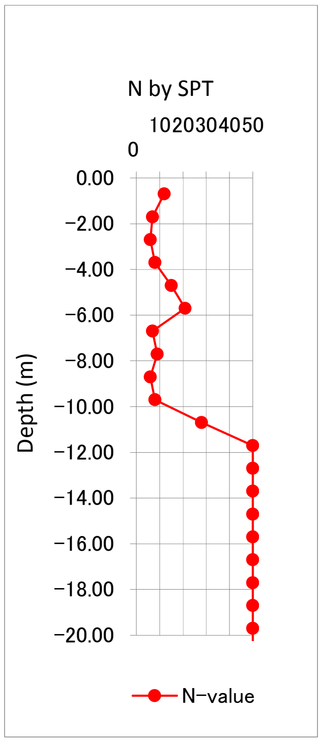
Figure 16.2.3-3 Ground Surface of an Abutment in Seismic Design

ii) Assessment of Soil Liquefaction

According to the design specifications, sandy layer requiring liquefaction Assessment is obviously obtained as following table.

Table 16.2.3-8 Assessment of Soil Liquefaction

Assessment of Liquefaction Potential								
GL-(m)	Soil Layers	N by SPT	Ground Water Level (-m)	Fc (%)	PI	D50 (mm)	D10 (mm)	Liquefiable
		<30		<35%	<15	<10mm	<1mm	
0.70	Bs	12	1.50	0.9		0.74	0.21	○
1.70	As	7	1.50	17.3		0.14		○
2.70	As	6	1.50	28.0		0.12		○
3.70	As	8	1.50	12.0		0.21		○
4.70	As	15	1.50	7.3		0.42	0.12	○
5.70	As	21	1.50	7.3		0.20	0.08	○
6.70	Ac	7	1.50	58.1	12			
7.70	Ac	9	1.50	77.1	33			
8.70	Ac	6	1.50	66.9	19			
9.70	Ac	8	1.50	51.0	45			
10.70	WGF	28	1.50	0.2		2.38	0.43	
11.70	GF	50	1.50	0.5		0.60	0.23	
12.70	GF	50	1.50					
13.70	GF	50	1.50					
14.70	GF	50	1.50					
15.70	GF	50	1.50					
16.70	GF	50	1.50					
17.70	GF	50	1.50					
18.70	GF	50	1.50					
19.70	GF	50	1.50					
20.70	GF	50	1.50					
21.70	GF	50	1.50					
22.70	GF	50	1.50					
23.70	GF	50	1.50					
24.70	GF	50	1.50					



Based on the results of liquefaction assessment, reduction of geotechnical parameters shall be conducted in accordance with the following tables.

Table 16.2.3-9 Assessment of Soil Liquefaction Parameters

Basic Soil Profile Information									
GL-(m)	Soil Layers	N by SPT	Fc (%)	γt γt1	Water unit weight	Ground Water Level (-m)	σU (Kpa)	σv (Kpa)	σv' (Kpa)
0.70	Bs	12	0.9	17	10.00	1.50	0.00	11.90	11.90
1.50	Bs	12	0.9	17	10.00	1.50	0.00	25.50	25.50
1.70	As	7	17.3	18	10.00	1.50	2.00	29.10	27.10
2.70	As	6	28.0	18	10.00	1.50	12.00	47.10	35.10
3.70	As	8	12.0	18	10.00	1.50	22.00	65.10	43.10
4.70	As	15	7.3	18	10.00	1.50	32.00	83.10	51.10
5.70	As	21	7.3	18	10.00	1.50	42.00	101.10	59.10

Table 16.2.3-10 Results on Liquefaction Resistance Factor (FL) & Reduction Factor (DE)

Calculation for FL								Reduction Factor D_E		
Depth	N1	C1	C2	Na	R	L	FL	**R(Ave.)**	**FL(Ave.)**	**D_E**
-0.70	24.91	1.000	0.000	24.908	0.412	0.376	1.097	**0.369**	**0.986**	**1.00**
-1.50	21.36	1.000	0.000	21.361	0.325	0.371	0.876			
-1.70	12.26	1.146	0.406	14.450	0.257	0.398	0.647	**0.327**	**0.652**	**2/3**
-2.70	9.71	1.361	1.002	14.210	0.255	0.489	0.521			
-3.70	12.02	1.040	0.111	12.617	0.240	0.542	0.443			
-4.70	21.06	1.000	0.000	21.057	0.321	0.574	0.559			
-5.70	27.65	1.000	0.000	27.653	0.561	0.594	0.944			

iii) Design Loads

Based on the results of seismic analysis, the abutment design is conducted for the following load combinations.

Results of Eigenvalue Analysis is

VERTICAL REACTIONS FOR SUBSTRUCTURE STABLE CALCULATION (KN)

	Dead load	Live load	Sum.
A1	10,390	4,260	14,650
A2	10,390	4,260	14,650

Note: Impact factor exclusive
Total Forces of Two Bridges (Two Arches)

HORISONTAL/ TRANSVERSAL REACTIONS BY RESPONSE SPECTRUM ANALYSIS UNDER L2 (at bearing)

	Longitudinal Direction		
	H(KN)	M(KNm)	V(KN)
A1 (E)	4,990	-	-
A2 (E)	4,990	-	-

Note: Total Forces of Two Bridges (Two Arches)

- Design Combination Loads

LONGITUDINAL DIRECTION(at bearing)

	DL		LL		EQ			**SUM of LONGITUDINAL**		
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	**N (kN)**	**H (kN)**	**M (kNm)**
A1 (Nmax)	10,390	1.25	4,260	0.50	4,990	-	1.00	**15,120**	**4,990**	-
(Nmin)	10,390	1.25	4,260	0.50	4,990	-	1.00	**15,120**	**4,990**	-
A2 (Nmax)	10,390	0.90	4,260	0.50	4,990	-	1.00	**11,490**	**4,990**	-
(Nmax)	10,390	0.90	4,260	0.50	4,990	-	1.00	**11,490**	**4,990**	-

iv) Design Result
Based on that the following dimensions are obtained as the abutment with foundation of Lambingan bridge.

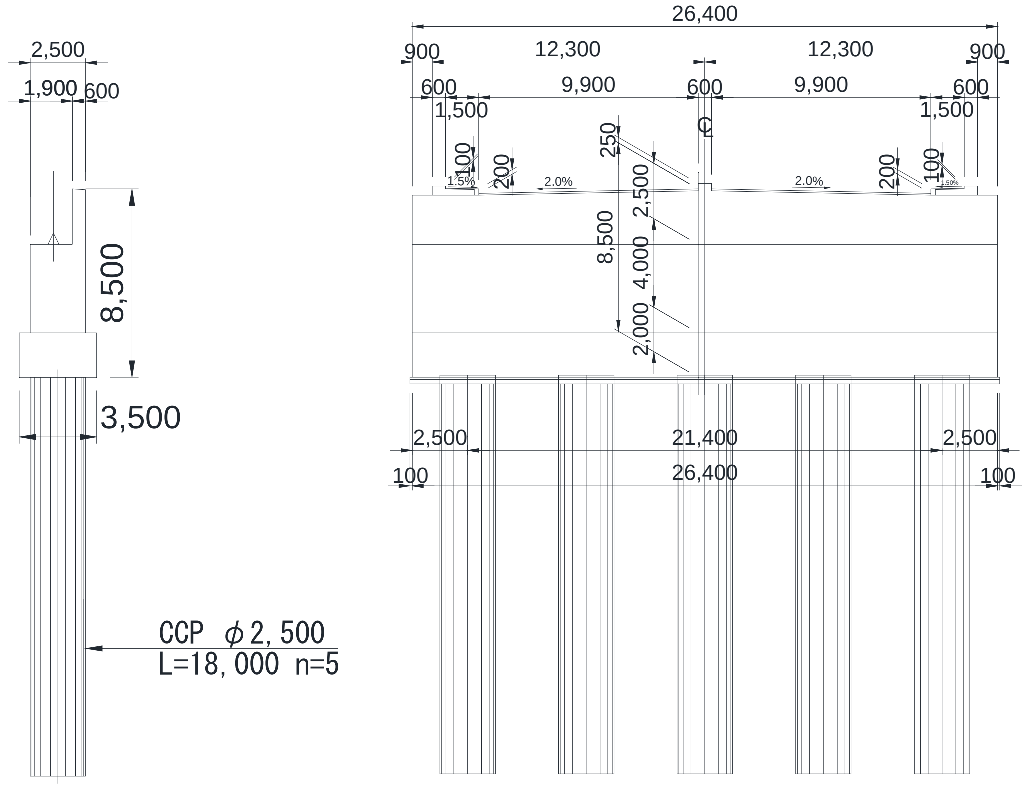


Figure 16.2.3-4 Side View & Sectional View of Abutment of Lambingan Bridge

(5) Unseating Prevention System

The superstructure is generally connected to the substructure through bearings. As such, the superstructure and the substructure are separated functionally and significantly critical state such as bridge falling down may be caused due to large relative displacements between them, in case of failure of bearings under unexpected seismic forces.

For a functional system preventing such severe state, detailed philosophy and articulate design concepts are explicitly specified in JRA as "Unseating Prevention System" based on accumulated data and experiences from large number of seismic damages. The aim is to provide multiple mechanisms that can complement each other efficiently, shown as follows.

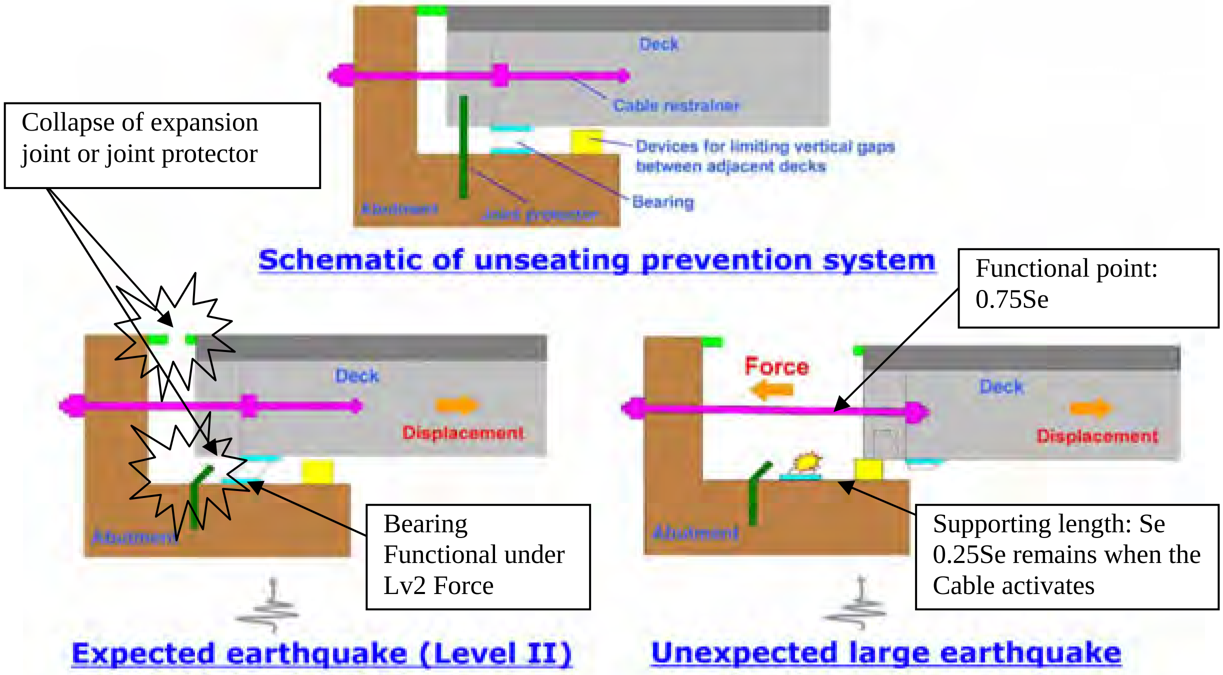


Figure 16.2.3-5 Philosophy of Unseating Prevention System in JRA

The Unseating Prevention System consists as following devices or functions:

Table 16.2.3-11 Devices and Functions of Unseating Prevention System

Devices	Function
Bearing (Type B)	"Type B" bearing in JRA, enables to resist the seismic forces of LV2 by itself.
Supporting Length	The final function of the unseating prevention system. The equation to determine the length is given as follows: Se=μr+μg : μr: Relative responses of girder, μg: Displacement of ground Minimum length: Semin=0.7+0.005l (m)
Longitudinal Restrainer	Design forces are given as 1.5Rd, where Rd is the reaction force of dead loads Maximum movable length: 0.75Se, where Se is supporting length. Generally for new bridges in Japan, cable type devices connecting between parapet of abutment and cross beam of superstructure are commonly utilized.
Expansion Joint	Expansion joint shall resist LV1 seismic forces, otherwise, the capability to resist LV2 seismic forces is not required.

Based on the philosophy and functions above, specification of each devices applied in this new bridge is shown as the results of outline design.

i) Bearing
For Lambingan bridge, following force distributing bearing is obviously advantageous for seismic behavior.

Table 16.2.3-12 Force Distribution Bearing

Supports	Nos.	Dimension	Thickness	G
Abutment 1	4	700mmx700mm	17mmx6layers	1.4 N/mm2
Abutment 2	4	700mmx700mm	17mmx6layers	1.4 N/mm2

From the point of view of the philosophy of unseating prevention system, those bearing shall be resist the LV2 seismic forces. As a part of outline design, following outline verification is conducted to clarify they can resist such the forces.

Table 16.2.3-13 Outline Verification of Bearing under LV2 Seismic Forces

		Value/ LV2	Allowance	Judge
Shear Strain	Longitudinal Dir.	1.8	2.5	OK
	Transversal Dir.	0.0	2.5	OK

ii) Supporting Length

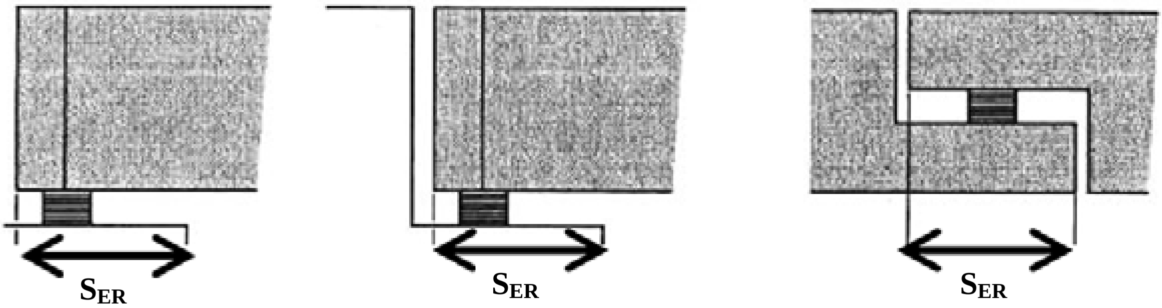


Figure 16.2.3-6 Supporting Length

Following equation gives the supporting length.
Se=0.7+0.005l (m) l: Span length
Se = 0.7+0.005*90 = 1.15m

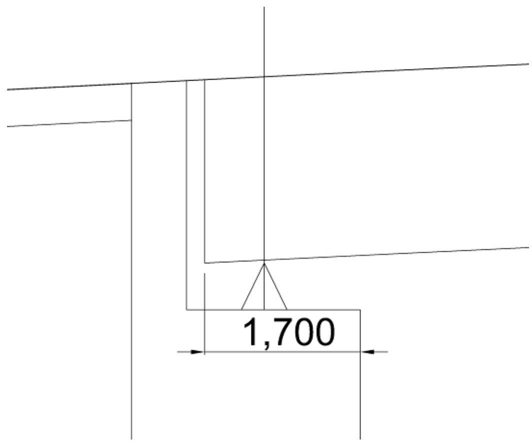


Figure 16.2.3-7 Secure the Length of "Se", Supporting Length

iii) Longitudinal Restrainer
The reaction forces by dead loads are 10390kN.
The following verification can be obtained.

Table 16.2.3-14 Verification of Longitudinal Restrainer

	1.5Rd: Design Forces	Allowance
PC Cable Type 19 x ϕ12.7mm 6-nos/ Abutment	2598 kN	2964 kN

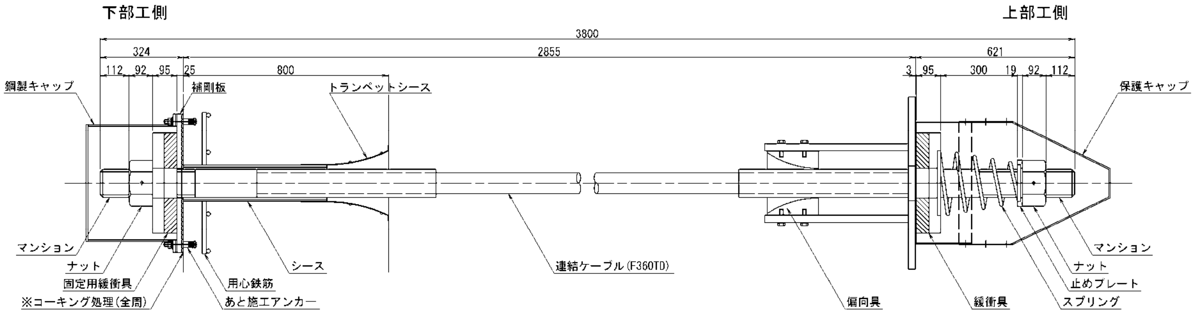


Figure 16.2.3-8 Longitudinal Restrainer for Lambingan Bridge

(6) Miscellaneous devices and others

Miscellaneous devices in the bridge are defined as following items:

- Bearing: Evaluated above clause
- Expansion joint
- Drainage
- Wearing coat

In this clause, the devices which are not explained in other clause are explained based on seismic behaviors and current bridge condition.

i) Expansion joint

For the design methodology of expansion joint, its seismic capacity shall be secured under LV1 forces and it does not have to be secured under LV2 forces. The expansion gap between girder end and abutment shall be determined from results of dynamic modal analysis under LV2 and another expansion gap of expansion joint itself shall be determined based on seismic analysis of LV1.

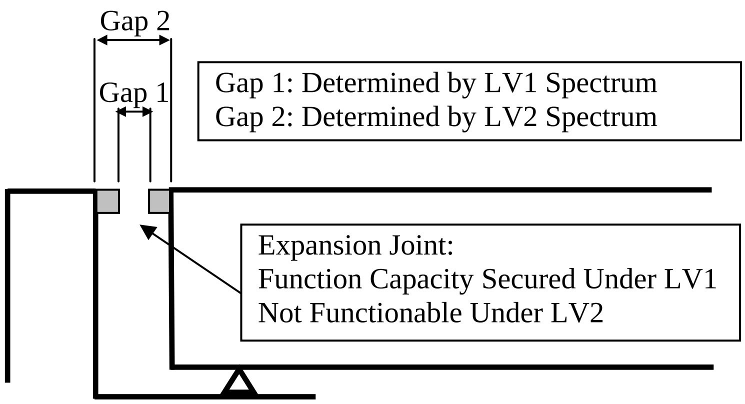


Figure 16.2.3-9 Design Methodology of Expansion Joint

However, expansion gap using laminated rubber bearing generally tends to be larger than common bearing and the size of the expansion joint tends to be larger and more expensive. Therefore, the both of expansion gap especially the gap between girder end and abutment should be carefully pay attention to displacement controlling during dynamic modal analysis, evaluating the size of expansion joint. When the gaps were so large comparing to general behavior, the stiffness of rubber bearing should be adjusted and should try the modal analysis again.

In this project, on the basis of above consideration, appropriate modal analysis are carried out, controlling caused displacements based on evaluation of stiffness of rubber bearing.
The final displacements to be used for determination of expansion joint are as follows.

- LV1: Gap 1: 10.5cm + 1.5cm (Excess allowance 15mm (JRA) ≒ 12cm
- LV2: Gap 2: 18.5cm + 1.5cm (Excess allowance 15mm (JRA) ≒ 20cm

Therefore, the expandable gap of the joint in this bridge shall be 12cm or more, and the gap between girder and abutment shall be 20cm or more, which are common results achieved under careful controlling in the dynamic analysis. Consequently, general steel type expansion joint can be adequately applied to this bridge.

ii) Drainage
Drainage system on the bridge is estimated based on current condition. In the next stage such as basic design or design stage, appropriate location of catch basins and drainage pipes shall be designed and drawn based on further investigation of accumulated rainfall data of corresponding area.

iii) Wearing coat
Lambingan bridge will consist of steel deck. The steel deck is definitely flexible member which may causes fracture and crack of wearing coat if the selection of the wearing coat engineeringly mistakes.
In Japan, we have a lot of steel deck bridges, in which generally utilized following wearing coat system on the steel deck bridge, consisting of two of layers, in order to appropriately follow the deformation of steel deck.

In detail design stage, comparison study of bridge wearing coat based on costs and structure as well as maintenance such as guss or epoxy asphalt, which are of Japan's advanced products, suitable for flexible steel deck should be conducted. And also, in Japan, as asphalt concrete, polymer modified asphalt is usually applied to wearing coat on bridges, which is excellent at flowability, flexibility, durability, rutting resistance and heat resistance.

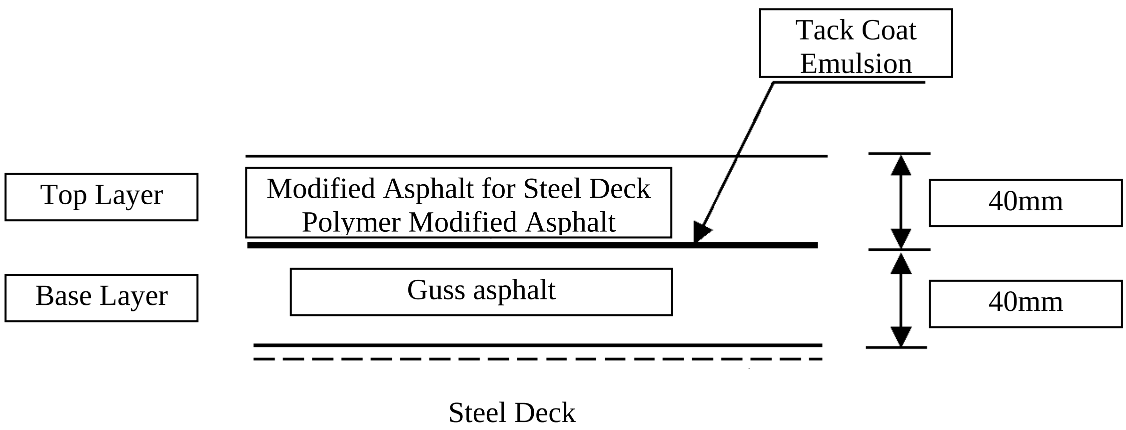


Figure 16.2.3-10 Wearing Coat System of Steel Deck

16.2.4 Summary of Outline Design Results

(1) Superstructure

Superstructure is designed based on AASHTO LRFD for the bridge type determined in multiple comparison study in consideration of various conditions. The bridge type is Steel Deck Lohse Arch Stiffening Box Girder. And laminated rubber bearing considering 3% of damping coefficient in dynamic modal analysis is applied in consideration of seismic behavior calculated dynamic modal analysis.

(2) Substructure and Foundation

Based on that the following dimensions are obtained as the abutment with foundation of Lambingan bridge.

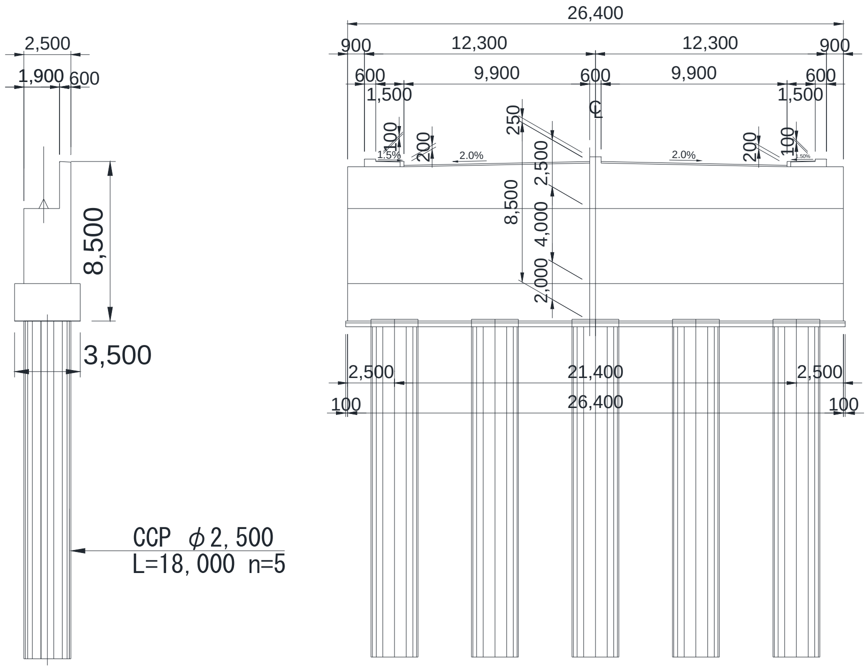


Figure 16.2.4-1 Side View & Sectional View of Abutment of Lambingan Bridge

(3) Further Verification to be Examined in the Next Phase

The following items may be necessary to be verified or evaluated further in the next phase such as basic or detail design stages.

- Optimization and re-verification of bridge length, span arrangement and bridge types, on the basis of latest existing road condition, newly future planning and detail river condition resulted by detail hydraulic analysis
- Applicability of utilization of high-damping bearing based on specific organization regarding non-linear time history response analysis based upon comparison study regarding bearing system
- Comparison study of bridge wearing coat based on costs and structure as well as maintenance such as guss or epoxy asphalt, which are of Japan's advanced products, suitable for flexible steel deck should be conducted. And also, in Japan, as asphalt concrete, polymer modified asphalt is usually applied to wearing coat on bridges, which is excellent at flowability, flexibility, durability, rutting resistance and heat resistance.

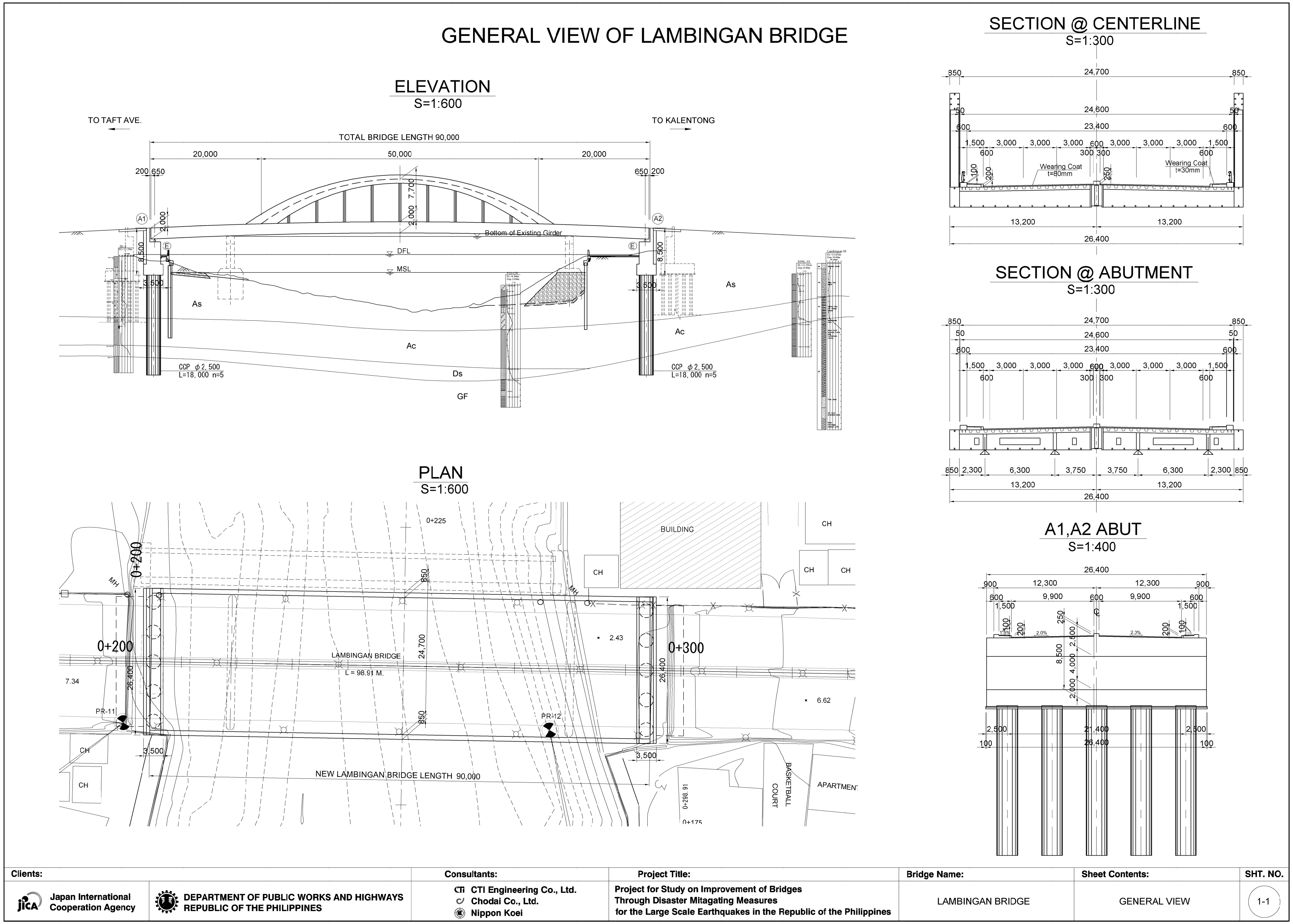


Figure 16.2.4-2 General View

(Structural Drawings are shown in Appendix-4)

16.3 Outline Design of Guadalupe Outer Side Bridge

16.3.1 Design Condition

The following items show design condition for the outline design of Guadalupe Bridge.

(1) Road Conditions

- Road Design Standard : AASHTO STANDARD VALUE
- Design Speed : V = 60 kmph
- Live Loads : AASHTO Live Loads HL93 and Lane Loads
- Road Width : Shown as follows:

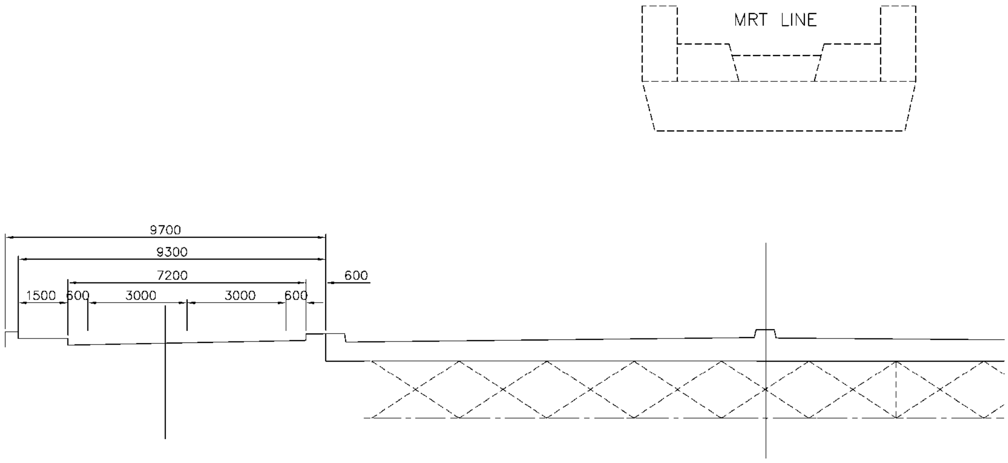


Figure 16.3.1-1 Cross Section/ Lane Arrangement of Guadalupe Bridge

(2) Soil Conditions

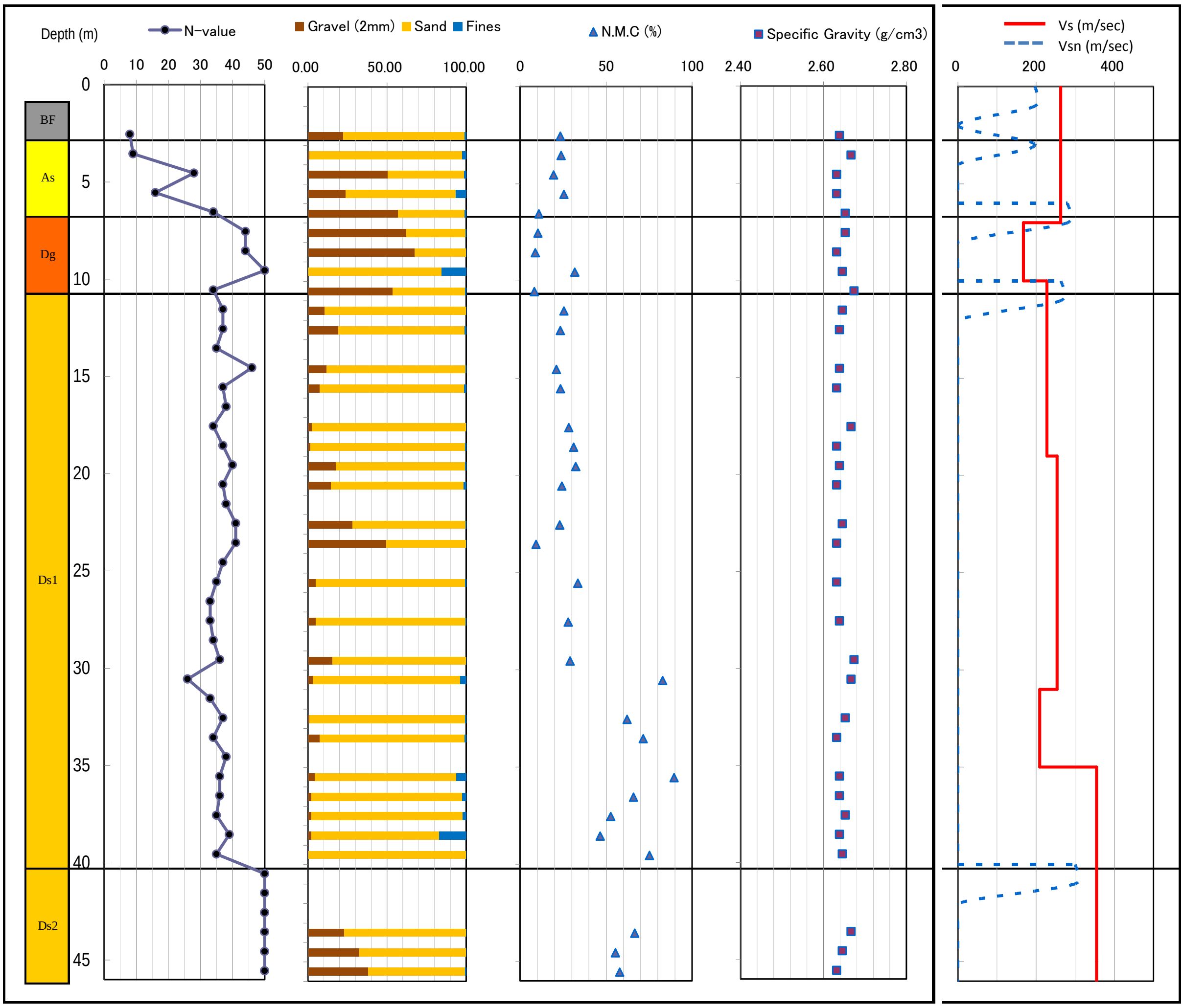
The results of ground investigation are shown in below illustrations and following tables. The sand with gravel layer that can be regarded as the bearing layer is distributed G.L. -15.0m to G.L.-30.0m depth, and has a thick surface layer predominant with sand on top. Specialty, liquefiable sand (AS) is thickly deposited from ground surface to GL-5.0m, of which N-value is 8 to 28, will be affected by liquefaction occurs with reduction of geotechnical parameter. .

Table 16.3.1-1 Summary for Soil Parameters (1)

Input by Tanaka - A2 Side - Guadalupe B1 E.L 4.3m

Number	Depth Upper	SPT Blows	S-wave	S-wave ave.	Vsn	Soil Layer	Soil Classification Observation	GSA-J Gravel (%)	GSA-J Sand (%)	GSA-J Fines (%)	N.M.C (%)	PI	Specific Gravity	Nd	γt (tf/m^2)	C (kN/m^2)	φ (°)	E0 (kN/m^2)	Vsn (m/sec)	Layer Thickness
1	0.55		263	263	197	BF	Fill soils							15	18	-	-	-	197	2
2	1.55		263				Fill soils								18					
3	2.55	8	263	263	197	As	Sand w/ gravel	22.2	76.7	1.1	23.4	N/A	2.64	15	17	0	35	10,500	197	4
4	3.55	9	263				Co. to med. sand	0.9	96.5	2.6	23.9	N/A	2.67		17					
5	4.55	28	263				Gravel /w sand	50.3	48.4	1.3	19.5	N/A	2.63		17					
6	5.55	16	263				Clayey gravel w/ sand	23.7	69.6	6.7	25.4	N/A	2.63		17					
7	6.55	34	263	192	280	Dg	Sand and gravel	57.0	41.8	1.2	10.9	N/A	2.65	43	18	0	39	30,100	280	4
8	7.55	44	168				Gravel /w sand	61.9	37.7	0.4	10.4	N/A	2.65		18					
9	8.55	44	168				Gravel	67.4	32.4	0.2	8.8	N/A	2.63		18					
10	9.55	50	168				Clayey gravel w/ sand	0.0	84.4	15.6	31.8	N/A	2.65		18					
11	10.55	34	228	257	264	Ds1	Co. sand w/ gravel	53.5	45.8	0.7	8.3	N/A	2.67	36	17	0	36	25,200	264	30
12	11.55	37	228				Med./fine sand w/ gravel	10.6	89.2	0.2	25.4	N/A	2.65		17					
13	12.55	37	228				Med./co. sand	19.1	79.8	1.1	23.4	N/A	2.64		17					
14	13.55	35	228												17					
15	14.55	46	228				Med./co. sand	11.9	87.6	0.5	21.0	N/A	2.64		17					
16	15.55	37	228				med. sand	7.4	91.3	1.3	23.5	N/A	2.63		17					
17	16.55	38	228												17					
18	17.55	34	228				med. sand	2.5	97.3	0.2	28.3	N/A	2.67		17					
19	18.55	37	228				med. sand	1.6	97.7	0.7	31.1	N/A	2.63		17					
20	19.55	40	254				Coarse sand	17.5	81.8	0.7	32.4	N/A	2.64		17					
21	20.55	37	254				Med./fine sand	14.5	83.8	1.8	24.2	N/A	2.63		17					
22	21.55	38	254												17					
23	22.55	41	254				Sand w/ gravel	28.1	71.4	0.5	23.0	N/A	2.65		17					
24	23.55	41	254				Med./fine sand w/ gravel	49.5	50.3	0.2	9.3	N/A	2.63		17					
25	24.55	37	254												17					
26	25.55	35	254				Fine to med. sand	4.9	94.3	0.8	33.5	N/A	2.63		17					
27	26.55	33	254												17					
28	27.55	33	254				Sand	5.0	94.5	0.5	27.9	N/A	2.64		17					
29	28.55	34	254												17					
30	29.55	36	254				Med. to fine sand	15.6	84.2	0.2	29.1	N/A	2.67		17					
31	30.55	26	254				Med. to fine sand	3.1	93.0	3.9	82.8	N/A	2.67		17					
32	31.55	33	209				Sand								17					
33	32.55	37	209				Med. to fine sand	1.1	98.0	0.9	62.2	N/A	2.65		17					
34	33.55	34	209				Med. to fine sand	7.5	91.3	1.2	71.6	N/A	2.63		17					
35	34.55	38	209				Sand								17					
36	35.55	36	355				Fine sand	4.4	89.5	6.2	89.5	N/A	2.64		17					
37	36.55	36	355				Fine sand	2.0	95.3	2.7	65.9	N/A	2.64		17					
38	37.55	35	355				Med to fine sand	2.0	95.7	2.3	52.7	N/A	2.65		17					
39	38.55	39	355				Fine to med. sand	2.2	80.5	17.3	46.5	N/A	2.64		17					
40	39.55	35	355				Sand	0.5	99.3	0.2	75.2	N/A	2.65		17					
41	40.55	50	355	355	300	Ds2	Sand							222	19	0	35	155,128	300	6
42	41.55	50	355				Sand								19					
43	42.55	50	355				Sand								19					
44	43.55	50	355				Med to fine sand	22.8	77.0	0.2	66.7	N/A	2.67		19					
45	44.55	50	355				Med to fine sand	32.4	67.4	0.2	55.4	N/A	2.65		19					
46	45.55	50	355				Med to fine sand	37.9	61.4	0.7	58.0	N/A	2.63		19					

Table 16.3.1-2 Summary for Soil Parameters (2)



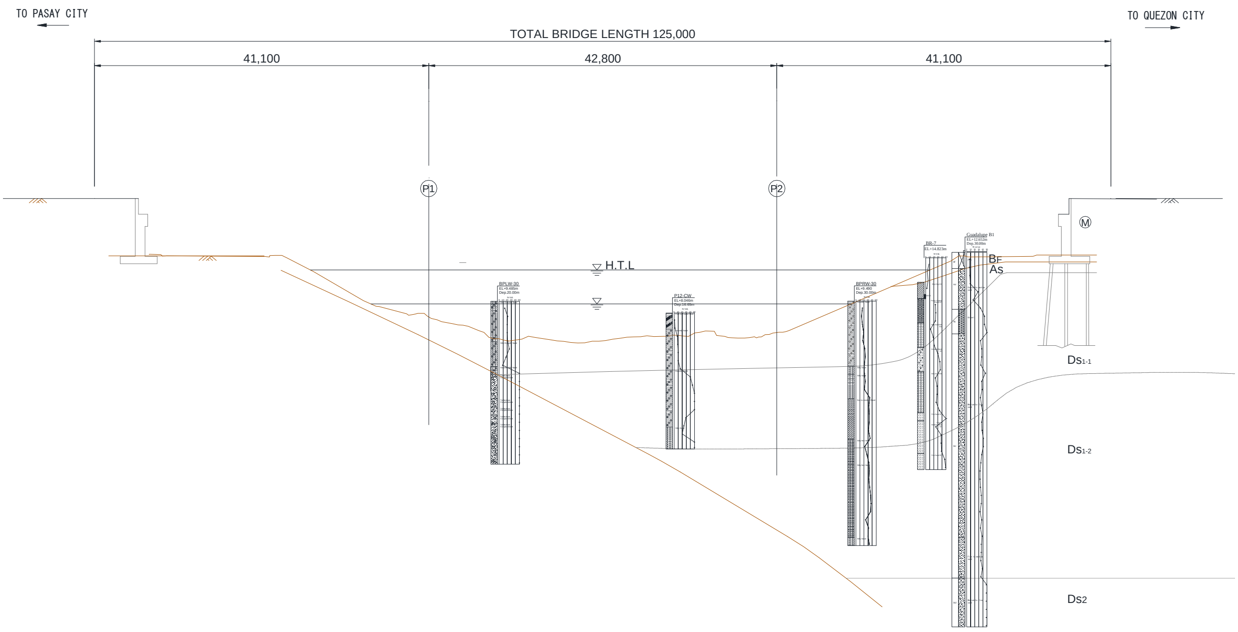


Figure 16.3.1-2 Soil Profile of Guadalupe Bridge (Included previous SPT)

(3) Hydraulic Conditions

- Design Water Level : EL= 1.48m
- Freeboard from Design Flood Level : H = 3.75m (To secure existing freeboard)

Hydraulic condition shall be carefully verified and examined by detail hydraulic analysis in the detail design stage, based on which the road and bridge planning shall be reevaluated in such the phase including comparison study of bridge types.

(4) Bridge Type

- Superstructure Type : Steel Deck Box Girder
- Bridge Length : L=125m
- Span Arrangement : 41.1m + 42.8m + 41.1
- Horizontal Alignment : R=∞
- Angle of Alignment : 90 Degrees
- Wearing coat : Guss asphalt and Polymer Modified Asphalt t=80mm for Vehicle lane, t=30mm for walkway
- Railing : Steel railing for vehicle and pedestrian
- Bearing : Steel Bearings
- Expansion Joint : Steel type
- Drainage Appliances : PVC pipe
- Bridge Falling Prevention Device: Cable type

- Substructure Type : RC wall type
- Foundation Type : Steel Pipe Sheet Pile Foundation
- Bearing Soil Condition : Clay with Gravel Layer (N>45)

(5) Design Cases of Outline Design

The outline design of superstructure shall be designed based on the above load condition, specified in AASHTO 2012. On the basis of various reactions and forces, substructures and foundation shall be designed throughout response spectrum analysis under the limit state of "Extreme Event I" specified in AASHTO 2012.

STEP 1. Outline Design of Superstructure
- Design Load, specified in AASHTO 2012
- Stress check

↓

STEP 2. Organization of Reaction Forces
- Organization of reaction forces for substructure design
- Organization of masses for response spectrum analysis

↓

STEP 3. Modal Analysis and Response Spectrum Analysis
- Eigenvalue analysis based on modal analysis
- Organization of response values by the seismic analysis

↓

STEP 4. Substructure and Foundation Design
- Extreme Event I

Figure 16.3.1-3 Flow of Outline Design

16.3.2 Outline Design of Superstructure

(1) Design Condition

- Superstructure Type : Steel Deck Box Girder
- Bridge Length : L=125m
- Angle of Alignment : 90 Degrees
- Wearing coat : Guss asphalt and Polymer Modified Asphalt
  t=80mm for Vehicle lane, t=30mm for walkway
- Railing : Steel railing for vehicle and pedestrian
- Bearing : Steel Bearings
- Road Width : Shown as follows:

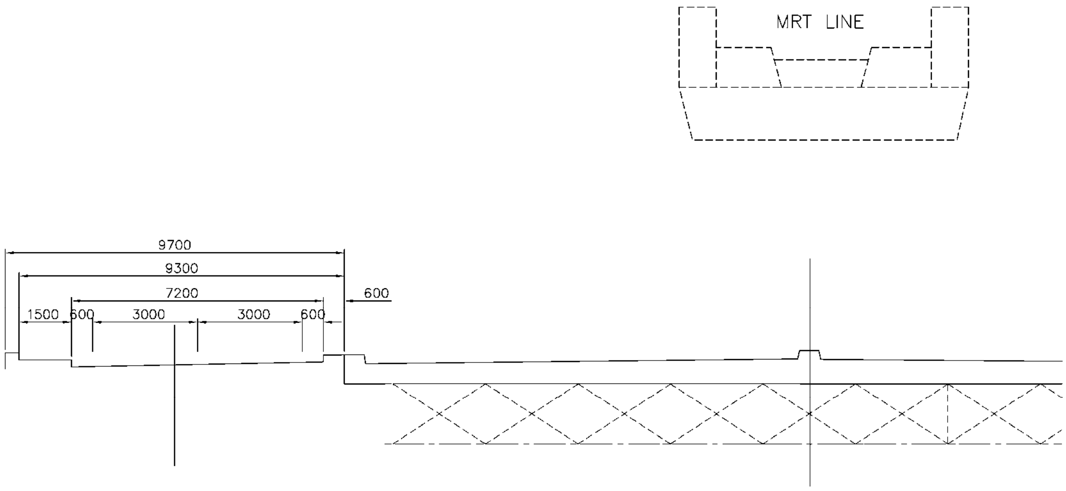


Figure 16.3.2-1 Cross Section/ Lane Arrangement of Guadalupe Side Bridge

(2) Design Loads

- Dead Loads : AASHTO 2012
- Live Loads : HL93 and Lane Loads in AASHTO 2012, utilized by influence line evaluation
- Limit State and Load Combination : Strength I in AASHTO 2012

Table 16.3.2-1 Load Combinations and Factors at Strength I in AASHTO 2012

Load Combination / Limit State	DC DD DW EH EV ES EL PS CR	LL IM CE BR PL LS	WA	WS	WL	FR	TU	TG	SE	Use One of These At a Time: EQ	CT	CV
Strength I	γ_p	1.75	1.00	-	-	1.00	0.5/1.2	γ_{tg}	γ_{se}	-	-	-

Source: LRFD 2012

(3) Analytical model

In the outline design, the following figure shows the analytical model for outline design of Guadalupe Side bridge. All elements in the analysis are beam element model which have 6 of DOFs

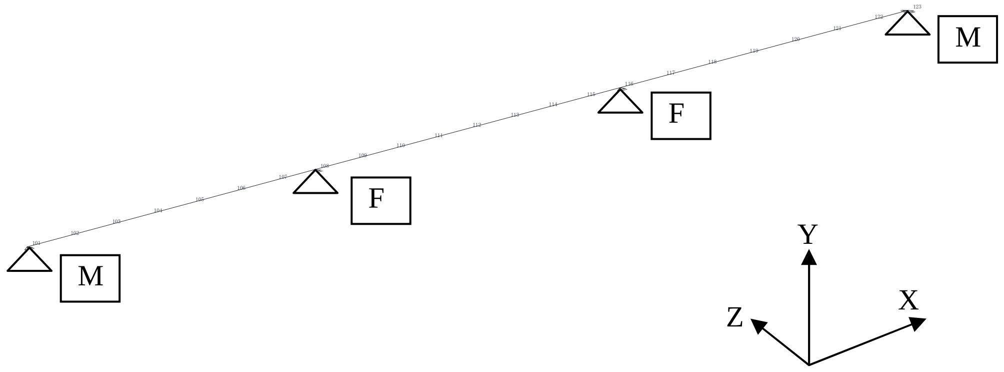
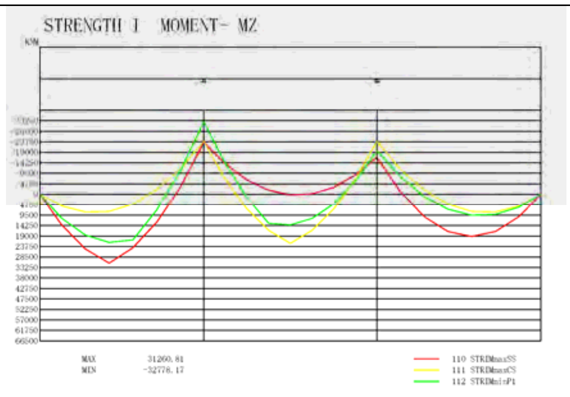
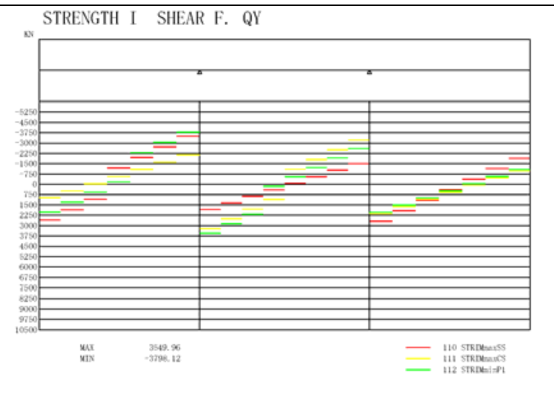


Figure 16.3.2-2 Analytical Model for Superstructure

(4) Sectional forces under Load Combination Strength I

Based on the analytical model, various sectional forces to be utilized for outline design can be obtained. In this report, two of figures regarding distribution of bending moments and shear forces under the combination sectional forces of "Strength I" in the following figures.

Table 16.3.2-2 Distribution of Sectional Forces under Combination of Strength I

Bending Moment in the Steel Deck "Strength I" Red: Focus on Side Span Yellow: Focus on Center Span Green: Focus on Mmin at P1	Shear Forces in the Arch Rib "Strength I" Red: Focus on Side Span Yellow: Focus on Center Span Green: Focus on Mmin at P
	
The maximum bending moment is about 32000KNm in the side span, 24000KNM in the center span and the minimum bending moment is about -33000KNm on the Piers. These values are quite common results for such the superstructure type.	The distribution of shear forces are between -3800kN to 3500kN, which are common results for such the superstructure.

(5) Stress Check

Based on the sectional forces of the load combination Strength I, stress checks are conducted for the following sections of the superstructure. In this report, the results of the sectional stress check are explained for 3 sections shown as in the following figures.

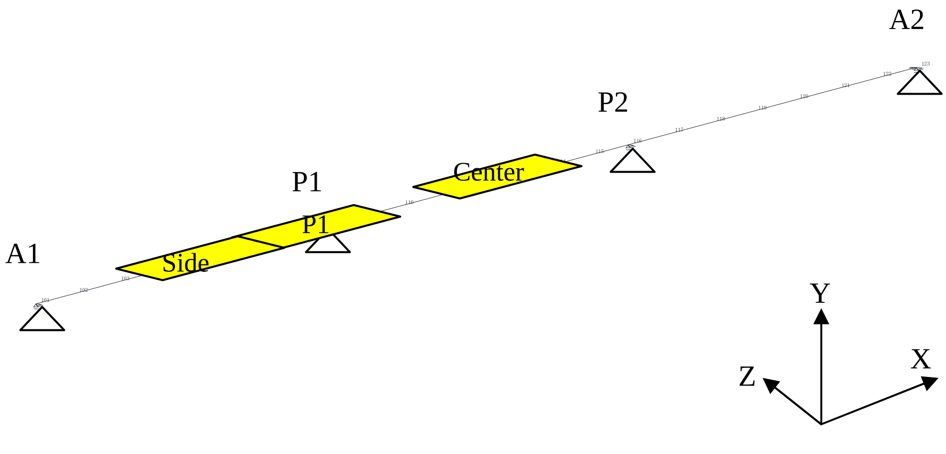
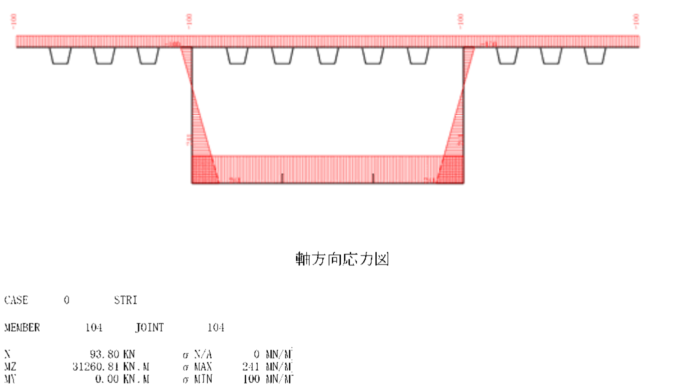
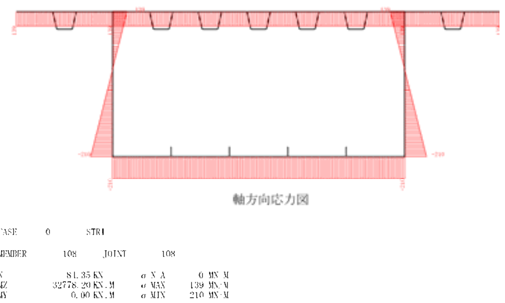


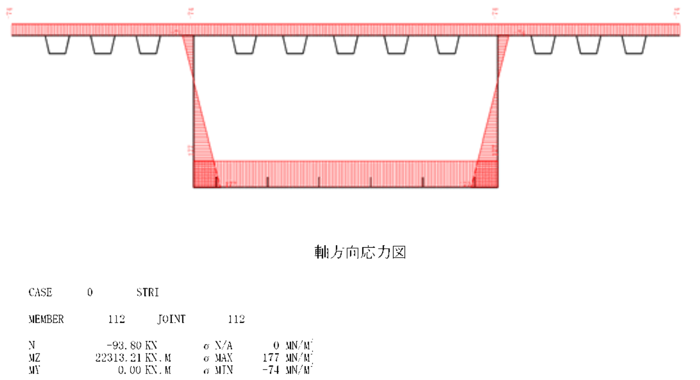
Figure 16.3.2-3 Sections for Stress Check

The results of stress checks are shown as follows.

<Bending Moment>

Table 16.3.2-3 Stress Check of Steel Deck for Bending Moment

	Distribution of Normal Stresses	Results
Side		Mz = 32000kNm <SM490Y> Upper flange : 14mm Web : 10mm Lower flange : 13mm σ max = 241Mpa < 355 Mpa
P1		Mz = -33000kNm <SM490Y> Upper flange : 14mm Web : 10mm Lower flange : 15mm σ max = -210Mpa < 355 Mpa

	Distribution of Normal Stresses	Results
Center		Mz = 23000 kNm <SM490Y> Upper flange : 14mm Web : 10mm Lower flange : 11mm σ max = 177Mpa < 355 Mpa

(6) Summary

Based on that the following dimensions are obtained as the superstructure of Guadalupe Side bridge

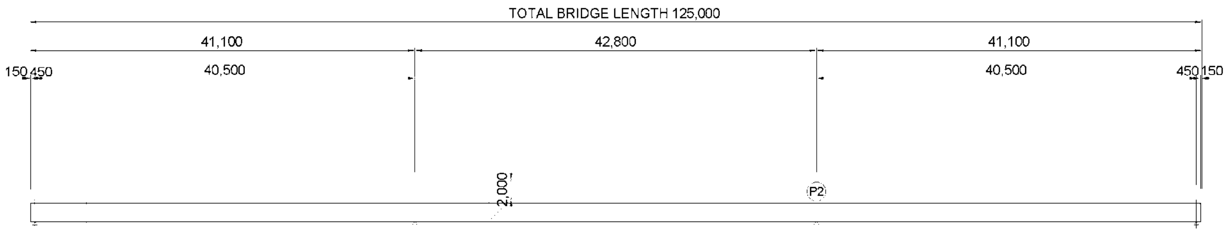


Figure 16.3.2-4 Side View of Superstructure of Guadalupe Side Bridge

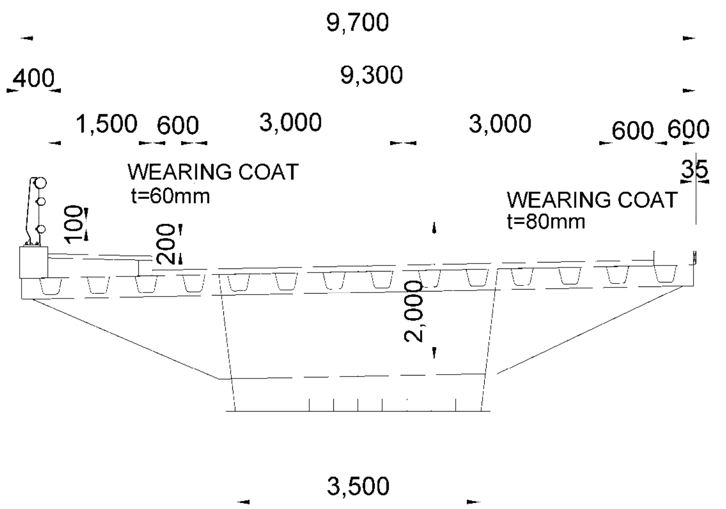


Figure 16.3.2-5 Sectional View of Superstructure of Guadalupe Side Bridge

Table 16.3.2-4 Summary of Calculated Results

Steel Deck	Material	U-Flange (mm)	Web (mm) H=2000mm	L-Flange (mm)
Sec.1 (Sec.11)	BOX-SM490Y	14	10	13
Sec.2 (Sec.10)	BOX-SM490Y	14	10	13
Sec.3 (Sec.9)	BOX-SM490Y	14	10	11
Sec.4 (Sec.8)	BOX-SM490Y	14	10	15
Sec.5 (Sec.7)	BOX-SM490Y	14	10	11
Sec.6	BOX-SM490Y	14	10	11

16.3.3 Seismic Design

In this project, as seismic analysis, modal response spectrum analysis is conducted for seismic design. Based on the response results, various structural members can be determined such as piers, foundations, bearings and expansion joints. Analytical model to be utilized for modal analysis commonly utilizes truss and beam type elements in the world.
Based on the results of the outline design of superstructure such as member dimension and masses, analytical model and results of modal response spectrum analysis are explained in this item.

Besides, in this design, abutments are not modeled in the seismic analysis because abutments may have enough strength and stiffness fixed by grounds for seismic vibration; if abutments are modeled in the analysis, excess damping efficiency would be expected to the whole of structural responses.

(1) Analytical Model

- Seismic Analysis: Response Spectrum Analysis based on Modal Eigenvalue Analysis
- Superstructure Type: Steel Deck Box Girder
- Bridge Length : L=125m
- Angle of Alignment: 90 Degrees
- Analytical Model :

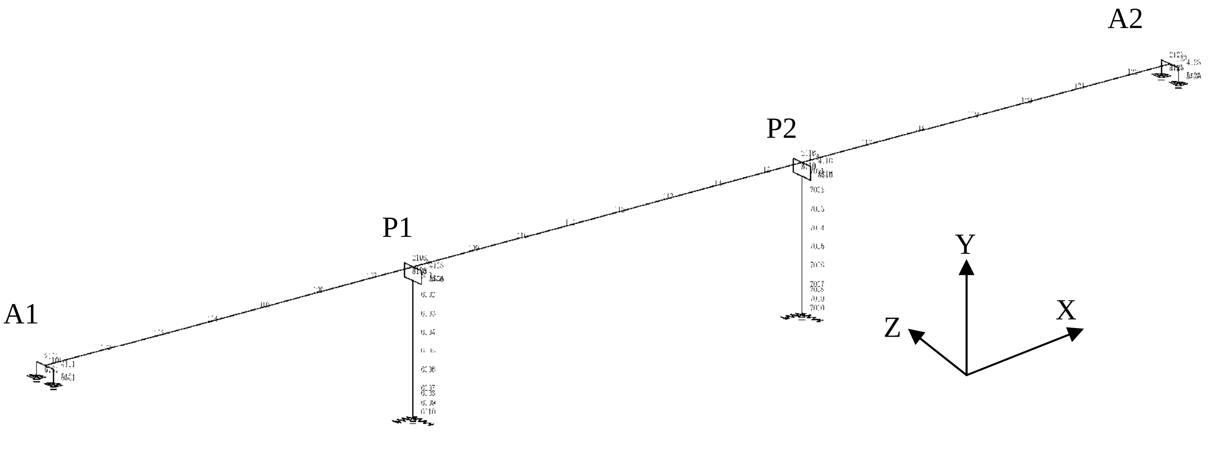


Figure 16.3.3-1 Analytical Mode of Seismic Analysis

Table 16.3.3-1 Support Condition

	X	Y	Z	RX	RY	RZ
Abutment 1	Move	Fix	Fix	Fix	Free	Free
Pier 1	Fix	Fix	Fix	Fix	Free	Free
Pier 2	Fix	Fix	Fix	Fix	Free	Free
Abutment 2	Move	Fix	Fix	Fix	Free	Free

- ➢ Abutments : Not Modeled
- ➢ Piers : Beam Type Elements for Wall Type Piers
- ➢ Bearing : Steel Bearing
- ➢ Foundation : Following springs shall be :

Table 16.3.3-2 Springs of Foundations

Foundations	X: Longitudinal kN/m	Z: Transversal kN/m	RX kNm/rad	RZ kNm/rad
Pier 1	1.06×10^7	1.02×10^7	4.37×10^8	4.99×10^8
Pier 2	7.37×10^6	7.09×10^6	3.75×10^8	4.28×10^8

- ➢ Damping coefficient :Following damping coefficients are applied:

Table 16.3.3-3 Damping Coefficient

Structural Element	Damping
Steel	0.01
Concrete	0.02
Steel Bearing	0.01
Foundation	0.10

(2) Comparison Studies of Seismic Capacity Improvement Schemes

In order to improve seismic capacity of this bridge, the following methodologies are applied based on technical comparison studies.

i) Application of Continuous Girder

In order to prevent bridge falling down and to reduce the number of bearings, expansion joint and to simplify related devices around pier top, continuous girders are generally applied for multiple span bridges in Japan. For Guadalupe bridge, 3-Span Steel Deck Box Girder is recommended based on above mentioned comparison study, which also meet such the improvement scheme.

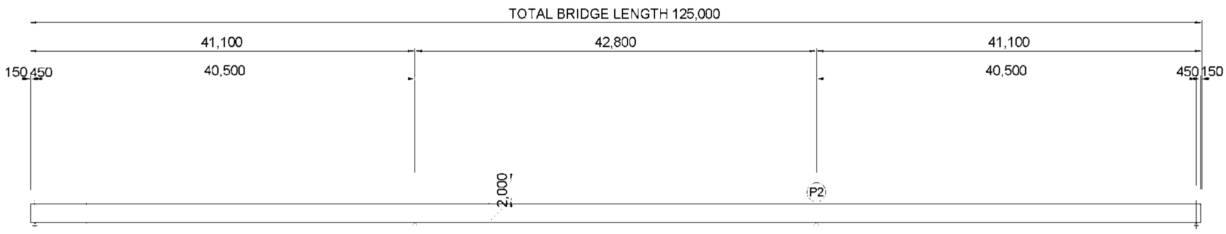


Figure 16.3.3-2 Application of Continuous Girder

ii) Adequate Bearing Type

Force distribution method by laminated rubber bearings (LRB) shown in the following figure are commonly utilized in viaducts and bridges in Japan as efficient devices to achieve appropriate seismic design. But comparing to other new bridges in this project, the dead loads of new superstructure of Guadalupe bridge is much lighter than the others. Therefore, the usual such the methodology may not apply to this bridge, which means that the appropriate and obvious advantages by application of LRB may not be achieved adequately.

Fig. Laminated Rubber Bearing

Fig. Steel Bearing

This bearing consists of rubber and steel plate layers. By changing the stiffness of the laminated rubber, such for thickness, number of layers and sizes, seismic horizontal forces can be freely and evenly shared to substructures. Therefore, the boundary condition between superstructure and substructure is "E" that means "elastic".

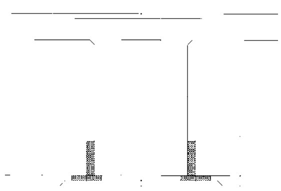
Otherwise, in Philippines, commonly thin-rubber bearing with anchor bars is utilized as bearing. By this bearing, only two ways of the boundary condition such as "Fix" or "Move" can be applied, which means that controlling of horizontal seismic forces or contribution forces to substructures depends on not horizontal stiffness of bearing but just only the period of its dynamic properties.

Otherwise, for steel deck girder bridge like Guadalupe bridge, it is appropriate to apply steel type bearing instead of above bearings, which shall resist LV2 seismic forces. Naturally the boundary condition will be two ways such as "FIX" and "MOVE", which can not apply force distribution method because stiffness of the bearing itself can not be changed. Generally, steel type bearings for viaducts are utilized under following conditions in Japan:

- Light weight superstructure
- No advantage to extend the period of the superstructure
- Slender piers can be maintained without high dumping
- Not require damping and force distribution, structurally

In this item, as an improvement scheme, technical comparison study between laminated rubber bearing, thin-rubber bearing with anchor bars and steel bearing is explained from the point of view of seismic behavior, shown as following table.

Table 16.3.3-4 Comparison Study of Bearing in Guadalupe Bridge

<table>
<tr><td>Bearing</td><td>Results of Evaluation</td></tr>
<tr><td>Laminated Rubber Bearing
Under Force Distribution Method
</td><td>Boundary Condition:
LD: Elastic (A1-P1-P2-A2), TD: Fix (A1 -P1-P2-A2)
Time Period
LD: 1.1S, TD: 0.39s
Modal Dumping of 1st mode
LD: 4.8%. TD: 4.8%
Total Horizontal Forces of Superstructure using Modal Dumping
LD: 6800kN of 8000kN
Seismic Force Distribution
LD: A1:P1:P2:A2=1:1:1:1</td></tr>
<tr><td>Pad Rubber Bearing with Dowel
Under Not Force Distribution
</td><td>Boundary Condition:
LD: Move (A1, A2), Fix (P1, P2), TD: Fix (A1-P1-P2-A2)
Time Period
LD: 0.84S, TD: 0.39s
Modal Dumping of 1st mode
LD: 2.1%. TD: 2.1%
Total Horizontal Forces of Superstructure using Modal Dumping
LD: 7200kN of 8000kN
Seismic Force Distribution
LD: A1:P1:P2:A2=0:1:1:0</td></tr>
<tr><td>Steel Bearing
Under Not Force Distribution
</td><td>Boundary Condition:
LD: Move (A1, A2), Fix (P1, P2), TD: Fix (A1-P1-P2-A2)
Time Period
LD: 0.84S, TD: 0.39s
Modal Dumping of 1st mode
LD: 2.1%. TD: 2.1%
Total Horizontal Forces of Superstructure using Modal Dumping
LD: 7200kN of 8000kN
Seismic Force Distribution
LD: A1:P1:P2:A2=0:1:1:0</td></tr>
<tr><td colspan="2">By using LRB, the period of longitudinal 1st mode achieves beyond 1.1s under controlling relative displacement of 22cm at girder end, which is slightly longer than the structure applying other bearings.

Therefore, the seismic forces based on design spectrum can not be significantly reduced between LRB and other bearings. And also, for steel bearing, the piers are adequately slender that the mode vectors in the 1st mode for such the piers are so large that the mode damping of 1st mode reaches more than 2.0%. Therefore, there may be no need to use LRB to pay high cost for reducing damping factor and extend the period. Consequently the total horizontal forces of superstructure using LRB seem as even as that of other bearings.
Therefore, structurally, application of steel bearings has enough function in Guadalupe bridge, the causes of such the result may be estimated as follows:
- Light weight superstructure
- Adequately slender piers under dead and live load condition
- Response mode vectors of piers are large, which brings large damping modal factor
- Consequently, enough period and enough damping modal factor even in steel bearing
- No advantage to extend the period of the superstructure by using LRB
- Slender piers can be maintained without high dumping
- Not require damping and force distribution, structurally</td></tr>
</table>

* LD: Longitudinal Direction, TD: Transversal Direction

(3) Summary of Seismic Analysis

i) Results of Eigenvalue Analysis

The following figure and table shows the results of eigenvalue analysis.

Table 16.3.3-5 Results of Eigenvalue Analysis

Modes	Frequency (Hz)	Period (s)	Ratio of Effective Mass		Mode Damping
			Longitudinal	Transversal	
1	1.186	0.843	**0.877**	0.000	0.021
2	2.568	0.389	0.000	**0.743**	0.021
3	4.003	0.250	0.000	0.000	0.016
4	8.373	0.119	0.000	**0.035**	0.013
5	10.762	0.093	0.000	0.000	0.010
6	11.078	0.090	0.000	0.000	0.010
7	15.816	0.063	0.000	**0.001**	0.013
8	15.845	0.063	**0.005**	0.000	0.023
9	16.159	0.062	**0.071**	0.000	0.023
10	22.262	0.045	**0.004**	0.000	0.011

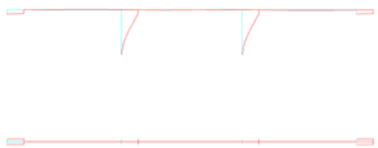
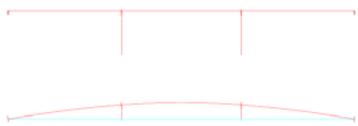
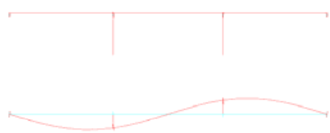
Mode 1	Mode 2	Mode 3
		

Figure 16.3.3-3 Results of Eigenvalue Analysis

According to the results, predominant mode for longitudinal direction is obviously obtained at 1st mode whose period is 0.8s and effective mass ration is 88% of modes for longitudinal direction. Therefore, the 1st mode is so important one for longitudinal direction. And for 2nd mode, the effective mass ratio is 0.74 with the period of 0.4s for transversal direction; hence the 2nd mode is also very important model shape for transversal direction.

ii) Response Displacement by Response Spectrum Analysis (EQ)

The following table shows the response displacement of relative displacements between substructure and superstructure.

Table 16.3.3-6 Relative Displacement between Substructure and Superstructure

Location	Longitudinal (mm)	Transversal (mm)
Abutment 1	157	0.00
Abutment 2	157	0.00

According to the results, the longitudinal displacements are well converged in realistic scale, for which common expansion joints can be applied.

(4) Seismic Design of Substructure and Foundation

i) Ground Surface in Seismic Design

The following figure shows the ground surface in seismic design.

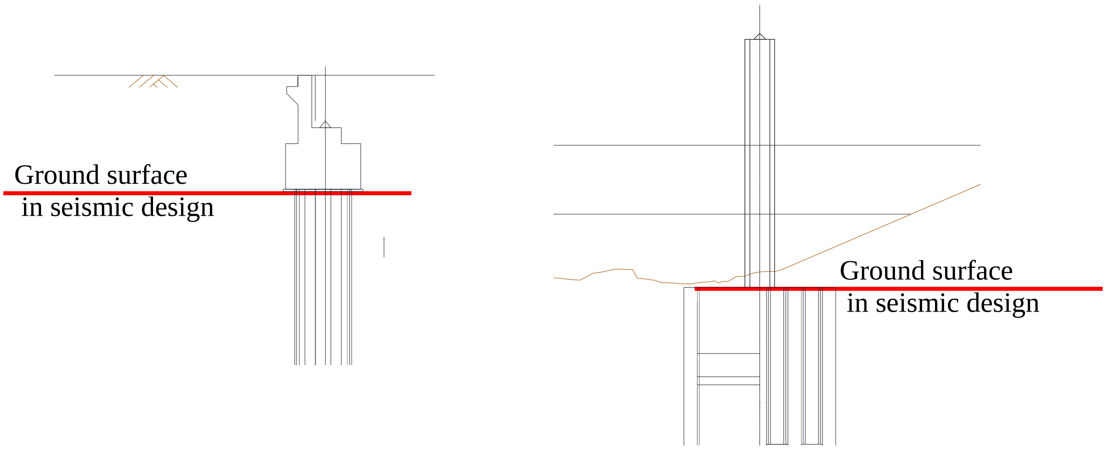


Figure 16.3.3-4 Ground Surface of an Abutment in Seismic Design

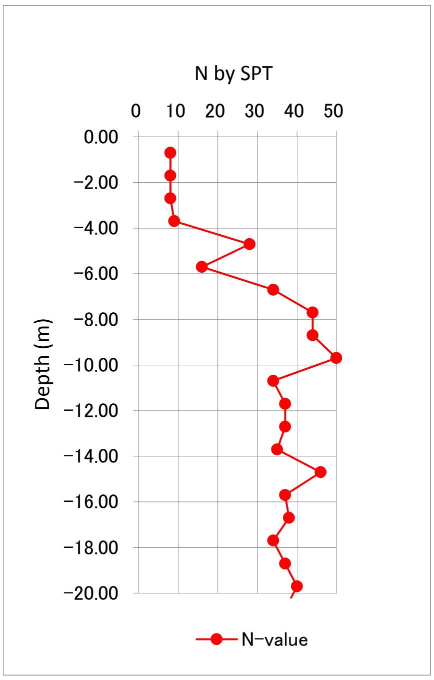
ii) Assessment of Soil Liquefaction

According to the design specifications, sandy layer requiring liquefaction Assessment is obviously obtained as following table.

Table 16.3.3-7 Assessment of Soil Liquefaction

Assessment of Liquefaction Potential

GL-(m)	Soil Layers	N by SPT	Ground Water Level (-m)	Fc (%)	PI	D50 (mm)	D10 (mm)	Liquefiable
		<30		<35%	<15	<10mm	<1mm	
0.70	BF	8	2.20	0.9				○
1.70	BF	8	2.20	17.3				○
2.70	As	8	2.20	28.0		0.52	0.18	○
3.70	As	9	2.20	12.0		0.49	0.15	○
4.70	As	28	2.20	7.3		2.06	0.25	○
5.70	As	16	2.20	7.3		0.56	0.09	○
6.70	Dg	34	2.20	58.1		3.54	0.29	
7.70	Dg	44	2.20	77.1		3.35	0.29	
8.70	Dg	44	2.20	66.9		5.76	1.70	
9.70	Dg	50	2.20	51.0		0.39		
10.70	Ds1	34	2.20	0.2		2.51	0.26	
11.70	Ds1	37	2.20	0.5		0.61	0.24	
12.70	Ds1	37	2.20			0.61	0.25	
13.70	Ds1	35	2.20					
14.70	Ds1	46	2.20			0.61	0.23	
15.70	Ds1	37	2.20			0.47	0.20	
16.70	Ds1	38	2.20					
17.70	Ds1	34	2.20			0.41	0.25	
18.70	Ds1	37	2.20			0.40	0.22	
19.70	Ds1	40	2.20			0.65	0.27	
20.70	Ds1	37	2.20			0.38	0.13	
21.70	Ds1	38	2.20					
22.70	Ds1	41	2.20			0.67	0.26	
23.70	Ds1	41	2.20			1.96	0.30	
24.70	Ds1	37	2.20					



Based on the results of liquefaction assessment, reduction of geotechnical parameters shall be conducted in accordance with the following tables.

Table 16.3.3-8 Assessment of Soil Liquefaction Parameters

Basic Soil Profile Information									
GL-(m)	Soil Layers	N by SPT	Fc (%)	γt γt1	Water unit weight	Ground Water Level (-m)	σU (Kpa)	σv (Kpa)	σv' (Kpa)
0.70	Bs	12		17	10.00	2.20	0.00	11.90	11.90
2.20	Bs	12		17	10.00	2.20	0.00	37.40	37.40
2.70	As	7		18	10.00	2.20	5.00	46.40	41.40
3.70	As	6		18	10.00	2.20	15.00	64.40	49.40
4.70	As	8		18	10.00	2.20	25.00	82.40	57.40
5.70	As	15		18	10.00	2.20	35.00	100.40	65.40

Table 16.3.3-9 Results on Liquefaction Resistance Factor (FL) & Reduction Factor (DE)

Calculation for FL								Reduction Factor D_E		
Depth	N1	C1	C2	Na	R	L	FL	**R(Ave.)**	**FL(Ave.)**	**D_E**
-0.70	24.91	1.000	0.000	24.908	0.412	0.376	1.097	**0.355**	**0.953**	**1.00**
-2.20	18.99	1.000	0.000	18.994	0.297	0.367	0.808			
-2.70	10.68	1.000	0.000	10.682	0.221	0.409	0.541	**0.269**	**0.569**	**1/3**
-3.70	8.54	1.000	0.000	8.543	0.198	0.468	0.423			
-4.70	10.68	1.000	0.000	10.675	0.221	0.507	0.436			
-5.70	18.83	1.000	0.000	18.833	0.295	0.533	0.554			

iii) Design Loads

Based on the results of seismic analysis, the abutment design is conducted for the following load combinations.

- Results of Eigenvalue Analysis

VERTICAL REACTIONS FOR SUBSTRUCTURE STABLE CALCULATION (KN)

	Dead load	Live load	Sum.
A1	1,040	1,100	2,140
A2	1,040	1,100	2,140

Note: Reaction forces for the upstream side bridge
Impact factor exclusive

- Design Combination Loads

- LONGITUDINAL DIRECTION (at bearing)

	DL		LL		EQ			SUM		
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	**N (kN)**	**H (kN)**	**M (kNm)**
A1 (Nmax)	1,040	1.25	1,100	0.50	156	-	1.00	**1,850**	**156**	-
(Nmax)	1,040	0.90	1,100	0.50	156	-	1.00	**1,490**	**156**	-
A2 (Nmax)	1,040	1.25	1,100	0.50	156	-	1.00	**1,850**	**156**	-
(Nmax)	1,040	0.90	1,100	0.50	156	-	1.00	**1,490**	**156**	-

*Friction coefficient shall be given for stable calculation of Abutments

- Results of Eigenvalue Analysis

VERTICAL REACTIONS FOR SUBSTRUCTURE STABLE CALCULATION (KN)

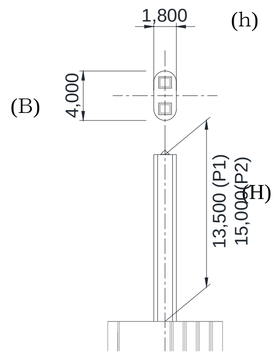
	Dead load	Live load	Sum.
P1	2,960	1,980	4,940
P2	2,960	1,980	4,940

Note: Reaction forces for the upstream side bridge
Impact factor exclusive

SECTIONAL FORCES BY RESPONSE SPECTRUM ANALYSIS FOR PIERS (at bottom of Piers)

	Longitudinal Direction			Transversal Direction		
	H(KN)	M(KNm)	V(KN)	H(KN)	M(KNm)	V(KN)
P1 (F)	4,170	50,070	0	4,060	49,640	0
P2 (F)	4,160	49,950	0	4,060	49,580	0

Note: Friction coefficient shall be given for stable calculation of Abutments



- Dead Loas (for Pier column)

	Pier Column					
	h	B	Aria	Height	Unit Weight	**Self Weight**
	(m)	(m)	(m2)	(m)	(kN/m3)	**(kN)**
P1	1.8	4.0	6.5	13.5	24.5	**2,152**
P2	1.8	4.0	6.5	15.0	24.5	**2,391**

- Design Combination Loads

LONGITUDINAL DIRECTION (at bottom of column,Nmax)

	DL		LL		EQ			SUM of LONGITUDINAL		
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	**N (kN)**	**H (kN)**	**M (kNm)**
P1(Nmax)	5,112	1.25	1,980	0.50	4,170	50,070	1.00	**7,380**	**4,170**	**50,070**
(Nmin)	5,112	0.90	1,980	0.50	4,170	50,070	1.00	**5,600**	**4,170**	**50,070**
P2(Nmax)	5,351	1.25	1,980	0.50	4,160	49,950	1.00	**7,680**	**4,160**	**49,950**
(Nmin)	5,351	0.90	1,980	0.50	4,160	49,950	1.00	**5,810**	**4,160**	**49,950**

TRANSVERSAL DIRECTION (at bottom of column,Nmax)

	DL		LL		EQ			SUM of TRANSVERSAL		
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	**N (kN)**	**H (kN)**	**M (kNm)**
P1(Nmax)	5,112	1.25	1,980	0.50	4,060	49,640	1.00	**7,380**	**4,060**	**49,640**
(Nmin)	5,112	0.90	1,980	0.50	4,060	49,640	1.00	**5,600**	**4,060**	**49,640**
P2(Nmax)	5,351	1.25	1,980	0.50	4,060	49,580	1.00	**7,680**	**4,060**	**49,580**
(Nmin)	5,351	0.90	1,980	0.50	4,060	49,580	1.00	**5,810**	**4,060**	**49,580**

vi)Design Result
Based on that the following dimensions are obtained as the substructure with foundation of Guadalupe bridge.

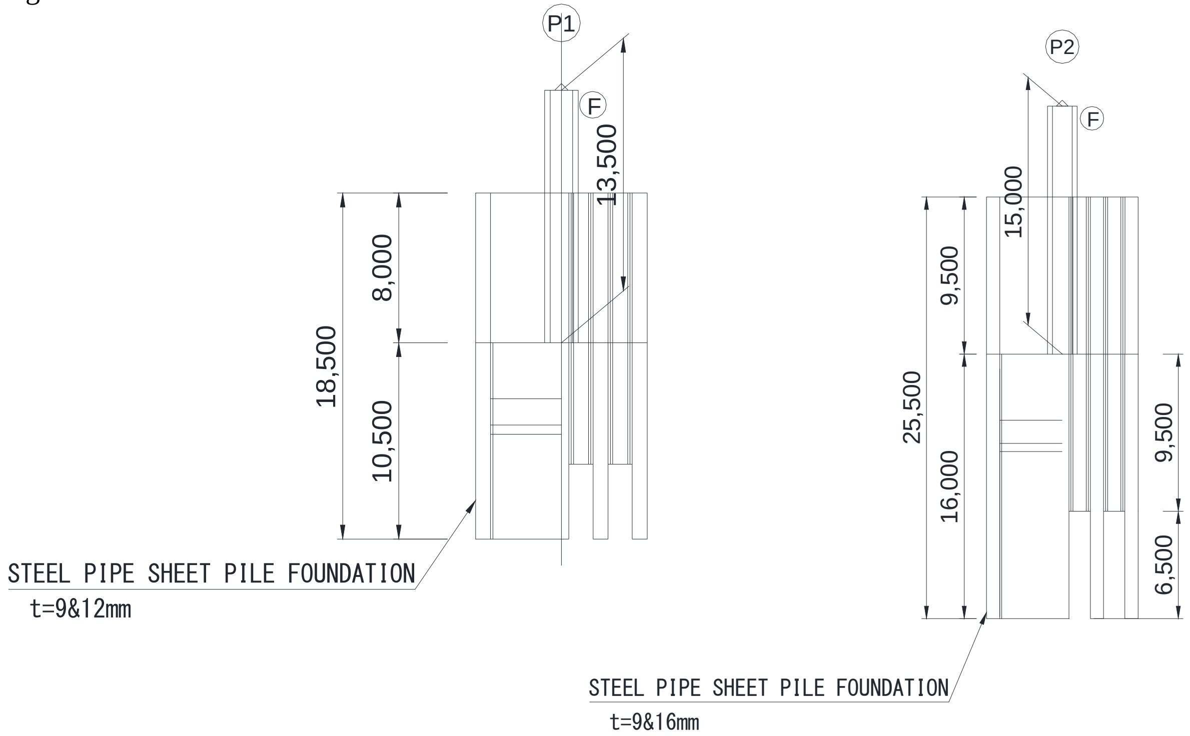


Figure 16.3.3-5 Side View of Pier of Guadalupe Bridge Substructure with Foundation.

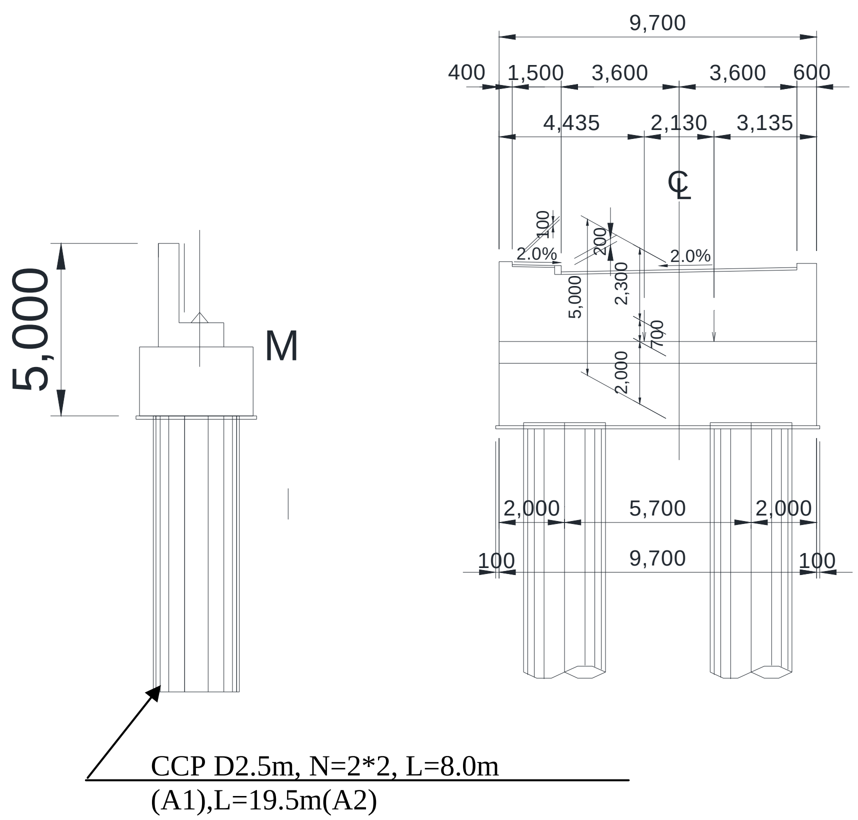


Figure 16.3.3-6 Side View & Sectional View of Abutment of Guadalupe Bridge Substructure with Foundation.

v) General of Steel Pipe Sheet Pile (SPSP) Foundation

A Steel pipe sheet pile consists of a steel pipe pile as the main component member, to with the joints illustrated in detail of following figure are attached. Compared to an ordinary steel sheet pile, it has an advantage of considerable rigidity; witch lends itself highly useful for wall structures such as earth retaining walls for deep excavation and deep water foundations.

Steel pipe sheet pile foundation is consisted of outside steel pipe sheet pile well and diaphragm steel pile sheet pile well. Open end steel pipe sheet piles are driven to the designated depth, loads from superstructure are transmitted to the upper slab and then to the sheet piles and finally to soil by friction and tip bearing.

Steel pipe sheet pile foundation lets the Outside sheet pipe well itself get up over the water surface, its joints being filled with cut off materials to serve as temporary cofferdam with temporary braces and wales. The inside of well is dried, and after a pile cap and a pier are erected there, the pipe pile temporary cofferdam planning cutting passion around above the top end of the footing is underwater cut and removed.

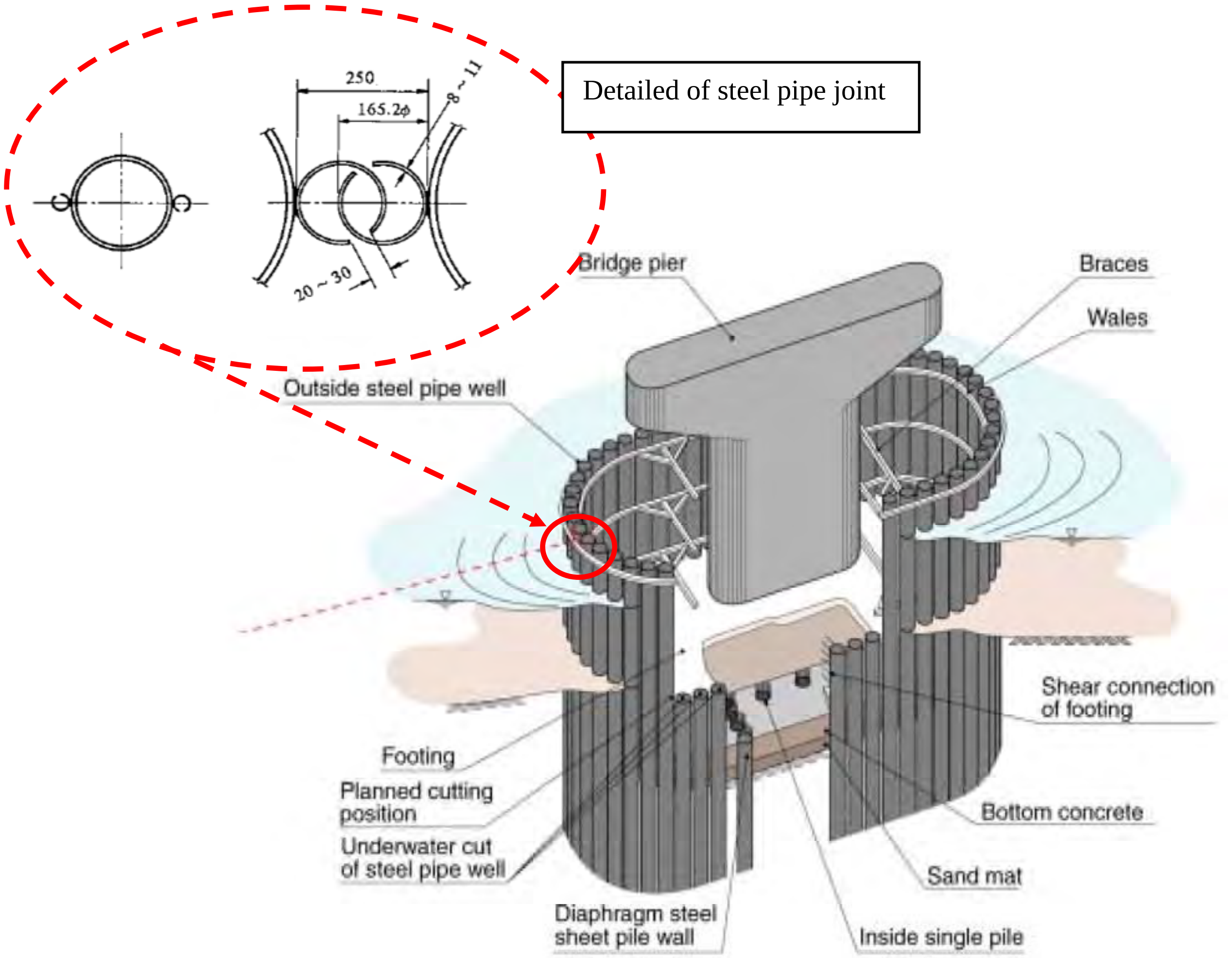


Figure 16.3.3-7 Conceptual View of Steel Sheet Pile Foundation

Based on that, the following design flow is obtained the design of steel pipe sheet pile foundation.

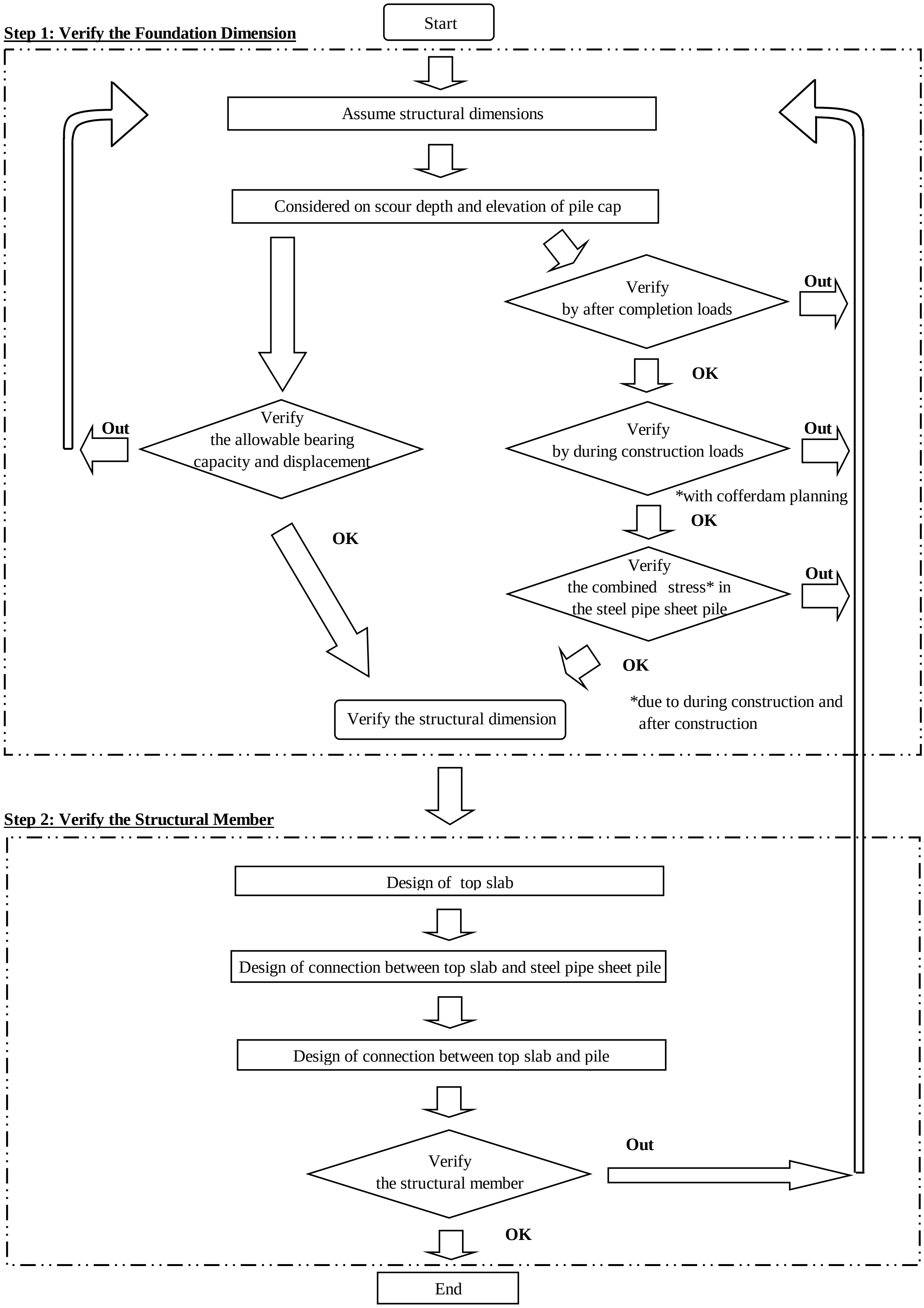


Figure 16.3.3-8 Design Flow for Basic Design of Steel Pipe Sheet Pile Foundation

Construction Step

The procedure for construction method of steel pipe sheet pile foundations and points of construction at each stage are shown as below.

1) Steel pipe sheet pile setting and driving

In setting and driving steel pipe sheet piles, to prevent the rotating and tilting of pipe pile, guide frames are installed inside and outside the circumference of the well are attached. Pile setting and driving work is performed by the pile driver on the boat or on the scaffold at site. Piles are set, one by one, by the Vibro hammer, at positions determined by the guides.

2) Joint work and bottom slab concrete casting

When the pipe pile driving is completed to the designed depth, earth and sand in the joint are removed and filling mortar into the joints to below the pile cap. Next comes pouring of cut off material into the joint of temporary cofferdam section by Nylon bags. Then the internal excavation to the prescribed depth by a clamshell and water pump is performed. Upon completion of the excavation, the ground surface is evened with sand gravel, and under water slab concrete is cast.

3) Braces setting with drying up

After curing of underwater bottom slab concrete, drying up of the inside of the temporary cofferdam is started. The water level is lowered to the lower level of the stage braces and a wale is set up, one by one.

4) Shear connection setting

In order to make the pile cap and the internal wall of steel pipe sheet pile foundation in one body, a shear connection is welded to the pipe pile.

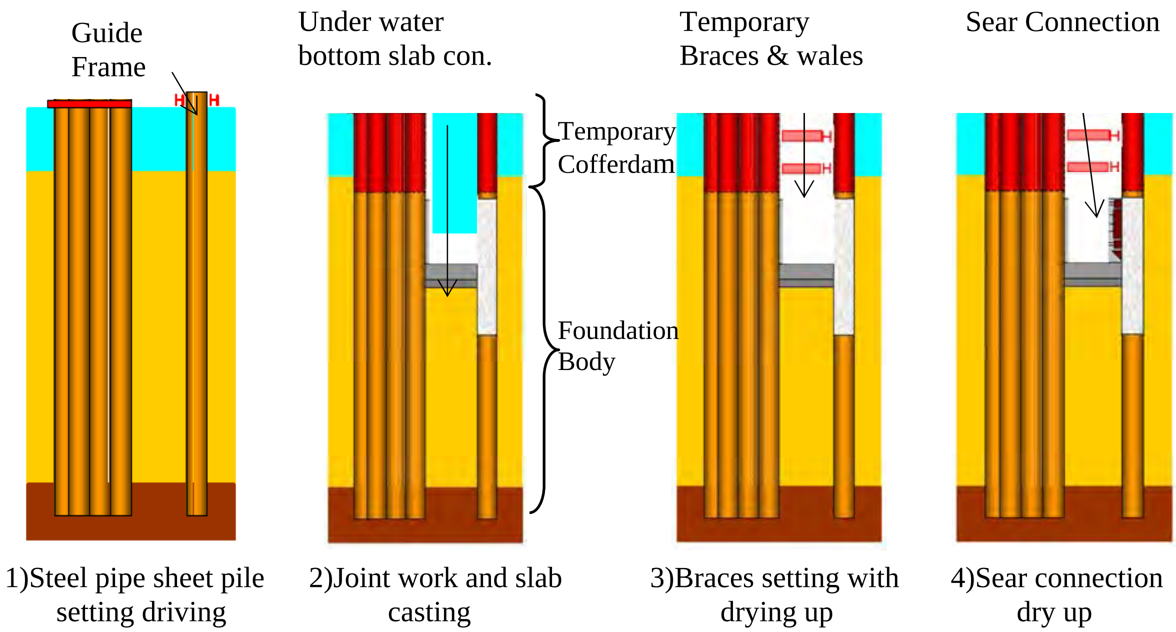


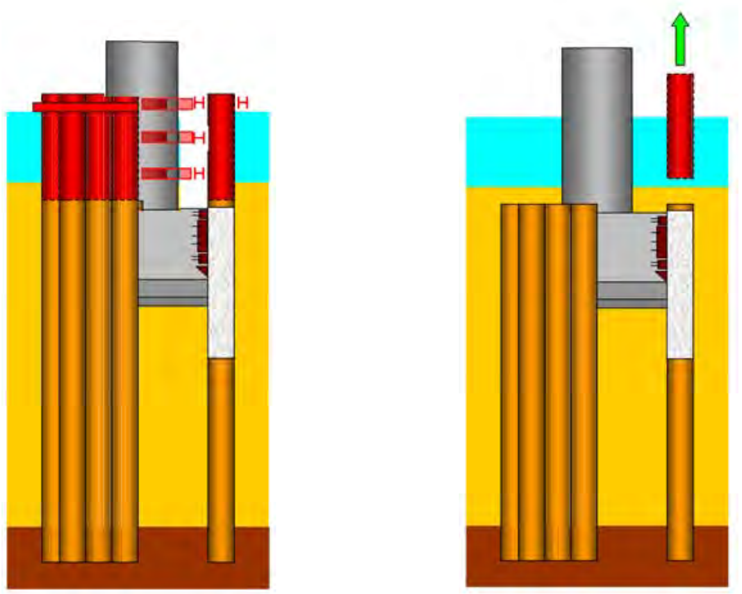
Figure 16.3.3-9 The Procedure for Construction Method of Steel Pipe Sheet Pile Foundations (1)

5) Pile cap and column construction

The arrangement on pile cap reinforcement as well as concrete casting follows. Then, a column is erected.

6) Temporary Braces & wales removal and underwater cutting of sheet pile

While water is poured inside the temporary cofferdam, braces and wales are removed, one by one. After external and internal pressure are balanced, steel pipe sheet pile are cut under the water at the top of the pile cap.



5) Pile cap and column construction

6) Temporary Braces & wales removal and underwater cutting of sheet pile

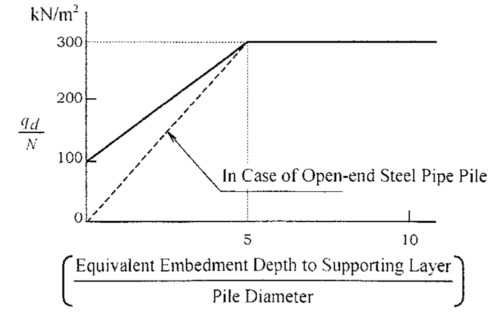
Figure 16.3.3-10 The Procedure For Construction Method of Steel Pipe Sheet Pile Foundations (2)

Vertical Bearing Capacity

Vertical bearing capacity (Ra) and safety factor (n) of Steel Pipe Sheet Pile foundation shall be calculated as follows.

$$R_a = \frac{1}{n} R_u$$

$$R_u = q_d \cdot A_1 + \frac{1}{n_1 + n_2 + n_3} \left\{ U_1 \sum L_i f_i + U_2 \sum L_j f_j \right\}$$



where

A1 : Tip closed section of sheet pile (m2)
qd : Tip resistance per unit area (kN/m2)
n1 : number of sheet piles in exterior wall
n2 : number of sheet piles in bulkhead
U1 : circumference envelop length of exterior wall (m)
U2 : circumference envelop length of interior wall and bulkhead (m)
Li : length of each layer considering side friction for exterior wall (m)
fi : maximum friction coefficient for exterior wall (kN/m2)
Lj : length of each layer considering side friction for exterior wall (m)
fj : maximum friction coefficient for interior wall (kN/m2)

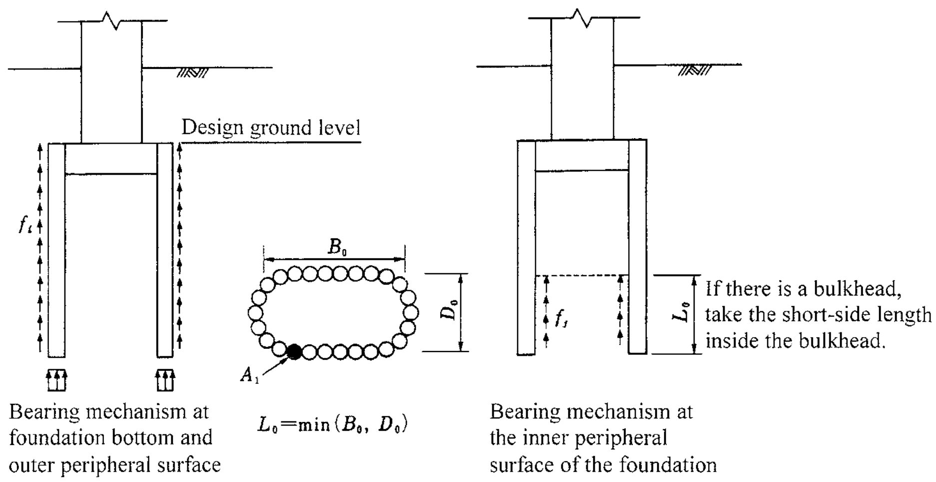


Figure 16.3.3-11 Region Where the Skin Friction Force at the Inter Peripheral Surface of the Well Portion of the Foundation Should Be Taken into Account

Design Model

The steel pipe sheet pile foundation has a very wide range of Le, which indicates the applicable scope of the design method, and generally belongs to elastic foundations of finite length. Judging from Le, some are regarded as an elastic foundation with a value less than 1, however, the steel pipe sheet pile foundation is a structure consisting of steel pipe sheet pile mutually joined by joint pipes of smaller rigidity than the steel pipe body and with mortar filled in the joint pipes, and a shear slippage deformation easily occurs in it. Therefore, verification of the slippage at the foundation bottom may be omitted. That is, stability should be verified on vertical bearing capacity and horizontal displacement.

An outline of the stability calculation model used in verification for ordinary, storm and seismic condition is shown in following table.

Table 16.3.3-10 Stability Calculation Model

		Verification for ordinary conditions, storm and Level 1 earthquake conditions	
		$B \leqq 30$m , $L/B > 1$ and $\beta L_e > 1$	$B > 30$m , $L/B \leqq 1$ or $\beta L_e \leqq 1$
Design model		Finite-length beam on an elastic floor (Beam Model)	Analysis by an imaginary well beam that considering shear slippage of the joints (Well Model)
Foundation body	Steel pipe sheet pile	Linear	
	Shear resistance of joint	Evaluation by composite efficiency and moment distribution factor	Bilinear
Ground hesitance element	Horizontal ground resistance at the foundation front face	Linear considering strain dependency	
	Horizontal shear ground resistance at the foundation peripheral faces	Included in the horizontal resistance of the front ground	
	Vertical shear ground resistance at the foundation outer and inner peripheral faces	Included in the bearing capacity of the steel pipe sheet pile	
	Vertical ground resistance at the foundation bottom face	Linear	Linear
	Horizontal shear ground resistance at the foundation bottom faces	Linear	Linear

Finite-length beam Model

The sectional forces, displacement and unit ground reaction force of a well-type steel pipe sheet pile foundation may be derived by regarding the steel pipe sheet pile foundation as a finite-length beam on an elastic model, as shown in following model.

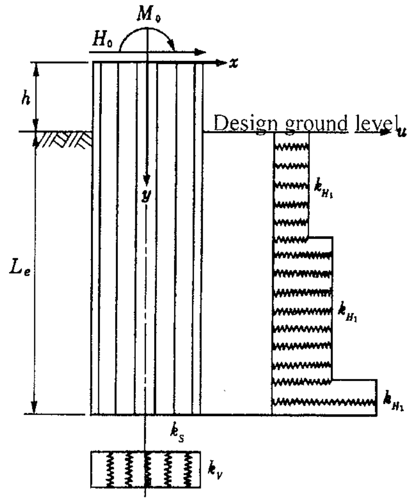


Figure 16.3.3-12 Calculation Model of Steel Pipe Sheet Pile Foundation

(5) Unseating Prevention System

The superstructure is generally connected to the substructure through bearings. As such, the superstructure and the substructure are separated functionally and significantly critical state such as bridge falling down may be caused due to large relative displacements between them, in case of failure of bearings under unexpected seismic forces.

For a functional system preventing such severe state, detailed philosophy and articulate design concepts are explicitly specified in JRA as “Unseating Prevention System” based on accumulated data and experiences from large number of seismic damages. The aim is to provide multiple mechanisms that can complement each other efficiently, shown as follows.

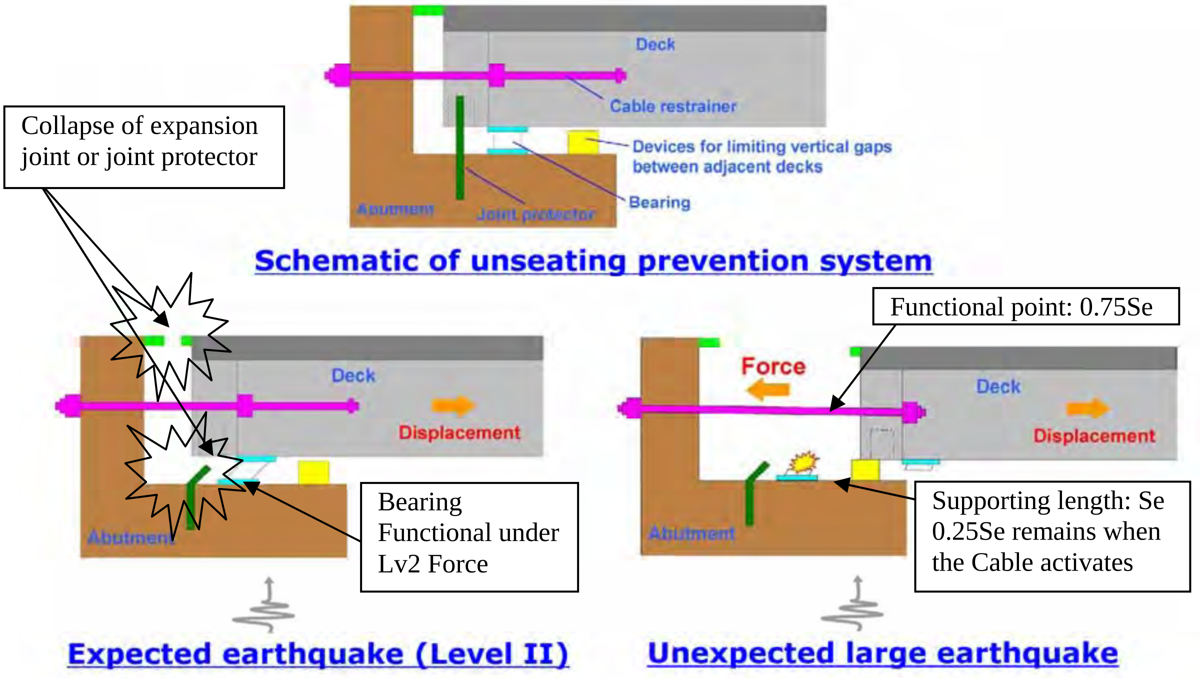


Figure 16.3.3-13 Philosophy of Unseating Prevention System in JRA

The Unseating Prevention System consists as following devices or functions:

Table 16.3.3-11 Devices and Functions of Unseating Prevention System

Devices	Function
Bearing (Type B)	"Type B" bearing in JRA, enables to resist the seismic forces of LV2 by itself.
Supporting Length	The final function of the unseating prevention system. The equation to determine the length is given as follows: Se=μr+μg : μr: Relative responses of girder, μg: Displacement of ground Minimum length: Semin=0.7+0.005l (m)
Longitudinal Restrainer	Design forces are given as 1.5Rd, where Rd is the reaction force of dead loads Maximum movable length: 0.75Se, where Se is supporting length. Generally for new bridges in Japan, cable type devices connecting between parapet of abutment and cross beam of superstructure are commonly utilized.
Expansion Joint	Expansion joint shall resist LV1 seismic forces, otherwise, the capability to resist LV2 seismic forces is not required.

Based on the philosophy and functions above, specification of each devices applied in this new bridge is shown as the results of outline design.

i) Bearing

For Guadalupe bridge, common steel bearing is advantageous because the weight of superstructure is extremely light weight. Therefore, in the stage of detail design, appropriate steel bearing that can resist surely LV2 seismic forces shall be selected.

ii) Supporting Length

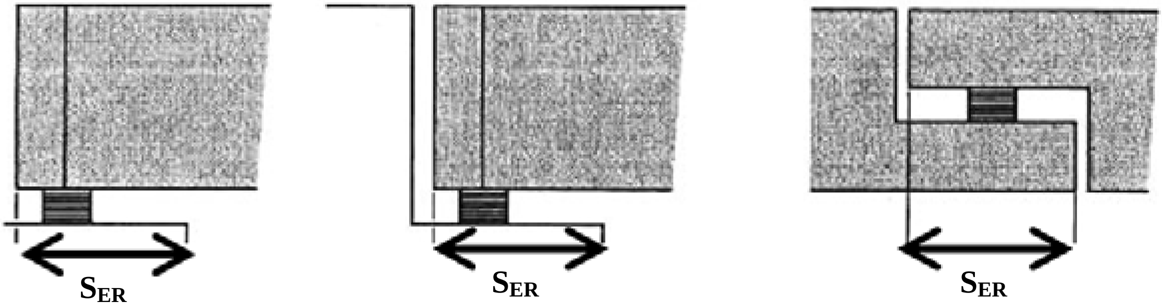


Figure 16.3.3-14 Supporting Length

Following equation gives the supporting length.
Se=0.7+0.005l (m) l: Span length
Se = 0.7+0.005*41 = 0.91m

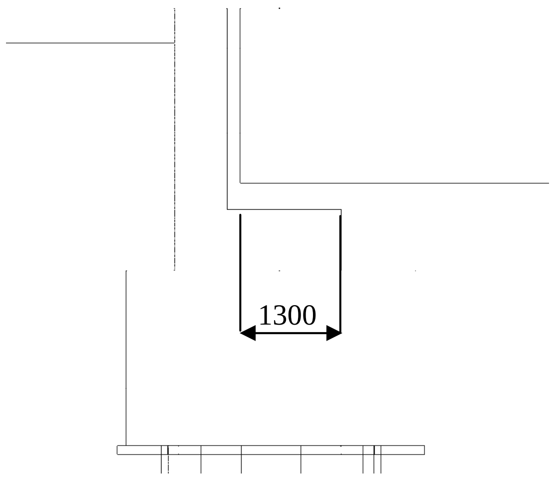


Figure 16.3.3-15 Secure the Length of "Se", Supporting Length

iii) Longitudinal Restrainer
The reaction forces by dead loads are 1040 kN.
The following verification can be obtained.

Table 16.3.3-12 Verification of Longitudinal Restrainer

	1.5Rd: Design Forces	Allowance
PC Cable Type 7 x ϕ 11.1mm 2-nos/ Abutment	780 kN	826 kN

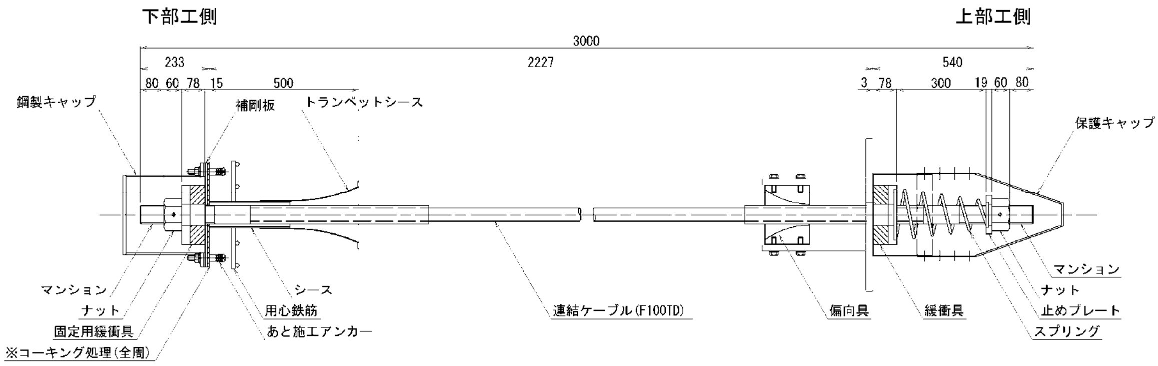


Figure 16.3.3-16 Longitudinal Restrainer for Guadalupe Bridge

(6) Miscellaneous devices and others

Miscellaneous devices in the bridge are defined as following items:

- Bearing: Evaluated above clause
- Expansion joint
- Drainage
- Wearing coat

In this clause, the devices which are not explained in other clause are explained based on seismic behaviors and current bridge condition.

i) Expansion joint
For the design methodology of expansion joint, its seismic capacity shall be secured under LV1 forces and it does not have to be secured under LV2 forces. The expansion gap between girder end and abutment shall be determined from results of dynamic modal analysis under LV2 and another expansion gap of expansion joint itself shall be determined based on seismic analysis of LV1.

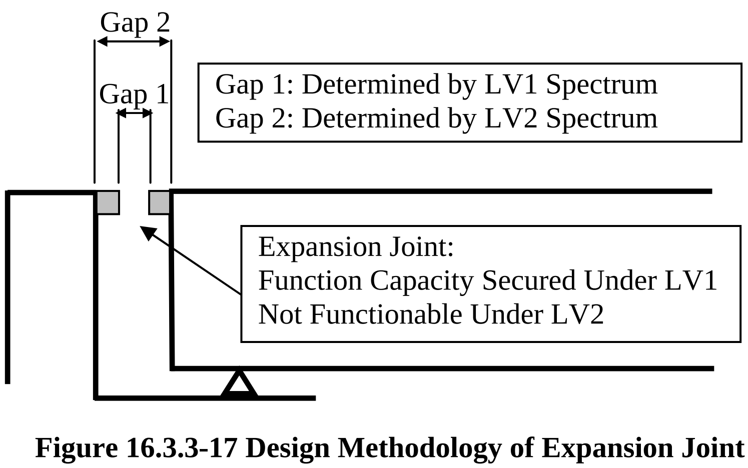


Figure 16.3.3-17 Design Methodology of Expansion Joint

The final displacements to be used for determination of expansion joint based on dynamic modal analysis are as follows.

- LV1: Gap 1: 9.5cm + 1.5cm (Excess allowance 15mm (JRA) ≒ 11cm
- LV2: Gap 2: 15.5cm + 1.5cm (Excess allowance 15mm (JRA) ≒ 17cm

Therefore, the expandable gap of the joint in this bridge shall be 11cm or more, and the gap between girder and abutment shall be 16cm or more, which are common results achieved under careful controlling in the dynamic analysis. Consequently, general steel type expansion joint can be adequately applied to this bridge.

ii) Drainage
Drainage system on the bridge is estimated based on current condition. In the next stage such as basic design or design stage, appropriate location of catch basins and drainage pipes shall be designed and drawn based on further investigation of accumulated rainfall data of corresponding area.

iii) Wearing coat
Guadalupe Side bridge will consist of steel deck. The steel deck is definitely flexible member which may causes fracture and crack of wearing coat if the selection of the wearing coat engineeringly mistakes.
In Japan, we have a lot of steel deck bridges, in which generally utilized following wearing coat system on the steel deck bridge, consisting of two of layers, in order to appropriately follow the deformation of steel deck.
In detail design stage, comparison study of bridge wearing coat based on costs and structure as well as maintenance such as guss or epoxy asphalt, which are of Japan's advanced products, suitable for flexible steel deck should be conducted. And also, in Japan, as asphalt concrete, polymer modified asphalt is usually applied to wearing coat on bridges, which is excellent at flowability, flexibility, durability, rutting resistance and heat resistance.

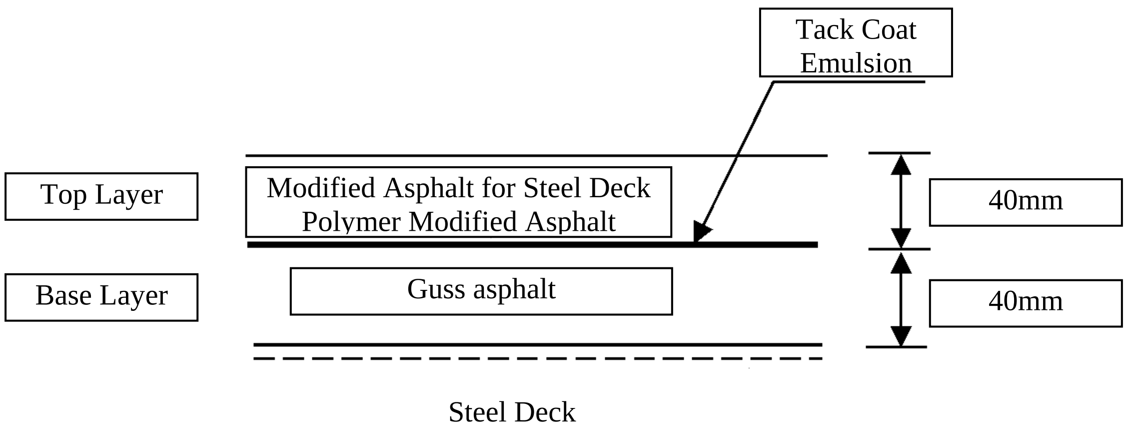


Figure 16.3.3-18 Wearing Coat System of Steel Deck

16.3.4 Summary of Outline Design Results

(1) Superstructure

Superstructure is designed based on AASHTO LRFD for the bridge type determined in multiple comparison study in consideration of various conditions. The bridge type is Steel Deck Box Girder. And steel bearing is applied in consideration of seismic behavior calculated dynamic modal analysis.

(2) Substructure and Foundation

The following substructures and foundations are resulted by the outline design.

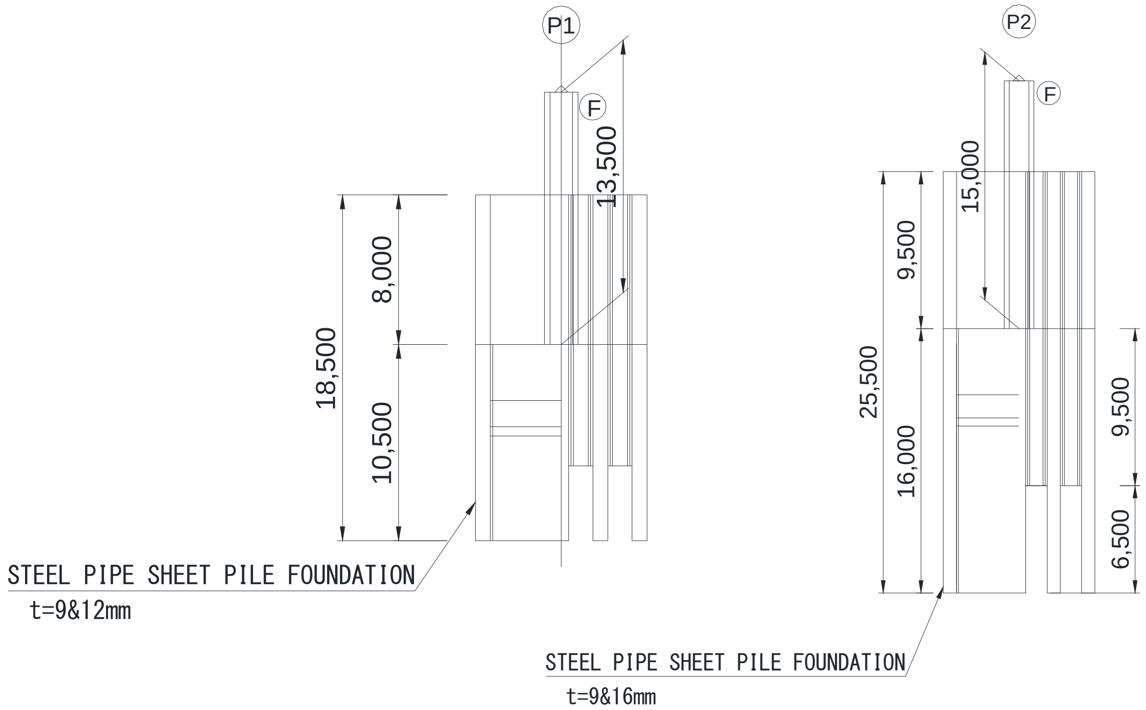


Figure 16.3.4-1 Side View of Pier of Guadalupe Bridge

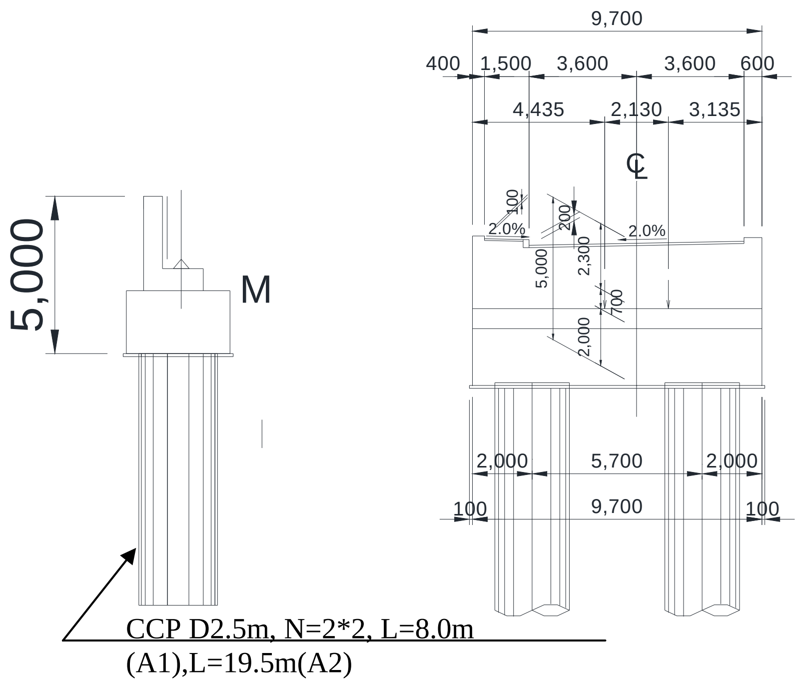


Figure 16.3.4-2 Side View & Sectional View of Abutment of Guadalupe Bridge

(3) Further Verification to Be Examined in The Next Phase

The following items may be necessary to be verified or evaluated further in the next phase such as basic or detail design stages.

- Optimization and re-verification of bridge length, span arrangement and bridge types, on the basis of latest existing road condition, newly future planning and detail river condition resulted by detail hydraulic analysis
- Applicability of utilization of high-damping bearing based on specific organization regarding non-linear time history response analysis based upon comparison study regarding bearing system
- Comparison study of bridge wearing coat based on costs and structure as well as maintenance such as guss or epoxy asphalt, which are of Japan's advanced products, suitable for flexible steel deck should be conducted. And also, in Japan, as asphalt concrete, polymer modified asphalt is usually applied to wearing coat on bridges, which is excellent at flowability, flexibility, durability, rutting resistance and heat resistance.

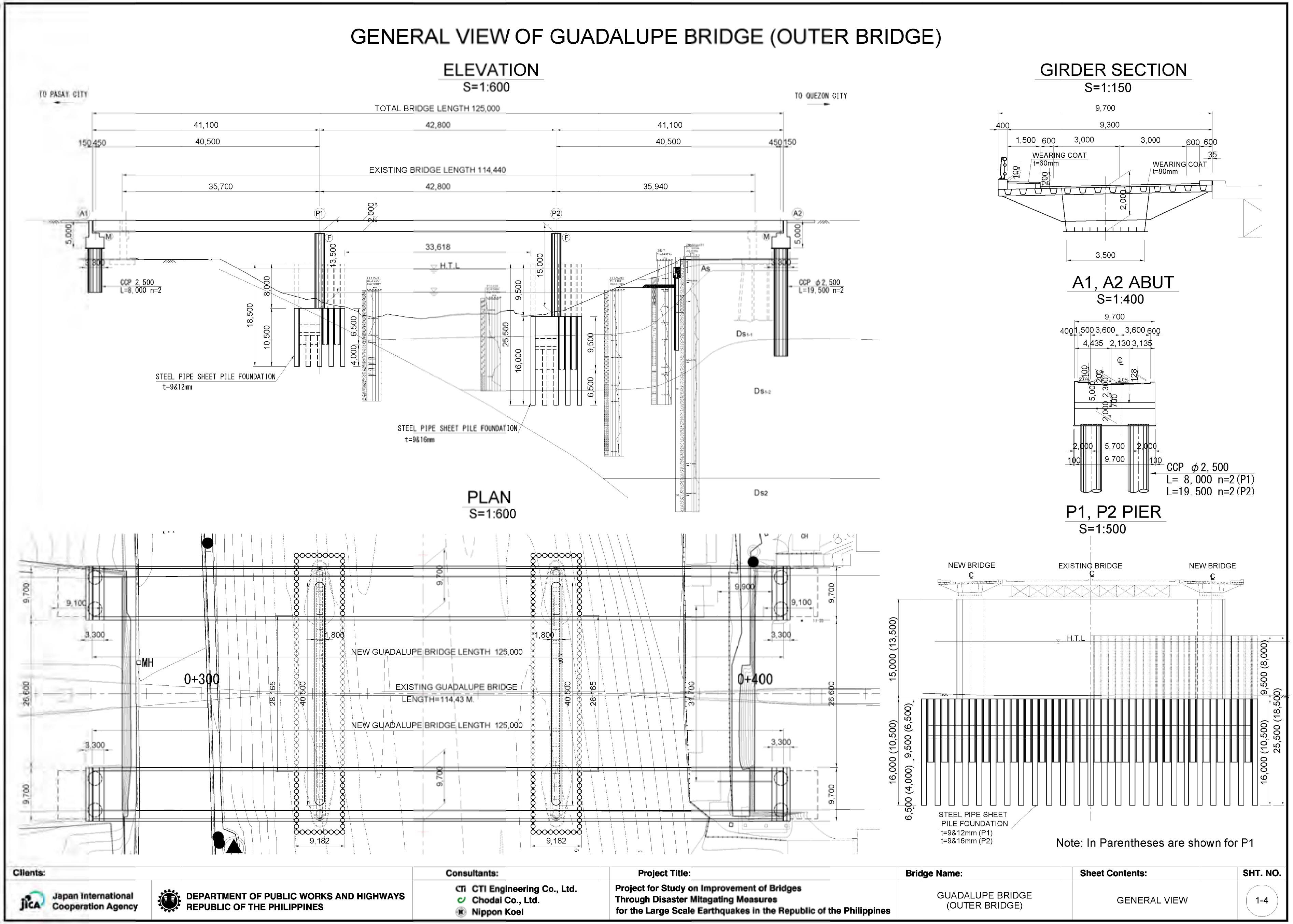


Figure 16.3.4-3 General View
(Structural Drawings are shown in Appendix-4)

16.4 Outline Design of Palanit Bridge

16.4.1 Design Condition

The following items show design condition for the outline design of Palanit Bridge.

(1) Road Conditions

- Road Design Standard : AASHTO STANDARD VALUE
- Design Speed : V = 60 kmph
- Live Loads : AASHTO Live Loads HL93 and Lane Loads
- Road Width : Shown as follows:

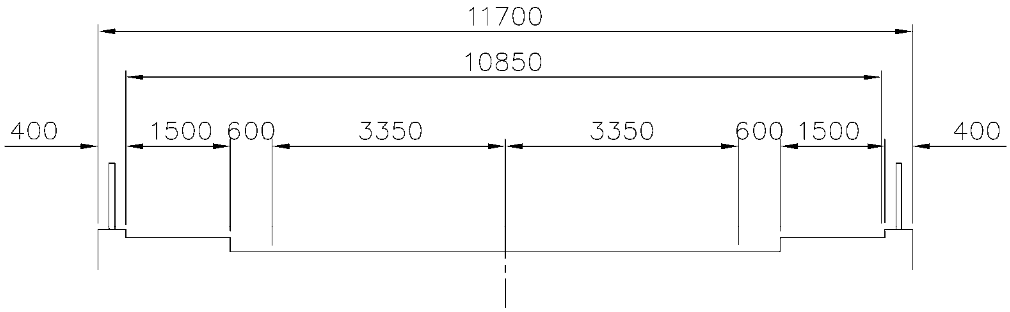


Figure 16.4.1-1 Cross Section/ Lane Arrangement of Palanit Bridge

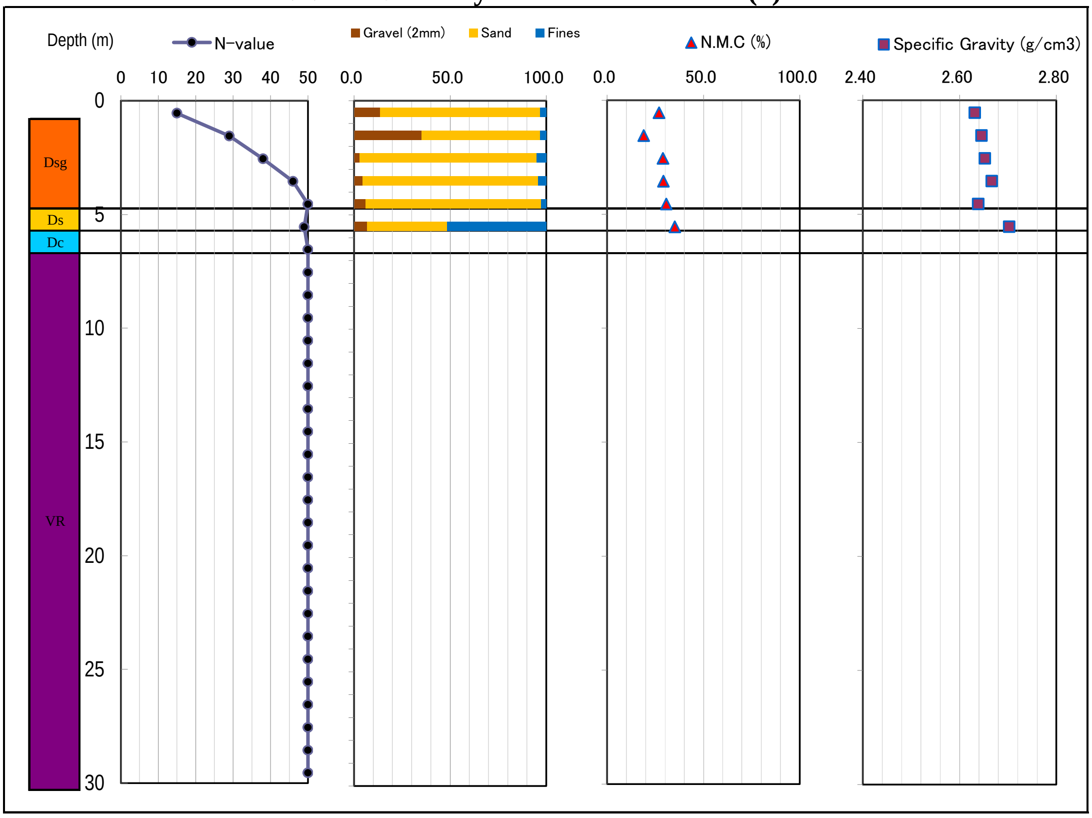
(2) Soil Conditions

The results of ground investigation are shown in below illustrations and following tables. The weathered rock layer that can be regarded as the bearing layer is distributed E.L. -2.0m to E.L.-6.5m depth, and has a thick surface layer predominant with gravely sand on top. Specialty, liquefiable sand (Dsg) is thickly deposited from ground surface to GL-2.0m, of which N-value is 15 to 29, will be affected by liquefaction occurs with reduction of geotechnical parameter. .

Table 16.4.1-1 Summary for Soil Parameters (1) at A1 side

Input by Tanaka - A2 Side - Palanit PAL-R1 (left bank) EL.2.40m										Input by EASCON						Soil Parameters						Layer Thickness (m)
Number	Depth		SPT		Nstd	S-wave	Vsn	Soil Layer	Soil	GSA			N.M.C (%)	PI	Specific Gravi	Nd	γt	C	φ	E0	Vsn	
	Upper	Lower	Blows	Penetration					Observation	Gravel (%)	Sand (%)	Fines (%)					(tf/m^2)	(kN/m^2)	(°)	(kN/m^2)	(m/sec)	
1	0.55	1.00	9	30	9	208	160	Asg	Gravely sand	38.00	59.3	2.7	21.0	N/A	2.64	8	17	0	33	5,600	160	2
2	1.55	2.00	8	30	8	208	160	Asg	Gravely sand	41.50	55.5	3.0	20.3	N/A	2.63		17					
3	2.55	3.00	50	5	300	208	798	VR	Rock							300	21	514	21	136,098	300	28
4	3.55	4.00	50	5	300	431	798	VR	Rock								21					
5	4.55	5.00	50	5	300	431	798	VR	Rock								21					
6	5.55	6.00	50	5	300	431	798	VR	Rock								21					
7	6.55	7.00	50	5	300	431	798	VR	Rock								21					
8	7.55	8.00	50	5	300	830	798	VR	Rock								21					
9	8.55	9.00	50	5	300	830	798	VR	Rock								21					
10	9.55	10.00	50	5	300	830	798	VR	Rock								21					
11	10.55	11.00	50	5	300	830	798	VR	Rock								21					
12	11.55	12.00	50	5	300	830	798	VR	Rock								21					
13	12.55	13.00	50	5	300	830	798	VR	Rock								21					
14	13.55	14.00	50	5	300	830	798	VR	Rock								21					
15	14.55	15.00	50	5	300	963	798	VR	Rock								21					
16	15.55	16.00	50	5	300	963	798	VR	Rock								21					
17	16.55	17.00	50	5	300	963	798	VR	Rock								21					
18	17.55	18.00	50	5	300	963	798	VR	Rock								21					
19	18.55	19.00	50	5	300	963	798	VR	Rock								21					
20	19.55	20.00	50	5	300	963	798	VR	Rock								21					
21	20.55	21.00	50	5	300	963	798	VR	Rock								21					
22	21.55	22.00	50	5	300	963	798	VR	Rock								21					
23	22.55	23.00	50	5	300	963	798	VR	Rock								21					
24	23.55	24.00	50	5	300	911	798	VR	Rock								21					
25	24.55	25.00	50	5	300	911	798	VR	Rock								21					
26	25.55	26.00	50	5	300	911	798	VR	Rock								21					
27	26.55	27.00	50	5	300	911	798	VR	Rock								21					
28	27.55	28.00	50	5	300	911	798	VR	Rock								21					
29	28.55	29.00	50	5	300	911	798	VR	Rock								21					
30	29.55	30.00	50	5	300	911	798	VR	Rock								21					

Table 16.4.1-2 Summary for Soil Parameters (2) at A1 side



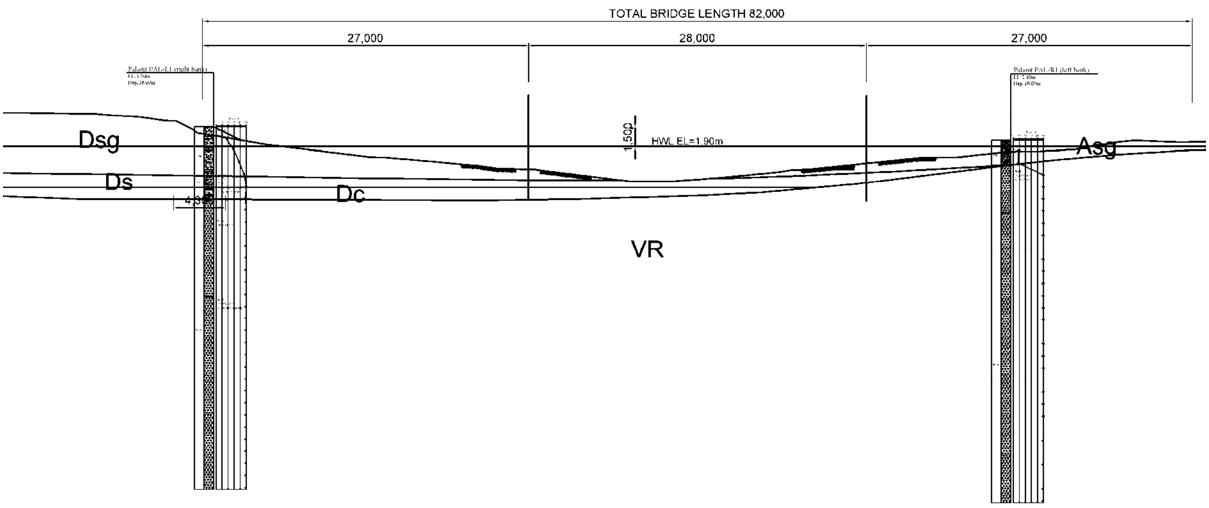


Figure 16.4.1-2 Soil Profile of Palanit Bridge (Included previous SPT)

(3) Hydraulic Conditions

- Design Flood Discharge : Q = 197 m3/s
- Design Water Level : EL= 1.90 m
- Freeboard from Design Flood Level : H = 1.50 m

Hydraulic condition shall be carefully verified and examined by detail hydraulic analysis in the detail design stage, based on which the road and bridge planning shall be reevaluated in such the phase including comparison study of bridge types.

(4) Bridge Type

- Superstructure Type : 3-Span PC-I Shape Girder
- Bridge Length : L=82m
- Span Arrangement : 27m +28m +27m
- Transversal Slope : 2.0%
- Horizontal Alignment : R=∞
- Angle of Alignment : 90 Degrees
- Wearing coat : Polymer Modified Asphalt, Coarse/ Dense-Graded t=80mm for Vehicle lane, t=30mm for walkway
- Railing : Steel railing for vehicle and pedestrian
- Bearing : NRB Rubber Bearings - Force Distribution Bearing
- Expansion Joint : Steel type
- Drainage Appliances : PVC pipe
- Bridge Falling Prevention Device: Cable type

- Substructure Type : Circular Type
- Foundation Type : Spread Sheet Foundation
- Bearing Soil Condition : Rock (N>50)

(5) Design Cases of Outline Design

The outline design of superstructure shall be designed based on the above load condition, specified in AASHTO 2012. On the basis of various reactions and forces, substructures and foundation shall be designed throughout response spectrum analysis under the limit state of "Extreme Event I" specified in AASHTO 2012.

STEP 1. Outline Design of Superstructure
- Design Load, specified in AASHTO 2012
- Stress check

↓

STEP 2. Organization of Reaction Forces
- Organization of reaction forces for substructure design
- Organization of masses for response spectrum analysis

↓

STEP 3. Modal Analysis and Response Spectrum Analysis
- Eigenvalue analysis based on modal analysis
- Organization of response values by the seismic analysis

↓

STEP 4. Substructure and Foundation Design
- Extreme Event I

Figure 16.4.1-3 Flow of Outline Design

16.4.2 Outline Design of Superstructure

(1) Design Condition

- Superstructure Type : Continuous PC-I girder
- Bridge Length : L=82m
- Angle of Alignment : 90 Degrees
- Wearing coat : Polymer Modified Asphalt, Coarse/ Dense-Graded
  t=80mm for Vehicle lane, t=30mm for walkway
- Railing : Steel railing for vehicle and pedestrian
- Bearing : NRB Rubber Bearings - Force Distribution Bearing
- Road Width : Shown as follows:

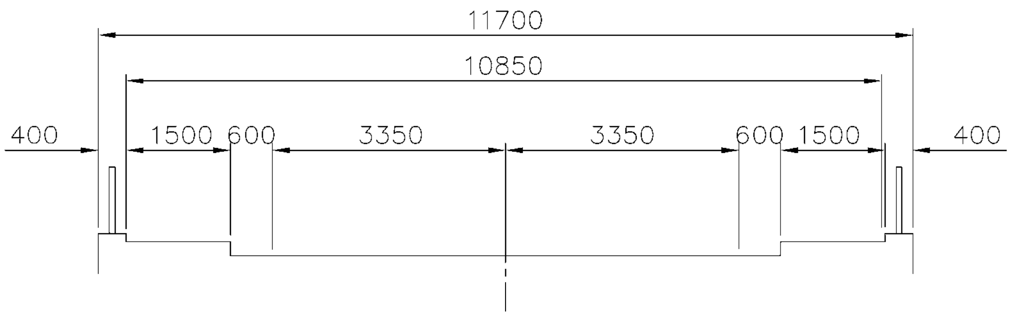


Figure 16.4.2-1 Cross Section/ Lane Arrangement of Palanit Bridge

(2) Design Loads

- Dead Loads : AASHTO 2012
- Live Loads : HL93 and Lane Loads in AASHTO 2012, utilized by influence line evaluation

(3) Design girder

For Palanit bridge, AASHTO girder type that is usually utilized in Philippines is applied to the superstructure. Therefore, complicated analysis and design methodology are not required for the outline design; hence, applied girder types and the results of the prestress estimation are shown in the following figures and tables.

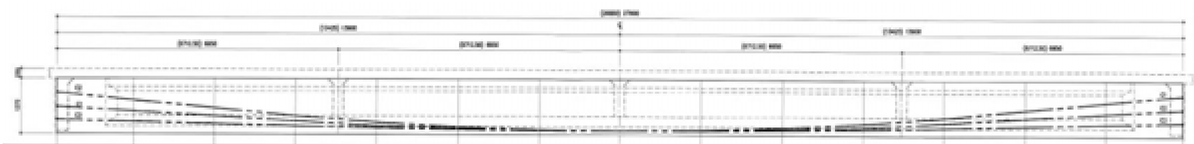


Figure 16.4.2-2 Designed and Applied AASHTO Girder Type-IV

(4) Design of AASHTO girder type-IV

For the AASHTO girder type-IV, following approximate amount of prestressing forces are calculated.

Table 16.4.2-1 Determination of Approximate Amount of Prestressing Force

At Service Condition Allowable Tension	Ft =3 Mpa
Assumed Centroid of Cables from Bottom	Dps = 0.1m
Eccentricity at Midspan	Emid = 0.528m
Approximate Prestressing Force	Pi = 4116.3 kN
Assumed Percentage of Prestress Loss	22%
Effective Prestressing Force	Pieff = 107.4
Number of Dia 15.2mm HTS G270)	40

(5) Summary

Based on that the following dimensions are obtained as the superstructure of Guadalupe Side bridge

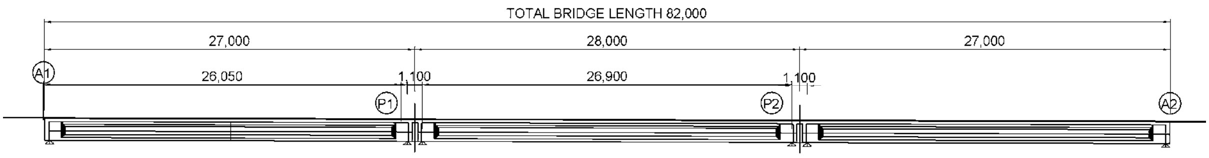


Figure 16.4.2-3 Side View of Superstructure of Palanit Bridge

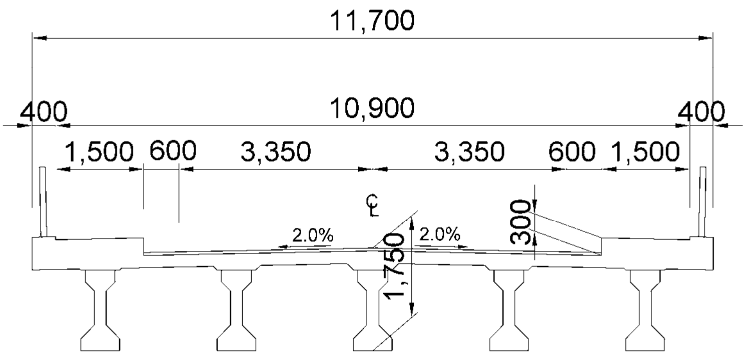


Figure 16.4.2-4 Sectional View of Superstructure of Palanit Bridge

16.4.3 Seismic Design

In this project, as seismic analysis, modal response spectrum analysis is conducted for seismic design. Based on the response results, various structural members can be determined such as piers, foundations, bearings and expansion joints. Analytical model to be utilized for modal analysis commonly utilizes truss and beam type elements in the world.
Based on the results of the outline design of superstructure such as member dimension and masses, analytical model and results of modal response spectrum analysis are explained in this item.

Besides, in this design, abutments are not modeled in the seismic analysis because abutments may have enough strength and stiffness fixed by grounds for seismic vibration; if abutments are modeled in the analysis, excess damping efficiency would be expected to the whole of structural responses.

(1) Analytical Model

- Seismic Analysis: Response Spectrum Analysis based on Modal Eigenvalue Analysis
- Superstructure Type: Continuous PC-I girder
- Bridge Length : L=82m
- Angle of Alignment: 90 Degrees
- Analytical Model :

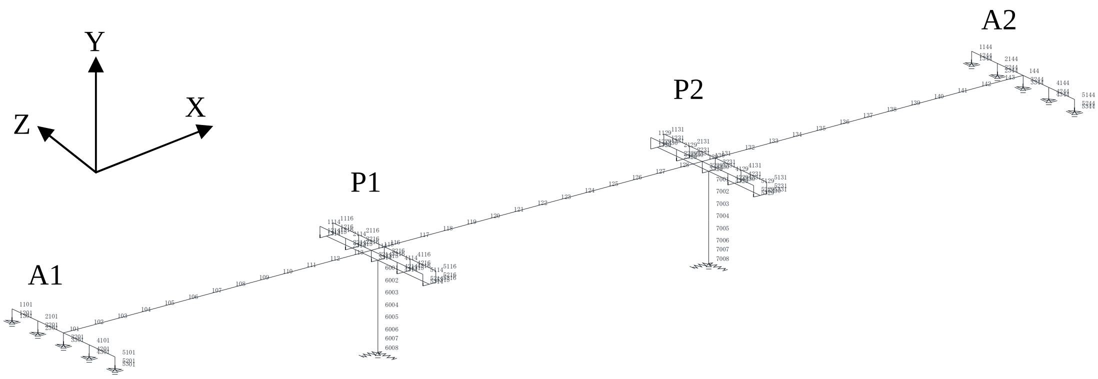


Figure 16.4.3-1 Analytical Mode of Seismic Analysis

Table 16.4.3-1 Support Condition

	X	Y	Z	RX	RY	RZ
Abutment 1	Elastic	Fix	Fix	Fix	Free	Free
Pier 1	Elastic	Fix	Elastic	Fix	Free	Free
Pier 2	Elastic	Fix	Elastic	Fix	Free	Free
Abutment 2	Elastic	Fix	Fix	Fix	Free	Free

- Abutments : Not Modeled
- Piers : Beam Type Elements for Circular Type Piers
- Bearing : Following Force Distribution Bearing:

Table 16.4.3-2 Force Distribution Bearing

Supports	Nos.	Rub. Dimension	Rub. Thickness	G
Abutment 1	5	320mmx320mm	10mmx5layers	1.4 N/mm2
Pier 1	5	320mmx320mm	10mmx5layers	1.4 N/mm2
Pier 2	5	320mmx320mm	10mmx5layers	1.4 N/mm2
Abutment 2	5	320mmx320mm	10mmx5layers	1.4 N/mm2

- Foundation : Following springs shall be :

Table 16.4.3-3 Springs of Foundations

Foundations	X: Longitudinal kN/m	Z: Transversal kN/m	RX kNm/rad	RZ kNm/rad
Pier 1	2.77×10^6	2.77×10^6	3.39×10^7	3.39×10^7
Pier 2	2.77×10^6	2.77×10^6	3.39×10^7	3.39×10^7

- Damping coefficient :Following damping coefficients are applied:

Table 16.4.3-4 Damping Coefficient

Structural Element	Damping
Concrete	0.02
Force Distribution Bearing	0.03
Foundation	0.10

(2) Comparison Studies of Seismic Capacity Improvement Schemes

In order to improve seismic capacity of this bridge, the following methodologies are applied based on technical comparison studies.

i) Application of Continuous Girder
In order to prevent bridge falling down and to reduce the number of bearings, expansion joint and to simplify related devices around pier top, continuous girders are generally applied for multiple span bridges in Japan. For Palanit bridge, AASHTO-IV girders are applied, for which continuous girder are applied for the superstructure based on connecting each girder at the piers. Therefore, this bridge can also meet such the improvement scheme.

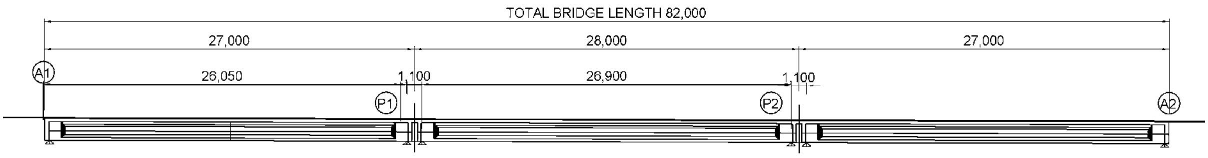


Figure 16.4.3-2 Application of Continuous Girder

ii) Adequate Bearing Type >

Force distribution method by laminated rubber bearings (LRB) shown in the following figure are commonly utilized in viaducts and bridges in Japan as efficient devices to achieve appropriate seismic design.

Fig. Laminated Rubber Bearing

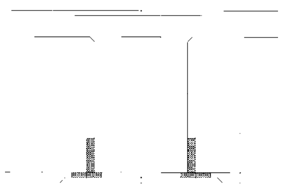
Fig. Steel Bearing

This bearing consists of rubber and steel plate layers. By changing the stiffness of the laminated rubber, such for thickness, number of layers and sizes, seismic horizontal forces can be freely and evenly shared to substructures. Therefore, the boundary condition between superstructure and substructure is "E" that means "elastic".

Otherwise, in Philippines, commonly thin-rubber bearing with anchor bars is utilized as bearing. By this bearing, only two ways of the boundary condition such as "Fix" or "Move" can be applied, which means that controlling of horizontal seismic forces or contribution forces to substructures depends on not horizontal stiffness of bearing but just only the period of its dynamic properties.

In this item, as an improvement scheme, technical comparison study between laminated rubber bearing, thin-rubber bearing with anchor bars and steel bearing is explained from the point of view of seismic behavior, shown as following table.

Table 16.4.3-5 Comparison Study of Bearing in Palanit Bridge

<table>
<tr><td>Bearing</td><td>Results of Evaluation</td></tr>
<tr><td>Laminated Rubber Bearing
Under Force Distribution Method
</td><td>Boundary Condition:
LD: Elastic (A1-P1-P2-A2), TD: Fix (A1,A2) Elastic (P1,P2)
Time Period
LD: 0.9S, TD: 0.6s
Modal Dumping of 1st mode
LD: 3%. TD: 1.6%
Total Horizontal Forces of Superstructure using Modal Dumping
LD: 10000kN of 15120kN, TD: 17000kN of 15120kN
Seismic Force Distribution
LD: A1:P1:P2:A2=1:2:2:1, TD: A1:P1:P2:A2=3:1:1:3</td></tr>
<tr><td>Pad Rubber Bearing with Dowel
Under Not Force Distribution
</td><td>Boundary Condition:
LD: Move (A1, A2), Fix (P1, P2), TD: Fix (A1-P1-P2-A2)
Time Period
LD: 0.7S, TD: 0.6s
Modal Dumping of 1st mode
LD: 1%. TD: 1%
Total Horizontal Forces of Superstructure using Modal Dumping
LD: 15500kN of 15120kN, TD: 17000kN of 15120kN
Seismic Force Distribution
LD: A1:P1:P2:A2=0:1:1:0,</td></tr>
<tr><td>Steel Bearing
Under Not Force Distribution
</td><td>Boundary Condition:
LD: Move (A1, A2), Fix (P1, P2), TD: Fix (A1-P1-P2-A2)
Time Period
LD: 0.7S, TD: 0.6s
Modal Dumping of 1st mode
LD: 1%. TD: 1%
Total Horizontal Forces of Superstructure using Modal Dumping
LD: 15500kN of 15120kN, TD: 17000kN of 15120kN
Seismic Force Distribution
LD: A1:P1:P2:A2=0:1:1:0,</td></tr>
<tr><td colspan="2">By using LRB, the period of transversal 1st mode achieves beyond 1.1s, which is much longer than the structure applying Pad Rubber bearing and Steel bearing.

Therefore, the seismic forces based on design spectrum are significantly reduced by extended period.

And also, dominant deformation of the 1st mode is obviously longitudinal deformation of superstructure, which caused by shearing deformation of the LRB; hence, the modal damping of 1st mode can achieve 3% despite just only 1%, modal damping, which is the damping factor of steel structure, of the structure using Pad Rubber bearing or Steel bearing.

Consequently the total horizontal forces of superstructure using LRB is greatly decreased comparing to the structure using common bearing due to extended period and higher structural mode dumping. Additionally, the seismic forces can be distributed evenly by LRB.
Therefore, structurally, superiority of application of LRB is extremely high.</td></tr>
</table>

* LD: Longitudinal Direction, TD: Transversal Direction

(3) Summary of Seismic Analysis

i) Results of Eigenvalue Analysis
The following figure and table shows the results of eigenvalue analysis.

Table 16.4.3-6 Results of Eigenvalue Analysis

Modes	Frequency (Hz)	Period (s)	Ratio of Effective Mass		Mode Damping
			Longitudinal	Transversal	
1	1.057	0.946	**0.731**	0.000	0.030
2	1.763	0.567	0.000	**0.615**	0.016
3	5.974	0.167	0.000	0.000	0.011
4	7.362	0.136	0.000	0.108	0.044
5	7.421	0.135	0.000	0.000	0.043
6	8.985	0.111	0.000	0.000	0.049
7	9.698	0.103	0.173	0.000	0.054
8	12.640	0.079	0.000	0.000	0.010
9	13.349	0.075	0.000	0.063	0.010
10	19.839	0.050	0.000	0.141	0.088

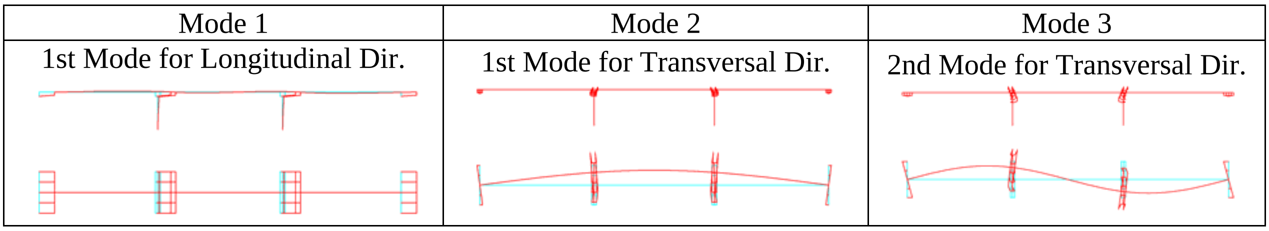


Figure 16.4.3-3 Results of Eigenvalue Analysis

According to the results, predominant mode for longitudinal direction is obviously obtained at 1st mode whose period is 0.94s and effective mass ration is 73% of modes for longitudinal direction. Therefore, the 1st mode for longitudinal direction is so important one, which have enough effective mass ratio. And for the 1st mode for transversal direction is the mode with the effective mass ratio of 0.615s and with the period of 0.57s for transversal direction. Both of the behaviors of longitudinal and transversal direction are efficiently functioned against strong seismic forces, using Force Distribution Bearings and appropriate dumping coefficient of them.

ii) Response Displacement by Response Spectrum Analysis (EQ)
The following table shows the response displacement of relative displacements between substructure and superstructure.

Table 16.4.3-7 Relative Displacement between Substructure and Superstructure

Locationt	Longitudinal (mm)	Transversal (mm)
Abutment 1	147	0.00
Abutment 2	147	0.00

The longitudinal displacements are well converged in realistic scale, for which common expansion joints can be applied.

(4) Seismic Design of Substructure and Foundation

i) Ground Surface in Seismic Design

The following figure shows the ground surface in seismic design.

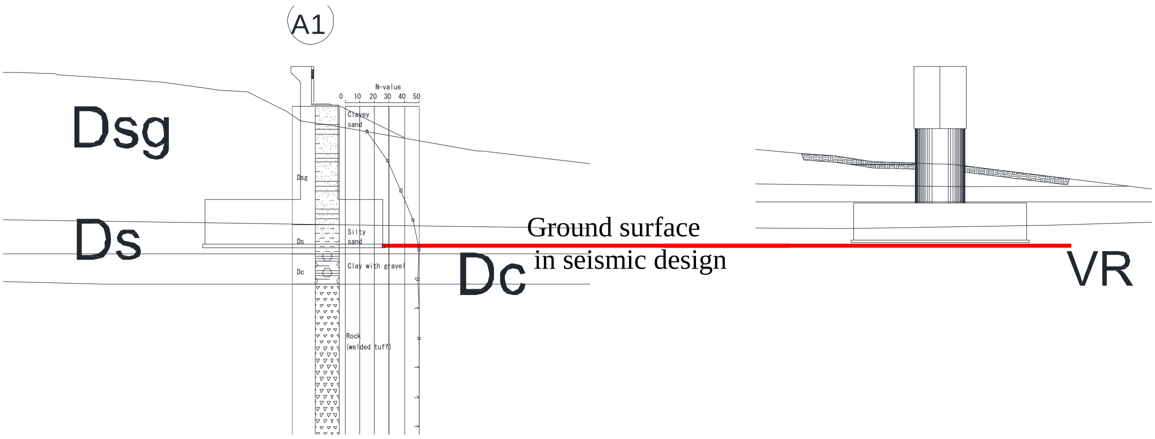


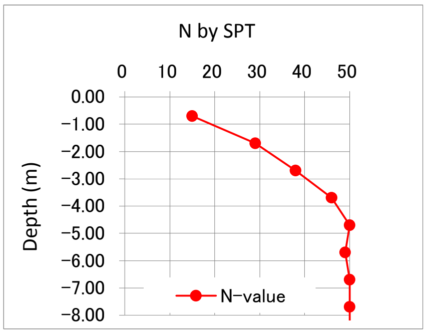
Figure 16.4.3-4 Ground Surface of Abutment & Pier in Seismic Design

ii) Assessment of Soil Liquefaction

According to the design specifications, sandy layer requiring liquefaction Assessment is obviously obtained as following table.

Table 16.4.3-8 Assessment of Soil Liquefaction

Assessment of Liquefaction Potential								
GL-(m)	Soil Layers	N by SPT	Ground Water Level (-m)	Fc (%)	PI	D50 (mm)	D10 (mm)	Liquefiable
		<30		<35%	<15	<10mm	<1mm	
0.70	Dsg	15	3.00	3.2	43	0.49	0.12	○
1.70	Dsg	29	3.00	3.2		1.27	0.14	○
2.70	Dsg	38	3.00	5.0		0.27	0.09	
3.70	Dsg	46	3.00	4.2		0.36	0.10	
4.70	Ds	50	3.00	2.8		0.49	0.13	
5.70	Dc	49	3.00	51.7				
6.70	VR	50	3.00					
7.70	VR	50	3.00					



Based on the results of liquefaction assessment, reduction of geotechnical parameters shall be conducted in accordance with the following tables.

Table 16.4.3-9 Assessment of Soil Liquefaction Parameters

Basic Soil Profile Information									
GL-(m)	Soil Layers	N by SPT	Fc (%)	γt γt1	Water unit weight	Ground Water Level (-m)	σU (Kpa)	σv (Kpa)	σv' (Kpa)
0.70	Dsg	15	3.2	18	10.00	3.00	0.00	12.60	12.60
1.70	Dsg	29	3.2	18	10.00	3.00	0.00	30.60	30.60

Table 16.4.3-10 Results on Liquefaction Resistance Factor (FL) & Reduction Factor (D_E)

Calculation for FL								Reduction Factor D_E		
Depth	N1	C1	C2	Na	R	L	FL	**R(Ave.)**	**FL(Ave.)**	**D_E**
-0.70	30.87	1.000	0.000	30.872	0.908	0.623	1.457	**7.799**	**12.692**	**1.00**
-1.70	49.01	1.000	0.000	49.006	14.689	0.614	23.926			

iii) Design Loads

Based on the results of seismic analysis, the abutment design is conducted for the following load combinations.

- Results of Eigenvalue Analysis

VERTICAL REACTIONS FOR SUBSTRUCTURE STABLE CALCULATION (KN)

	Dead load	Live load	Sum.
A1	2,520	690	3,210
A2	2,520	690	3,210

Note: Impact factor exclusive

HORISONTAL/ TRANSVERSAL REACTIONS BY RESPONSE SPECTRUM ANALYSIS UNDER L2 (at bearing)

	H(KN)	M(KNm)	V(KN)
A1 (E)	1,780	-	-
A2 (E)	1,780	-	-

Note:

- Design Combination Loads

LONGITUDINAL DIRECTION(at bearing)

	DL		LL		EQ			SUM		
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	N (kN)	H (kN)	M (kNm)
A1（Nmax)	2,520	1.25	690	0.50	1,780	-	1.00	**3,500**	**1,780**	-
（Nmax)	2,520	0.90	690	0.50	1,780	-	1.00	**2,620**	**1,780**	-
A2（Nmax)	2,520	1.25	690	0.50	1,780	-	1.00	**3,500**	**1,780**	-
（Nmax)	2,520	0.90	690	0.50	1,780	-	1.00	**2,620**	**1,780**	-

- Results of Eigenvalue Analysis

VERTICAL REACTIONS FOR SUBSTRUCTURE STABLE CALCULATION (KN)

	Dead load	Live load	Sum.
P1	5,040	1,550	6,590
P2	5,040	1,550	6,590

Note: Impact factor exclusive

SECTIONAL FORCES BY RESPONSE SPECTRUM ANALYSIS FOR PIERS (at bottom of Piers)

	Longitudinal Direction			Transversal Direction		
	H(KN)	M(KNm)	V(KN)	H(KN)	M(KNm)	V(KN)
P1 (E)	3,830	11,620	0	4,470	17,600	0
P2 (E)	3,830	11,620	0	4,470	17,600	0

- Dead Loas (for Pier)

	1) Cross Beam					2) Circular Column				1)+2)
	Beam Width	Beam Thicknes	Beam Height	Beam Height	Cross Aria	Diameter	Cross Aria	Column Heigh	Unit Weight	Self Weight
	(m)	(m)	(m)	(m)	(m2)	(m)	(m2)	(m)	(kN/m3)	(kN)
P1	11.6	2.0	2.0	1.2	19.36	2.0	3.1	4.0	24.5	1,257
P2	11.6	2.0	2.0	1.2	19.36	2.0	3.1	4.0	24.5	1,257

- Design Combination Loads

LONGITUDINAL DIRECTION (at bottom of column)

	DL		LL		EQ			**SUM of LONGITUDINAL**		
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	**N (kN)**	**H (kN)**	**M (kNm)**
P1(Nmax)	6,297	1.25	1,550	0.50	3,830	11,620	1.00	**8,650**	**3,830**	**11,620**
(Nmin)	6,297	0.90	1,550	0.50	3,830	11,620	1.00	**6,450**	**3,830**	**11,620**
P2(Nmax)	6,297	1.25	1,550	0.50	3,830	11,620	1.00	**8,650**	**3,830**	**11,620**
(Nmin)	6,297	0.90	1,550	0.50	3,830	11,620	1.00	**6,450**	**3,830**	**11,620**

TRANSVERSAL DIRECTION (at bottom of column)

	DL		LL		EQ			**SUM of TRANSVERSAL**		
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	**N (kN)**	**H (kN)**	**M (kNm)**
P1(Nmax)	6,297	1.25	1,550	0.50	4,470	17,600	1.00	**8,650**	**4,470**	**17,600**
(Nmin)	6,297	0.90	1,550	0.50	4,470	17,600	1.00	**6,450**	**4,470**	**17,600**
P2(Nmax)	6,297	1.25	1,550	0.50	4,470	17,600	1.00	**8,650**	**4,470**	**17,600**
(Nmin)	6,297	0.90	1,550	0.50	4,470	17,600	1.00	**6,450**	**4,470**	**17,600**

iv) Design Result

Based on that the following dimensions are obtained as the abutment with foundation of Lambingan bridge.

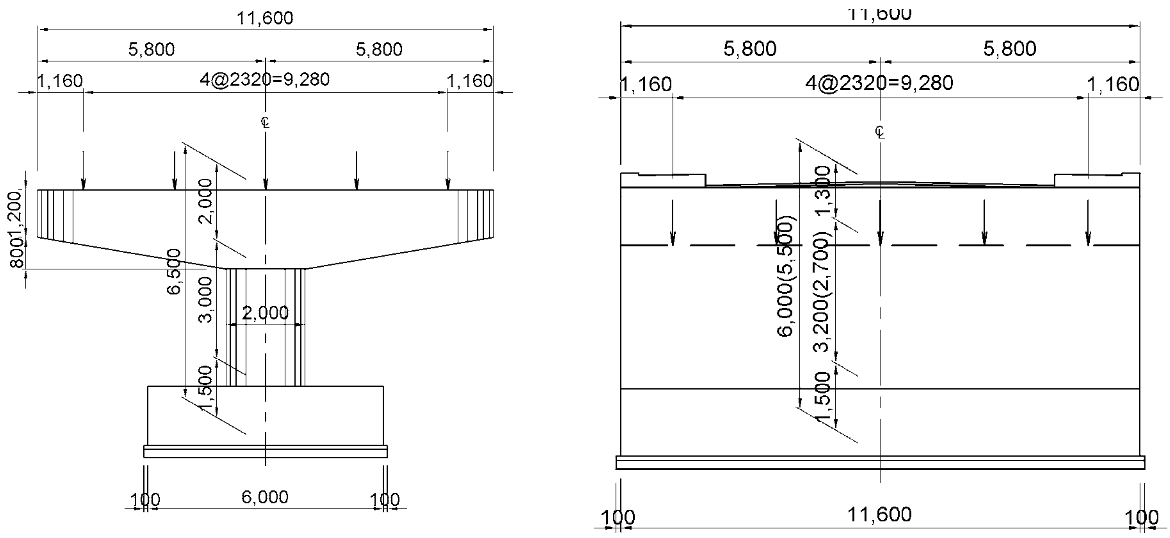


Figure 16.4.3-5 Sectional View of Pier & Abutment of Palanit Bridge

(5) Unseating Prevention System

The superstructure is generally connected to the substructure through bearings. As such, the superstructure and the substructure are separated functionally and significantly critical state such as bridge falling down may be caused due to large relative displacements between them, in case of failure of bearings under unexpected seismic forces.

For a functional system preventing such severe state, detailed philosophy and articulate design concepts are explicitly specified in JRA as “Unseating Prevention System” based on accumulated data and experiences from large number of seismic damages. The aim is to provide multiple mechanisms that can complement each other efficiently, shown as follows.

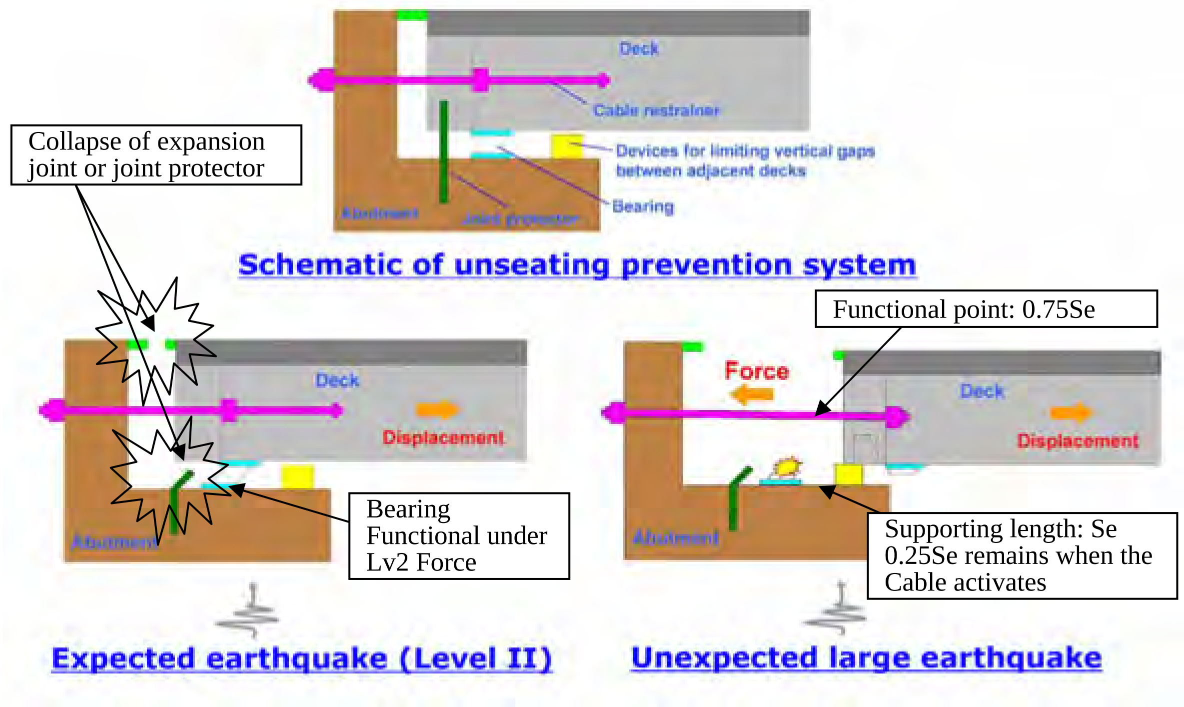


Figure 16.4.3-6 Philosophy of Unseating Prevention System in JRA

The Unseating Prevention System consists as following devices or functions:

Table 16.4.3-11 Devices and Functions of Unseating Prevention System

Devices	Function
Bearing (Type B)	"Type B" bearing in JRA, enables to resist the seismic forces of LV2 by itself.
Supporting Length	The final function of the unseating prevention system. The equation to determine the length is given as follows: Se=μr+μg : μr: Relative responses of girder, μg: Displacement of ground Minimum length: Semin=0.7+0.005l (m)
Longitudinal Restrainer	Design forces are given as 1.5Rd, where Rd is the reaction force of dead loads Maximum movable length: 0.75Se, where Se is supporting length. Generally for new bridges in Japan, cable type devices connecting between parapet of abutment and cross beam of superstructure are commonly utilized.
Expansion Joint	Expansion joint shall resist LV1 seismic forces, otherwise, the capability to resist LV2 seismic forces is not required.

Based on the philosophy and functions above, specification of each devices applied in this new bridge is shown as the results of outline design.

i) Bearing
For abutment of Palanit bridge, following force distributing bearing is obviously advantageous for seismic behavior.

Table 16.4.3-12 Force Distribution Bearing

Supports	Nos.	Dimension	Thickness	G
Abutment 1	5	320mmx320mm	10mmx5layers	1.4 N/mm2
Abutment 2	5	320mmx320mm	10mmx5layers	1.4 N/mm2

From the point of view of the philosophy of unseating prevention system, those bearing shall be resisted the LV2 seismic forces. As a part of outline design, following outline verification is conducted to clarify they can resist such the forces.

Table 16.4.3-13 Outline Verification of Bearing under LV2 Seismic Forces

		Value/ LV2	Allowance	Judge
Shear Strain	Longitudinal Dir.	1.5	2.5	OK
	Transversal Dir.	0.0	2.5	OK

ii) Supporting Length

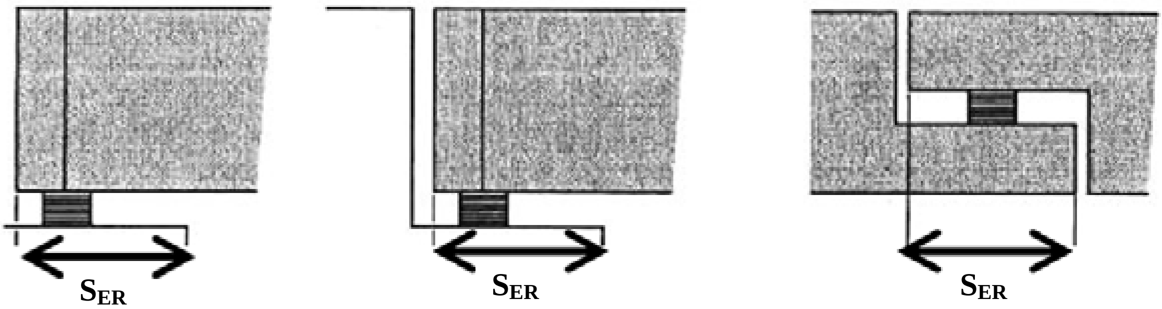


Figure 16.4.3-7 Supporting Length

Following equation gives the supporting length.
Se=0.7+0.005l (m) l: Span length
Se = 0.7+0.005*27 = 0.84m

Therefore, the length of Se should be secured over 85cm.

iii) Longitudinal Restrainer
The reaction force by dead loads is 2520 kN.
The following verification can be obtained.

Table 16.4.3-14 Verification of Longitudinal Restrainer

	1.5Rd: Design Forces	Allowance
PC Cable Type 7 x ϕ 12.4mm 4-nos/ Abutment	945 kN	952 kN

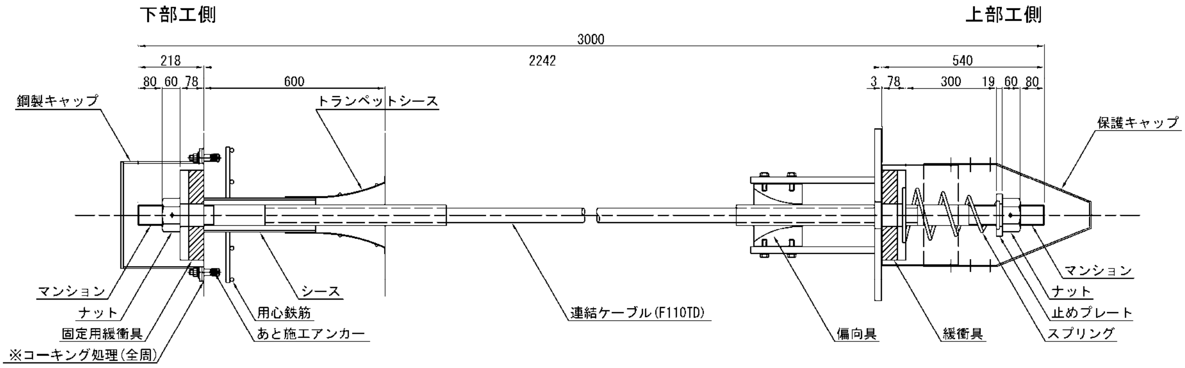


Figure 16.4.3-8 Longitudinal Restrainer for Palanit Bridge

(6) Miscellaneous devices and others

Miscellaneous devices in the bridge are defined as following items:

- Bearing: Evaluated above clause
- Expansion joint
- Drainage
- Wearing coat

In this clause, the devices which are not explained in other clause are explained based on seismic behaviors and current bridge condition.

i) Expansion joint

For the design methodology of expansion joint, its seismic capacity shall be secured under LV1 forces and it does not have to be secured under LV2 forces. The expansion gap between girder end and abutment shall be determined from results of dynamic modal analysis under LV2 and another expansion gap of expansion joint itself shall be determined based on seismic analysis of LV1.

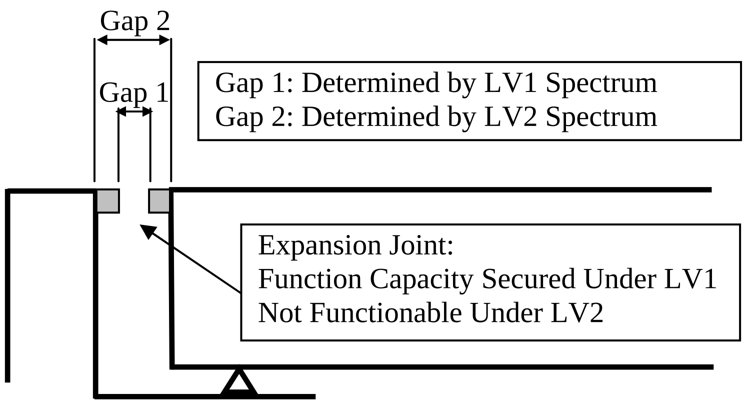


Figure 16.4.3-9 Design Methodology of Expansion Joint

However, expansion gap using laminated rubber bearing generally tends to be larger than common bearing and the size of the expansion joint tends to be larger and more expensive. Therefore, the both of expansion gap especially the gap between girder end and abutment should be carefully pay attention to displacement controlling during dynamic modal analysis, evaluating the size of expansion joint. When the gaps were so large comparing to general behavior, the stiffness of rubber bearing should be adjusted and should try the modal analysis again.

In this project, on the basis of above consideration, appropriate modal analysis are carried out, controlling caused displacements based on evaluation of stiffness of rubber bearing.

The final displacements to be used for determination of expansion joint are as follows.

- LV1: Gap 1: 10.3cm + 1.5cm (Excess allowance 15mm (JRA) ≒ 12cm
- LV2: Gap 2: 14.7cm + 1.5cm (Excess allowance 15mm (JRA) ≒ 20cm

Therefore, the expandable gap of the joint in this bridge shall be 12cm or more, and the gap between girder and abutment shall be 20cm or more, which are common results achieved under careful controlling in the dynamic analysis. Consequently, general steel type expansion joint can be adequately applied to this bridge.

ii) Drainage
Drainage system on the bridge is estimated based on current condition. In the next stage such as basic design or design stage, appropriate location of catch basins and drainage pipes shall be designed and drawn based on further investigation of accumulated rainfall data of corresponding area.

iii) Wearing coat
Palanit bridge will have concrete slab deck on the girder. Therefore, usual asphalt concrete can be applied as follows.

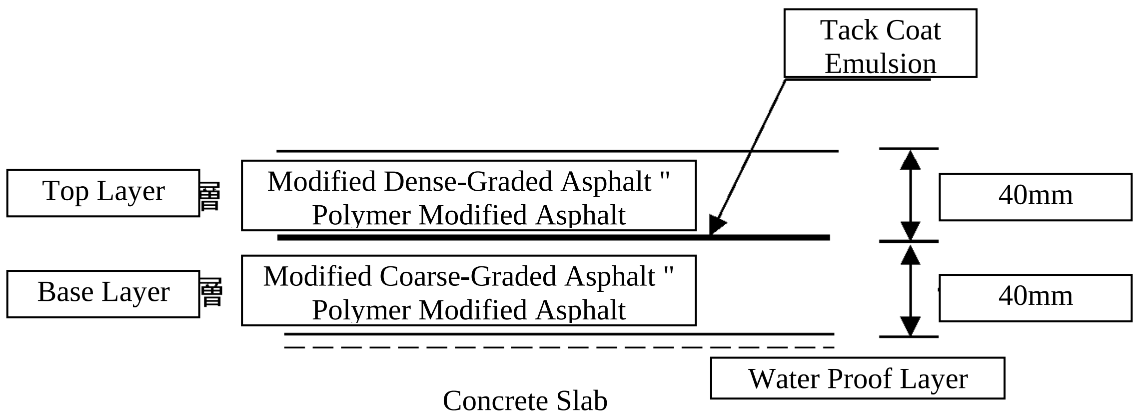


Figure 16.4.3-10 Wearing Coat System of Concrete Slab

16.4.4 Summary of Outline Design Results

(1) Superstructure

The bridge type is PC-I girder bridge, and laminated rubber bearing considering 3% of damping coefficient in dynamic modal analysis is applied. As the girder of superstructure, AASHTO Type IV girders are applied based on AASHTO LRFD design specification.

(2) Substructure and Foundation

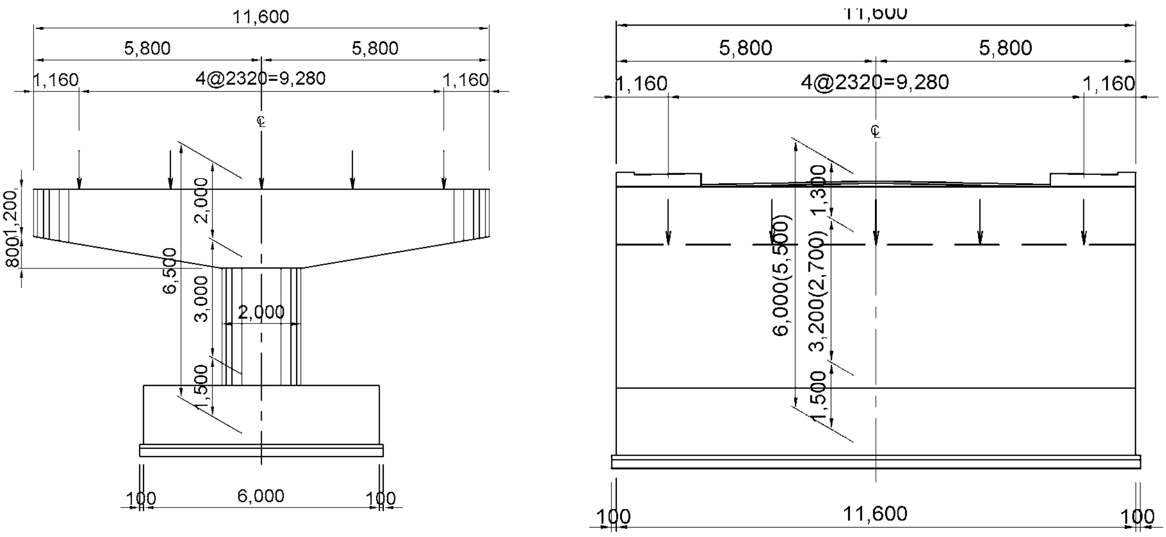


Figure 16.4.4-1 Sectional View of Pier & Abutment of Palanit Bridge

(3) Further Verification to be Examined in the Next Phase

The following items may be necessary to be verified or evaluated further in the next phase such as basic or detail design stages.

- Optimization and re-verification of bridge length, span arrangement and bridge types, on the basis of latest existing road condition, newly future planning and detail river condition resulted by detail hydraulic analysis
- Utilization of high-damping bearing based on specific organization regarding non-linear time history response analysis

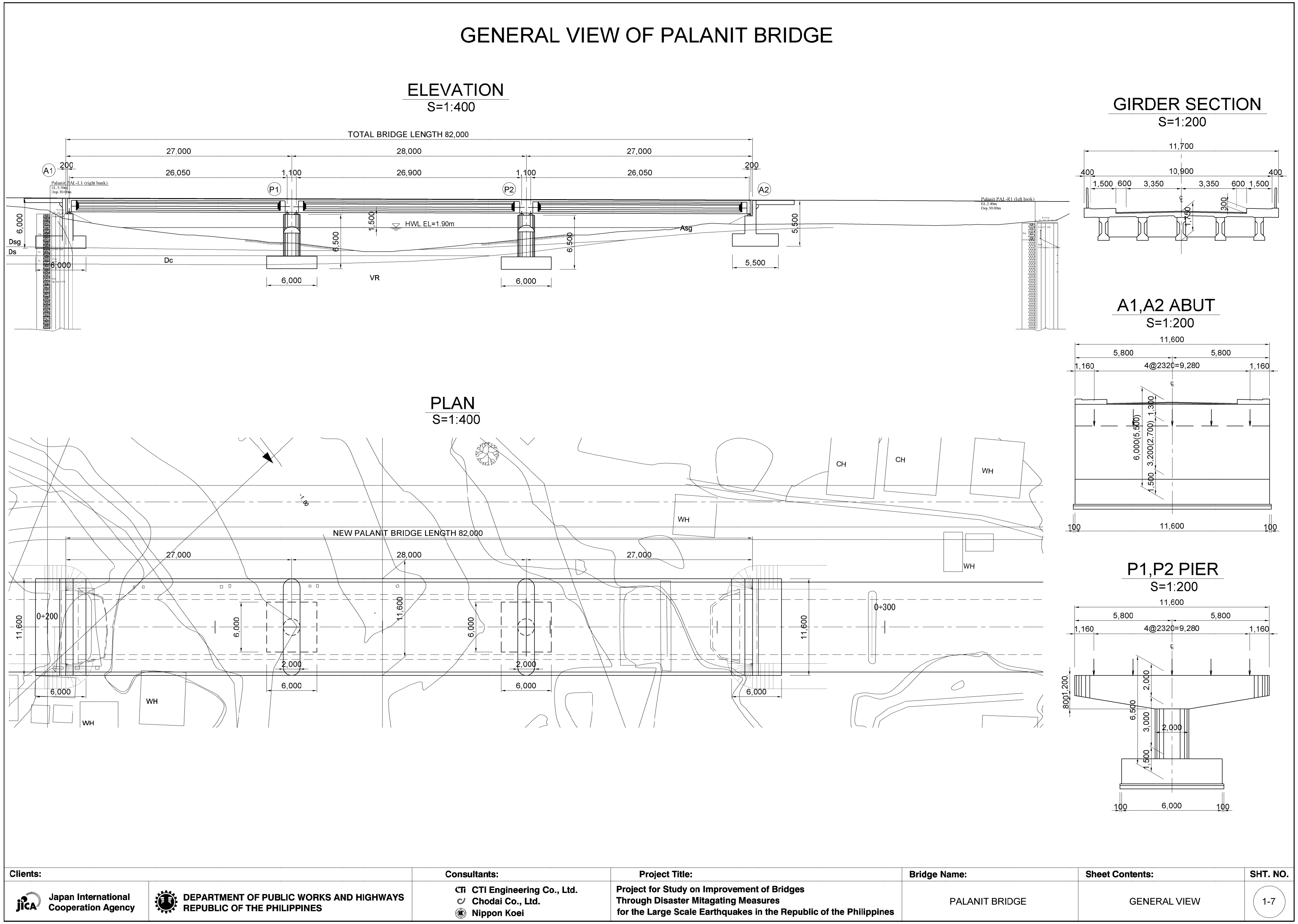


Figure 16.4.4-2 General View

(Structural Drawings are shown in Appendix-4)

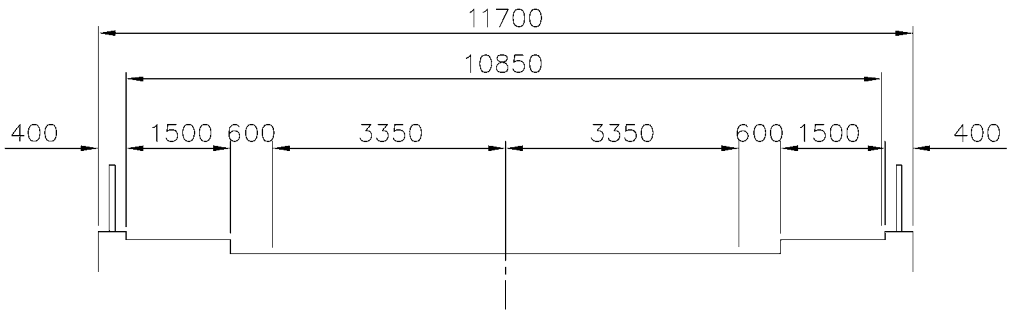
16.5 Outline Design of Mawo Bridge

16.5.1 Design Condition

The following items show design condition for the outline design of Mawo Bridge.

(1) Road Conditions

- Road Design Standard : AASHTO STANDARD VALUE
- Design Speed : V = 60 kmph
- Live Loads : AASHTO Live Loads HL93 and Lane Loads
- Road Width : Shown as follows:



Note: Basically, 400mm width of the end curb can be applied to the road condition at Mawo bridge; however, for finback bridge, wider width of the curb shall be secured not to affect the fin-back-shaped structural members in case of vehicle collision.

Figure 16.5.1-1 Cross Section/ Lane Arrangement of Mawo Bridge

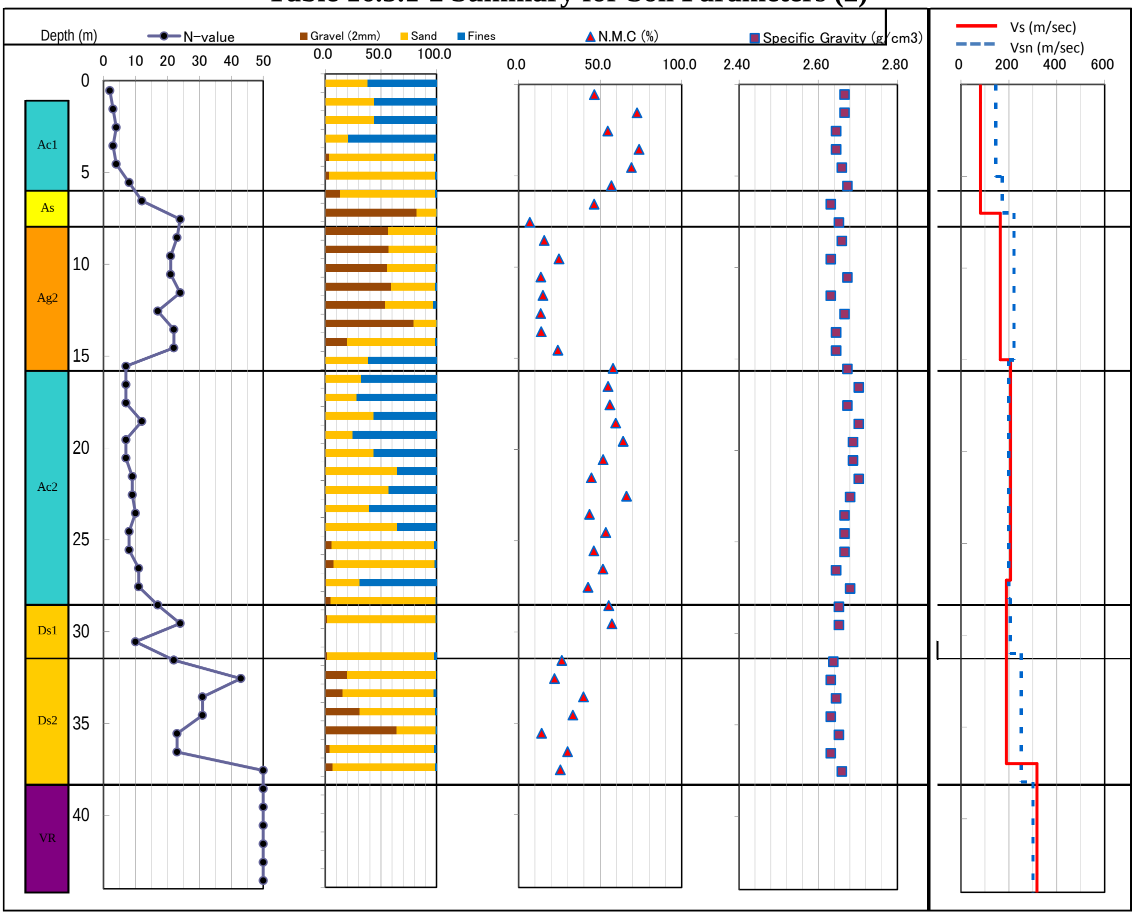
(2) Soil Conditions

The results of ground investigation are shown in below illustrations and following tables. The weathered rock layer that can be regarded as the bearing layer is distributed E.L. -8.0m to E.L.-38.0m depth, and has a thick surface layer predominant with clay with sand on top. Specialty, liquefiable sand (Ag) is thickly deposited from ground surface to GL-15m, of which N-value is 8 to 24, will be affected by liquefaction occurs with reduction of geotechnical parameter. .

Table 16.5.1-1 Summary for Soil Parameters (1)

- A2 Side - Mawo MAW-L1 (right bank) EL.1.20m															Soil Parameters						Layer Thickness (m)
Number	Depth Upper	SPT Blows	S-wave	Vsn	Soil Layer	Soil Classification Observation	GSA Gravel (%)	GSA Sand (%)	GSA Fines (%)	N.M.C (%)	PI	Specific Gravity	D50≦ 10mm	D10≦ 1mm	Nd	γt (tf/m^2)	C (kN/m^2)	φ (°)	E0 (kN/m^2)	Vsn (m/sec)	
1	0.55	2	81	144	Ac1	Clay	0.0	37.7	62.3	46.4	16	2.67	N/A	N/A	3	14	19	0	2,100	144	5
2	1.55	3	81	144	Ac1	Clay	0.0	43.7	56.3	72.6	2	2.67	N/A	N/A		14					
3	2.55	4	81	144	Ac1	Silty clay	0.0	43.7	56.3	54.7	2	2.65	N/A	N/A		14					
4	3.55	3	81	144	Ac1	Claye silt	0.0	20.3	79.7	73.8	8	2.65	N/A	N/A		14					
5	4.55	4	81	144	Ac1	Silty v.f. sand	3.1	94.7	2.2	69.1	N/A	2.66	0.3	0.1		14					
6	5.55	8	81	172	As	Silty fine sand	3.2	95.3	1.5	56.9	N/A	2.67	0.2	0.1	10	17	0	34	7,000	172	2
7	6.55	12	81	172	As	Fine sand with silt	13.2	85.3	1.5	46.3	N/A	2.63	0.6	0.1		17					
8	7.55	24	164	221	Ag2	Gravel w/ silt	82.1	17.7	0.2	6.9	N/A	2.65	12.6	0.5	21	18	0	36	14,700	221	8
9	8.55	23	164	221	Ag2	Gravel w/ silt	56.3	43.1	0.6	15.7	N/A	2.66	2.8	0.2		18					
10	9.55	21	164	221	Ag2	Gravel	56.7	43.1	0.2	24.7	N/A	2.63	2.6	0.8		18					
11	10.55	21	164	221	Ag2	Sandy gravel	55.6	43.3	1.1	13.7	N/A	2.67	4.6	0.2		18					
12	11.55	24	164	221	Ag2	Gravel with sand	59.0	39.7	1.3	14.9	N/A	2.63	3.2	0.2		18					
13	12.55	17	164	221	Ag2	Gravel with silt	53.7	43.2	3.1	13.5	N/A	2.67	2.8	0.1		18					
14	13.55	22	164	221	Ag2	Gravel with silt	79.6	20.2	0.2	13.9	N/A	2.65	10.3	0.5		18					
15	14.55	22	164	221	Ag2	Corse sand with gravel	19.4	79.0	1.6	24.1	N/A	2.65	0.9	0.2		18					
16	15.55	7	207	200	Ac2	Silty clay	0.0	38.2	61.8	58.0	12	2.67	N/A	N/A	8	18	50	0	5,600	200	13
17	16.55	7	207	200	Ac2	Sandy silt	0.0	32.0	68.0	54.9	9	2.70	N/A	N/A		18					
18	17.55	7	207	200	Ac2	Clayey silt	0.0	27.9	72.1	56.0	4	2.67	N/A	N/A		18					
19	18.55	12	207	200	Ac2	Sandy clay	0.0	43.4	56.6	59.6	2	2.70	N/A	N/A		18					
20	19.55	7	207	200	Ac2	Clay with sand	0.0	24.3	75.7	64.1	4	2.69	N/A	N/A		18					
21	20.55	7	207	200	Ac2	Clay with sand	0.0	43.4	56.6	51.9	5	2.69	N/A	N/A		18					
22	21.55	9	207	200	Ac2	Clayey sand	0.0	64.4	35.6	44.6	4	2.70	0.3	N/A		18					
23	22.55	9	207	200	Ac2	Clay with sand	0.0	56.6	43.4	66.2	4	2.68	0.1	N/A		18					
24	23.55	10	207	200	Ac2	Clay with sand	0.0	39.5	60.5	43.5	4	2.67	N/A	N/A		18					
25	24.55	8	207	200	Ac2	Sandy silt	0.0	64.7	35.3	53.4	N/A	2.67	0.1	N/A		18					
26	25.55	8	207	200	Ac2	Clay with sand	5.5	92.1	2.4	46.1	N/A	2.67	0.4	0.1		18					
27	26.55	11	207	200	Ac2	Clay with sand	7.3	91.1	1.6	51.8	N/A	2.65	0.3	0.1		18					
28	27.55	11	190	200	Ac2	Sandy clay	0.7	30.0	69.3	42.6	12	2.68	N/A	N/A		18					
29	28.55	17	190	206	Ds1	Sand with clay	4.7	94.4	0.9	55.3	N/A	2.65	0.3	0.1	17	17	0	32	11,900	206	3
30	29.55	24	190	206	Ds1	Fine to med. sand	1.4	97.8	0.8	57.3	N/A	2.65	0.3	0.1		17					
31	30.55	10	190	206	Ds1											17					
32	31.55	22	190	251	Ds2	Fine to med. sand	1.4	96.3	2.3	26.5	N/A	2.64	0.4	0.1164	31	19	0	34	21,700	251	7
33	32.55	43	190	251	Ds2	Fine to med. sand	19.4	80.2	0.4	22.0	N/A	2.63	0.7	0.2033		19					
34	33.55	31	190	251	Ds2	Sand with gravel	15.2	82.3	2.5	39.8	N/A	2.65	0.5	0.1219		19					
35	34.55	31	190	251	Ds2	Gravelly sand	30.7	67.9	1.4	33.3	N/A	2.63	0.8	0.1614		19					
36	35.55	23	190	251	Ds2	Silty grravel	64.2	34.6	1.2	14.1	N/A	2.65	11.3	0.2151		19					
37	36.55	23	190	251	Ds2	Silty sand with gravel	3.3	94.5	2.2	30.0	N/A	2.63	0.5	0.1225		19					
38	37.55	50	317	251	Ds2	Med. sand with gravel	6.3	92.2	1.5	25.6	N/A	2.66	0.4	0.1348		19					
39	38.55	50	317	300	VR	Rock									300	21	170	38	136,098	300	6
40	39.55	50	317	300	VR	Rock										21					
41	40.55	50	317	300	VR	Rock										21					
42	41.55	50	317	300	VR	Rock										21					
43	42.55	50	317	300	VR	Rock										21					
44	43.55	50	317	300	VR	Rock										21					

Table 16.5.1-2 Summary for Soil Parameters (2)



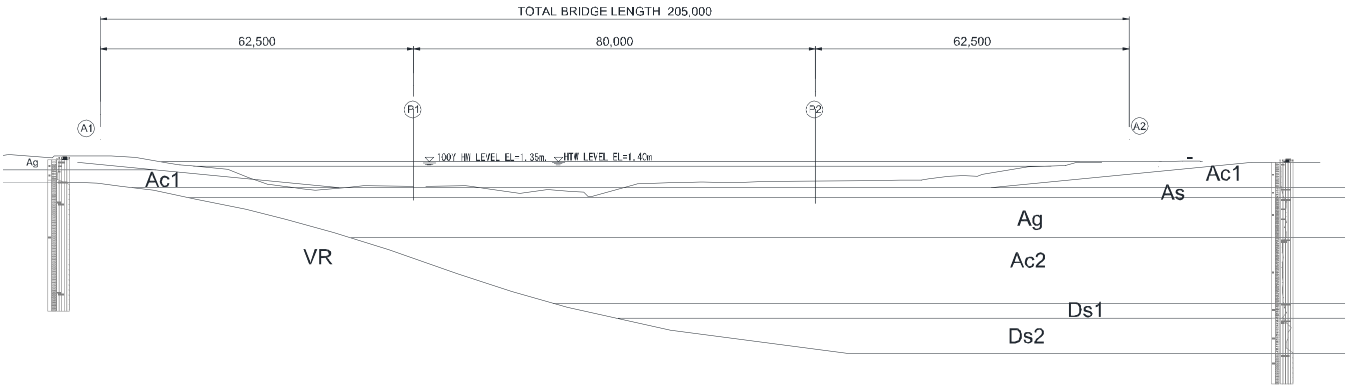


Figure 16.5.1-2 Soil Profile of Mawo Bridge (Included previous SPT)

(3) Hydraulic Conditions

- Design Flood Discharge : Q = 1,245m3/s
- Design Water Level : EL= 1.40 m
- Freeboard from Design Flood Level : H = 1.50 m

Note: Hydraulic condition shall be re-evaluated based on detail hydraulic analysis and site interview in the next stage such as detail design stage.

Around this area, the water flows is comparatively complicated condition hydraulically. And the results of hydraulic analysis will affect critically the results of bridge planning. Therefore, in detail design stage, detail hydraulic analysis based upon further investigation shall be implemented by river/ hydraulic engineers. Based on the detail examination, the free board to be secured, level of high water level, abutment location and reevaluation of comparison study may be preferable to be re-implemented in the detail design stage.

(4) Bridge Type

- Superstructure Type : 3-Span Continuous PC Fin-Back Box Girder
- Bridge Length : L=205m
- Span Arrangement : 62.5m + 80.0m + 62.5m
- Transversal Slope : 2.0%
- Horizontal Alignment : R=∞
- Angle of Alignment : 90 Degrees
- Wearing coat : Polymer Modified Asphalt, Coarse/ Dense-Graded t=80mm for Vehicle lane, t=30mm for walkway
- Railing : Steel railing for vehicle and pedestrian
- Bearing : NRB Rubber Bearings - Force Distribution Bearing
- Expansion Joint : Steel type
- Drainage Appliances : PVC pipe
- Bridge Falling Prevention Device: Cable type

- Substructure Type : Wall Type
- Foundation Type : Cast-in-place Pile
- Bearing Soil Condition : Rock (N>50)

(5) Design Cases of Outline Design

The outline design of superstructure shall be designed based on the above load condition, specified in AASHTO 2012. On the basis of various reactions and forces, substructures and foundation shall be designed throughout response spectrum analysis under the limit state of "Extreme Event I" specified in AASHTO 2012.

STEP 1. Outline Design of Superstructure
- Design Load, specified in AASHTO 2012
- Stress check

↓

STEP 2. Organization of Reaction Forces
- Organization of reaction forces for substructure design
- Organization of masses for response spectrum analysis

↓

STEP 3. Modal Analysis and Response Spectrum Analysis
- Eigenvalue analysis based on modal analysis
- Organization of response values by the seismic analysis

↓

STEP 4. Substructure and Foundation Design
- Extreme Event I

Figure 16.5.1-3 Flow of Outline Design

16.5.2 Outline Design of Superstructure

(1) Design Condition

- ➢ Superstructure Type : Continuous PC Fin-Back Box Girder
- ➢ Bridge Length : L=205m
- ➢ Angle of Alignment : 90 Degrees
- ➢ Wearing coat : Polymer Modified Asphalt, Coarse/ Dense-Graded
  t=80mm for Vehicle lane, t=30mm for walkway
- ➢ Railing : Steel railing for vehicle and pedestrian
- ➢ Bearing : NRB Rubber Bearings - Force Distribution Bearing
- ➢ Road Width : Shown as follows:

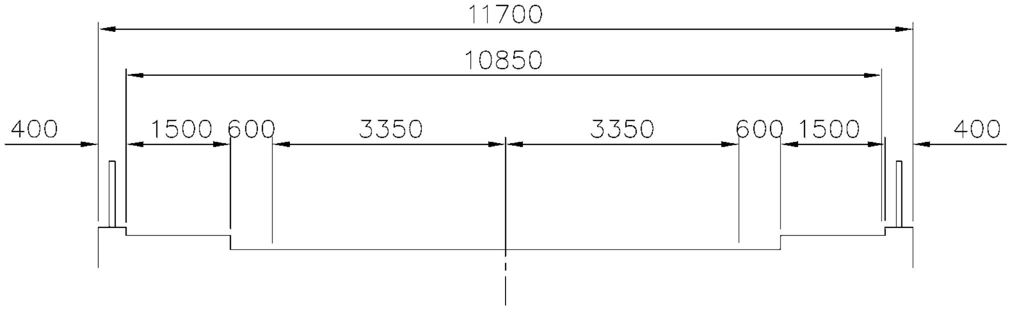


Figure 16.5.2-1 Cross Section/ Lane Arrangement of Mawo Bridge

(2) Design Loads

- Dead Loads : AASHTO 2012
- Live Loads : HL93 and Lane Loads in AASHTO 2012, utilized by influence line evaluation

(3) Design Philosophy

Concrete box girder will be casted in place with cantilever method. Side span will be constructed using scaffolding. The cantilever shall be erected with acceptable support condition, fixed by temporary PC bars at interior supports. As prestressing system and cables, 12S15.2mm (SWPR7BL) is applied in this bridge.

(4) Summary

Based on that the following dimensions are obtained as the superstructure of Mawo Side bridge

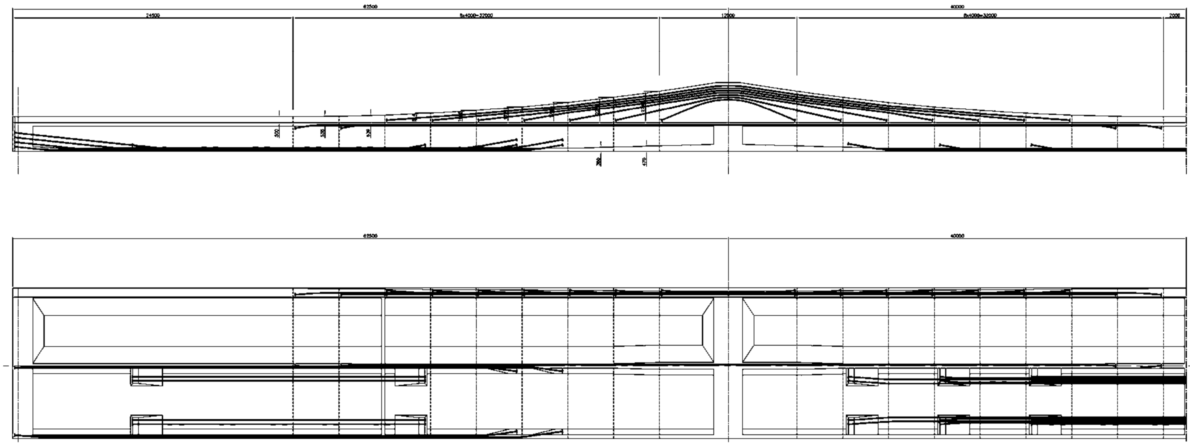


Figure 16.5.2-2 Side View and PC Cable Arrangement of Superstructure of Mawo Bridge

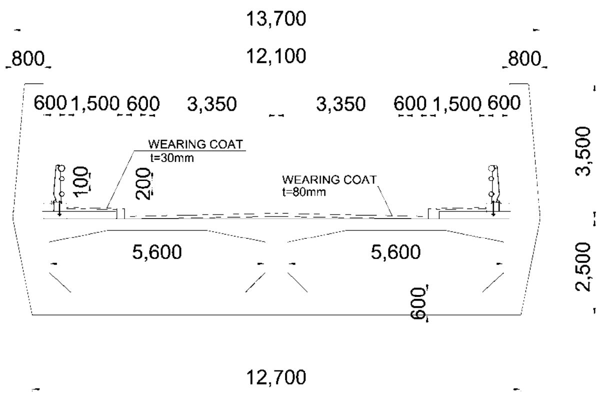


Figure 16.5.2-3 Sectional View of Superstructure of Mawo Side Bridge

Table 16.5.2-1 Reaction Forces of Superstructure

	Dead load	Live load	Sum.
A1	8350	1700	10050
P1	33900	3800	37700
P2	33900	3800	37700
A2	8350	1700	10050

16.5.3 Seismic Design

In this project, as seismic analysis, modal response spectrum analysis is conducted for seismic design. Based on the response results, various structural members can be determined such as piers, foundations, bearings and expansion joints. Analytical model to be utilized for modal analysis commonly utilizes truss and beam type elements in the world.
Based on the results of the outline design of superstructure such as member dimension and mass, analytical model and results of modal response spectrum analysis are explained in this item.

Besides, in this design, abutments are not modeled in the seismic analysis because abutments may have enough strength and stiffness fixed by grounds for seismic vibration; if abutments are modeled in the analysis, excess damping efficiency would be expected to the whole of structural responses.

(1) Analytical Model

- Seismic Analysis: Response Spectrum Analysis based on Modal Eigenvalue Analysis
- Superstructure Type: PC Fin-Back Box Girder
- Bridge Length : L=205m
- Angle of Alignment: 90 Degrees
- Analytical Model :

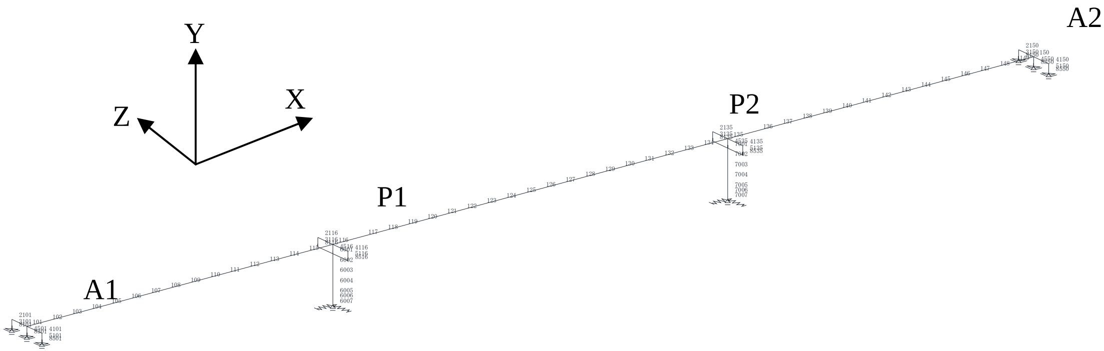


Figure 16.5.3-1 Analytical Mode of Seismic Analysis

Table 16.5.3-1 Support Condition

	X	Y	Z	RX	RY	RZ
Abutment 1	Elastic	Fix	Fix	Fix	Free	Free
Pier 1	Elastic	Fix	Elastic	Fix	Free	Free
Pier 2	Elastic	Fix	Elastic	Fix	Free	Free
Abutment 2	Elastic	Fix	Fix	Fix	Free	Free

- Abutments : Not Modeled
- Piers : Beam Type Elements for Circular Type Piers
- Bearing : Following Force Distribution Bearing:

Table 16.5.3-2 Force Distribution Bearing

Supports	Nos.	Rub. Dimension	Rub. Thickness	G
Abutment 1	3	1500mmx1500mm	37mmx5layers	1.4 N/mm2
Pier 1	3	1500mmx1500mm	37mmx5layers	1.4 N/mm2
Pier 2	3	1500mmx1500mm	37mmx5layers	1.4 N/mm2
Abutment 2	3	1500mmx1500mm	37mmx5layers	1.4 N/mm2

- Foundation : Following springs shall be :

Table 16.5.3-3 Springs of Foundations

Foundations	X: Longitudinal kN/m	Z: Transversal kN/m	RX kNm/rad	RZ kNm/rad
Pier 1	$7.41x10^6$	$7.41x10^6$	$1.58x10^8$	$1.05x10^8$
Pier 2	$7.40x10^6$	$7.40x10^6$	$2.02x10^8$	$1.238x10^8$

- Damping coefficient : Following damping coefficients are applied:

Table 16.5.3-4 Damping Coefficient

Structural Element	Damping
Concrete	0.02
Force Distribution Bearing	0.03
Foundation	0.10

(2) Comparison Studies of Seismic Capacity Improvement Schemes

In order to improve seismic capacity of this bridge, the following methodologies are applied based on technical comparison studies.

i) Application of Continuous Girder

In order to prevent bridge falling down and to reduce the number of bearings, expansion joint and to simplify related devices around pier top, continuous girders are generally applied for multiple span bridges in Japan. For Mawo bridge, 3-span composite PC Fin-back is recommended based on above mentioned comparison study, which also meet such the improvement scheme.

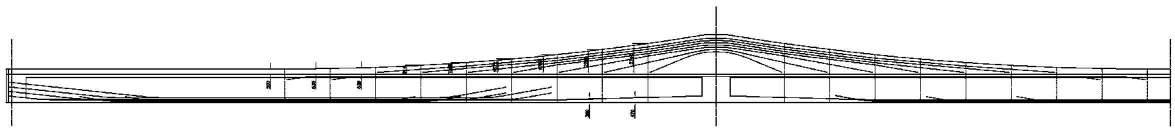


Figure 16.5.3-2 Application of Continuous Girder

ii) Adequate Bearing Type >

Force distribution method by laminated rubber bearings (LRB) shown in the following figure are commonly utilized in viaducts and bridges in Japan as efficient devices to achieve appropriate seismic design.

Fig. Laminated Rubber Bearing

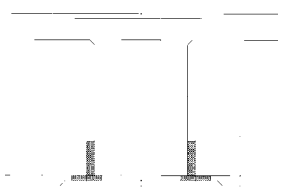
Fig. Steel Bearing

This bearing consists of rubber and steel plate layers. By changing the stiffness of the laminated rubber, such for thickness, number of layers and sizes, seismic horizontal forces can be freely and evenly shared to substructures. Therefore, the boundary condition between superstructure and substructure is "E" that means "elastic".

Otherwise, in Philippines, commonly thin-rubber bearing with anchor bars are utilized as bearing. By this bearing, only two ways of the boundary condition such as "Fix" or "Move" can be applied, which means that controlling of horizontal seismic forces or contribution forces to substructures depends on not horizontal stiffness of bearing but just only the period of its dynamic properties.

In this item, as an improvement scheme, technical comparison study between laminated rubber bearing, thin-rubber bearing with anchor bars and steel bearing is explained from the point of view of seismic behavior, shown as following table.

Table 16.5.3-5 Comparison Study of Bearing in Mawo Bridge

Bearing	Results of Evaluation
Laminated Rubber Bearing Under Force Distribution Method 	Boundary Condition: LD: Elastic (A1-P1-P2-A2), TD: Fix (A1,A2) Elastic (P1,P2) Time Period LD: 1.31S, TD: 1.14s Modal Dumping of 1st mode LD: 3%. TD: 2% Total Horizontal Forces of Superstructure using Modal Dumping LD: 69000kN of 85000kN, TD: 73000kN of 84500kN Seismic Force Distribution LD: A1:P1:P2:A2=1:1:1:1, TD: A1:P1:P2:A2=1:1:1:1
Pad Rubber Bearing with Dowel Under Not Force Distribution 	Boundary Condition: LD: Move (A1, A2), Fix (P1, P2), TD: Fix (A1-P1-P2-A2) Time Period LD: 0.8S, TD: 0.9s Modal Dumping of 1st mode LD: 2%. TD: 2% Total Horizontal Forces of Superstructure using Modal Dumping LD: 88000kN of 85000kN, TD: 92000kN of 84500kN Seismic Force Distribution LD: A1:P1:P2:A2=0:1:1:0,

<table>
<tr><td>Steel Bearing
Under Not Force Distribution
</td><td>Boundary Condition:
LD: Move (A1, A2), Fix (P1, P2), TD: Fix (A1-P1-P2-A2)
Time Period
LD: 0.8S, TD: 0.9s
Modal Dumping of 1st mode
LD: 2%. TD: 2%
Total Horizontal Forces of Superstructure using Modal Dumping
LD: 88000kN of 85000kN, TD: 92000kN of 84500kN
Seismic Force Distribution
LD: A1:P1:P2:A2=0:1:1:0,</td></tr>
<tr><td colspan="2">By using LRB, the period of transversal 1st mode achieves beyond 1.3s, which is much longer than the structure applying Pad Rubber bearing and Steel bearing.

Therefore, the seismic forces based on design spectrum are significantly reduced by extended period.

And also, dominant deformation of the 1st mode is obviously longitudinal deformation of superstructure, which caused by shearing deformation of the LRB; hence, the modal damping of 1st mode can achieve 3% despite just only 2%, modal damping, which is the damping factor of steel structure, of the structure using Pad Rubber bearing or Steel bearing.

Consequently the total horizontal forces of superstructure using LRB is greatly decreased comparing to the structure using common bearing due to extended period and higher structural mode dumping. Additionally, the seismic forces can be distributed evenly by LRB.
Therefore, structurally, superiority of application of LRB is extremely high.</td></tr>
</table>

* LD: Longitudinal Direction, TD: Transversal Direction

(3) Summary of Seismic Analysis

i) Results of Eigenvalue Analysis

The following figure and table shows the results of eigenvalue analysis.

Table 16.5.3-6 Results of Eigenvalue Analysis

Modes	Frequency (Hz)	Period (s)	Ratio of Effective Mass		Mode Damping
			Longitudinal	Transversal	
1	0.762	1.312	**0.764**	0.000	0.031
2	0.876	1.142	0.000	**0.617**	0.021
3	2.574	0.388	0.000	0.000	0.011
4	5.561	0.180	0.000	0.071	0.010
5	6.849	0.146	0.097	0.000	0.086
6	7.896	0.127	0.000	0.126	0.096
7	8.053	0.124	0.099	0.000	0.083
8	8.551	0.117	0.000	0.000	0.011
9	8.829	0.113	0.000	0.101	0.096
10	9.894	0.101	0.000	0.000	0.011

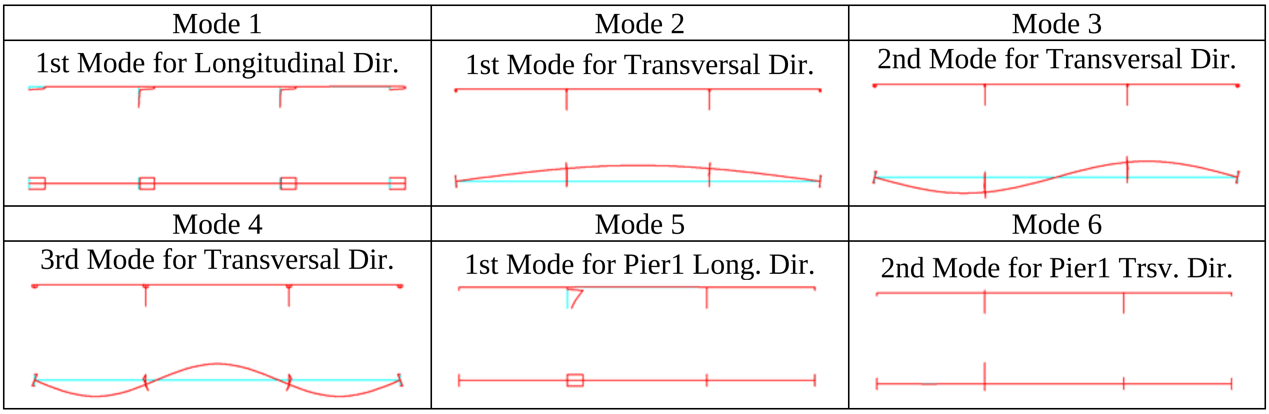


Figure 16.5.3-3 Results of Eigenvalue Analysis

According to the results, predominant mode for longitudinal direction is obviously obtained at 1st mode whose period is 1.31s and effective mass ration is 76% of modes for longitudinal direction. Therefore, the 1st mode for longitudinal direction is so important one, which have enough effective mass ratio. And for the 1st mode for transversal direction is the mode with the effective mass ratio of 0.62 and with the period of 1.14s for transversal direction. Both of the behaviors of longitudinal and transversal direction are efficiently functioned against strong seismic forces, using Force Distribution Bearings and appropriate dumping coefficient of them.

ii) Response Displacement by Response Spectrum Analysis (EQ)
The following table shows the response displacement of relative displacements between substructure and superstructure.

Table 16.5.3-7 Relative Displacement between Substructure and Superstructure

Locationt	Longitudinal (mm)	Transversal (mm)
Abutment 1	349	0.00
Abutment 2	349	0.00

The longitudinal displacements are well converged in realistic scale, for which common expansion joints can be applied.

(4) Seismic Design of Substructure and Foundation

i) Ground Surface in Seismic Design

The following figure shows the ground surface in seismic design.

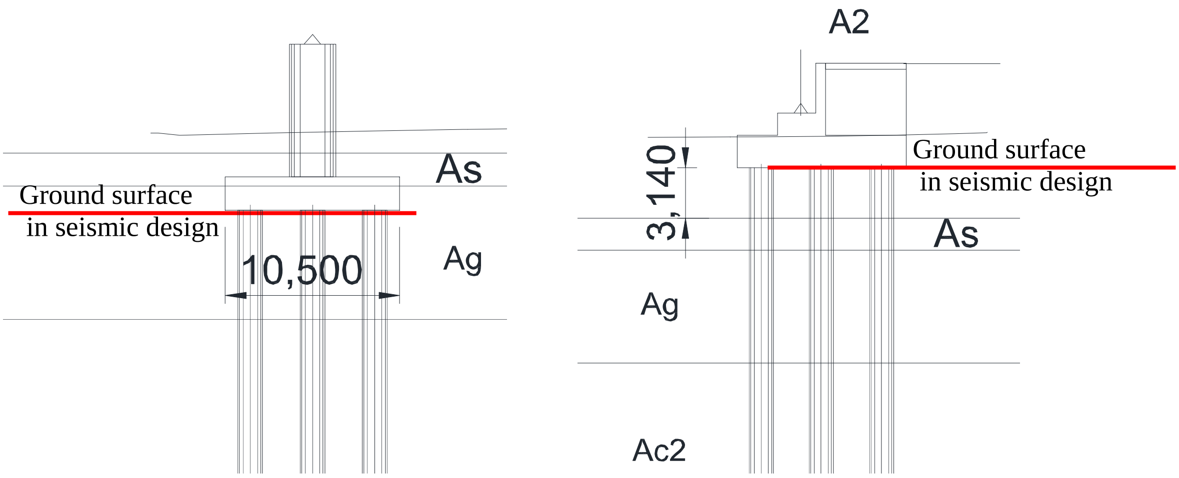


Figure 16.5.3-4 Ground Surface of an Abutment in Seismic Design

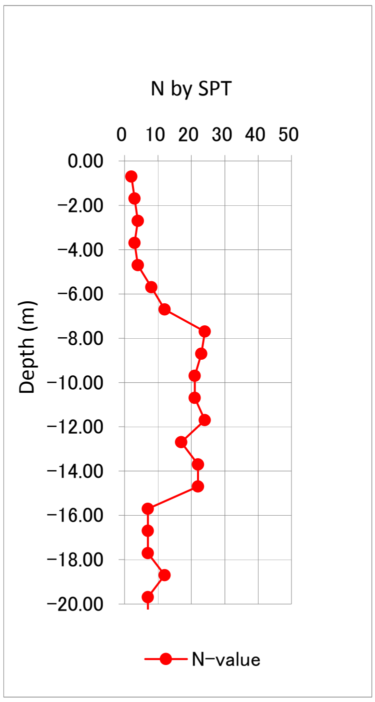
ii) Assessment of Soil Liquefaction

According to the design specifications, sandy layer requiring liquefaction Assessment is obviously obtained as following table.

Table 16.5.3-8 Assessment of Soil Liquefaction

Assessment of Liquefaction Potential

GL-(m)	Soil Layers	N by SPT	Ground Water Level (-m)	Fc (%)	PI	D50 (mm)	D10 (mm)	Liquefiable
		<30		<35%	<15	<10mm	<1mm	
0.70	Ac1	2	0.50	62.3	16			
1.70	Ac1	3	0.50	56.3	2			
2.70	Ac1	4	0.50	56.3	2			
3.70	Ac1	3	0.50	79.7	8			
4.70	Ac1	4	0.50	2.2		0.30	0.10	
5.70	As	8	0.50	1.5		0.24	0.11	○
6.70	As	12	0.50	1.5		0.63	0.13	○
7.70	Ag	24	0.50	0.2		12.61	0.49	○
8.70	Ag	23	0.50	0.6		2.75	0.21	○
9.70	Ag	21	0.50	0.2		2.57	0.76	○
10.70	Ag	21	0.50	1.1		4.57	0.20	○
11.70	Ag	24	0.50	1.3		3.24	0.23	○
12.70	Ag	17	0.50	3.1		2.85	0.14	○
13.70	Ag	22	0.50	0.2		10.30	0.47	○
14.70	Ag	22	0.50	1.6		0.86	0.16	○
15.70	Ac2	7	0.50	61.8	12.0			
16.70	Ac2	7	0.50	68.0	9.0			
17.70	Ac2	7	0.50	72.1	4.0			
18.70	Ac2	12	0.50	56.6	2.0			
19.70	Ac2	7	0.50	75.7	4.0			
20.70	Ac2	7	0.50	56.6	5.0			
21.70	Ac2	9	0.50	35.6	4.0	0.34		
22.70	Ac2	9	0.50	43.4	4.0	0.09		
23.70	Ac2	10	0.50	60.5	4.0			
24.70	Ac2	8	0.50	35.3		0.14		



Based on the results of liquefaction assessment, reduction of geotechnical parameters shall be conducted in accordance with the following tables.

Table 16.5.3-9 Assessment of Soil Liquefaction Parameters

Basic Soil Profile Information										
GL-(m)	Soil Layers	N by SPT	Fc (%)	S=1 G=2 C=3	γt γt1	Water unit weight	Ground Water Level (-m)	σU (Kpa)	σv (Kpa)	σv' (Kpa)
5.70	As	8	1.5	1	18	10.00	0.50	52.00	87.80	35.80
6.70	As	12	1.5	1	18	10.00	0.50	62.00	105.80	43.80
7.70	Ag	24	0.2	1	19	10.00	0.50	72.00	124.80	52.80
8.70	Ag	23	0.6	1	19	10.00	0.50	82.00	143.80	61.80
9.70	Ag	21	0.2	1	19	10.00	0.50	92.00	162.80	70.80
10.70	Ag	21	1.1	1	19	10.00	0.50	102.00	181.80	79.80
11.70	Ag	24	1.3	1	19	10.00	0.50	112.00	200.80	88.80
12.70	Ag	17	3.1	1	19	10.00	0.50	122.00	219.80	97.80
13.70	Ag	22	0.2	1	19	10.00	0.50	132.00	238.80	106.80
14.70	Ag	22	1.6	1	19	10.00	0.50	142.00	257.80	115.80

Table 16.5.3-10 Results on Liquefaction Resistance Factor (FL) & Reduction Factor (D_E)

Calculation for FL								Reduction Factor D_E		
Depth	N1	C1	C2	Na	R	L	FL	**R(Ave.)**	**FL(Ave.)**	**D_E**
-5.70	12.85	1.000	0.000	12.854	0.243	0.852	0.285	**0.265**	**0.316**	**0.00**
-6.70	17.93	1.000	0.000	17.926	0.287	0.826	0.348			
-7.70	33.22	1.000	0.000	33.225	1.348	0.794	1.697	**0.533**	**0.719**	**1.00**
-8.70	29.67	1.000	0.000	29.666	0.750	0.769	0.975			
-9.70	25.36	1.000	0.000	25.355	0.430	0.747	0.576			
-10.70	23.83	1.000	0.000	23.832	0.377	0.727	0.519			
-11.70	25.69	1.000	0.000	25.693	0.445	0.708	0.628			
-12.70	17.22	1.000	0.000	17.223	0.281	0.691	0.407			
-13.70	21.15	1.000	0.000	21.154	0.322	0.675	0.477			
-14.70	20.13	1.000	0.000	20.129	0.309	0.659	0.469			

iii) Design Loads

Based on the results of seismic analysis, the abutment design is conducted for the following load combinations.

LONGITUDINAL DIRECTION (at bearing)

	DL		LL		EQ		**SUM**		
	N (kN)	φ	N (kN)	φ	H (kN)	φ	**N (kN)**	**H (kN)**	**M (kNm)**
A1（Nmax	8,350	1.25	1,700	0.50	17,780	1.00	**11,290**	**17,780**	-
（Nmax	8,350	1.25	1,700	0.50	17,800	1.00	**11,290**	**17,800**	-
A2（Nmax	8,350	1.25	1,700	0.50	17,780	1.00	**11,290**	**17,780**	-
（Nmax	8,350	1.25	1,700	0.50	17,780	1.00	**11,290**	**17,780**	

- Results of Eigenvalue Analysis

VERTICAL REACTIONS FOR SUBSTRUCTURE STABLE CALCULATION (KN)

	Dead load	Live load	Sum.
P1	33,900	3,800	37,700
P2	33,900	3,800	37,700

Note: Impact factor exclusive

SECTIONAL FORCES BY RESPONSE SPECTRUM ANALYSIS FOR PIERS (at bottom of Piers)

	Longitudinal Direction			Transversal Direction		
	H(KN)	M(KNm)	V(KN)	H(KN)	M(KNm)	V(KN)
P1 (F)	17,350	134,700	0	4,060	49,640	0
P2 (F)	17,590	137,100	0	4,060	49,580	0

- Dead Loas (for Pier column)

	Pier Column					
	h	B	Aria	Height	Unit Weight	Self Weight
	(m)	(m)	(m2)	(m)	(kN/m3)	(kN)
P1	2.8	13.7	36.7	8.0	24.5	7,189
P2	2.8	13.7	36.7	8.0	24.5	7,189

- Design Combination Loads

LONGITUDINAL DIRECTION (at bottom of column,Nmax)

	DL		LL		EQ			**SUM of LONGITUDINAL**		
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	N (kN)	H (kN)	M (kNm)
P1(Nmax)	41,089	1.25	3,800	0.50	17,350	134,700	1.00	**53,270**	**17,350**	**134,700**
(Nmin)	41,089	0.90	3,800	0.50	17,350	134,700	1.00	**38,890**	**17,350**	**134,700**
P2(Nmax)	41,089	1.25	3,800	0.50	17,590	137,100	1.00	**53,270**	**17,590**	**137,100**
(Nmin)	41,089	0.90	3,800	0.50	17,590	137,100	1.00	**38,890**	**17,590**	**137,100**

- Results of Eigenvalue Analysis

VERTICAL REACTIONS FOR SUBSTRUCTURE STABLE CALCULATION (KN)

	Dead load	Live load	Sum.
P1	33,900	3,800	37,700
P2	33,900	3,800	37,700

Note: Impact factor exclusive

SECTIONAL FORCES BY RESPONSE SPECTRUM ANALYSIS FOR PIERS (at bottom of Piers)

	Longitudinal Direction			Transversal Direction		
	H(KN)	M(KNm)	V(KN)	H(KN)	M(KNm)	V(KN)
P1 (F)	17,350	134,700	0	4,060	49,640	0
P2 (F)	17,590	137,100	0	4,060	49,580	0

- Dead Loas (for Pier column)

	Pier Column					
	h	B	Aria	Height	Unit Weight	Self Weight
	(m)	(m)	(m2)	(m)	(kN/m3)	(kN)
P1	2.8	13.7	36.7	8.0	24.5	7,189
P2	2.8	13.7	36.7	8.0	24.5	7,189

- Design Combination Loads

LONGITUDINAL DIRECTION (at bottom of column,Nmax)

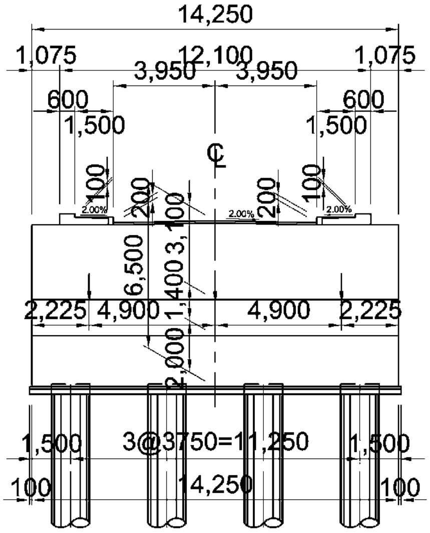
	DL		LL		EQ			**SUM of LONGITUDINAL**		
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	N (kN)	H (kN)	M (kNm)
P1(Nmax)	41,089	1.25	3,800	0.50	17,350	134,700	1.00	**53,270**	**17,350**	**134,700**
(Nmin)	41,089	0.90	3,800	0.50	17,350	134,700	1.00	**38,890**	**17,350**	**134,700**
P2(Nmax)	41,089	1.25	3,800	0.50	17,590	137,100	1.00	**53,270**	**17,590**	**137,100**
(Nmin)	41,089	0.90	3,800	0.50	17,590	137,100	1.00	**38,890**	**17,590**	**137,100**

TRANSVERSAL DIRECTION (at bottom of column,Nmax)

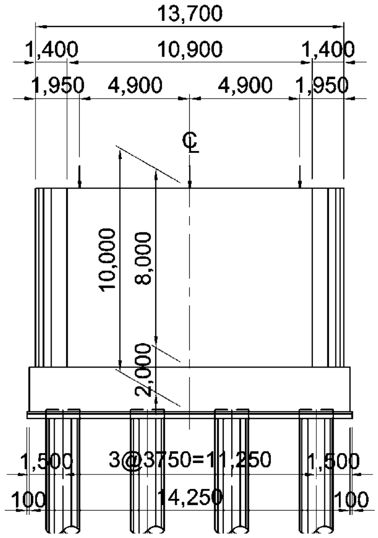
	DL		LL		EQ			**SUM of TRANSVERSAL**		
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	N (kN)	H (kN)	M (kNm)
P1(Nmax)	41,089	1.25	3,800	0.50	4,060	49,640	1.00	**53,270**	**4,060**	**49,640**
(Nmin)	41,089	0.90	3,800	0.50	4,060	49,640	1.00	**38,890**	**4,060**	**49,640**
P2(Nmax)	41,089	1.25	3,800	0.50	4,060	49,580	1.00	**53,270**	**4,060**	**49,580**
(Nmin)	41,089	0.90	3,800	0.50	4,060	49,580	1.00	**38,890**	**4,060**	**49,580**

iv) Design Result

Based on that the following dimensions are obtained as the abutment with foundation of Mawo bridge.



A1: Spread foundation
A2: CCP D=1.5m,L=36.5m,N=12



P1: CCP D=1.5m, L=14.0m,N=12
P2: CCP D=1.5m, L=31m, N=12

Figure 16.5.3-5 Sectional View of Abutment & Pier of Mawo Bridge

(5) Unseating Prevention System

The superstructure is generally connected to the substructure through bearings. As such, the superstructure and the substructure are separated functionally and significantly critical state such as bridge falling down may be caused due to large relative displacements between them, in case of failure of bearings under unexpected seismic forces.

For a functional system preventing such severe state, detailed philosophy and articulate design concepts are explicitly specified in JRA as “Unseating Prevention System” based on accumulated data and experiences from large number of seismic damages. The aim is to provide multiple mechanisms that can complement each other efficiently, shown as follows.

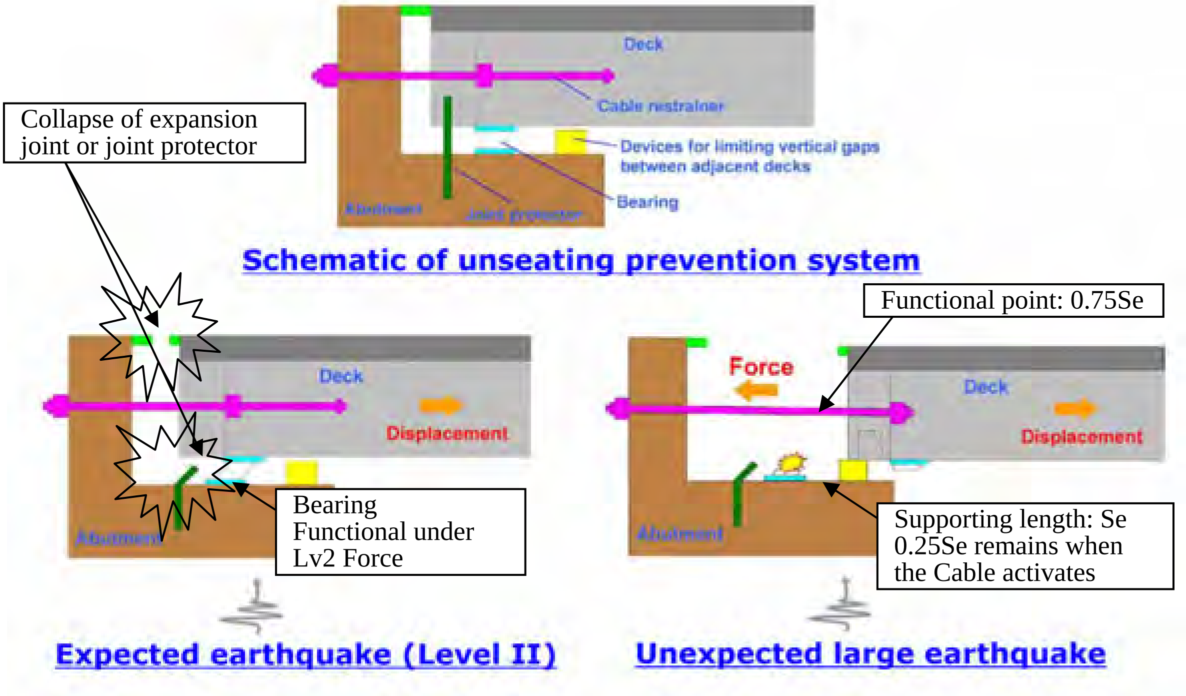


Figure 16.5.3-6 Philosophy of Unseating Prevention System in JRA

The Unseating Prevention System consists as following devices or functions:

Table 16.5.3-11 Devices and Functions of Unseating Prevention System

Devices	Function
Bearing (Type B)	"Type B" bearing in JRA, enables to resist the seismic forces of LV2 by itself.
Supporting Length	The final function of the unseating prevention system. The equation to determine the length is given as follows: Se=μr+μg : μr: Relative responses of girder, μg: Displacement of ground Minimum length: Semin=0.7+0.005l (m)
Longitudinal Restrainer	Design forces are given as 1.5Rd, where Rd is the reaction force of dead loads Maximum movable length: 0.75Se, where Se is supporting length. Generally for new bridges in Japan, cable type devices connecting between parapet of abutment and cross beam of superstructure are commonly utilized.
Expansion Joint	Expansion joint shall resist LV1 seismic forces, otherwise, the capability to resist LV2 seismic forces is not required.

Based on the philosophy and functions above, specification of each devices applied in this new bridge is shown as the results of outline design.

i) Bearing

For the abutments of Mawo bridge, following force distributing bearing is obviously advantageous for seismic behavior.

Table 16.5.3-12 Force Distribution Bearing

Supports	Nos.	Dimension	Thickness	G
Abutment 1	3	1500mmx1500mm	37mmx5layers	1.4 N/mm2
Abutment 2	3	1500mmx1500mm	37mmx5layers	1.4 N/mm2

From the point of view of the philosophy of unseating prevention system, those bearing shall be resist the LV2 seismic forces. As a part of outline design, following outline verification is conducted to clarify they can resist such the forces.

Table 16.5.3-13 Outline Verification of Bearing under LV2 Seismic Forces

		Value/ LV2	Allowance	Judge
Shear Strain	Longitudinal Dir.	1.9	2.5	OK
	Transversal Dir.	0.0	2.5	OK

ii) Supporting Length

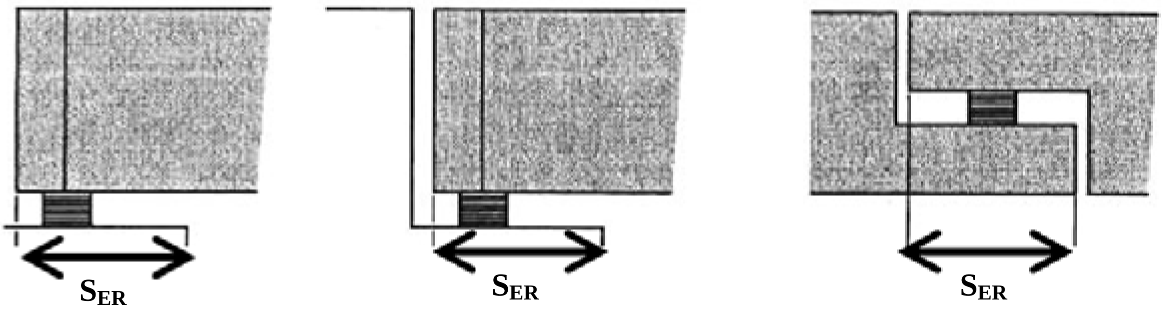


Figure 16.5.3-7 Supporting length

Following equation gives the supporting length.

Se=0.7+0.005l (m) l: Span length

Se = 0.7+0.005*62.5 = 1.1m

Therefore, the supporting length shall be secured over 1.1m.

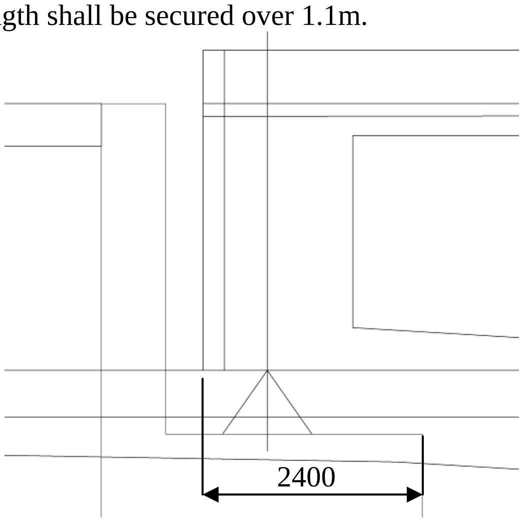


Figure 16.5.3-8 Secure the Length of "Se", Supporting Length

iii) Longitudinal Restrainer
The reaction force by dead loads is 8350kN.
The following verification can be obtained.

Table 16.5.3-14 Verification of Longitudinal Restrainer

	1.5Rd: Design Forces	Allowance
PC Cable Type 19 x 12.7Hmm 4-nos/ Abutment	3131 kN	3667 kN

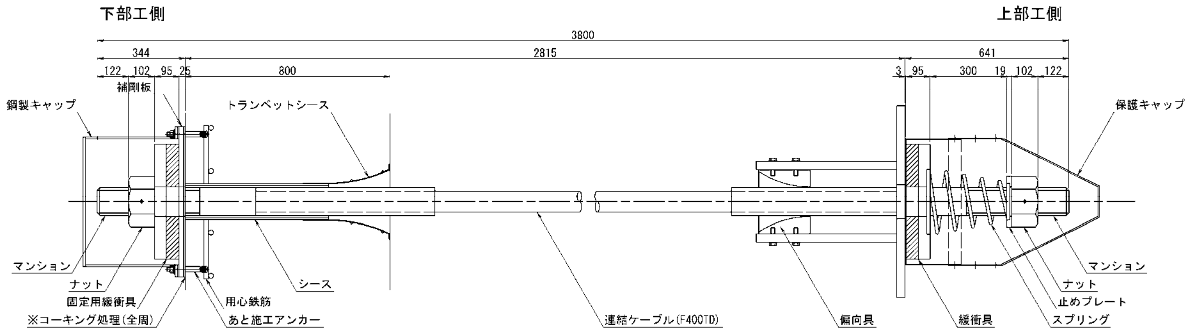


Figure 16.5.3-9 Longitudinal Restrainer for Mawo Bridge

(6) Miscellaneous devices and others

Miscellaneous devices in the bridge are defined as following items:

- Bearing: Evaluated above clause
- Expansion joint
- Drainage
- Wearing coat

In this clause, the devices which are not explained in other clause are explained based on seismic behaviors and current bridge condition.

i) Expansion joint
For the design methodology of expansion joint, its seismic capacity shall be secured under LV1 forces and it does not have to be secured under LV2 forces. The expansion gap between girder end and abutment shall be determined from results of dynamic modal analysis under LV2 and another expansion gap of expansion joint itself shall be determined based on seismic analysis of LV1.

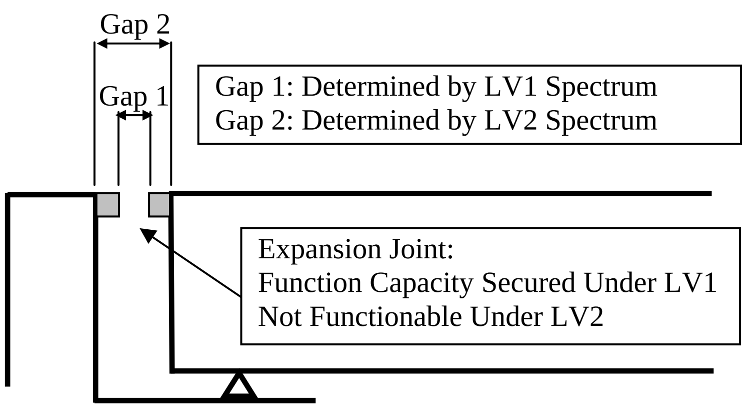


Figure 16.5.3-10 Design Methodology of Expansion Joint

However, expansion gap using laminated rubber baring generally tends to be larger than common bearing and the size of the expansion joint tends to be larger and more expensive. Therefore, the both of expansion gap especially the gap between girder end and abutment should be carefully pay attention to displacement controlling during dynamic modal analysis, evaluating the size of expansion joint. When the gaps were so large comparing to general behavior, the stiffness of rubber bearing should be adjusted and should try the modal analysis again.

In this project, on the basis of above consideration, appropriate modal analysis are carried out, controlling caused displacements based on evaluation of stiffness of rubber bearing.
The final displacements to be used for determination of expansion joint are as follows.

- LV1: Gap 1: 16.5cm + 1.5cm (Excess allowance 15mm (JRA) ≒ 18cm
- LV2: Gap 2: 34.9cm + 1.5cm (Excess allowance 15mm (JRA) ≒ 40cm

Therefore, the expandable gap of the joint in this bridge shall be 18cm or more, and the gap between girder and abutment shall be 30cm or more, which are common results achieved under careful controlling in the dynamic analysis. Consequently, general steel type expansion joint can be adequately applied to this bridge.

ii) Drainage
Drainage system on the bridge is estimated based on current condition. In the next stage such as basic design or design stage, appropriate location of catch basins and drainage pipes shall be designed and drawn based on further investigation of accumulated rainfall data of corresponding area.

iii) Wearing coat
Mawo bridge will have concrete slab deck on the girder. Therefore, usual asphalt concrete can be applied as follows.

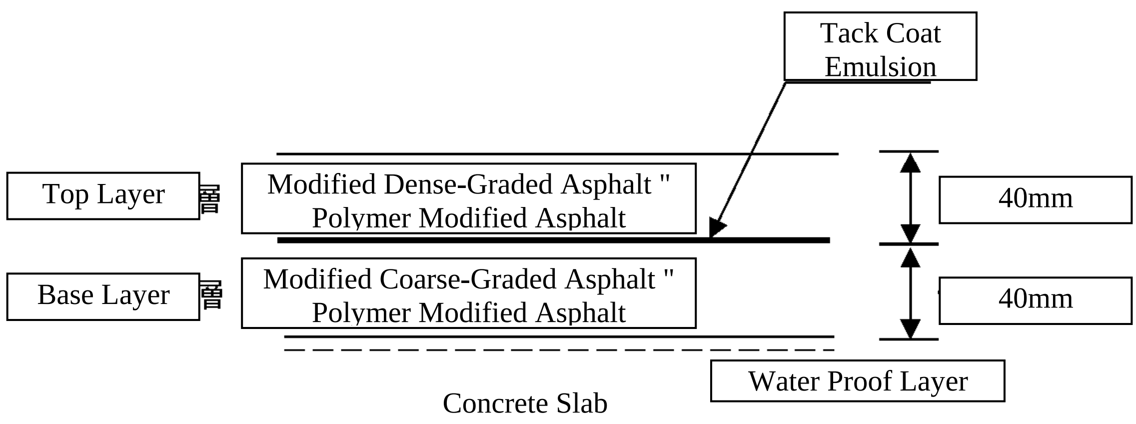


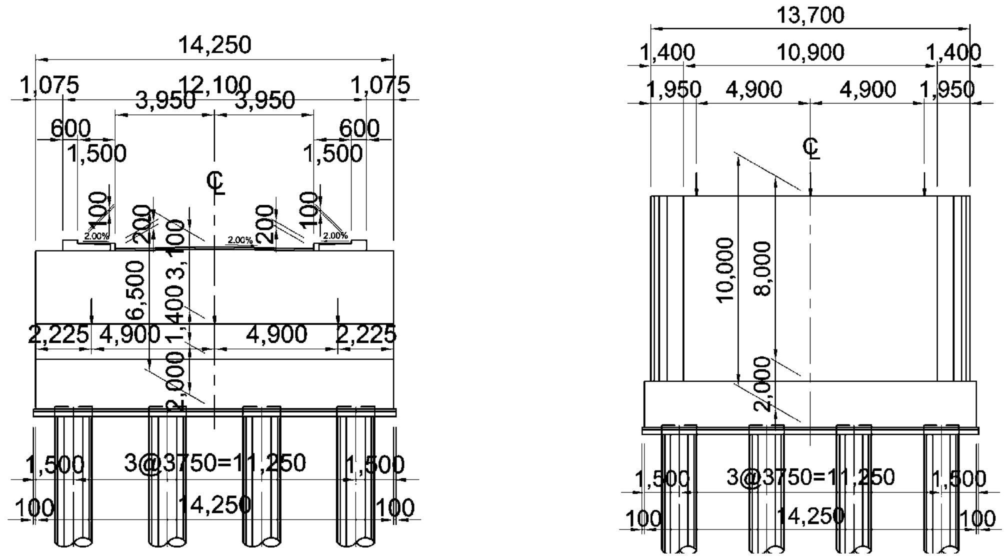
Figure 16.5.3-11 Wearing Coat System of Concrete Slab

16.5.4 Summary of Outline Design Results

(1) Superstructure

Superstructure is designed based on AASHTO LRFD for the bridge type determined in multiple comparison study in consideration of various conditions. The bridge type is PC Fin-Back Box Girder. And laminated rubber bearing considering 3% of damping coefficient in dynamic modal analysis is applied in consideration of seismic behavior calculated dynamic modal analysis.

(2) Substructure and Foundation



A1: Spread foundation
A2: CCP D=1.5m,L=36.5m,N=12

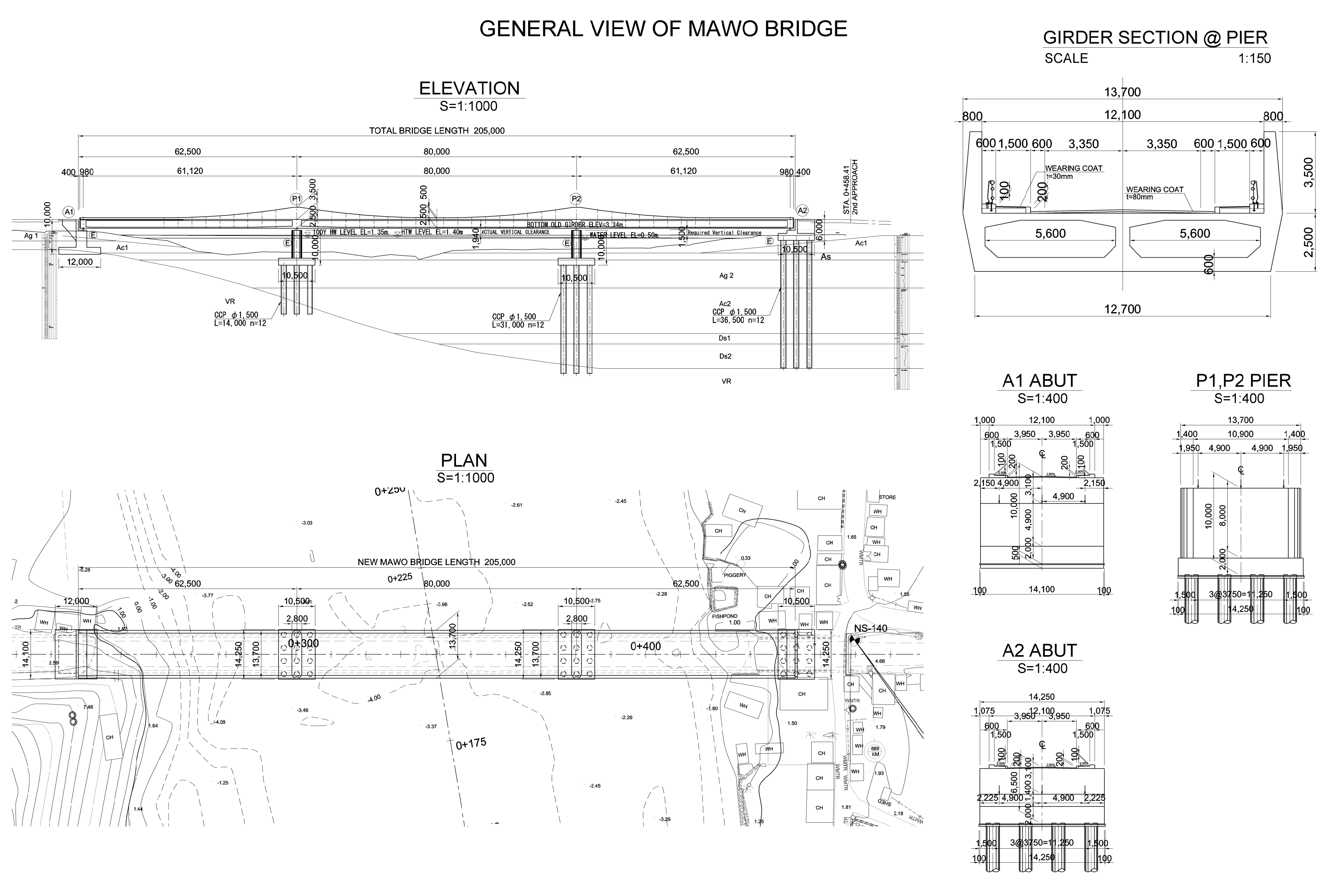
P1: CCP D=1.5m, L=14.0m,N=12
P2: CCP D=1.5m, L=31m, N=12

Figure 16.5.4-1 Sectional View of Abutment & Pier of Mawo Bridge

(3) Further Verification to Be Examined in the Next Phase

The following items may be necessary to be verified or evaluated further in the next phase such as basic or detail design stages.

- Optimization and re-verification of bridge length, span arrangement and bridge types, on the basis of latest existing road condition, newly future planning and detail river condition resulted by detail hydraulic analysis
- Utilization of high-damping bearing based on specific organization regarding non-linear time history response analysis



Clients:		Consultants:	Project Title:	Bridge Name:	Sheet Contents:	SHT. NO.
Japan International Cooperation Agency	DEPARTMENT OF PUBLIC WORKS AND HIGHWAYS REPUBLIC OF THE PHILIPPINES	CTI Engineering Co., Ltd. Chodai Co., Ltd. Nippon Koei	Project for Study on Improvement of Bridges Through Disaster Mitagating Measures for the Large Scale Earthquakes in the Republic of the Philippines	MAWO BRIDGE	GENERAL VIEW	1-10

Figure 16.5.4-2 General View
(Structural Drawings are shown in Appendix-4)

16.6 Outline Design of Wawa Bridge

16.6.1 Design Condition

The following items show design condition for the outline design of Wawa Bridge.

(1) Road Conditions

- Road Design Standard : AASHTO STANDARD VALUE
- Design Speed : V = 60 kmph
- Live Loads : AASHTO Live Loads HL93 and Lane Loads
- Road Width : Shown as follows:

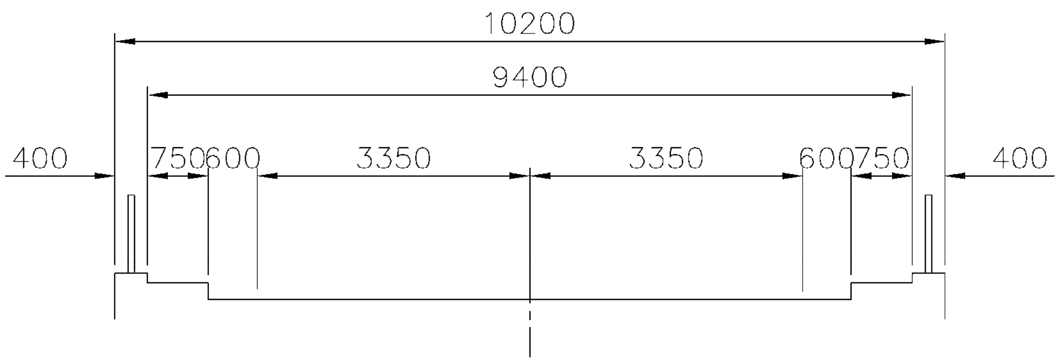


Figure 16.6.1-1 Cross Section/ Lane Arrangement of Wawa Bridge

(2) Soil Conditions

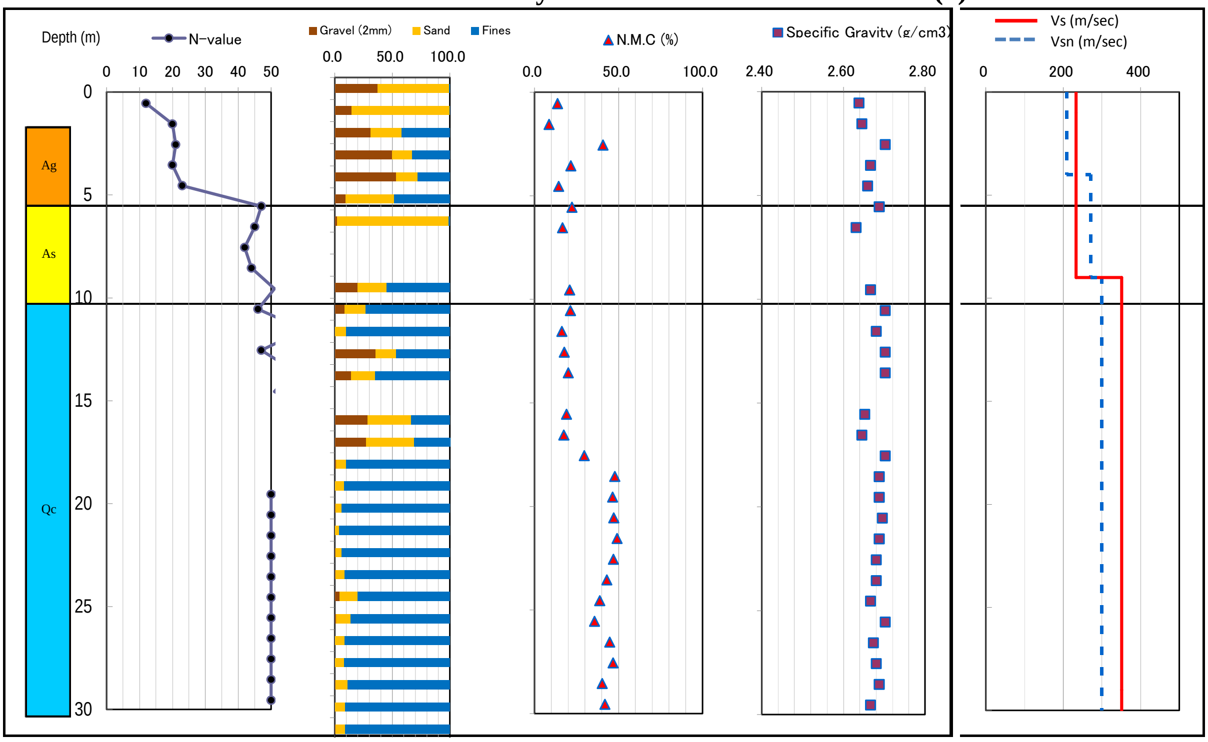
The results of ground investigation are shown in below illustrations and following tables. The clay with gravel layer that can be regarded as the bearing layer is distributed E.L. -5.0m to E.L.-10.0m depth, and has a thick surface layer predominant with sand on top. Specialty, liquefiable sand (AS) is thickly deposited from ground surface to GL-8.0m, of which N-value is 12 to 23, will be affected by liquefaction occurs with reduction of geotechnical parameter. .

Table 16.6.1-1 Summary for Soil Parameters at A2side (1)

A1 Side - Wawa WAW-R1 (right bank) EL.38.50m

Numbe	Depth	SPT	S-wave Ave.	Vsn	Soil Layer	Soil Classification	GSA-J			N.M. C (%)	PI	Specific Gravity	D50 ≤ 10mm	D10 ≤ 1mm	Nd	γt	C	φ	E0	Vsn	Layer Thickness (m)
	Upper	Blows				Observation	Gravel (%)	Sand (%)	Fines (%)							(tf/m^2)	(kN/m^2)	(°)	(kN/m^2)	(m/sec)	
1	0.55	12		210	Ag	Sand and gravel	37.2	62.2	0.6	13.7	N/A	2.64	1.35	0.23		18					
2	1.55	20	233	210	Ag	Clayey gravel and sand	14.6	85.0	0.4	8.6	N/A	2.65	0.52	0.17	18	18	0	36	12,600	210	4
3	2.55	21		210	Ag	Clay w/ gravel	30.9	27.3	41.8	40.8	14	2.70	0.17	N/A		18					
4	3.55	20		210	Ag	Clay w/ gravel	49.7	17.7	32.6	21.5	N/A	2.67	1.91	N/A		18					
5	4.55	23		271	As	Clayey gravel and sand	53.3	18.8	27.9	14.4	N/A	2.66	3.93	N/A		19					
6	5.55	47		271	As	Clayey gravel and sand	9.4	42.4	48.2	22.2	7	2.69	0.08	N/A		19					
7	6.55	45	233	271	As	V.f. sand w/ fines and gravel	2.0	96.6	1.4	16.7	N/A	2.63	0.31	0.1	39	19	0	39	27,300	271	5
8	7.55	42		271	As	Weathered rock (boulder)							RK	RK		19					
9	8.55	44		271	As	Weathered rock (boulder)							RK	RK		19					
10	9.55	51		300	Qc	Clay	19.7	25.6	54.7	20.9	16	2.67	N/A	N/A		18					
11	10.55	46		300	Qc	Clay	8.5	18.3	73.2	21.4	11	2.70	N/A	N/A		18					
12	11.55	60		300	Qc	Clay w/ gravel	0.0	9.7	90.3	16.4	12	2.68	N/A	N/A		18					
13	12.55	47		300	Qc	Clay w/ gravel	35.0	18.3	46.7	17.7	13	2.70	0.16	N/A		18					
14	13.55	57		300	Qc	Clay w/ gravel	14.3	20.5	65.2	20.1	13	2.70	N/A	N/A		18					
15	14.55	51		300	Qc	boulder							RK	RK		18					
16	15.55	62		300	Qc	Gravel	28.4	37.8	33.8	19.1	N/A	2.65	0.61	N/A		18					
17	16.55	69		300	Qc	Gravel	26.9	41.8	31.3	17.5	N/A	2.65	0.62	N/A		18					
18	17.55	63		300	Qc	Silt	1.2	8.7	90.1	29.6	14	2.70	N/A	N/A		18					
19	18.55	69		300	Qc	Clay	0.4	7.5	92.1	47.7	15	2.69	N/A	N/A		18					
20	19.55	50	351	300	Qc	Clay	0.7	5.4	93.9	46.5	14	2.69	N/A	N/A	127	18	793	0	88,810	368	21
21	20.55	50		300	Qc	Gravelly clay	0.2	3.4	96.4	47.1	17	2.70	N/A	N/A		18					
22	21.55	50		300	Qc	Gravelly clay	0.7	5.3	94.0	49.2	16	2.69	N/A	N/A		18					
23	22.55	50		300	Qc	Gravelly clay	0.6	7.7	91.7	47.0	13	2.68	N/A	N/A		18					
24	23.55	50		300	Qc	Gravelly clay	4.0	15.8	80.2	43.0	15	2.68	N/A	N/A		18					
25	24.55	50		300	Qc	Gravelly clay	1.1	12.6	86.3	38.8	12	2.67	N/A	N/A		18					
26	25.55	50		300	Qc	Gravelly clay	0.0	8.5	91.5	35.7	13	2.70	N/A	N/A		18					
27	26.55	50		300	Qc	Gravelly clay	0.3	7.6	92.1	44.8	14	2.67	N/A	N/A		18					
28	27.55	50		300	Qc	Gravelly clay	0.7	10.4	88.9	46.7	12	2.68	N/A	N/A		18					
29	28.55	50		300	Qc	Gravelly clay	0.3	8.7	91.0	40.3	13	2.69	N/A	N/A		18					
30	29.55	50		300	Qc	Gravelly clay	0.7	8.2	91.1	41.9	15	2.67	N/A	N/A		18					

Table 16.6.1-2 Summary for Soil Parameters at A1side (2)



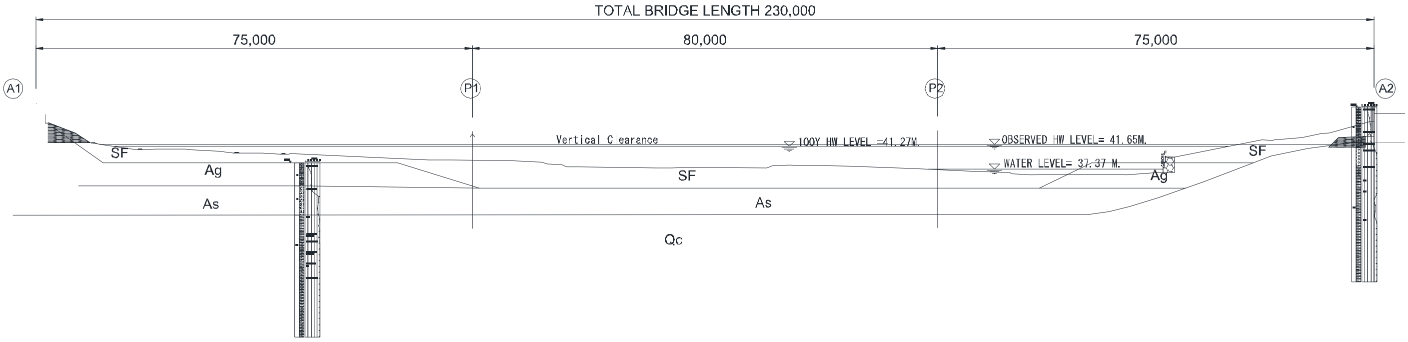


Figure 16.6.1-2 Soil Profile of Wawa Bridge (included previous SPT)

(3) Hydraulic Conditions

- Design Flood Discharge : Q = 1,770m3/s
- Design Water Level : EL= 41.27m
- Freeboard from Design Flood Level : H = 1.50 m

Note: Hydraulic condition shall be re-evaluated based on detail hydraulic analysis and site interview in the next stage such as detail design stage.
Around this area, the water flows is comparatively complicated condition hydraulically. And the results of hydraulic analysis will affect critically the results of bridge planning. Therefore, in detail design stage, detail hydraulic analysis based upon further investigation shall be implemented by river/ hydraulic engineers. Based on the detail examination, the free board to be secured, level of high water level, abutment location and reevaluation of comparison study may be preferable to be re-implemented in the detail design stage.

(4) Bridge Type

- Superstructure Type : 3-Span Continuous Composite Steel Truss
- Bridge Length : L=230m
- Span Arrangement : 75.0m + 80.0m + 75.0m
- Transversal Slope : 2.0%
- Horizontal Alignment : R=∞
- Angle of Alignment : 90 Degrees
- Wearing coat : Polymer Modified Asphalt, Coarse/ Dense-Graded
  t=80mm for Vehicle lane, t=30mm for walkway
- Railing : Steel railing for vehicle and pedestrian
- Bearing : NRB Rubber Bearings - Force Distribution Bearing
- Expansion Joint : Steel type
- Drainage Appliances : PVC pipe
- Bridge Falling Prevention Device: Cable type

- Substructure Type : Wall Type
- Foundation Type : Cast-in-place Pile
- Bearing Soil Condition : Clay with Gravel Layer (N>45)

(5) Design Cases of Outline Design

The outline design of superstructure shall be designed based on the above load condition, specified in AASHTO 2012. On the basis of various reactions and forces, substructures and foundation shall be designed throughout response spectrum analysis under the limit state of "Extreme Event I" specified in AASHTO 2012.

STEP 1. Outline Design of Superstructure
- Design Load, specified in AASHTO 2012
- Stress check

↓

STEP 2. Organization of Reaction Forces
- Organization of reaction forces for substructure design
- Organization of masses for response spectrum analysis

↓

STEP 3. Modal Analysis and Response Spectrum Analysis
- Eigenvalue analysis based on modal analysis
- Organization of response values by the seismic analysis

↓

STEP 4. Substructure and Foundation Design
- Extreme Event I

Figure 16.6.1-3 Flow of Outline Design

16.6.2 Outline Design of Superstructure

(1) Design Condition

➢ Superstructure Type	:	Composite Steel Truss Steel Deck Box Girder
➢ Bridge Length	:	L=230m
➢ Angle of Alignment	:	90 Degrees
➢ Wearing coat	:	Polymer Modified Asphalt, Coarse/ Dense-Graded t=80mm for Vehicle lane, t=30mm for walkway
➢ Railing	:	Steel railing for vehicle and pedestrian
➢ Bearing	:	NRB Rubber Bearings - Force Distribution Bearing
➢ Road Width	:	Shown as follows:

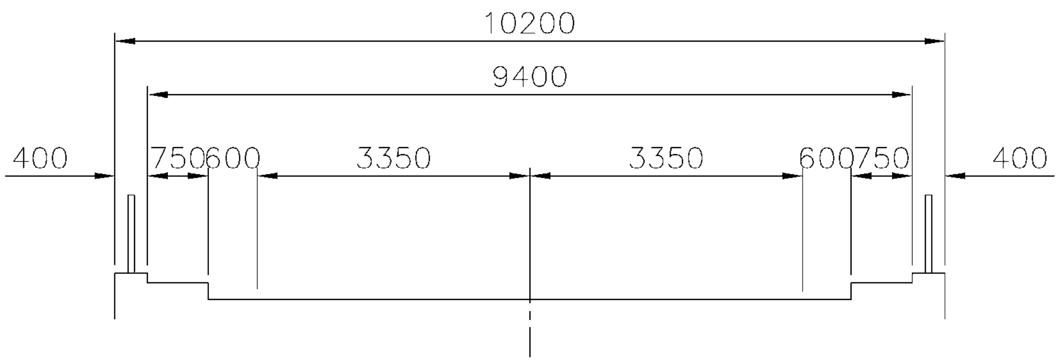


Figure 16.6.2-1 Cross Section/ Lane Arrangement of Wawa Bridge

(2) Design Loads

- Dead Loads : AASHTO 2012
- Live Loads : HL93 and Lane Loads in AASHTO 2012, utilized by influence line evaluation
- Limit State and Load Combination : Strength I in AASHTO 2012

Table 16.6.2-1 Load Combinations and Factors at Strength I in AASHTO 2012

Load Combination	DC DD DW EH EV ES EL PS CR	LL IM CE BR PL LS	WA	WS	WL	FR	TU	TG	SE	Use One of These At a Time		
Limit State										EQ	CT	CV
Strength I	γ_p	1.75	1.00	-	-	1.00	0.5/1.2	γ_{tg}	γ_{se}	-	-	-

Source: LRFD 2012

(3) Analytical Model

In the outline design, the following figure shows the analytical model for outline design of Wawa bridge. All elements in the analysis are truss and beam element model which have 6 of DOFs

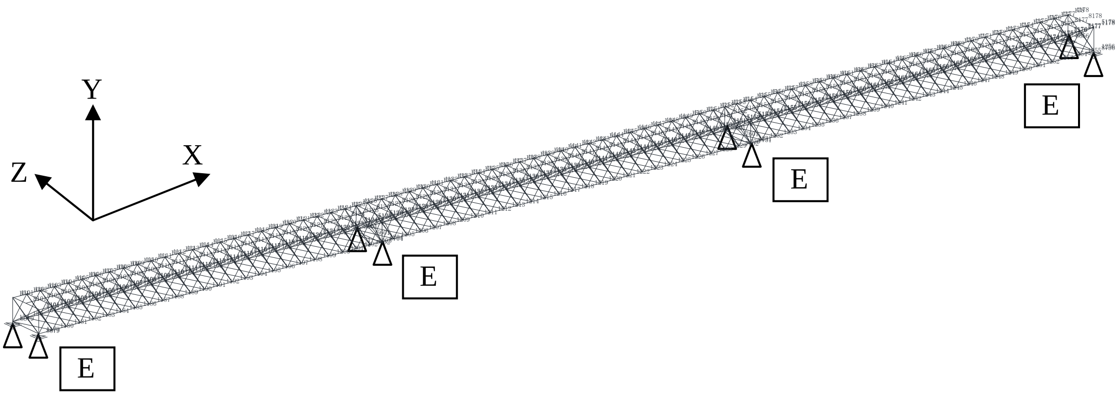
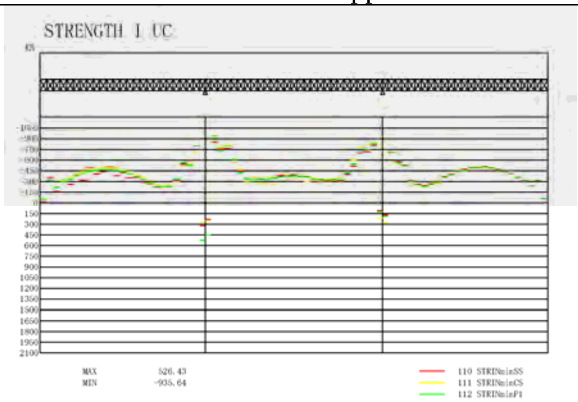
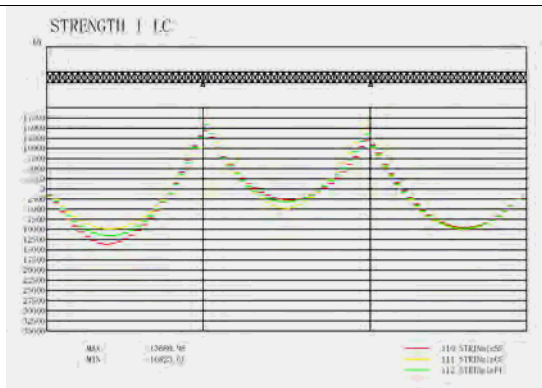
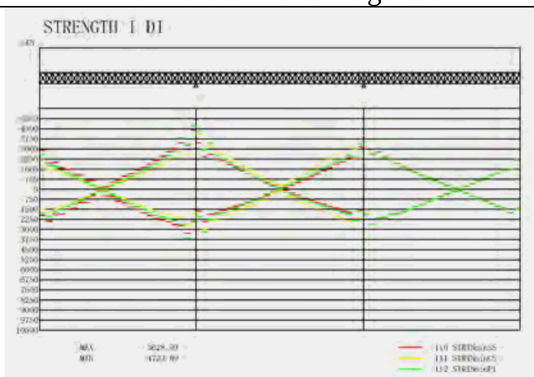


Figure 16.6.2-2 Analytical Model for Superstructure

(4) Sectional forces under Load Combination Strength I

Based on the analytical model, various sectional forces to be utilized for outline design can be obtained. In this report, following figures regarding axial forces for all chords focusing on "side span, "center span" and "pier 1" are shown under the combination sectional forces of "Strength I" in the following figures.

Table 16.6.2-2 Distribution of Axial Forces under Combination of Strength I

Axial Forces in the Upper Chord	Axial Forces in the Lower Chord
	
Axial Forces in the Diagonals	
	The distribution of axial forces in both chords and diagonals are common values for such the truss bridges, which can be designed in ordinal dimension of steel members. Especially, the thickness of diagonals utilizing "lattice truss" can be reduced comparing to common truss bridges because of reduction of buckling length.

* Red: Focus on Side Span, Yellow: Focus on Center Span, Green: Focus on P1

(5) Stress Check

Based on the axial forces of the load combination Strength I, stress checks are conducted for all the members consisting of lattice truss. In this report, the results of following the section are introduced. The all of the calculation results are shown in calculation report.

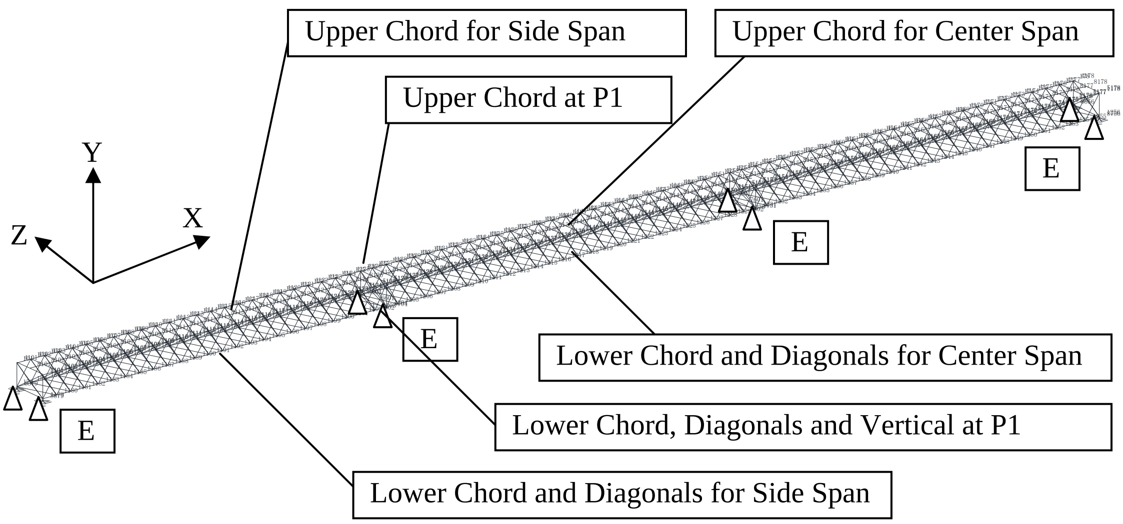


Figure 16.6.2-3 Members for Stress Check

The results of stress checks are shown as follows.

Table 16.6.2-3 Stress Check of Truss

Live Loads	Upper Chord	Lower Chord	Diagonals	Verticals
Focus on Side Span	N= -510 kN <SM490W A = 0.0488 m^2 σ = -11 Mpa < 355 Mpa (Tens) < -352 Mpa (Comp)	N=-13700 kN <SM490W A = 0.0488 m^2 σ = 285 Mpa < 355 Mpa (Tens) < -172 Mpa (Comp)	N=-2910 kN <SM490W A = 0.01782 m^2 σ = -165 Mpa < 355 Mpa (Tens) < -240 Mpa (Comp)	N=-1355 kN <SM490W A = 0.01464 m^2 σ = -93 Mpa < 355 Mpa (Tens) < -163 Mpa (Comp)
Focus on P1	N=-940 kN <SM490W A = 0.0497 m^2 σ = -20 Mpa < 355 Mpa (Tens) < -173 Mpa (Comp)	N=-15900 kN <SM490W A = 0.0568 m^2 σ = -280 Mpa < 355 Mpa (Tens) < -355 Mpa (Comp)	N=-2570 kN <SM490W A = 0.01782 m^2 σ = -144 Mpa < 355 Mpa (Tens) < -240 Mpa (Comp)	N=-3510 kN <SM490W A = 0.01901 m^2 σ = -185 Mpa < 355 Mpa (Tens) < -283 Mpa (Comp)
Focus on Center Span	N=-880 kN <SM490W A = 0.0497 m^2 σ = -18 Mpa < 355 Mpa (Tens) < -173 Mpa (Comp)	N=-16830 kN <SM490W A = 0.0568 m^2 σ = -300 Mpa < 355 Mpa (Tens) < -355 Mpa (Comp)	N=-4106 kN <SM490W A = 0.01978 m^2 σ = -210 Mpa < 355 Mpa (Tens) < -240Mpa (Comp)	N=-3100 kN <SM490W A = 0.01901 m^2 σ = -165 Mpa < 355 Mpa (Tens) < -283 Mpa (Comp)

(6) Summary

Based on that the following dimensions are obtained as the superstructure of Wawa Side bridge

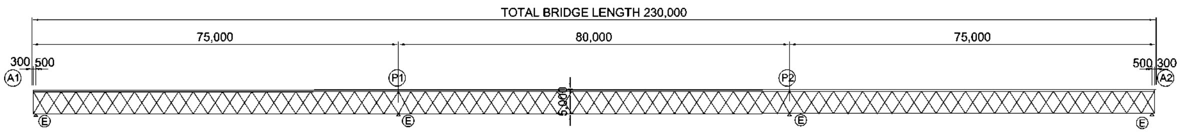


Figure 16.6.2-4 Side View of Superstructure of Wawa Bridge

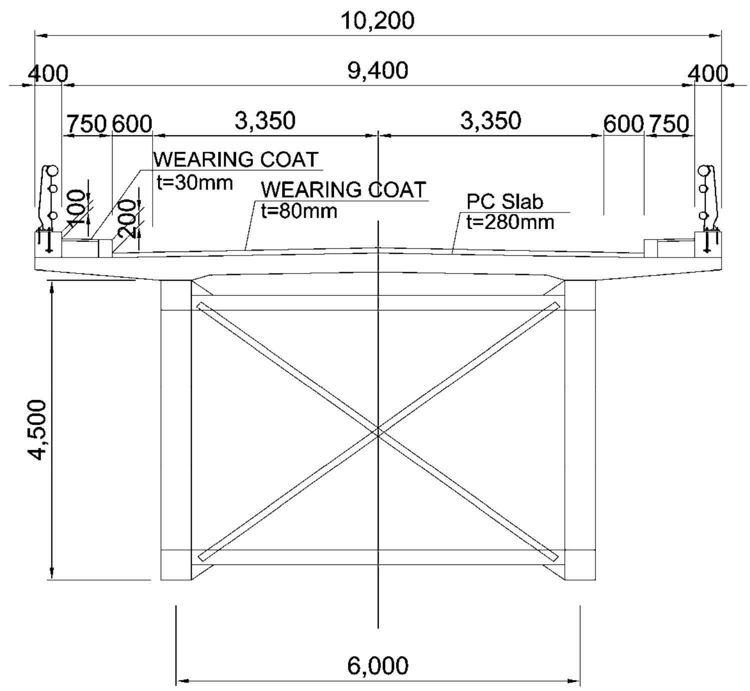


Figure 16.6.2-5 Sectional View of Superstructure of Wawa Side Bridge

Table 16.6.2-4 Reaction Forces of Superstructure

Reaction forces for Substructure Stable Calculation (KN)

	Dead load	Live load	Sum.
A1	4920	1210	6130
P1	14930	2530	17460
P2	14930	2530	17460
A2	4920	1210	6130

Note: Impact factor exclusive

16.6.3 Seismic Design

In this project, as seismic analysis, modal response spectrum analysis is conducted for seismic design. Based on the response results, various structural members can be determined such as piers, foundations, bearings and expansion joints. Analytical model to be utilized for modal analysis commonly utilizes truss and beam type elements in the world.
Based on the results of the outline design of superstructure such as member dimension and masses, analytical model and results of modal response spectrum analysis are explained in this item.

Besides, in this design, abutments are not modeled in the seismic analysis because abutments may have enough strength and stiffness fixed by grounds for seismic vibration; if abutments are modeled in the analysis, excess damping efficiency would be expected to the whole of structural responses.

(1) Analytical Model

- Seismic Analysis: Response Spectrum Analysis based on Modal Eigenvalue Analysis
- Superstructure Type: Composite Steel Truss Steel Deck Box Girder
- Bridge Length : L=230m
- Angle of Alignment: 90 Degrees
- Analytical Model :

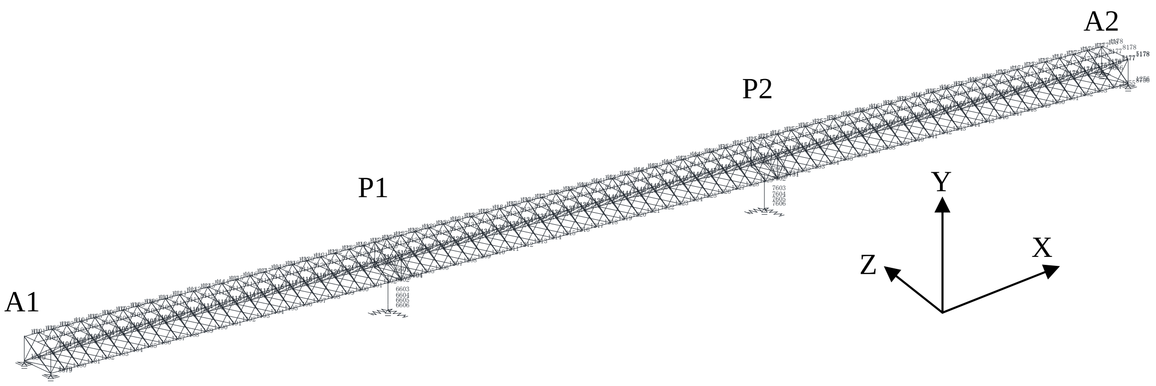


Figure 16.6.3-1 Analytical Mode of Seismic Analysis

Table 16.6.3-1 Support Condition

	X	Y	Z	RX	RY	RZ
Abutment 1	Elastic	Fix	Fix	Fix	Free	Free
Pier 1	Elastic	Fix	Elastic	Fix	Free	Free
Pier 2	Elastic	Fix	Elastic	Fix	Free	Free
Abutment 2	Elastic	Fix	Fix	Fix	Free	Free

- Abutments : Not Modeled
- Piers : Beam Type Elements for Wall Type Piers
- Bearing : Following Force Distribution Bearing:

Table 16.6.3-2 Force Distribution Bearing

Supports	Nos.	Rub. Dimension	Rub. Thickness	G
Abutment 1	2	900mmx900mm	22mmx5layers	1.4 N/mm2
Pier 1	2	900mmx900mm	22mmx5layers	1.4 N/mm2
Pier 2	2	900mmx900mm	22mmx5layers	1.4 N/mm2
Abutment 2	2	900mmx900mm	22mmx5layers	1.4 N/mm2

- Foundation : Following springs shall be :

Table 16.6.3-3 Springs of Foundations

Foundations	X: Longitudinal kN/m	Z: Transversal kN/m	RX kNm/rad	RZ kNm/rad
Pier 1	7.69×10^6	7.69×10^6	2.75×10^8	1.69×10^8
Pier 2	7.69×10^6	7.69×10^6	2.75×10^8	1.69×10^8

- Damping coefficient :Following damping coefficients are applied:

Table 16.6.3-4 Damping Coefficient

Structural Element	Damping
Steel	0.01
Concrete	0.02
Force Distribution Bearing	0.03
Foundation	0.10

(2) Comparison Studies of Seismic Capacity Improvement Schemes

In order to improve seismic capacity of this bridge, the following methodologies are applied based on technical comparison studies.

i) Application of Continuous Girder
In order to prevent bridge falling down and to reduce the number of bearings, expansion joint and to simplify related devices around pier top, continuous girders are generally applied for multiple span bridges in Japan. For Wawa bridge, 3-Span Composite Steel Truss is recommended based on above mentioned comparison study, which also meet such the improvement scheme.

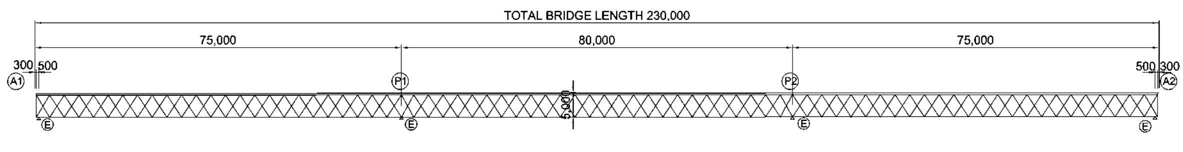


Figure 16.6.3-2 Application of Continuous Girder

ii) Adequate Bearing Type >

Force distribution method by laminated rubber bearings (LRB) shown in the following figure are commonly utilized in viaducts and bridges in Japan as efficient devices to achieve appropriate seismic design.

Fig. Laminated Rubber Bearing

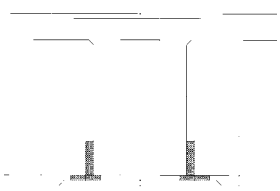
Fig. Steel Bearing

This bearing consists of rubber and steel plate layers. By changing the stiffness of the laminated rubber, such for thickness, number of layers and sizes, seismic horizontal forces can be freely and evenly shared to substructures. Therefore, the boundary condition between superstructure and substructure is "E" that means "elastic".

Otherwise, in Philippines, commonly thin-rubber bearing with anchor bars is utilized as bearing. By this bearing, only two ways of the boundary condition such as "Fix" or "Move" can be applied, which means that controlling of horizontal seismic forces or contribution forces to substructures depends on not horizontal stiffness of bearing but just only the period of its dynamic properties.

In this item, as an improvement scheme, technical comparison study between laminated rubber bearing, thin-rubber bearing with anchor bars and steel bearing is explained from the point of view of seismic behavior, shown as following table.

Table 16.6.3-5 Comparison Study of Bearing in Wawa Bridge

Bearing	Results of Evaluation
Laminated Rubber Bearing Under Force Distribution Method 	Boundary Condition: LD: Elastic (A1-P1-P2-A2), TD: Fix (A1,A2) Elastic (P1,P2) Time Period LD: 1.4S, TD: 1.6s Modal Dumping of 1st mode LD: 3%. TD: 3% Total Horizontal Forces of Superstructure using Modal Dumping LD: 21000kN of 40000kN, TD: 20400kN of 40000kN Seismic Force Distribution LD: A1:P1:P2:A2=1:1:1:1, TD: A1:P1:P2:A2=1:1.5:1.5:1
Pad Rubber Bearing with Dowel Under Not Force Distribution 	Boundary Condition: LD: Move (A1, A2), Fix (P1, P2), TD: Fix (A1-P1-P2-A2) Time Period LD: 0.85S, TD: 0.88s Modal Dumping of 1st mode LD: 1%. TD: 1% Total Horizontal Forces of Superstructure using Modal Dumping LD: 42000kN of 40000kN, TD: 41500kN of 40000kN Seismic Force Distribution LD: A1:P1:P2:A2=0:1:1:0,
Steel Bearing Under Not Force Distribution 	Boundary Condition: LD: Move (A1, A2), Fix (P1, P2), TD: Fix (A1-P1-P2-A2) Time Period LD: 0.85S, TD: 0.88s Modal Dumping of 1st mode LD: 1%. TD: 1% Total Horizontal Forces of Superstructure using Modal Dumping LD: 42000kN of 40000kN, TD: 41500kN of 40000kN Seismic Force Distribution LD: A1:P1:P2:A2=0:1:1:0,

By using LRB, the period of transversal 1st mode achieves beyond 1.5s, which is much longer than the structure applying Pad Rubber bearing and Steel bearing.

Therefore, the seismic forces based on design spectrum are significantly reduced by extended period.

And also, dominant deformation of the 1st mode is obviously transversal deformation of superstructure, which caused by shearing deformation of the LRB; hence, the modal damping of 1st mode can achieve 3% despite just only 1%, modal damping, which is the damping factor of steel structure, of the structure using Pad Rubber bearing or Steel bearing.

Consequently the total horizontal forces of superstructure using LRB is greatly decreased comparing to the structure using common bearing due to extended period and higher structural mode dumping. Additionally, the seismic forces can be distributed evenly by LRB.
Therefore, structurally, superiority of application of LRB is extremely high.

* LD: Longitudinal Direction, TD: Transversal Direction

(3) Summary of Seismic Analysis

i) Results of Eigenvalue Analysis
The following figure and table shows the results of eigenvalue analysis.

Table 16.6.3-6 Results of Eigenvalue Analysis

Modes	Frequency (Hz)	Period (s)	Ratio of Effective Mass		Mode Damping
			Longitudinal	Transversal	
1	0.643	1.556	0.000	**0.717**	0.027
2	0.707	1.414	**0.882**	0.000	0.030
3	1.160	0.862	0.000	0.000	0.015
4	2.168	0.461	0.000	0.931	0.010
5	3.802	0.263	0.000	0.000	0.011
6	5.569	0.180	0.000	0.046	0.010
7	7,392	0.135	0.202	0.000	0.010
8	9.594	0.104	0.000	0.023	0.010
9	11.973	0.084	0.000	0.000	0.010
10	14.925	0.067	0.000	0.005	0.010

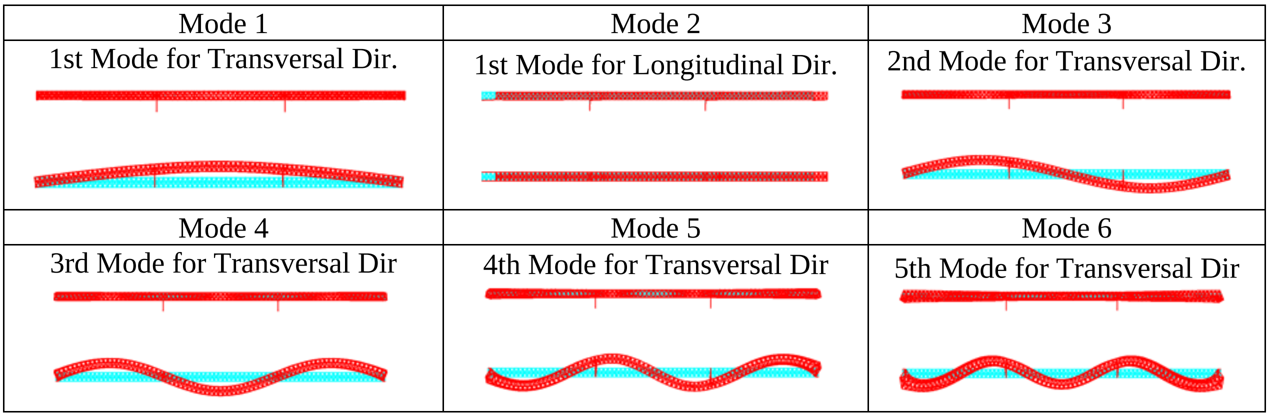


Figure 16.6.3-3 Results of Eigenvalue Analysis

According to the results, predominant mode for longitudinal direction is obviously obtained at 1st mode whose period is 1.42s and effective mass ration is 89% of modes for longitudinal direction. Therefore, the 1st mode for longitudinal direction is so important one, which have enough effective mass ratio. And for the 1st mode for transversal direction is the mode with the effective mass ratio of 0.717s and with the period of 1.56s for transversal direction. Both of the behaviors of longitudinal and transversal direction are efficiently functioned against strong seismic forces, using Force Distribution Bearings and appropriate dumping coefficient of them.

ii) Response Displacement by Response Spectrum Analysis (EQ)
The following table shows the response displacement of relative displacements between substructure and superstructure.

Table 16.6.3-7 Relative Displacement between Substructure and Superstructure

Location	Longitudinal (mm)	Transversal (mm)
Abutment 1	265	0.00
Abutment 2	265	0.00

The longitudinal displacements are well converged in realistic scale, for which common expansion joints can be applied.

(4) Seismic Design of Substructure and Foundation

i) Ground Surface in Seismic Design
The following figure shows the ground surface in seismic design.

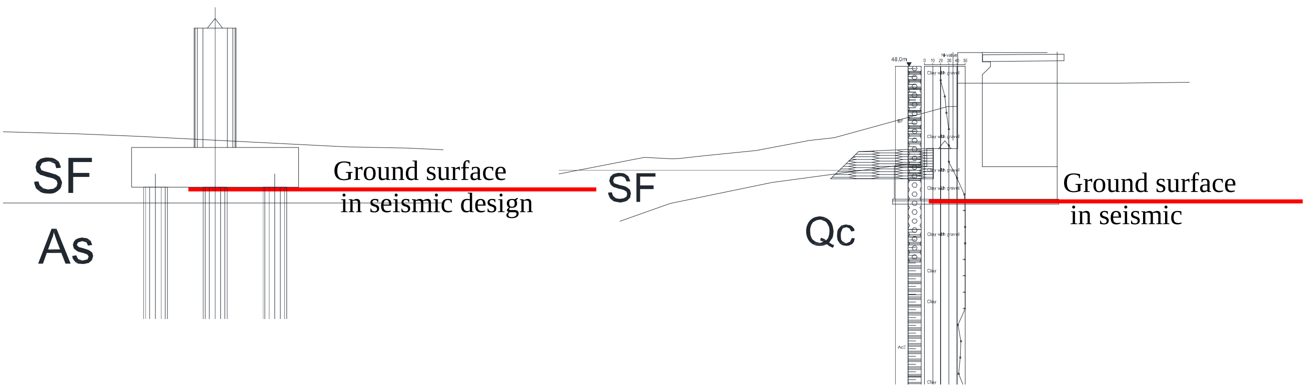


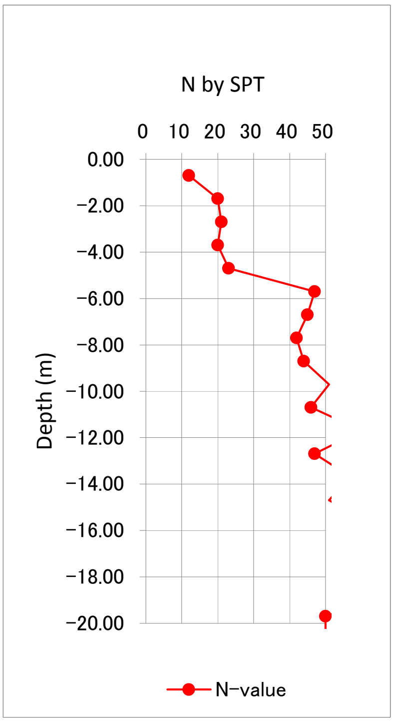
Figure 16.6.3-4 Ground Surface of an Abutment in Seismic Design

ii) Assessment of Soil Liquefaction

According to the design specifications, sandy layer requiring liquefaction Assessment is obviously obtained as following table.

Table 16.6.3-8 Assessment of Soil Liquefaction

Assessment of Liquefaction Potential								
GL-(m)	Soil Layers	N by SPT	Ground Water Level (-m)	Fc (%)	PI	D50 (mm)	D10 (mm)	Liquefiable
		<30		<35%	<15	<10mm	<1mm	
0.70	Ag	12	0.00	0.6		1.35	0.23	○
1.70	Ag	20	0.00	0.4		0.52	0.17	○
2.70	Ag	21	0.00	41.8	14	0.17		
3.70	Ag	20	0.00	32.6		1.91		
4.70	As	23	0.00	27.9		3.93		○
5.70	As	47	0.00	48.2	7	0.08		
6.70	As	45	0.00	1.4		0.31	0.10	
7.70	As	42	0.00					
8.70	As	44	0.00					
9.70	Qc	51	0.00	54.7	16			
10.70	Qc	46	0.00	73.2	11			
11.70	Qc	60	0.00	90.3	12.0			
12.70	Qc	47	0.00	46.7	13.0	0.16		
13.70	Qc	57	0.00	65.2	13.0			
14.70	Qc	51	0.00					
15.70	Qc	62	0.00	33.8		0.61		
16.70	Qc	69	0.00	31.3		0.62		
17.70	Qc	63	0.00	90.1	14.0			
18.70	Qc	69	0.00	92.1	15.0			
19.70	Qc	50	0.00	93.9	14.0			
20.70	Qc	50	0.00	96.4	17.0			
21.70	Qc	50	0.00	94.0	16.0			
22.70	Qc	50	0.00	91.7	13.0			
23.70	Qc	50	0.00	80.2	15.0			
24.70	Qc	50	0.00	86.3	12.0			



Based on the results of liquefaction assessment, reduction of geotechnical parameters shall be conducted in accordance with the following tables.

Table 16.6.3-9 Result Assessment of Soil Liquefaction Parameters

Basic Soil Profile Information									
GL-(m)	Soil Layers	N by SPT	Fc (%)	γt γt1	Water unit weight	Ground Water Level (-m)	σU (Kpa)	σv (Kpa)	σv' (Kpa)
0.70	Ag	12	0.9	19	10.00	1.50	0.00	13.30	13.30
1.00	Ag	12.0	0.9	19	10.00	1.50	0.00	19.00	19.00
4.70	As	23	27.9	20	10.00	1.50	32.00	90.30	58.30

Table 16.6.3-10 Results on Liquefaction Resistance Factor (FL) & Reduction Factor (DE)

Calculation for FL								Reduction Factor D_E		
Depth	N1	C1	C2	Na	R	L	FL	**R(Ave.)**	**FL(Ave.)**	**D_E**
-0.70	24.49	1.000	0.000	24.490	0.398	0.376	1.057	**0.376**	**0.801**	**1.000**
-1.00	22.92	1.000	0.000	22.921	0.354	0.374	0.946			
-4.70	30.48	1.358	0.994	42.380	5.970	0.547	10.912	**5.970**	**10.912**	**1.000**

iii) Design Loads

Based on the results of seismic analysis, the abutment design is conducted for the following load combinations.

- Results of Eigenvalue Analysis

VERTICAL REACTIONS FOR SUBSTRUCTURE STABLE CALCULATION (KN)

	Dead load	Live load	Sum.
A1	4,920	1,210	6,130
A2	4,920	1,210	6,130

Note: Impact factor exclusive

HORISONTAL/ TRANSVERSAL REACTIONS BY RESPONSE SPECTRUM ANALYSIS UNDER L2 (at bearing)

	H(KN)	M(KNm)	V(KN)
A1 (E)	5,340	-	-
A2 (E)	5,340	-	-

- Design Combination Loads(at bearing)

LONGITUDINAL DIRECTION (at bearing)

	DL		LL		EQ			SUM		
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	**N (kN)**	**H (kN)**	**M (kNm)**
A1（Nmax)	4,920	1.25	1,210	0.50	5,340	-	1.00	**6,760**	**5,340**	-
（Nmax)	4,920	1.25	1,210	0.50	5,340	-	1.00	**6,760**	**5,340**	-
A2（Nmax)	4,920	1.25	1,210	0.50	5,340	-	1.00	**6,760**	**5,340**	-
（Nmax)	4,920	1.25	1,210	0.50	5,340	-	1.00	**6,760**	**5,340**	-

- Results of Eigenvalue Analysis

VERTICAL REACTIONS FOR SUBSTRUCTURE STABLE CALCULATION (KN)

	Dead load	Live load	Sum.
P1	14,930	2,530	17,460
P2	14,930	2,530	17,460

Note: Impact factor exclusive

SECTIONAL FORCES BY RESPONSE SPECTRUM ANALYSIS FOR PIERS (at bottom of Piers)

	Longitudinal Direction			Transversal Direction		
	H(KN)	M(KNm)	V(KN)	H(KN)	M(KNm)	V(KN)
P1 (E)	5,390	27,690	0	5,430	38,470	0
P2 (E)	5,390	27,690	0	5,430	38,470	0

- Dead Loas (for Pier column)

	Pier Column					
	h	B	Aria	Height	Unit Weight	Self Weight
	(m)	(m)	(m2)	(m)	(kN/m3)	(kN)
P1	2.1	6.5	12.7	6.0	24.5	1,868
P2	2.1	6.5	12.7	6.0	24.5	1,868

- Design Combination Loads

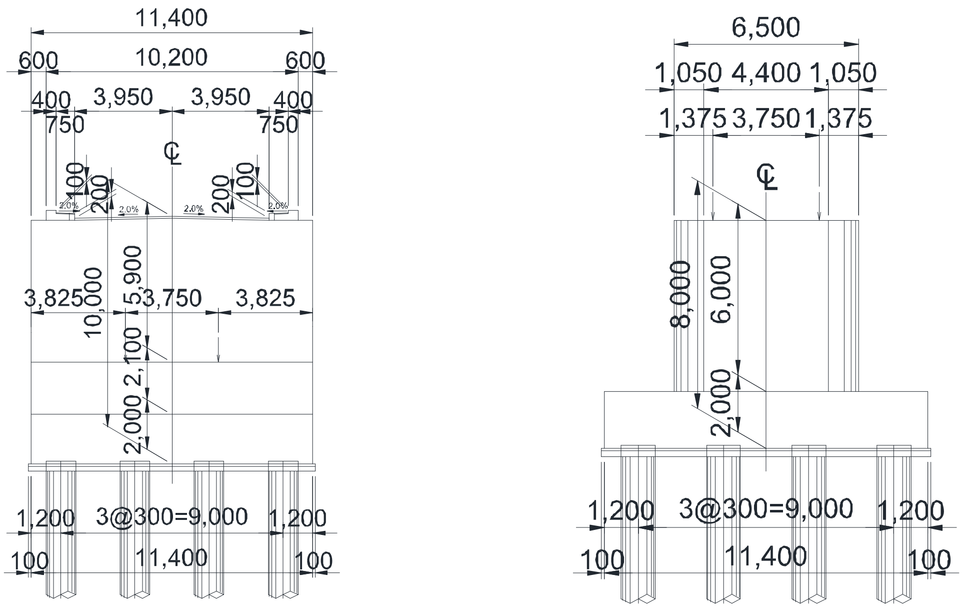
LONGITUDINAL DIRECTION (at bottom of column,Nmax)

	DL		LL		EQ			SUM of LONGITUDINAL		
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	**N (kN)**	**H (kN)**	**M (kNm)**
P1(Nmax)	16,798	1.25	2,530	0.50	5,390	27,690	1.00	**22,270**	**5,390**	**27,690**
(Nmin)	16,798	0.90	2,530	0.50	5,390	27,690	1.00	**16,390**	**5,390**	**27,690**
P2(Nmax)	16,798	1.25	2,530	0.50	5,390	27,690	1.00	**22,270**	**5,390**	**27,690**
(Nmin)	16,798	0.90	2,530	0.50	5,390	27,690	1.00	**16,390**	**5,390**	**27,690**

TRANSVERSAL DIRECTION (at bottom of column,Nmax)

	DL		LL		EQ			SUM of TRANSVERSAL		
	N (kN)	φ	N (kN)	φ	H (kN)	M (kNm)	φ	**N (kN)**	**H (kN)**	**M (kNm)**
P1(Nmax)	16,798	1.25	2,530	0.50	5,430	38,470	1.00	**22,270**	**5,430**	**38,470**
(Nmin)	16,798	0.90	2,530	0.50	5,430	38,470	1.00	**16,390**	**5,430**	**38,470**
P2(Nmax)	16,798	1.25	2,530	0.50	5,430	38,470	1.00	**22,270**	**5,430**	**38,470**
(Nmin)	16,798	0.90	2,530	0.50	5,430	38,470	1.00	**16,390**	**5,430**	**38,470**

iv) Design Result
Based on that the following dimensions are obtained as the abutment with foundation of Wawa bridge.



A1: CPP D1.2m,L=7m, n=12
A2: Spread foundation

CPP D1.2m,L=7m, n=12(P1,P2)

Figure 16.6.3-5 Sectional View of Substructure of Wawa Bridge

(5) Unseating Prevention System

The superstructure is generally connected to the substructure through bearings. As such, the superstructure and the substructure are separated functionally and significantly critical state such as bridge falling down may be caused due to large relative displacements between them, in case of failure of bearings under unexpected seismic forces.

For a functional system preventing such severe state, detailed philosophy and articulate design concepts are explicitly specified in JRA as “Unseating Prevention System” based on accumulated data and experiences from large number of seismic damages. The aim is to provide multiple mechanisms that can complement each other efficiently, shown as follows.

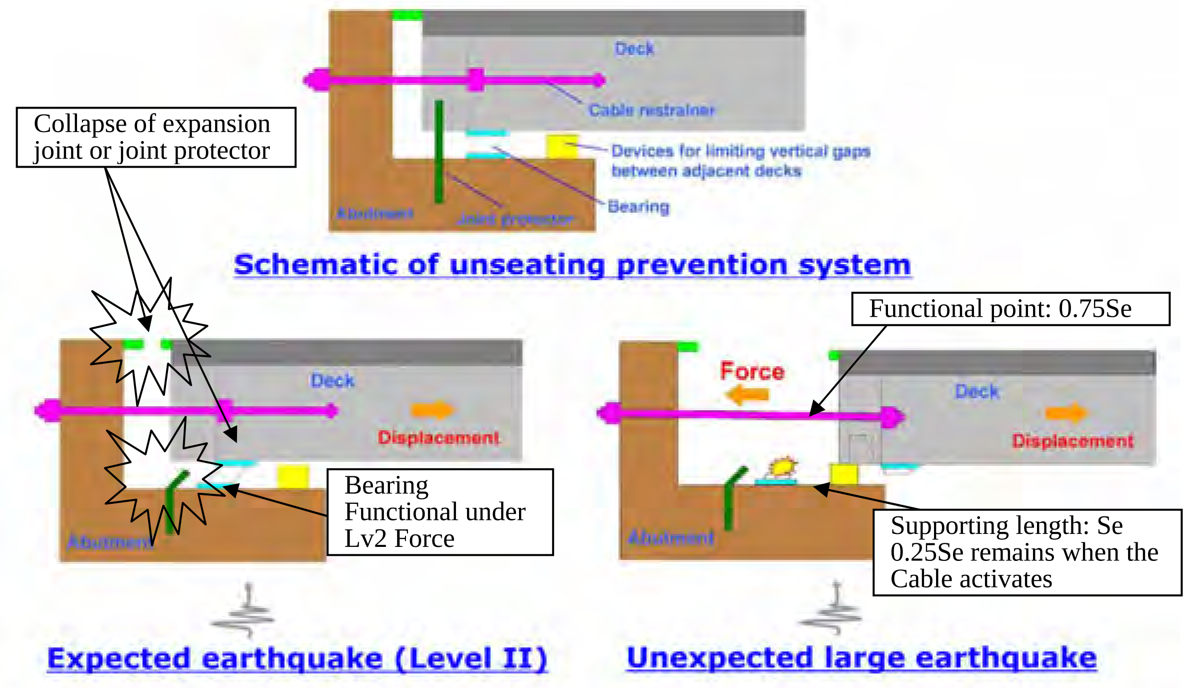


Figure 16.6.3-6 Philosophy of Unseating Prevention System in JRA

The Unseating Prevention System consists as following devices or functions:

Table 16.6.3-11 Devices and Functions of Unseating Prevention System

Devices	Function
Bearing (Type B)	"Type B" bearing in JRA, enables to resist the seismic forces of LV2 by itself.
Supporting Length	The final function of the unseating prevention system. The equation to determine the length is given as follows: Se=μr+μg : μr: Relative responses of girder, μg: Displacement of ground Minimum length: Semin=0.7+0.005l (m)
Longitudinal Restrainer	Design forces are given as 1.5Rd, where Rd is the reaction force of dead loads Maximum movable length: 0.75Se, where Se is supporting length. Generally for new bridges in Japan, cable type devices connecting between parapet of abutment and cross beam of superstructure are commonly utilized.
Expansion Joint	Expansion joint shall resist LV1 seismic forces, otherwise, the capability to resist LV2 seismic forces is not required.

Based on the philosophy and functions above, specification of each devices applied in this new bridge is shown as the results of outline design.

i) Bearing
For the abutments of Wawa bridge, following force distributing bearing is obviously advantageous for seismic behavior.

Table 16.6.3-12 Force Distribution Bearing

Supports	Nos.	Dimension	Thickness	G
Abutment 1	2	900mmx900mm	22mmx5layers	1.4 N/mm2
Abutment 2	3	900mmx900mm	22mmx5layers	1.4 N/mm2

From the point of view of the philosophy of unseating prevention system, those bearing shall be resist the LV2 seismic forces. As a part of outline design, following outline verification is conducted to clarify they can resist such the forces.

Table 16.6.3-13 Outline Verification of Bearing under LV2 Seismic Forces

		Value/ LV2	Allowance	Judge
Shear Strain	Longitudinal Dir.	2.4	2.5	OK
	Transversal Dir.	0.0	2.5	OK

ii) Supporting Length

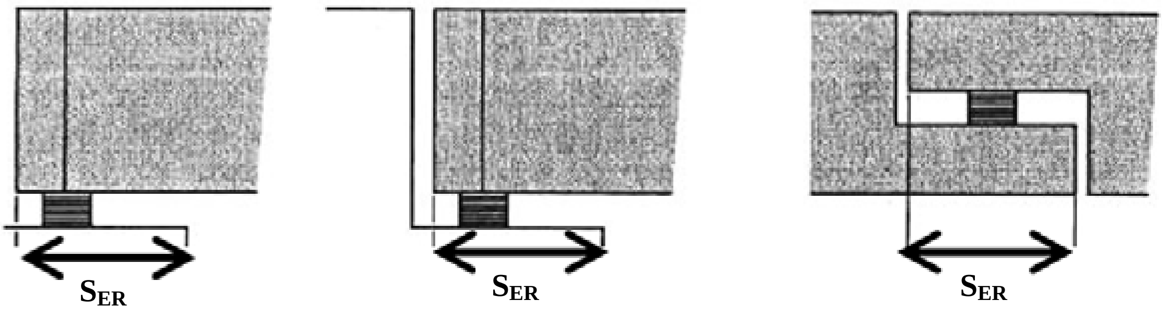


Figure 16.6.3-7 Supporting Length

Following equation gives the supporting length.
Se=0.7+0.005l (m) l: Span length
Se = 0.7+0.005*75 = 1.1m
Therefore, the supporting length shall be secured over 1.1m.

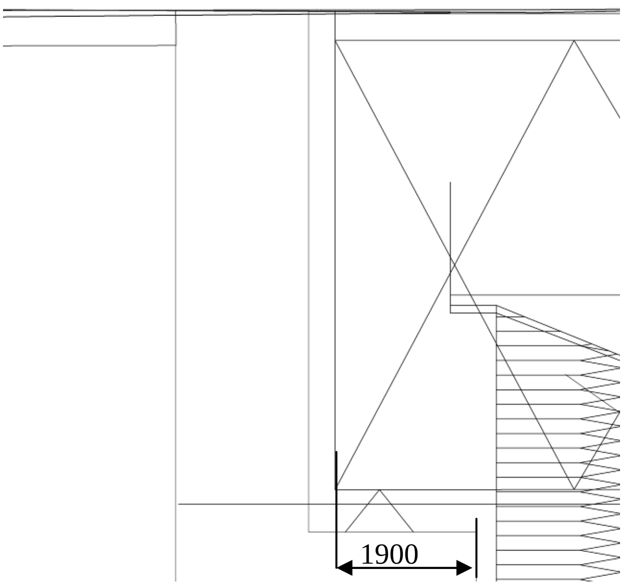


Figure 16.6.3-8 Secure the Length of "Se", Supporting Length

iii) Longitudinal Restrainer
The reaction force by dead loads is 4920kN.
The following verification can be obtained.

Table 16.6.3-14 Verification of Longitudinal Restrainer

	1.5Rd: Design Forces	Allowance
PC Cable Type 19 x 12.4mm 3-nos/ Abutment	2460 kN	2584 kN

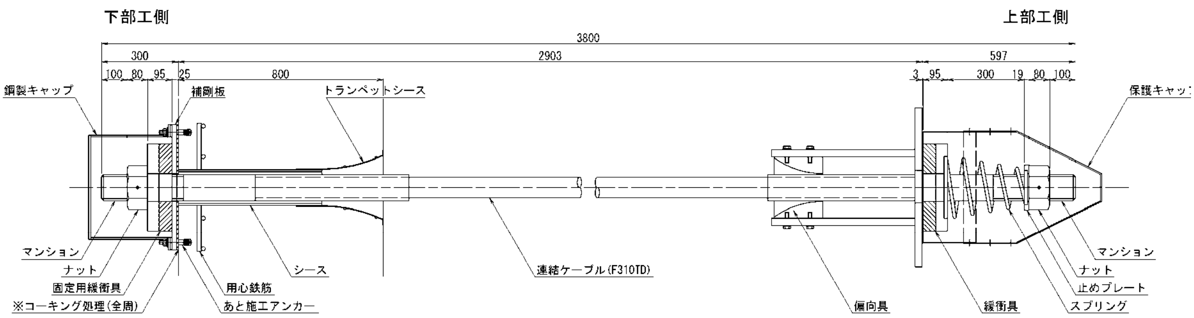


Figure 16.6.3-9 Longitudinal Restrainer for Wawa Bridge

(6) Miscellaneous Devices and Others

Miscellaneous devices in the bridge are defined as following items:

- Bearing: Evaluated above clause
- Expansion joint
- Drainage
- Wearing coat

In this clause, the devices which are not explained in other clause are explained based on seismic behaviors and current bridge condition.

i) Expansion joint
For the design methodology of expansion joint, its seismic capacity shall be secured under LV1 forces and it does not have to be secured under LV2 forces. The expansion gap between girder end and abutment shall be determined from results of dynamic modal analysis under LV2 and another expansion gap of expansion joint itself shall be determined based on seismic analysis of LV1.

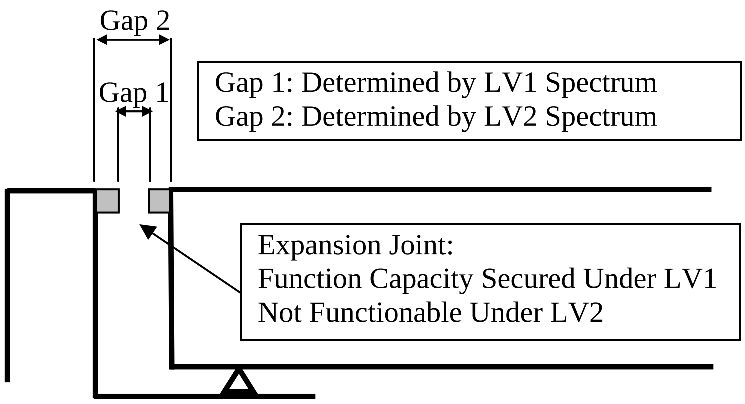


Figure 16.6.3-10 Design Methodology of Expansion Joint

However, expansion gap using laminated rubber bearing generally tends to be larger than common bearing and the size of the expansion joint tends to be larger and more expensive. Therefore, the both of expansion gap especially the gap between girder end and abutment should be carefully pay attention to displacement controlling during dynamic modal analysis, evaluating the size of expansion joint. When the gaps were so large comparing to general behavior, the stiffness of rubber bearing should be adjusted and should try the modal analysis again.

In this project, on the basis of above consideration, appropriate modal analysis are carried out, controlling caused displacements based on evaluation of stiffness of rubber bearing.
The final displacements to be used for determination of expansion joint are as follows.

- LV1: Gap 1: 13.3cm + 1.5cm (Excess allowance 15mm (JRA) ≒ 15cm
- LV2: Gap 2: 26.5cm + 1.5cm (Excess allowance 15mm (JRA) ≒ 28cm

Therefore, the expandable gap of the joint in this bridge shall be 15cm or more, and the gap between girder and abutment shall be 28cm or more, which are common results achieved under careful controlling in the dynamic analysis. Consequently, general steel type expansion joint can be adequately applied to this bridge.

ii) Drainage
Drainage system on the bridge is estimated based on current condition. In the next stage such as basic design or design stage, appropriate location of catch basins and drainage pipes shall be designed and drawn based on further investigation of accumulated rainfall data of corresponding area.

iii) Wearing coat
Wawa bridge will have concrete slab deck on the girder. Therefore, usual asphalt concrete can be applied as follows.

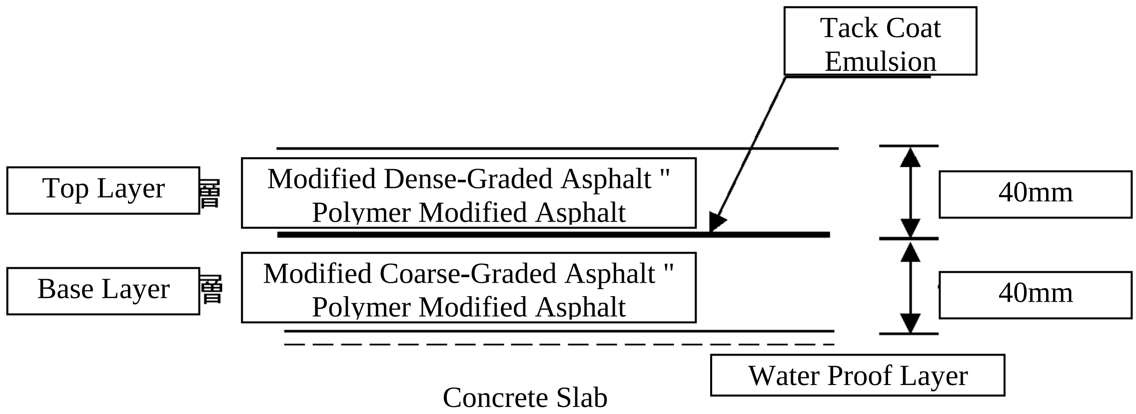


Figure 16.6.3-11 Wearing Coat System of Concrete Slab

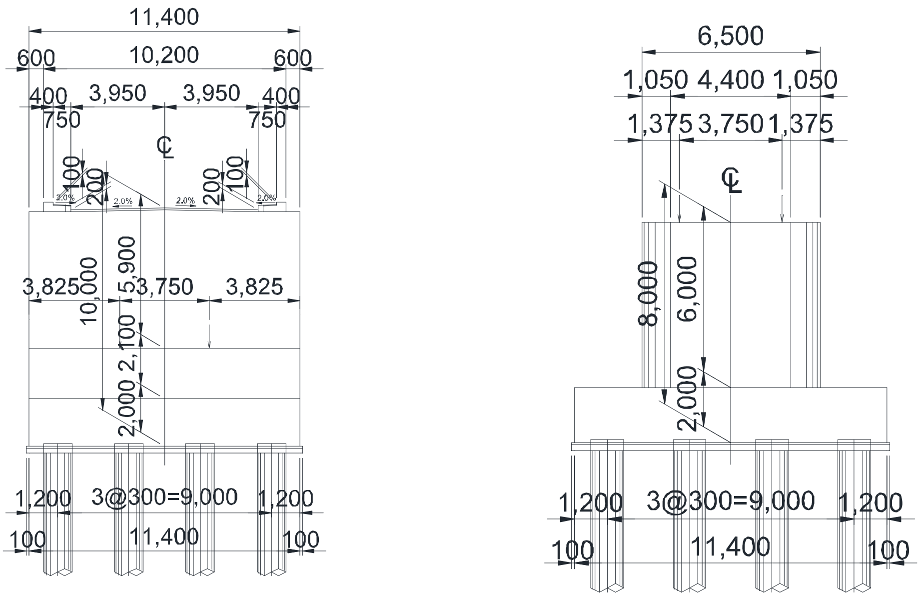
16.6.4 Summary of Outline Design Results

(1) Superstructure

Superstructure is designed based on AASHTO LRFD for the bridge type determined in multiple comparison study in consideration of various conditions. The bridge type is Continuous Composite Steel Truss. And laminated rubber bearing considering 3% of damping coefficient in dynamic modal analysis is applied in consideration of seismic behavior calculated dynamic modal analysis.

(2) Substructure and Foundation

Based on that the following dimensions are obtained as the abutment with foundation of Wawa bridge.



A1: CPP D1.2m,L=7m, n=12
A2: Spread foundation

CPP D1.2m,L=7m, n=12(P1,P2)

Figure 16.6.4-1 Sectional View of Substructure of Wawa Bridge

(3) Further Verification to be Examined in the Next Phase

The following items may be necessary to be verified or evaluated further in the next phase such as basic or detail design stages.

- Optimization and re-verification of bridge length, span arrangement and bridge types, on the basis of latest existing road condition, newly future planning and detail river condition resulted by detail hydraulic analysis
- Applicability of utilization of high-damping bearing based on specific organization regarding non-linear time history response analysis based upon comparison study regarding bearing system

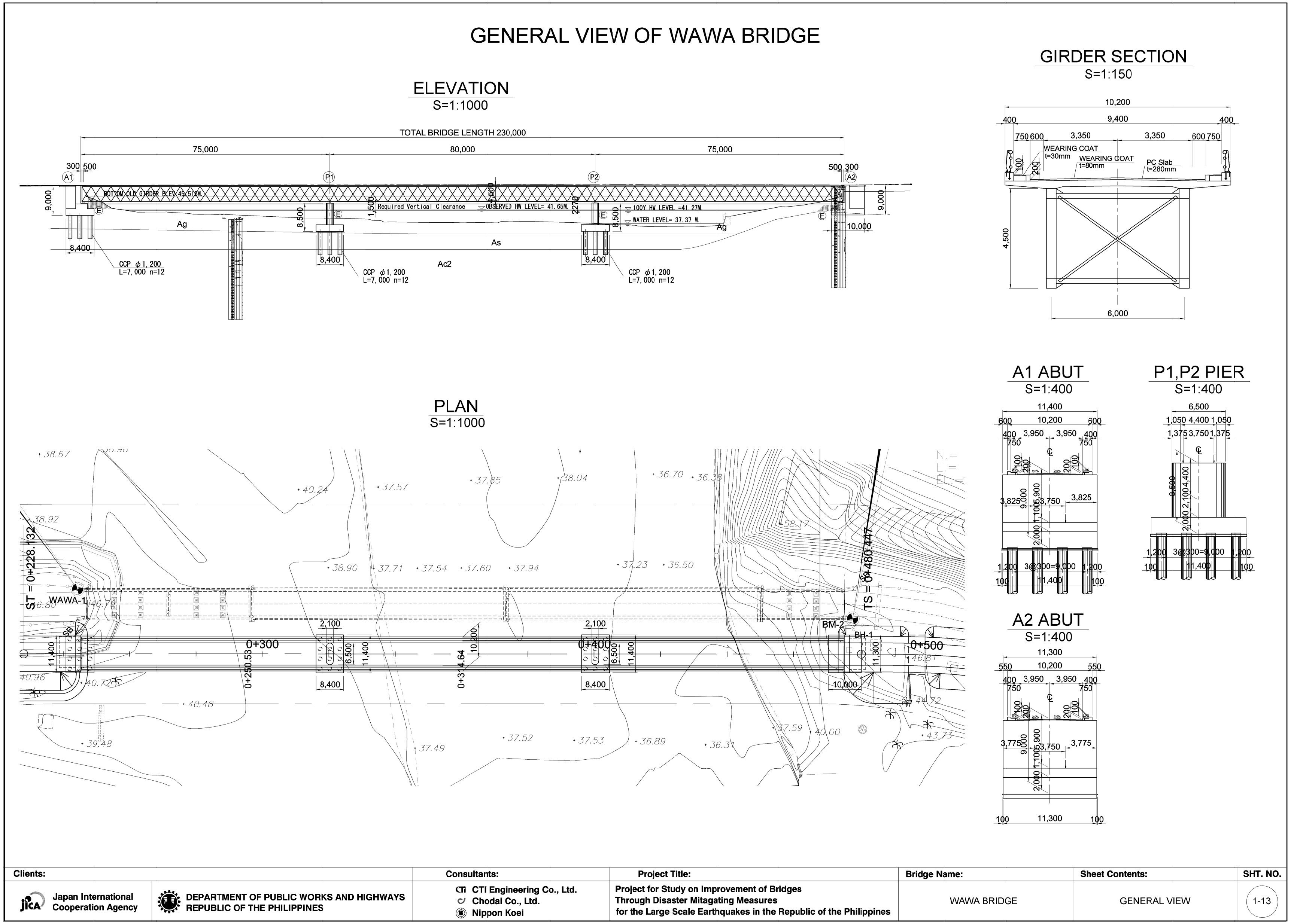


Figure 16.6.4-2 General View

(Structural Drawings are shown in Appendix-4)

CHAPTER 17 BRIDGE SEISMIC RETROFIT OUTLINE DESIGN OF SELECTED BRIDGES

17.1 Design Criteria and Conditions for Bridge Retrofit Design

17.1.1 Design Criteria

The seismic retrofit planning and design were conducted in accordance with the provisions of Bridge Seismic Design Specifications (BSDS), which was prepared in this project.

17.1.2 General Conditions for Bridge Retrofit Design

(1) Design Load Conditions

1) Load Combination and Load Factors

In the design example, the following load combination is applied.
Load combination: [1.0DL] + [0.5LL] + [1.0EQ]
Where,
DL: Dead Load
LL: Live Load
EQ: Earthquake load

2) Unit Weight

The following unit weights are applied in the design.
- Reinforced concrete: γc= 24.0 (kN/m3); rounded up for modification
  Note: - (Unit weight of concrete)= 2320(kg/m3); normal density concrete
  - (Unit weight of re-bars in 1m^3 of concrete)= 200 (kg/m3)
- Wearing surface: γ_{ws}= 22.5 (kN/m3)
- Water: γw= 10.0 (kN/m3)
- Soil: γt= (result of soil tests)

(2) Material Properties

The following material properties are applied in the design.

Table 17.1.2-1 Material Properties

Material	Strength	Remarks
Concrete	fc'=28.0 (MPa); Compressive Strength at 28 days	-To be applied to all the substructure members
Re-bars	Fy=415 (N/mm2); Grade60	-To be applied to all the substructure members - Applicable diameter: D16, D20, D25, D28, D32, D36

- Young's modulus
  - Concrete: Ec=4800$\sqrt{fc'}$ =25,000 (MPa); rounded down for modification
  - Steel: Es=20,000 (MPa)

(3) Construction Conditions

Basically, seismic retrofit planning was conducted under the following conditions.

- Existing roads are open to traffic with no traffic regulations during construction except for the retrofit work of abutments, in which at least one-lane-closure was indispensable for its implementation.
- Construction with one-lane-closure was assumed as night work in the planning.
- No temporary detour bridge installation during the construction
- Construction field is limited within the "right of way (ROW) range".

17.2 Outline Design of Lilo-an Bridge

17.2.1 Structural Data of the Existing Bridge

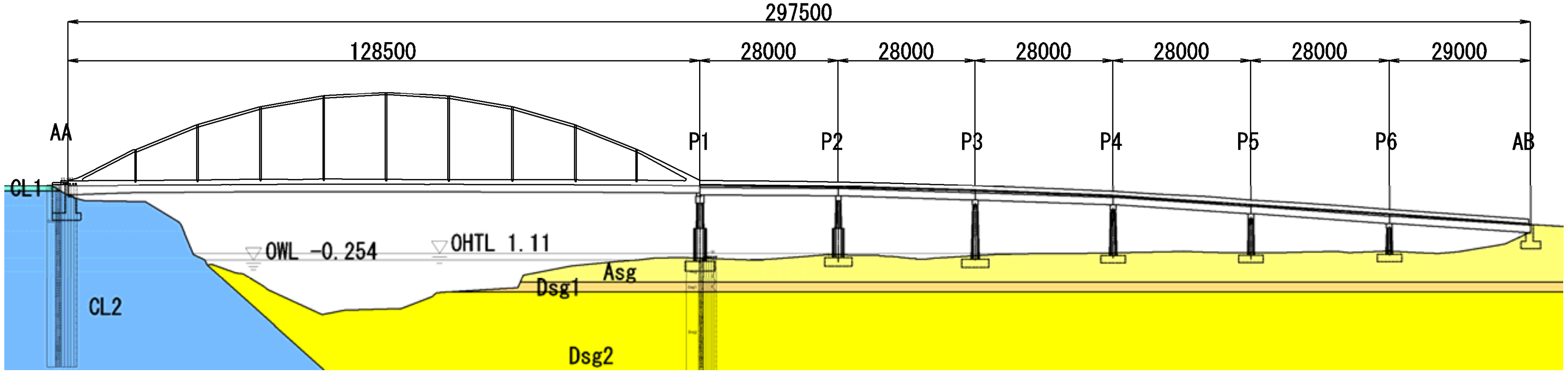
(1) Outline of the Existing Bridge

1) Construction year: 1979
2) Total bridge length: 297.5m (topographic survey result)
   (each bridge length)
   - Simply-supported steel Langer arch bridge: 128.5m
   - Simply-supported PC I-girder bridge: 28.0+28.0+28.0+28.0+28.0+29.0m

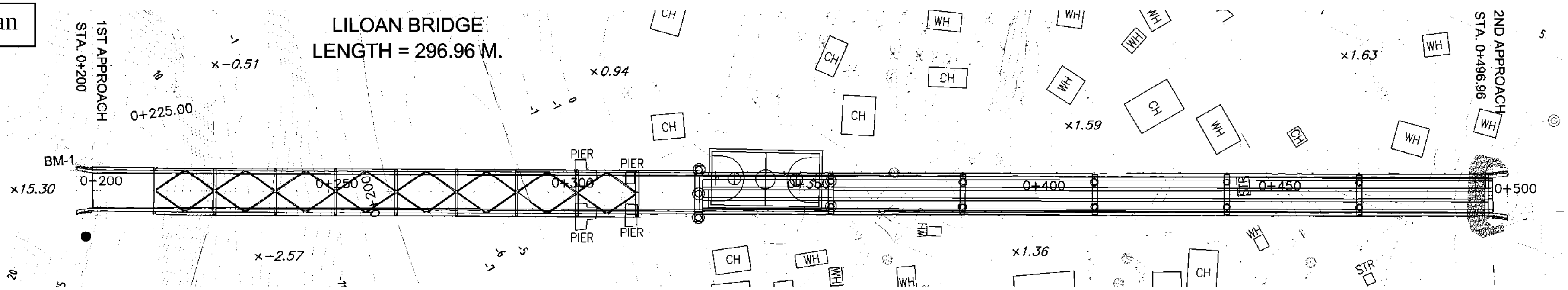
Other structural information is unknown.

(2) General View

Profile

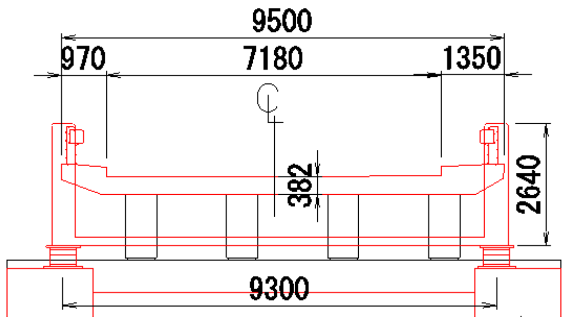


Plan

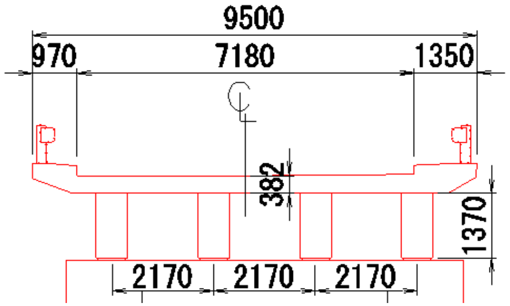


Cross-section

Simply-supported steel Langer arch bridge

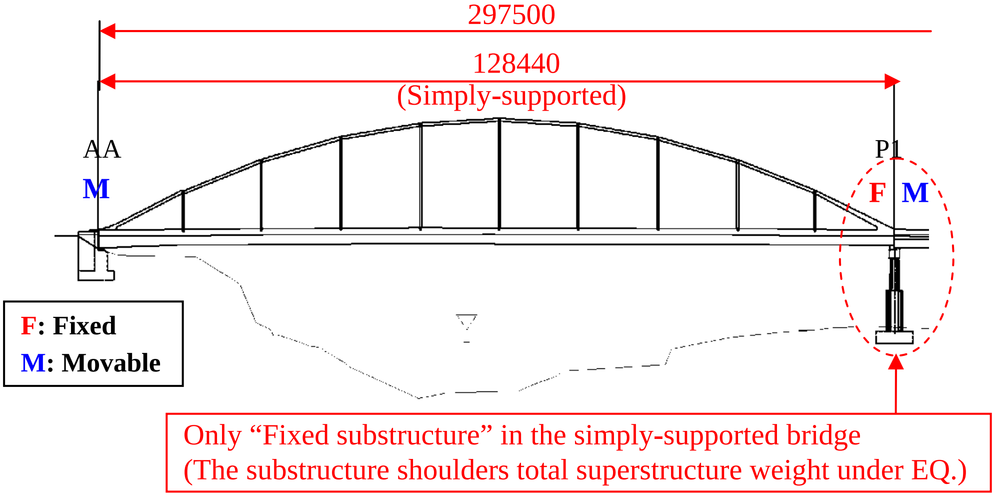
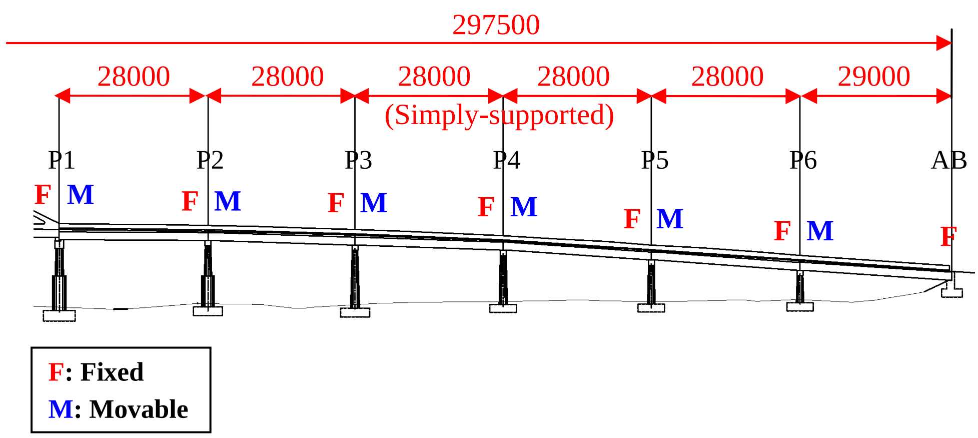


Simply-supported PC I-girder bridge



Source: topographic survey of this project

(3) Bridge/Span Length, Bridge Continuity and Bearing Restraint Conditions

Items	Contents
Span-1	
Span-7 to Span-9	

(4) Existing Pier Condition

<table>
<tr><th>Items</th><th>Contents</th></tr>
<tr><td>Concrete Strength</td><td>

[Results of Concrete Strength Test]

1) Compressive Strength Test (P3) : 25.0 (MPa)

2) Schmidt Rebound Hammer Test

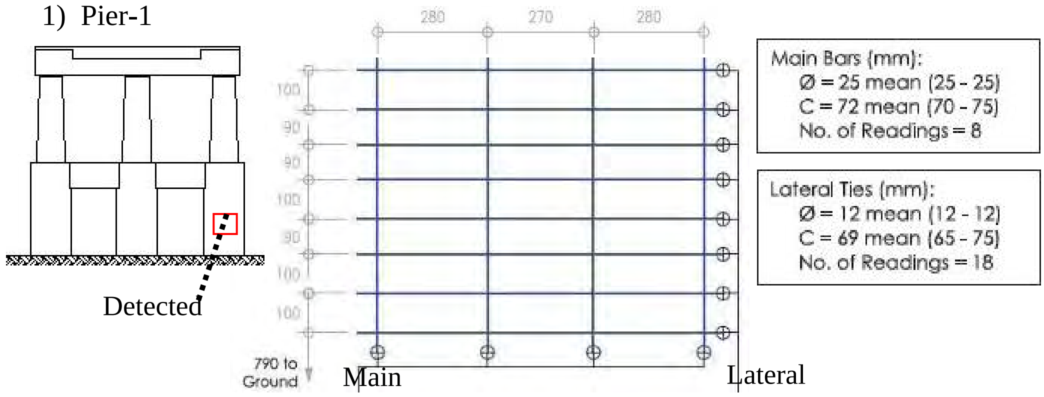
- P1: 25.0 (MPa)
- P3: 28.3 (MPa)

Ave.: 26.5

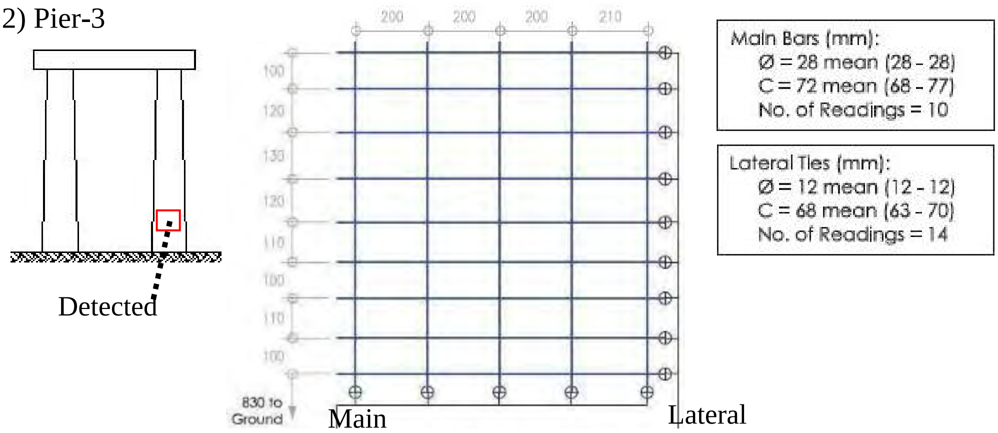
</td></tr>
<tr><td>Rebar Condition</td><td>

[Results of Rebar Detection]

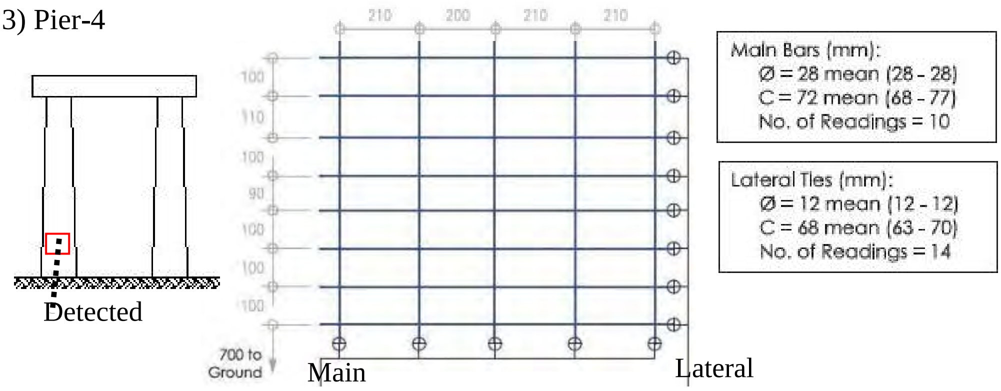
1) Pier-1



2) Pier-3

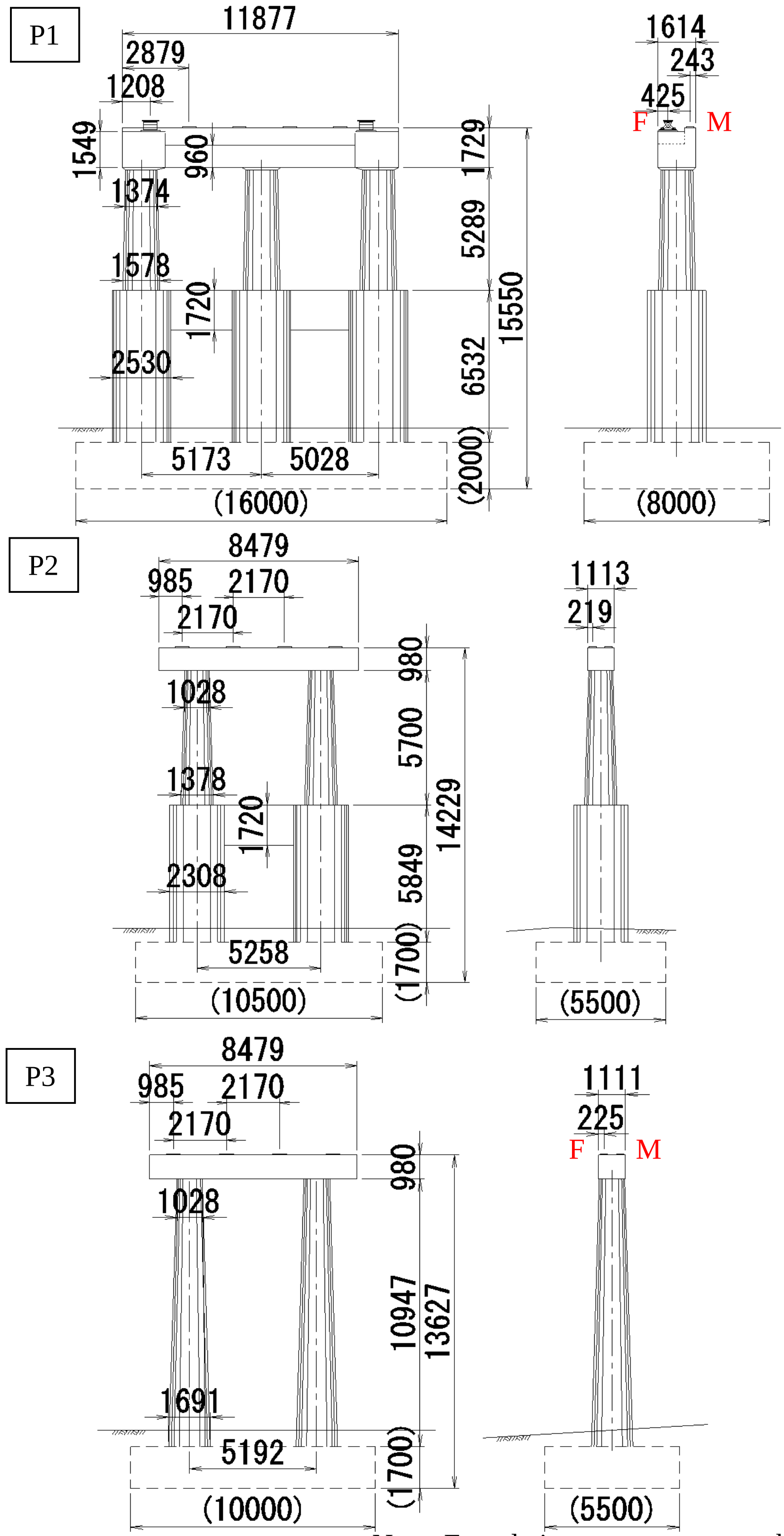


3) Pier-4

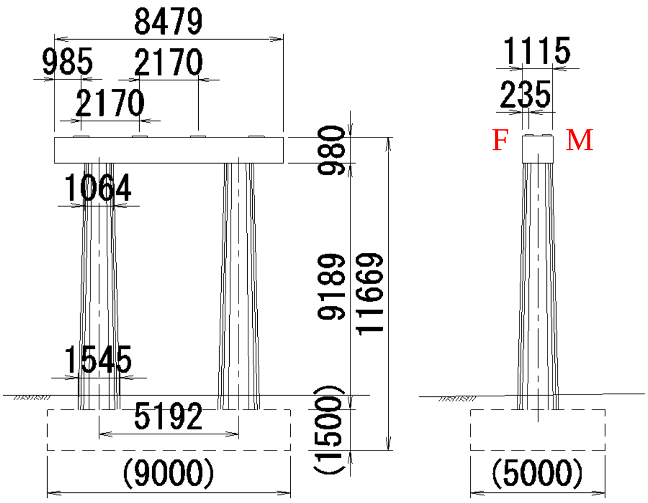
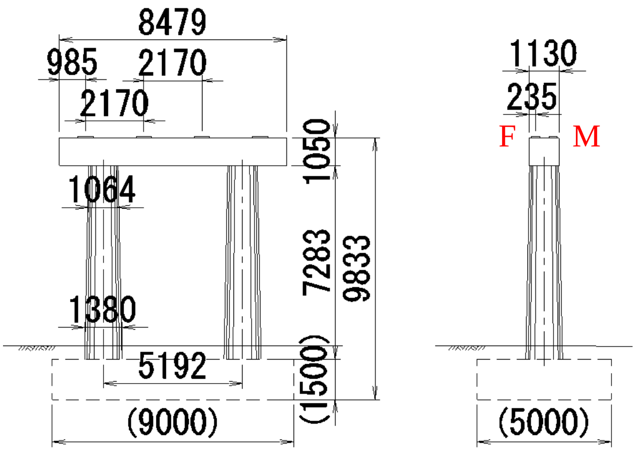
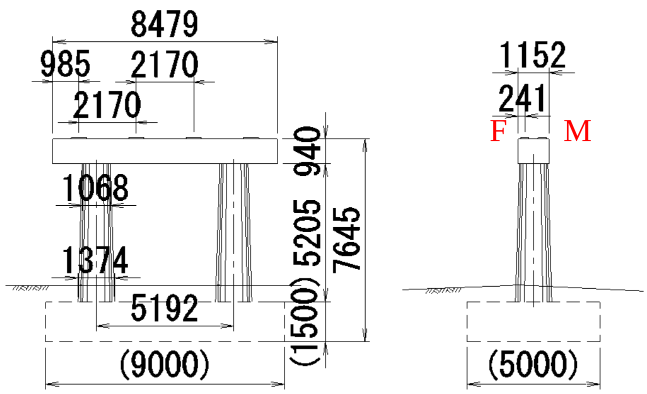


Note: Existing rebars are ignored in the seismic retrofit design.: only for capacity check

</td></tr>
</table>

<table>
<tr><th>Items</th><th>Contents</th></tr>
<tr><td>Dimension</td><td>


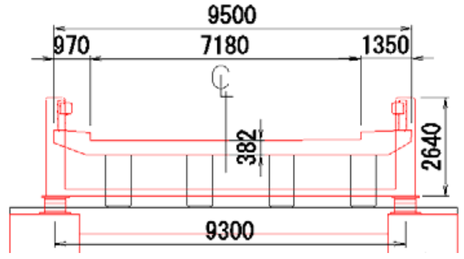
Note: Foundation structures are unknown: to be ignored in the foundation design
</td></tr>
</table>

Items	Contents
Dimension	P4  P5  P6  Note: - Cross-sectional view was assumed with Pier-3 condition - Foundation structures are unknown: to be ignored in the foundation design

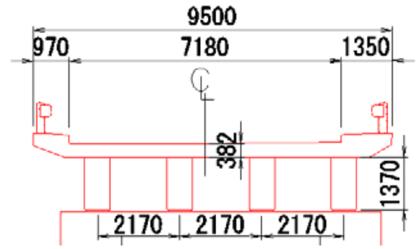
17.2.2 Design Conditions

(1) Design Loads

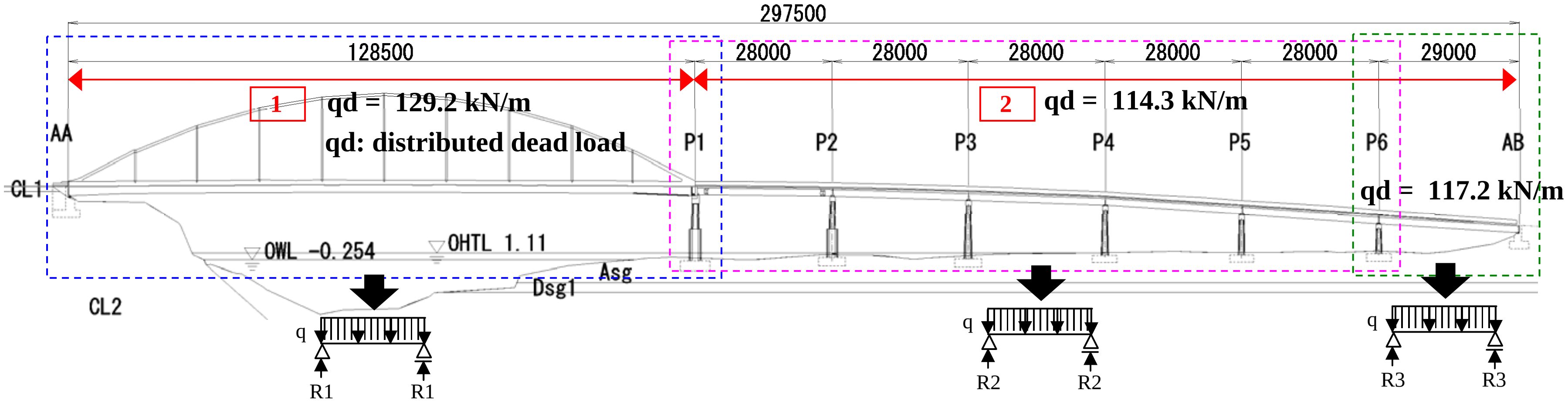
1 **Simply-supported steel Langer arch bridge**



2 **Simply-supported PC I-girder bridge**



Brige Type	Substructure			Sub-total (kN)		Total "Rd" (kN)		0.5Rl (kN)	Rd+0.5Rl (kN)
Simply-supported steel Langer arch bridge	Abut- A	-	M	8300	R1	8300	R1	1300	9600
	Pier- 1	L	F	8300	R1	9900	R1 + R2	1500	11400
Simply-supported PC I-girder bridge		R	M	1600	R1				
	Pier- 2	L	F	1600	R1	3200	R2 + R2	850	4050
Simply-pupported PC I-girder bridge		R	M	1600	R1				
	Pier- 3	L	F	1600	R1	3200	R2 + R2	850	4050
Simply-pupported PC I-girder bridge		R	M	1600	R2				
	Pier- 4	L	F	1600	R1	3200	R2 + R2	850	4050
Simply-pupported PC I-girder bridge		R	M	1600	R2				
	Pier- 5	L	F	1600	R1	3200	R2 + R2	850	4050
Simply-pupported PC I-girder bridge		R	M	1600	R2				
	Pier- 6	L	F	1600	R1	3300	R2 + R3	850	4150
Simply-pupported PC I-girder bridge		R	M	1700	R2				
	Abut- B	-	F	1700	R1	1700	R3	650	2350

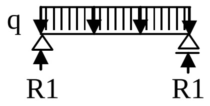


Design calculation of Lilo-an Bridge is not available. Therefore, the reaction forces of the target bridges are assumed as follows.

(1) Reaction Forces of "Simply-supported Steel Langer Arch Bridge"

- Simply-supported
- Bridge Length: L = 128.5 m
- Total Width of Road: W = 9.5 m

Superstructure Components	Unit Load		Quantity			Line Load (kN/m)	Remarks
Railing	kN/m	0.6	num	2		1.2	
Deck Slub	kN/m3	24.0	m	9.5	0.382	87.1	W = 9.5m, t = 0.382m
Steel Members	kg/m2	420	m	9.5		39.9	Steel Langer Arch Bridge



Sum: q = 128.2 kN/m

Loading Length: 128.5 m

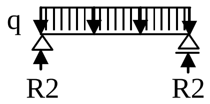
Total Load: 16,500 kN

R1 = 16,500 / 2 = 8300 kN (rounded up)

(2) Reaction Forces of "Simply-supported PC I-girder Bridge"

- Simply-supported
- Bridge Length: L = 28.0 m
- Total Width of Road: W = 9.5 m

Superstructure Components	Unit Load		Quantity			Line Load (kN/m)	Remarks
Railing	kN/m	0.6	num	2		1.2	
Deck Slub	kN/m3	24.0	m	9.5	0.382	87.1	W = 9.5m, t = 0.382m
PC I-girder	kN/no.	617	m	4		24.7	PC-I Girder Bridge



Sum: q = 113.0 kN/m

Loading Length: 28.0 m

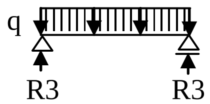
Total Load: 3,200 kN

R2 = 3,200 / 2 = 1600 kN

(3) Reaction Forces of "Simply-supported PC I-girder Bridge"

- Simply-supported
- Bridge Length: L = 29.0 m
- Total Width of Road: W = 9.5 m

Superstructure Components	Unit Load		Quantity			Line Load (kN/m)	Remarks
Railing	kN/m	0.6	num	2		1.2	
Deck Slub	kN/m3	24.0	m	9.5	0.382	87.1	W = 9.5m, t = 0.382m
PC I-girder	t/no.	639	m	4		25.6	PC-I Girder Bridge



Sum: q = 113.9 kN/m

Loading Length: 29.0 m

Total Load: 3,400 kN

R3 = 3,400 / 2 = 1700 kN

The following figure summarizes “site-specific design spectrum of 50-, 100-, 500-, and 1000-year return period for Lilo-an Bridge site” which were developed in this study.

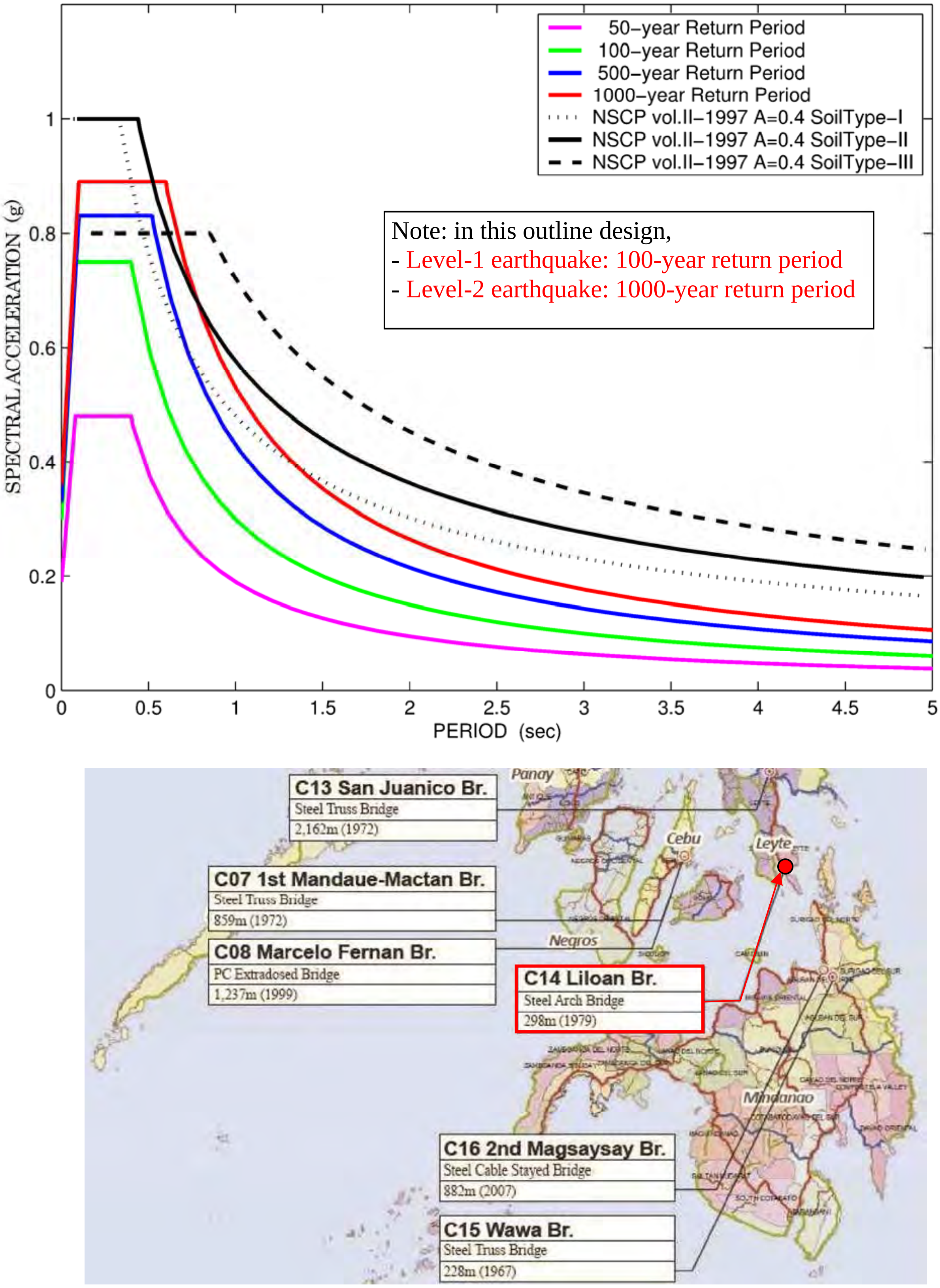


Figure 17.2.2-1 Site-Specific Design Spectrum of 50-, 100-, 500-, and 1000-Year Return Periods for Lilo-an Bridge Site

The following table summarizes “the load distribution of existing bridge under earthquakes” and “application point of seismic inertial forces”.

Table 17.2.2-1 Load Distribution under EQ and Application Point of Seismic Inertial Forces

Substructure		Rd (kN)	Longitudinal Direction				Transeverse Direction			
			Restraint Condition	Wu (kN)		h (m)	Restraint Condition	Wu (kN)		h (m)
Abut- A	-	8300	M	-	-	-	F	-	-	-
Pier- 1	L	8300	F	16600	16600	0	F	8300	9900	1.6
	R	1600	M	0		0	F	1600		
Pier- 2	L	1600	F	3200	3200	0	F	1600	3200	1.4
	R	1600	M	0		0	F	1600		
Pier- 3	L	1600	F	3200	3200	0	F	1600	3200	1.4
	R	1600	M	0		0	F	1600		
Pier- 4	L	1600	F	3200	3200	0	F	1600	3200	1.4
	R	1600	M	0		0	F	1600		
Pier- 5	L	1600	F	3200	3200	0	F	1600	3200	1.4
	R	1600	M	0		0	F	1600		
Pier- 6	L	1600	F	3200	3200	0	F	1600	3300	1.4
	R	1700	M	0		0	F	1700		
Abut- B	-	1700	F	3400	3400	0	F	-	-	-

L: Left side bearing to the longitudinal direction
R: Right side bearing to longitudinal direction
h (m): Height from the top of the substructure body to the height of the superstructural inertia force
Wu (kN): Weight of the superstructure portion supported by the substructure body concerned

(2) Soil Conditions

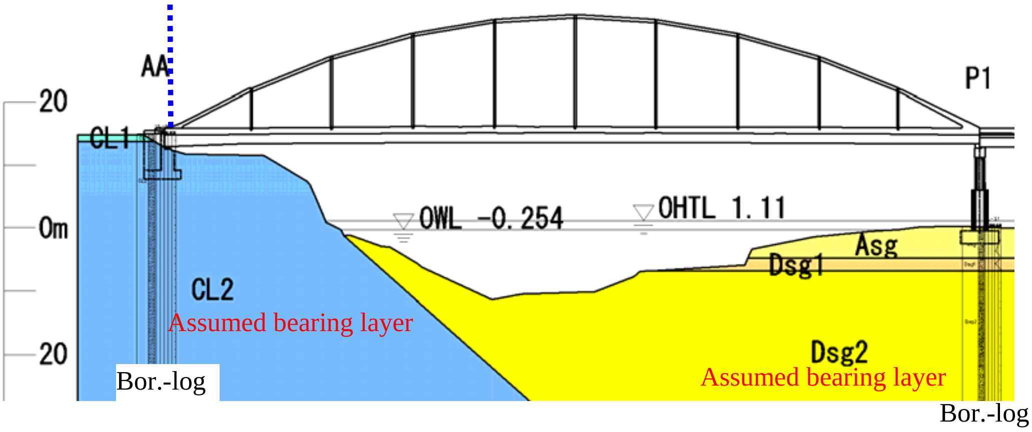
Soil condition of Lillo-an Bridge is summarized as follows.

Design Soil Parameters for "Lilo-an Bridge" (LIL-N1 Site) → Soil Type: I

Layer Name	Soil Type	N	γt	C	Φ	α E_0		Vs	DE
						α = 4	α = 8		L2
		-	(kN/m^3)	(kN/m^2)	(°)	(kN/m^2)	(kN/m^2)	(m/sec)	-
CL1	Clay	50	18	313	0	140,000	280,000	292	-
CL2	Rock	50	21	514	21	532,000	1,064,000	292	-

α = 4 (Service state), 8 (Under earthquake)

Based on results of SPT & laboratory tests

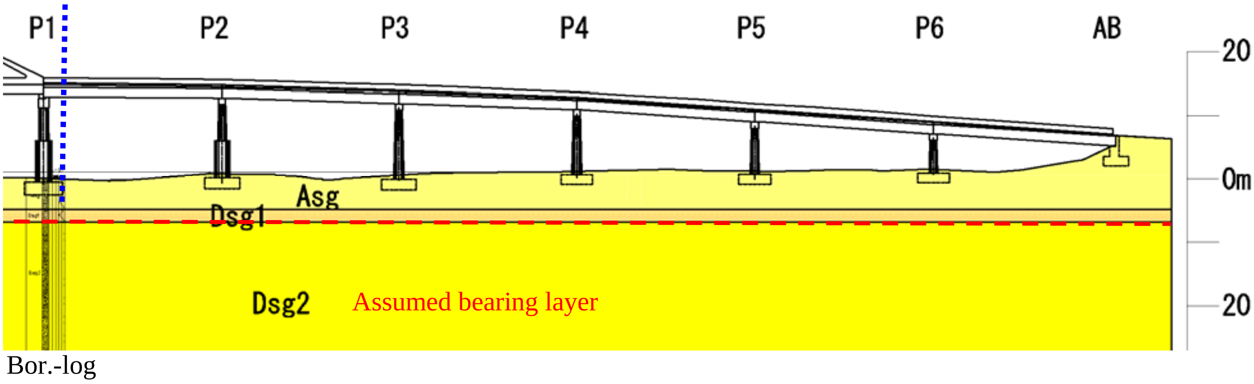


Design Soil Parameters for "Lilo-an Bridge" (LIL-S1 Site) → Soil Type: I

Layer Name	Soil Type	N	γt	C	Φ	α E_0		Vs	DE
						α = 4	α = 8		L2
		-	(kN/m^3)	(kN/m^2)	(°)	(kN/m^2)	(kN/m^2)	-	-
Asg	Gravel	36	18	0	41	100,800	201,600	264	-
Dsg1	Sandy gravel	32	20	0	39	89,600	179,200	292	-
Dsg2	Sandy gravel	50	20	0	38	140,000	280,000	295	-

α = 4 (Service state), 8 (Under earthquake)

Based on results of SPT & laboratory tests



(3) Hydrological Condition

The Hydrological condition of 1st Mandaue-Mactan Bridge is as follows.
1. Observed water level (OWL): -0.254m from mean sea level (MSL: 0m)
2. Observed high tide level (OHTL): 1.11m from mean sea level (MSL: 0m)
3. Navigation Clearance under the arch bridge: not defined (no large ships go under the bridge.)
The above conditions are illustrated in the following figure.

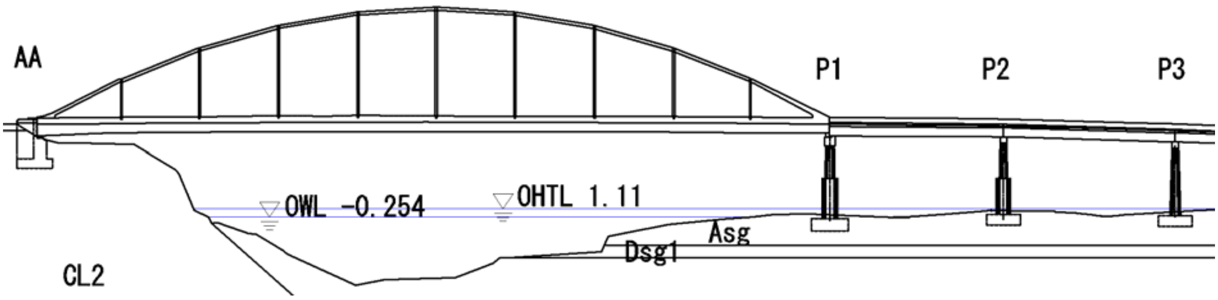


Figure 17.2.2-2 Hydrological Condition of Lilo-an Bridge

17.2.3 Seismic Capacity Verification of Existing Structures

(1) Summary of Seismic Capacity Verification

Seismic capacity verification of existing structures was conducted for Pier-1 and Pier-2, in accordance with provisions of LRFD for pier columns, and JRA for pier foundations. The following figure highlights the result of the seismic capacity verification of the existing structures. The detail of the verification is shown from the next page.

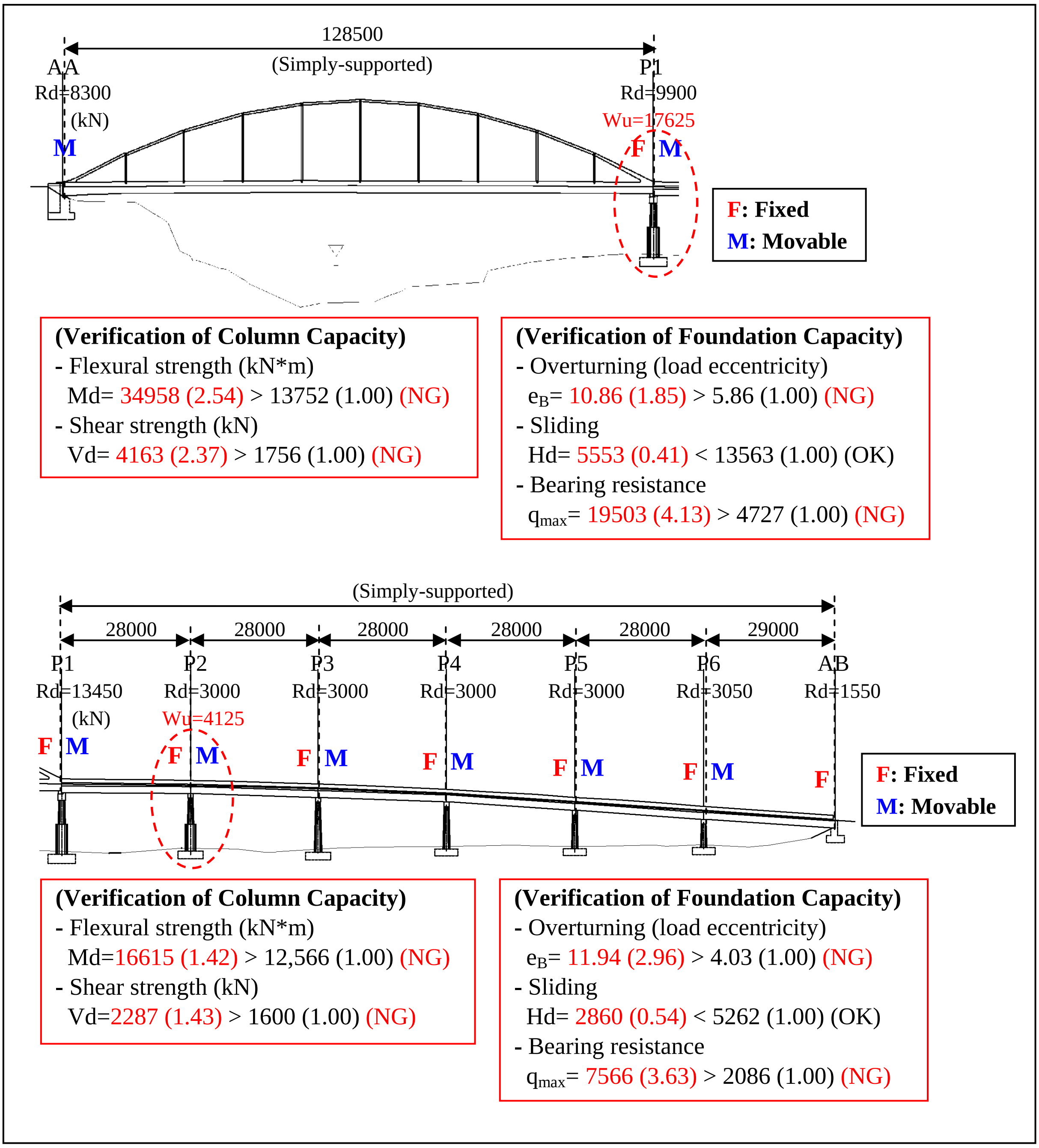
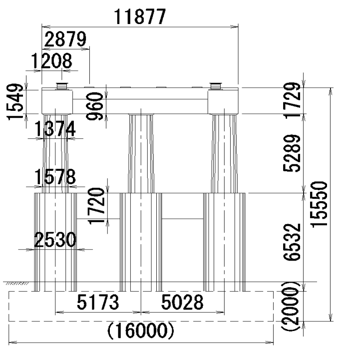
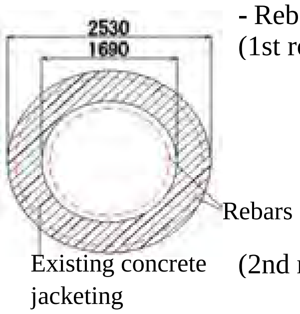


Figure 17.2.3-1 Summary of Seismic Capacity Verification

(2) Seismic Capacity Verification of Pier-1

1) Verification of Column Seismic Capacity





- Rebars:
(1st row)

(Longitudinal reinforcement)	
- diameter: 25 (mm)	- fy= 415 (N/mm2)
- No. of rebars: 26	- spacing: 280 (mm)
- concrete cover thickness: 75 (mm)	
(Transverse reinforcement)	
- diameter: 13 (mm)	- fy= 415 (N/mm2)
- spacing: 100 (mm)	

(2nd row)

(Longitudinal reinforcement)	
- diameter: 28 (mm)	- fy= 415 (N/mm2)
- No. of rebars: 26	- spacing: 200 (mm)
- concrete cover thickness: 490 (mm)	

Note: Pier-1 & 3 condition is applied.

- Bearing restraint condition: Fixed(L) + Movable(R)
- Reaction force: Rd+0.5Rl = 11,400 (kN)
- Weight of single column Wp = 1,245 (kN)
- Horizontal seismic coefficient: Csm= 0.38 (longitudinal dir.: T=1.39 (s))
  0.89 (transverse dir.T=0.54 (s))
- R-factor: 1.5 (importance: critical)
- Direction of seismic force: Transverse dir.
- Loads for capacity verification

Seismic Forces (per Column) in Longitudinal dir.

	W	Csm	H=Wu*Csm	h	M=H*h
	(kN)	-	(kN)	(m)	(kN*m)
Superstructure	5,533	0.38	2,110	13.4	28,270
Pier	1,245	0.38	475	6.2	2,944
Sum		V_L=	2,584	M_L=	31,213

Seismic Forces (per Column) in Transverse dir.

	W	Csm	H=Wu*Csm	h	M=H*h
	(kN)	-	(kN)	(m)	(kN*m)
Superstructure	3,350	0.89	2,982	15.0	44,723
Pier	1,245	0.89	1,108	6.2	6,872
Sum		V_T=	4,090	M_T=	51,595

Where,
W (kN): Weight of the structures under consideration
H (kN): Horizontal seismic inertial force
h (m): Height from the botttom of column to the height of seismic inertial force
M (kN*m): Bending moment
V_L, V_T (kN): Shear force which acts at the bottom of pier columns/walls
M_L, M_T (kN): Bending moment which acts at the bottom of pier columns/walls

- Forces for verification (transverse dir.)

Nd= Rd+0.5*Rl+Wp — Nd: Design axial force for M-N interaction diagram
= 5,045 (kN)

Vd= $(Vmax^2+(0.3Vmin)^2)^{0.5}$ — Vd: Shear force for capacity verification
= 4,163 (kN) — (Vmax= 4,090 Vmin= 2,584)

Md= $(Mmax^2+(0.3Mmin)^2)^{0.5}/R$ — Md: Bending moment for capacity verification
34,958 (kN*m) — (Mmax= 51,595 Mmin= 31,213)

Note: - R-factor is applied to only bending moment.
- Load combination: consideration of 30% of perpendicular force

- Verification of "flexural strength"

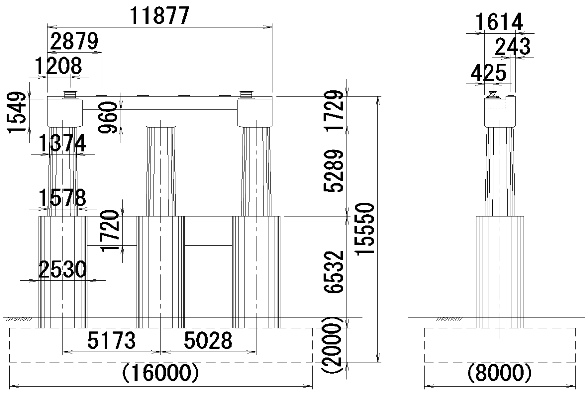
Md= 34,958 > 13,752 (=ϕ*Mn) (NG)
(2.54) (1.00)

- Verification of "shear strength"

Vd= 4,163 > 1,756 (=ϕ*Vn) (NG)
(2.37) (1.00)

2) Verification of Foundation Stability

2. Verification of foundation stability



- Direction of seismic force: Longitudinal dir.
- Load condition

Rd+0.5Rl=	11,400	(kN)
Wu=	16,600	(kN)
Wp=	3,736	(kN)
As=	0.36	(g)
Mu=	45,840	(kN*m)
Ws=	1,463	(kN) Ws: weight of sand
Fw=	-3,241	(kN) Fw: buoyancy

Seismic forces of footing

	Wf	0.5*As	Ff=Wf*(0.5*As)	hf/2	Mf=H*h
	(kN)	(g)	(kN)	(m)	(kN*m)
Footing	6,144	0.18	1,106	0.63	700

Where,
Wu (kN): Weight of superstructure under consideration
Wp (kN): Weight of pier column & coping
Wf (kN): Weight of pier footing
Ff (kN): Horizontal seismic inertial force
hf (m): height of footing
Mf (kN*m): Bending moment

- Forces for capacity verification

Vd= Rd+0.5Rl+Wp+Wf+Ws+Fw
= 19,503 (kN)
Hd= Mp/h+Ff (Mp= 59,592 Ff= 1,106)
= 5,553 (kN)
Md= Mp+Mf=1.3Mu+Mf
= 60,292 (kN*m) (Mu= 45,840 Mf= 700)
Note: h= 13.4 (m) h: Height from the botttom of column to the point of seismic inertial force

- Verification of "overturning (load eccentricity) "

e_B= 10.86 > 5.86 = 0.733*L (NG)
(1.85) (1.00)

Where,
L= 8.0 (m) L: footing length

- Verification of "sliding"

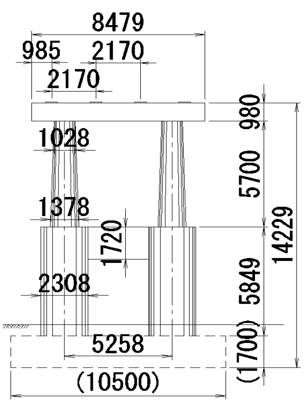
Hd= 5,553 < 13,563 =Rr (OK)
(0.41) (1.00)

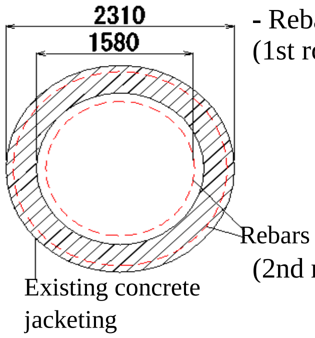
- Verification of "bearing resistance"

qmax=Vd= 19,503 > 4,727 = φb*qn (NG)
(4.13) (1.00)

(3) Seismic Capacity Verification of Pier-2

1) Verification of Column Seismic Capacity





- Rebars: (1st row)

(Longitudinal reinforcement)	
- diameter: 25 (mm)	- fy= 415 (N/mm2)
- No. of rebars: 24	- spacing: 280 (mm)
- concrete cover thickness: 75 (mm)	
(Transverse reinforcement)	
- diameter: 13 (mm)	- fy= 415 (N/mm2)
- spacing: 100 (mm)	

(2nd row)

(Longitudinal reinforcement)	
- diameter: 28 (mm)	- fy= 415 (N/mm2)
- No. of rebars: 24	- spacing: 200 (mm)
- concrete cover thickness: 490 (mm)	

Note: Pier-1 & 3 condition is applied.

- Bearing restraint condition: Fixed(L) + Movable(R)
- Reaction force: Rd+0.5Rl = 4,150 (kN)
- Weight of single column Wp = 878 (kN)
- Horizontal seismic coefficient: Csm= 0.54 (longitudinal dir.: T=0.98 (s))
  0.89 (transverse dir.T=0.51 (s))
- R-factor: 1.5 (importance: critical)
- Loads for capacity verification

Seismic Forces (per Column) in Longitudinal dir.

	W	Csm	H=Wu*Csm	h	M=H*h
	(kN)	-	(kN)	(m)	(kN*m)
Superstructure	1,650	0.54	893	12.5	11,179
Pier	878	0.54	475	5.3	2,518
Sum		V_L=	1,368	M_L=	13,697

Seismic Forces (per Column) in Transverse dir.

	W	Csm	H=Wu*Csm	h	M=H*h
	(kN)	-	(kN)	(m)	(kN*m)
Superstructure	1,650	0.89	1,469	13.9	20,442
Pier	878	0.89	781	5.3	4,140
Sum		V_T=	2,250	M_T=	24,582

Where,
W (kN): Weight of the structures under consideration
H (kN): Horizontal seismic inertial force
h (m): Height from the botttom of column to the height of seismic inertial force
M (kN*m): Bending moment
V_L, V_T (kN): Shear force which acts at the bottom of pier columns/walls
M_L, M_T (kN): Bending moment which acts at the bottom of pier columns/walls

- Forces for verification (transverse dir.)

Nd= Rd+0.5*Rl+Wp — Nd: Design axial force for M-N interaction diagram
= 5,028 (kN)

Vd= $(Vmax^2+(0.3Vmin)^2)^{0.5}$ — Vd: Shear force for capacity verification
= 2,287 (kN) — (Vmax= 2,250 Vmin= 1,368)

Md= $(Mmax^2+(0.3Mmin)^2)^{0.5}/R$ — Md: Bending moment for capacity verification
16,615 (kN*m) — (Mmax= 24,582 Mmin= 13,697)

Note: - R-factor is applied to only bending moment.
- Load combination: consideration of 30% of perpendicular force

- Verification of "flexural strength"

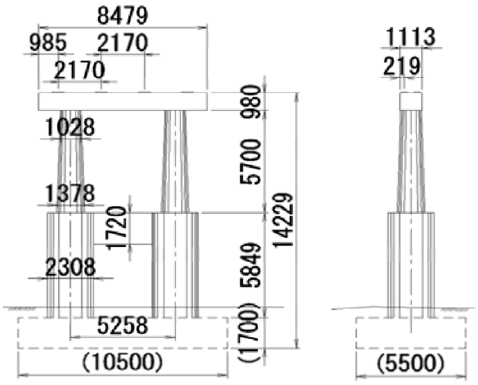
Md= 16,615 > 11,739 (NG)
(1.42) (1.00)

- Verification of "shear strength"

Vd= 2,287 > 1,600 (NG)
(1.43) (1.00)

2) Verification of Foundation Stability

2. Verification of foundation stability



- Direction of seismic force: Longitudinal dir.
- Load condition

Rd+0.5Rl=	4,150	(kN)
Wu=	3,300	(kN)
Wp=	1,755	(kN)
As=	0.36	(g)
Mu=	26,087	(kN*m)
Ws=	533	(kN) Ws: weight of sand
Fw=	-1,229	(kN) Fw: buoyancy

Seismic forces of footing

	Wf	0.5*As	Ff=Wf*(0.5*As)	hf/2	Mf=H*h
	(kN)	(g)	(kN)	(m)	(kN*m)
Footing	2,356	0.18	424	0.58	245

Where,
Wu (kN): Weight of superstructure under consideration
Wp (kN): Weight of pier column & coping
Wf (kN): Weight of pier footing
Ff (kN): Horizontal seismic inertial force
hf (m): height of footing
Mf (kN*m): Bending moment

- Forces for capacity verification

Vd= Rd+0.5Rl+Wp+Wf+Ws+Fw
= 7,566 (kN)
Hd= Mp/h+Ff (Mp= 33,913 Ff= 424)
= 2,860 (kN)
Md= Mp+Mf=1.3Mu+Mf
= 34,158 (kN*m) (Mu= 26,087 Mf= 245)
Note: h= 13.9 (m) h: Height from the botttom of column to the point of seismic inertial force

- Verification of "overturning (load eccentricity) "

e_B= 11.94 > 4.03 = 0.733*L (NG)
(2.96) (1.00)

Where,
L= 5.5 (m) L: footing length

- Verification of "sliding"

Hd= 2,860 < 5,262 =Rr (OK)
(0.54) (1.00)

- Verification of "bearing resistance"

qmax=Vd= 7,566 > 2,086 = φb*qn (NG)
(3.63) (1.00)

17.2.4 Comparative Studies on Seismic Capacity Improvement Schemes

(1) Outline of Comparison Studies on Seismic Capacity Improvement Schemes

Selection of “Seismic Capacity Improvement Methods” was done in accordance with the following flowchart process.

Control of Seismic Inertial Force by Changing Bearing Restraint Condition

↓

Seismic Capacity Improvement Scheme for Pier Columns

↓

Seismic Capacity Improvement Scheme for Pier Copings

↓

Seismic Capacity Improvement Scheme for Foundations
(1) Improvement scheme for foundation of piers
(2) Improvement scheme for abutments

↓

Planning for Unseating Prevention System
(1) Planning for replacement of bearings
(2) Planning for seat extender
(3) Planning for unseating prevention devices
(4) Planning for structural limiting (shear keys)

↓

Planning for Repair Works
(not part of seismic capacity improvement)

↓

Outline design

Figure 17.2.4-1 Outline of Comparative Studies on Seismic Capacity Improvement Schemes

(2) Control of Seismic Inertial Force by Changing Bearing Restraint Conditions

Old long-span simply-supported bridges are likely to have only one substructure with fixed bearings in longitudinal direction. The fixed substructures are regarded as the weakest point of the bridge structures, for the fixed piers shoulder total superstructure weight under earthquakes. In this case, recombination of bearing restraint conditions should be considered with the application of seismic devices (ex. elastomeric bearing and seismic damper) in order to control the scale of seismic inertial forces on each substructure and save the only fixed substructures. The mechanism of seismic device application is shown below.

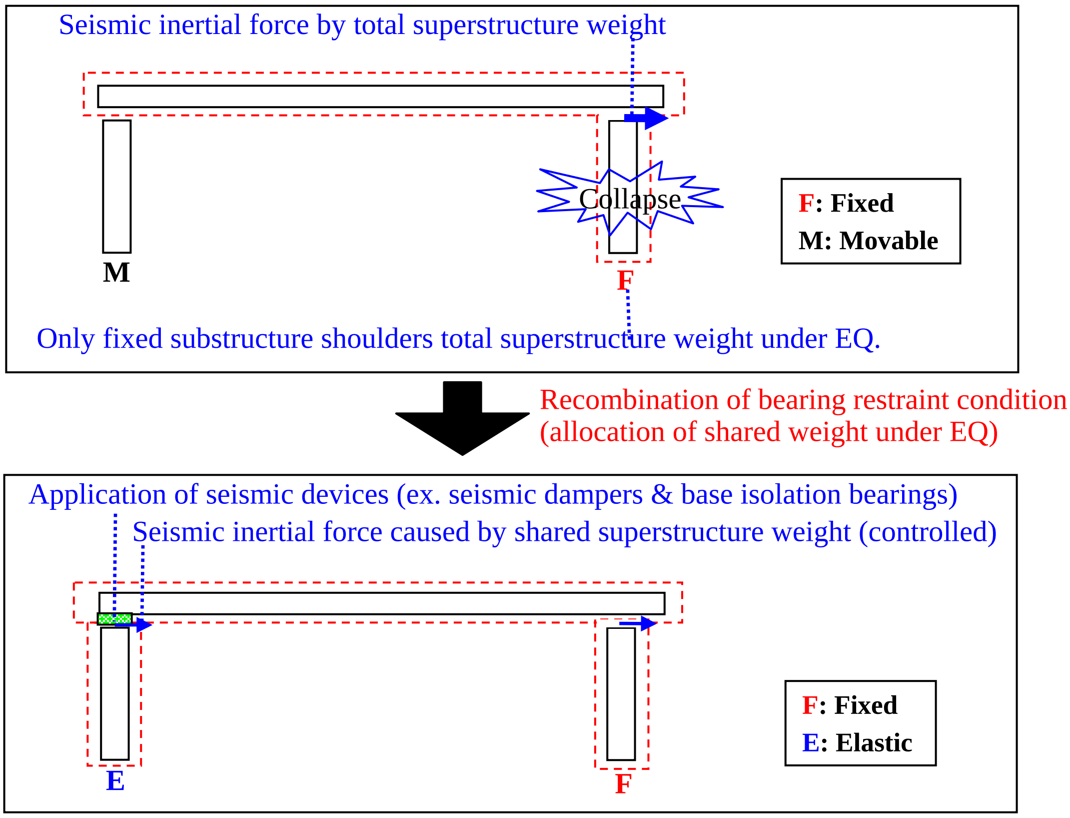


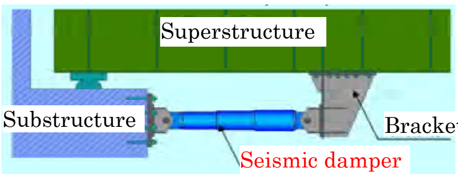
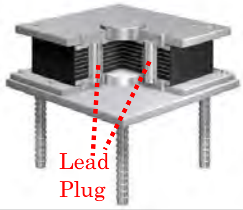
Figure 17.2.4-2 Control of Seismic Inertial Force by Application of Seismic Devices

In this study, the following two seismic devices are compared for the application.

- Alternative-1: Seismic damper (hydraulic cylinder type)
- Alternative-2: Elastomeric bearing

As a result, “seismic damper” is recommended to mitigate the seismic inertial force of fixed piers. Generally speaking, elastomeric bearings are more cost-effective than seismic dampers. However, they have problems in “structural characteristics” and “constraints of construction”. Elastomeric bearings are recommended to be applied to newly-constructed bridges, considering its structural and constructive restrictive conditions. The detail of the comparison is shown in the following table.

Table 17.2.4-1 Comparison of Seismic Devices

	Seismic damper (hydraulic cylinder type)		Base isolation bearing	
Outline of improvement method	 (Source: Japan Bridge Association)		 High damping rubber type / Rubber with lead type (Source: Japan Bridge Association)	
	- To absorb seismic energy and control seismic inertial force on substructures		- To reduce seismic inertial force to substructures - To make natural period longer; base isolation - To avoid the “Sympathetic Vibration” of substructure and superstructure.	
Structural characteristic	- Easy to control the seismic inertial force on substructures - Possible to restrain/control the amout of structural movement under EQ - Possible to be used as unseating prevention device	A	- Difficult To control the seismic inertial force on substructures - New bearings are wider and taller than old ones: need of larger space for the installation - Impossible to restrain/control the amout of structural movement under EQ; superstructures and backwalls collide with each other under EQ	D
Constraints of construction	- Quick and easy installtation - No need of removal of existing structures for the installation	A	- Need of partial removal of existing pier coping to fit new bearings into the space; new bearing height is higer than that of old one	C
Duration	Short	A	Typical priod of bearing installation	B
Recommend-ation	Recommended		Not applicable	

Recommendation for installation location of seismic dampers is shown in the following figure.

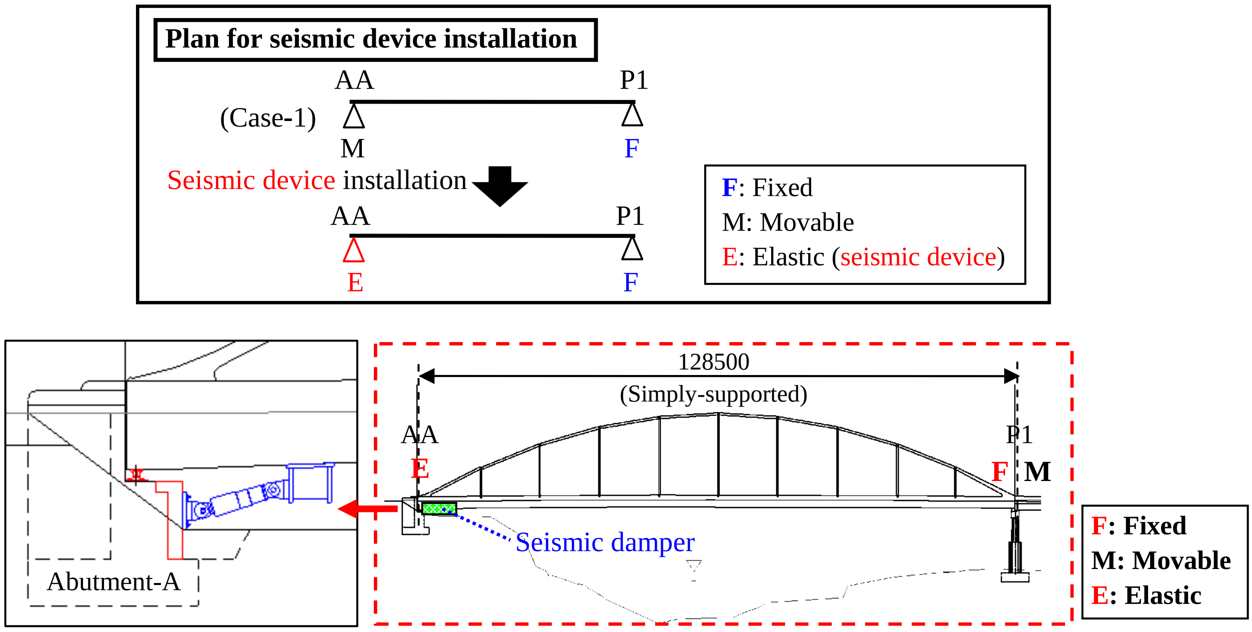


Figure 17.2.4-3 Recommendation for Location of Seismic Damper Installation

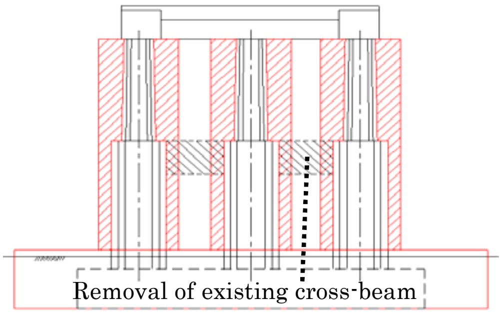
(3) Seismic Capacity Improvement Scheme for Pier Columns

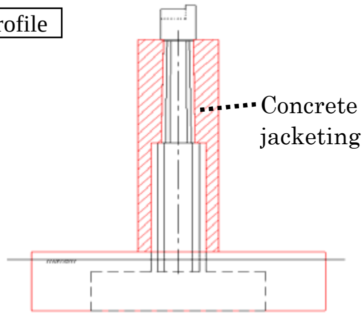
The following two improvement schemes were compared for pier columns so as to improve flexural resistance and shear resistance.

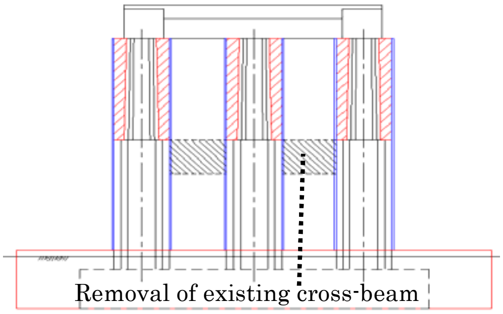
- Alternative-1: Concrete jacketing
- Alternative-2: Steel plate jacketing

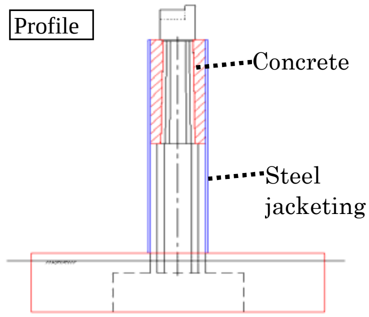
As a result of evaluation, “Concrete jacketing” was selected for its structural advantage and overall suitability for its implementation. The detail of the comparison is shown in the next page.

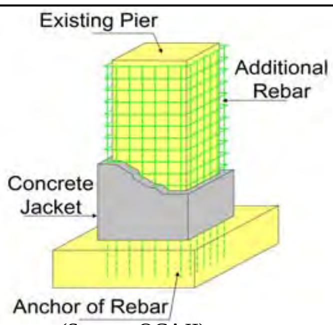
Table 17.2.4-2 Comparison of Improvement Schemes for Pier Columns

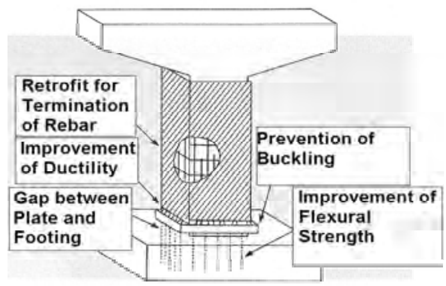
<table>
<tr><th></th><th colspan="2">Concrete jacketing</th><th colspan="2">Steel plate jacketing</th></tr>
<tr><td rowspan="2">Outline of improvement method</td><td colspan="2">Cross-section


Profile


Thickness: 586mm (at bottom)</td><td colspan="2">Cross-section


Profile


Thickness: 40mm</td></tr>
<tr><td colspan="2">

(Source: OCAJI)</td><td colspan="2">

(Source: OCAJI)</td></tr>
<tr><td>Structural characteristic</td><td>- Improvement of flexural strength and shear strength
- Large impact to clearance/river flow
- Effect of weight increase on foundation structures</td><td>B</td><td>- Improvement of flexural strength and shear strength
- Low impact to clearance/river flow
- Unable to develop required flexural strength</td><td>D</td></tr>
<tr><td rowspan="2">Constraints of construction</td><td>Typical durability of cast-in-place concrete structure: no need of maintenance</td><td>B</td><td>Need of constant maintenance to prevent corossions: every 30 years</td><td>C</td></tr>
<tr><td>- Need of cast-in-place concrete construction
-Need of smoothing of existing column surface
- Need of installation of shear connectors
- Need of larger construction space</td><td>B</td><td>- Need of installation of shear connectors
- Need of splicing of steel plates
-Need of smoothing of existing column surface
- Need of painting for corossion protection</td><td>B</td></tr>
<tr><td>Duration</td><td>Long</td><td>B</td><td>Long</td><td>B</td></tr>
<tr><td>Recommend-ation</td><td colspan="2">Recommended</td><td colspan="2"></td></tr>
</table>

Compared under “Pier-1 condition”.

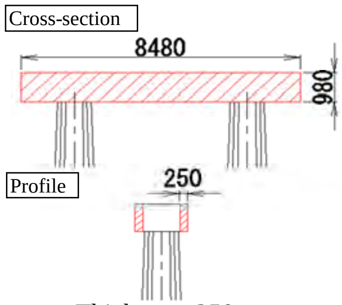
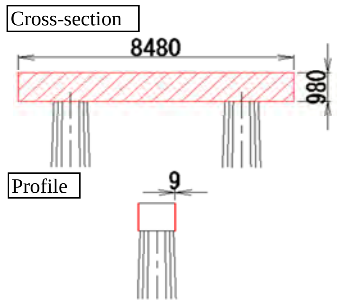
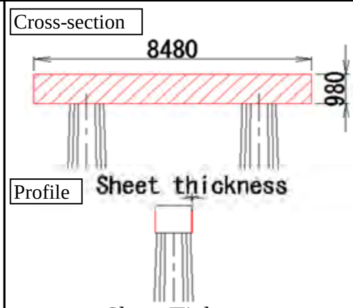
(4) Seismic Capacity Improvement Scheme for Pier Coping

The following three improvement schemes were compared for pier copings so as to improve flexural resistance and shear resistance.

- Alternative-1: Concrete jacketing
- Alternative-2: Steel plate jacketing
- Alternative-3: Carbon fiber sheet jacketing

As a result of evaluation, “concrete jacketing” was selected for its structural advantage and overall suitability for its implementation. The detail of the comparison is shown below.

Table 17.2.4-3 Comparison of Improvement Schemes for Pier Copings

	Concrete jacketing		Steel plate jacketing		Carbon fiber sheet jacketing	
Outline of improvement method	 Thickness: 250mm		 Thickness: 9mm		 Sheet Tickness	
Structural characteristic	- Improvement of both flexural strength and shear strength - Able to extend seat width; 250mm - Effect of weight increase on	A	- Improvement of shear strength - Unable to improve flexural strength without additional structure at the coping edges	C	- Improvement of shear strength - Limited Improvement of flexural strength	B
Durability / Maintenance	No need of maintenance	A	Need of constant maintenance to prevent corossions: every 30 years	C	Need of constant maintenance for surface protection: every 30 years	C
Constraints of construction	- Need of cast-in-place concrete construction -Need of smoothing of existing column surface - Need of installation of shear connectors - Need of larger construction space	B	- Need of installation of shear connectors - Need of splicing of steel plates -Need of smoothing of existing column surface - Need of painting for corossion protection	B	- No need of installation of shear connectors: to be attached - Temprature and humidity must be checked for the construction quality - Need of placement of polymer cement mortar as surface protection	A
Duration	Long	B	Long	B	Short	A
Recommend-ation	Recommended					

Compared with “Pier-2 condition”.

(5) Seismic Capacity Improvement Scheme for Foundations

1) Foundation Improvement Scheme for Piers

For improvement of existing pier foundation capacities, "additional piles for reinforcement" is recommended for piers. In case of Lilo-an Bridge, construction types for the foundation retrofit work are categorized into the following two.

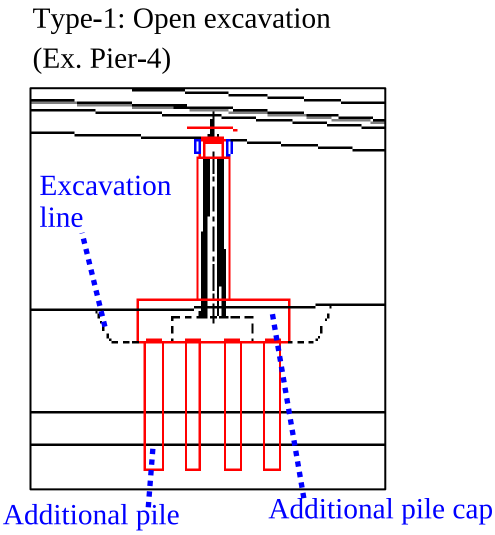


Figure 17.2.4-4 Construction Types for the Foundation Retrofit Work

Many pile-driving methods are available these days. However, in case of additional pile driving, applicability under lower vertical clearance must be considered in the selection of pile driving method.

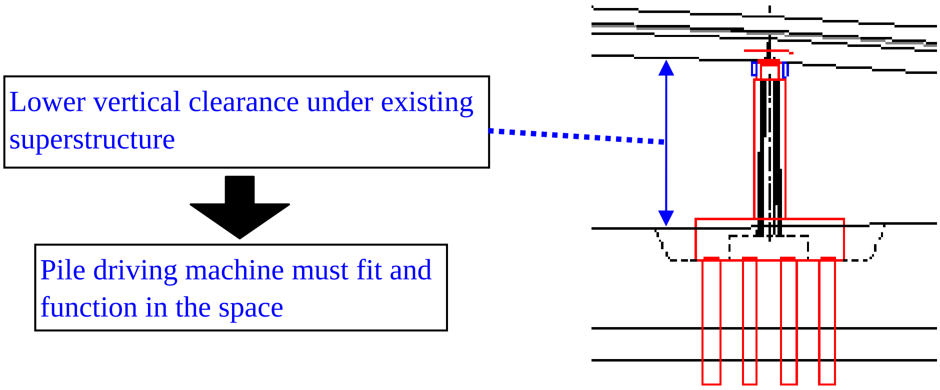


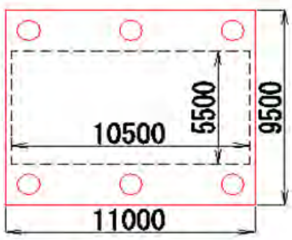
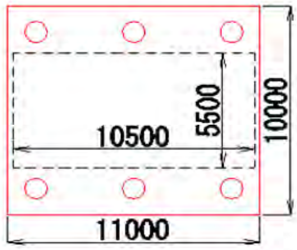
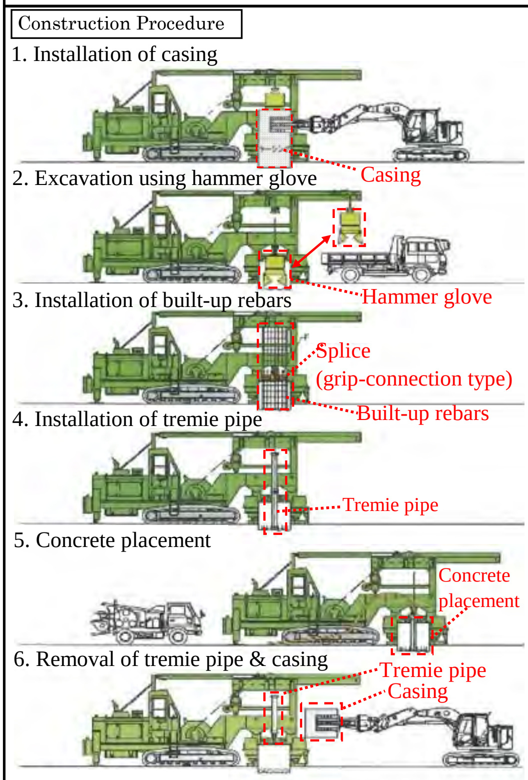
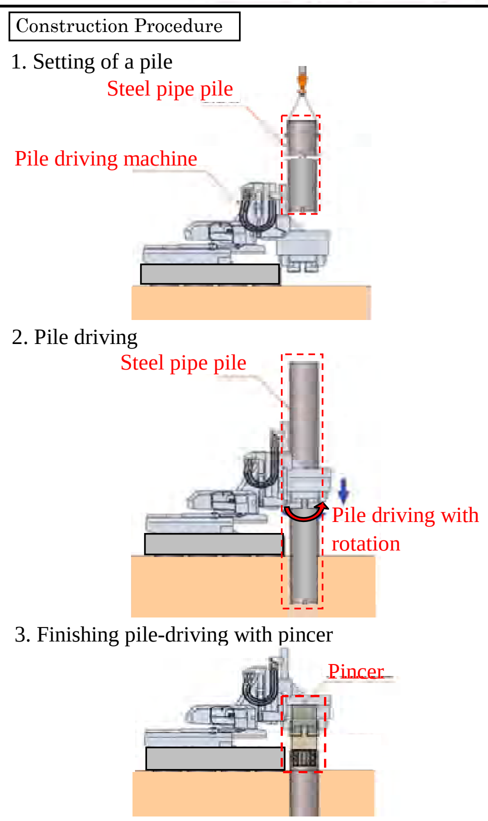
Figure 17.2.4-5 Restrictive Condition for Additional Pile Driving

In consideration with the above restrictive condition, the following two applicable pile driving methods were compared.

- Alternative-1: Cast-in-place concrete pile foundation (revolving all casing method)
- Alternative-2: Steel pipe pile foundation (jacked pile method)

As a result of the comparison, "cast-in-place pile foundation (revolving all casing method)" is recommended for its suitability for the construction site which has gravel layers with cobbles. The detail of comparison study is shown in the next page.

Table 17.2.4-4 Comparison of Improvement Schemes for Foundations

	Cast-in-place concrete pile (CCP) foundation (revolving all casing method)		Steel pipe pile (SPP) foundation (revolving type press-in method)	
Outline of improvement method	CCP φ1000 		SPP φ1000 	
	 (Source: NETIS plus)		 (Source: NETIS plus)	
Cost*	1.00	A	1.08	B
Structural characteristic	- Rigid concrete structure: valid against soft layer movement and liquefaction-induced lateral spreaading - Cast-in-place concrete pile: Structural reliability depends on construction quality - Maximum pile diameter: φ1500	A	- Non-rigid structure: vulnerable to lateral forces such as liquefaction-induced lateral spreaading - Fabricated product: reliable structure - Maximum pile diameter: φ1500	B
Constraints of construction	- Minimum reuired vertical clearance: 5m - Cast-in-place concrete pile: need of accurate field work - Need of many rebar splices - Able to penetrate solid substance such as rock	A	- Minimum reuired vertical clearance: 5-6m: it depends on the pile diameter size - Need of many welding splices - Able to penetrate solid substance such as rock	A
Duration	Long	C	Short: twice as short as CIP pile foundation	A
Recommend-ation	Recommended			

Cost*: The cost is compared with "Pier-2 condition".

2) Seismic Capacity Improvement Scheme for Abutments

Structural information of existing abutments is unknown as shown in the following figure. Therefore, study on improvement schemes for abutments was conducted assuming the underground structures. The unknown structure must be revealed by certain survey during detail design stage. As explained in "Change of Bearing Restraint Conditions", seismic dampers will be installed at the abutment as shown below; the abutment undertakes seismic inertial force of superstructure through the damper.

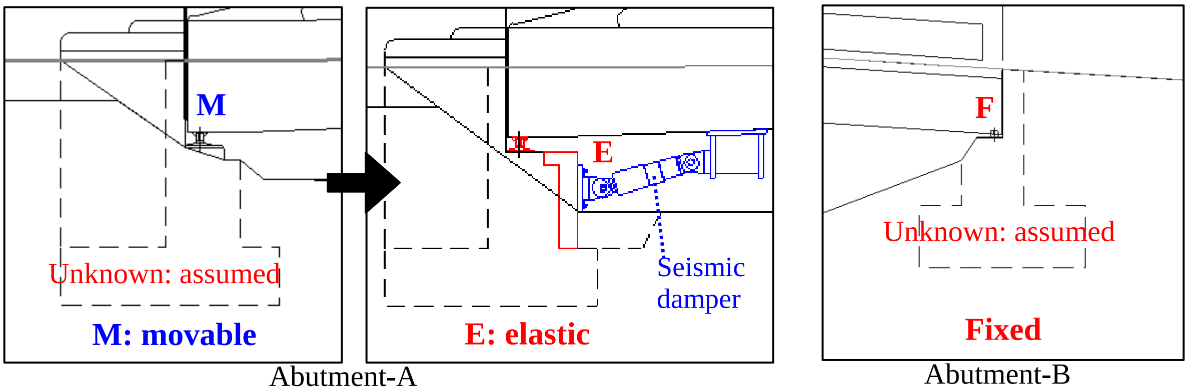


Figure 17.2.4-6 Assumed Abutment Conditions for Comparison Study

First of all, as the result of study on countermeasure for Abutment-A, expansion of spread foundation for improvement of foundation stability is recommended. Abutment-A is already structurally stable with spread footing on rock. However, larger seismic inertial force will act on the abutment through the installed seismic dampers. The foundation stability must be improved so as to resist against the larger force. In case of lack of stability with expansion of spread footing, ground anchor method can be additionally applied for the support. The image of the improvement work is illustrated below. Need of the ground anchor application will be confirmed in the detail design stage.

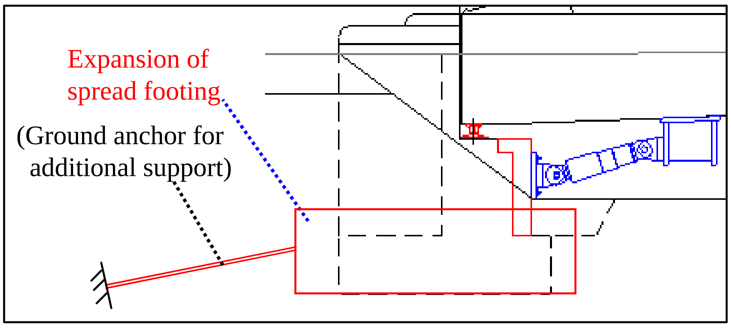


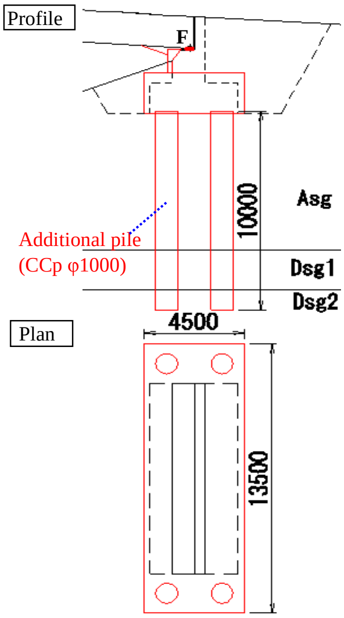
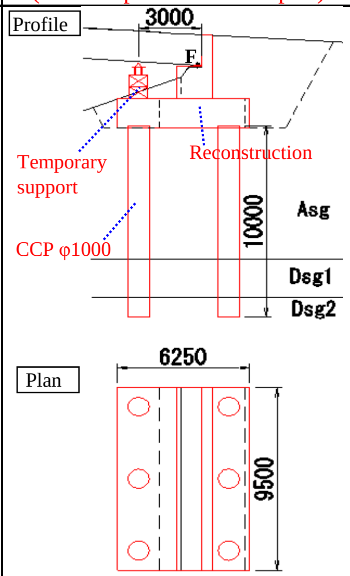
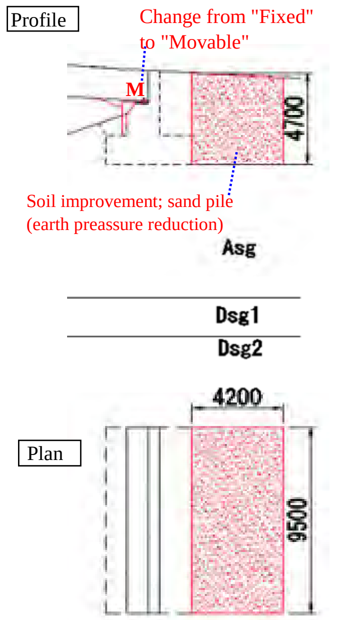
Figure 17.2.4-7 Improvement Work Image of Abutment-A

Secondary, Abutment-B must be improved with appropriate improvement method because the abutment is not supported by rock unlike Abutment-A, but alluvial gravel layer, which is not reliable as bearing layer of spread foundation. Therefore, the following three improvement schemes were compared for the stability improvement.

- Alternative-1: Soil improvement work with application of movable bearings
- Alternative-2: Additional piles for reinforcement
- Alternative-3: Total reconstruction

As a result of the comparison shown below, "additional piles for reinforcement" is recommended as improvement scheme of Abutment-B for the cost-effectiveness and overall suitability.

Table 17.2.4-5 Comparison of Improvement Schemes for Abutments

	Additional piles for reinforcement		Total reconstruction (Cast-in-place concrete pile)		Soil improvement with application of movable bearings	
Outline of improvement method						
Cost*	1.00	A	3.00	B	3.67	C
Structural characteristic	- Improved stability with additional piles - Application of cast-in-place pile foundation - Need of firm connection between existing pile cap and additional pile cap	B	- Total reconstruction of abutment - Application of cast-in-place pile foundation	A	- Soil improvement (sand pile) for earth preassure reduction - Change of bearing restraint condition from "Fix" to "Movable": reduction of seismic inertia force	B
Constraints of construction	- No need of large-scale excavation - No need of pile driving under superstructure	A	- Need of large-scale excavation &demolishing of exiting structure - Installation of temporary support within very close to the existing bearings to prevent cracks in PC I-girders - Need of pile driving under superstructure: small vertical clearance - Need of lane-closure during construction - Risk of superstructure damage during construction	C	- No need of excavation & demolishing work - Need of lane-closure during construction	A
Necessary information	- Need of existing pile cap information for detail design	B	- Need of existing pile cap information for detail design	B	- Need of existing pile cap information for detail design of sand pile	B
Duration	Short	A	Long	C	Medium	B
Recommend-ation	Recommended					

(6) Planning for Unseating Prevention System

1) Planning Procedure of Unseating Prevention System

Besides the strengthening of bridge structures, installation of unseating prevention system is very important in order to prevent superstructure-fall-down, which could happen in case retrofitted structures are devastated by unexpectedly large-scale earthquakes. In this study, the planning of unseating prevention system was done in accordance with the following procedure.

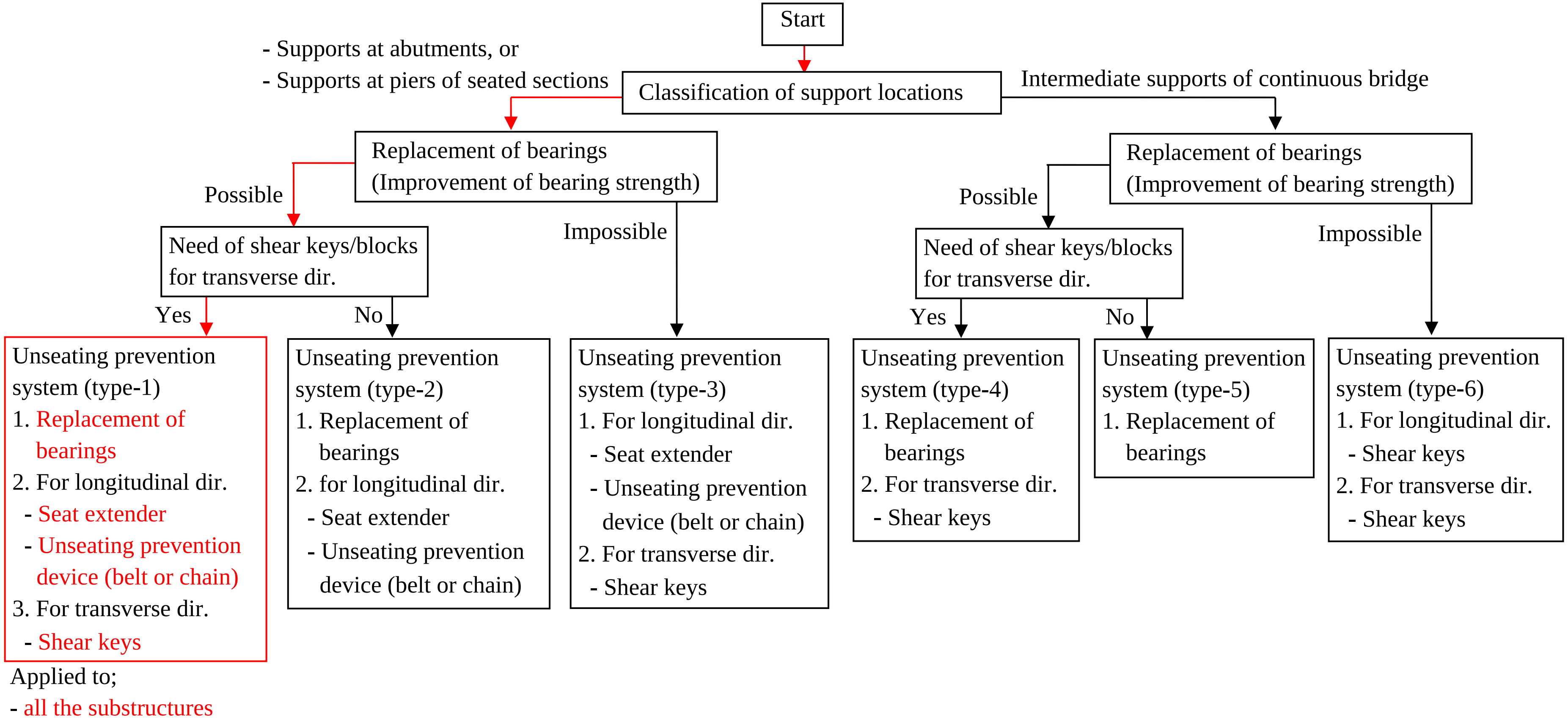


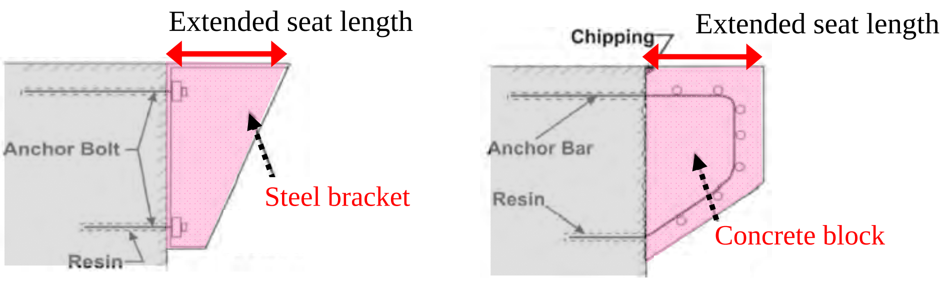
Figure 17.2.4-8 Basic Concept of Unseating Prevention System Planning

2) Planning for Replacement of Bearings

All the existing bearings will be replaced with new bearings which can resist level-2-scale earthquakes and fit in the space between existing structures.

3) Planning for Seat Extender

Seat length of abutments and piers at seated sections must be checked with the following formula.
Required minimum seat length: $S_{EM} = 0.7 + 0.005L$ (m)
Where,
L: seat-length-related span length (m)
Seat extenders will be installed at abutments and piers at seated sections, where seat length doesn't satisfy the above requirement. In consideration of construction efficiency and quality, "steel bracket" was selected for piers which needs seat length extension. Concrete blocks will be installed at abutments where no scaffolding is required for the installation of seat extenders.



Steel bracket (applied to piers)
- Easy and quick installation
- Fabricated product; good quality

Concrete block (applied to abutments)
- Cast-in-place concrete structure;
  - Inefficient installation at higher locations
  - Structural reliability depends on construction quality

Figure 17.2.4-9 Concrete Block and Steel Bracket

4) Planning for Unseating Prevention Devices

Unseating prevention devices will be installed at abutments and piers at seated sections. The device type was selected in accordance with the following rules.

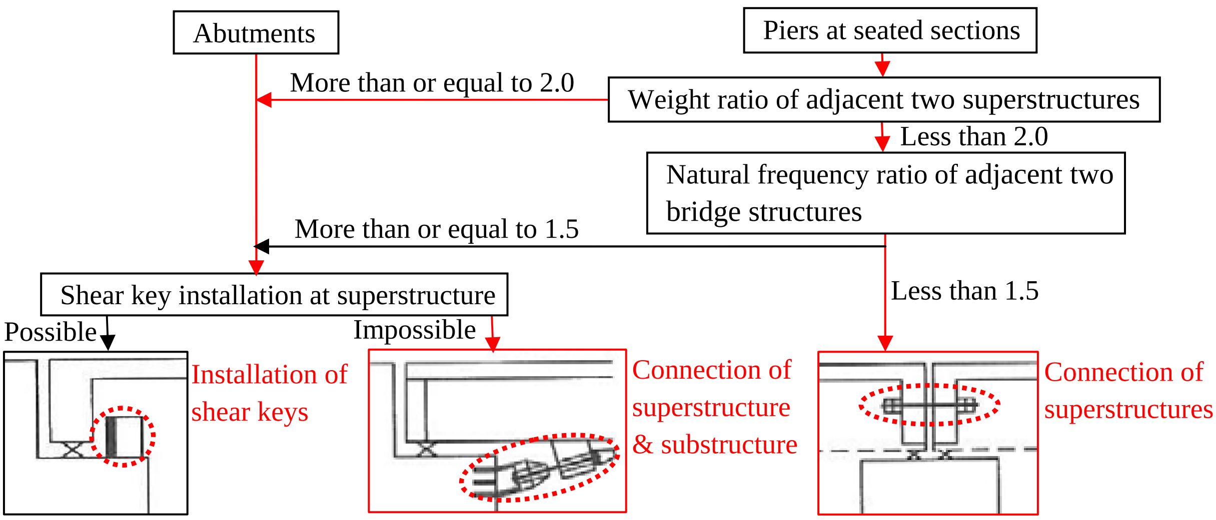
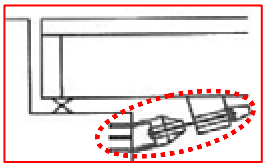


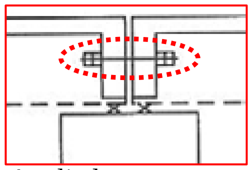
Figure 17.2.4-10 Selection of Unseating Prevention Device Type

Connection of superstructure and substructure



Applied to;
- Abutments
- Pier-1

Connection of superstructures



Applied to;
- Pier-2
- Pier-3
- Pier-4
- Pier-5
- Pier-6

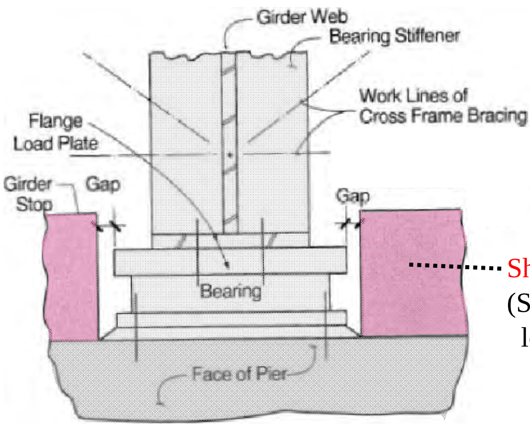
Note: no unseating prevention devices are needed at Abutment-A

Figure 17.2.4-11 Selection of Unseating Prevention Device Type (continued)

5) Planning for Structure Limiting Horizontal Displacement (Shear Keys)

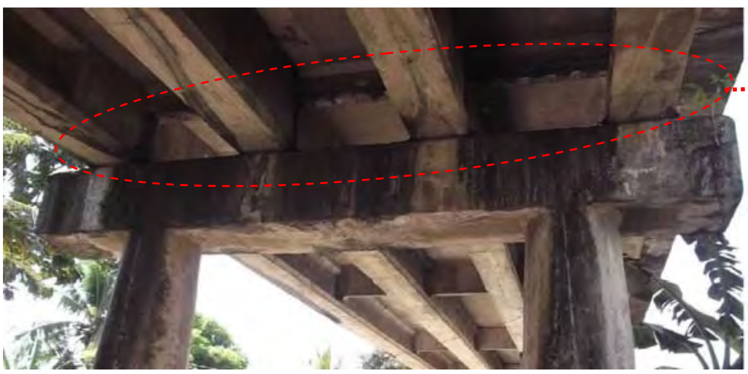
Structure limiting horizontal displacement (shear keys) for transverse direction will be installed at all the substructures for the following reasons.

- Existing bearings of steel arch bridge can't be replaced with new one due to its heavy weight. Therefore, the following devices should be installed as fail-safe system at Abutment-A and Pier-1(L).
  a) For longitudinal direction: unseating prevention chain
  b) For transverse direction: shear keys
- Although all the existing bearings of pier-2 through Pier-6 can be replaced with new one which can resist level-2 scale earthquakes, there's no cross beam at end supports. Therefore, shear keys should be installed at pier-2 through Pier-6 to improve horizontal rigidity of superstructures at end supports.



Shear keys/blocks
(Structure limiting horizontal displacement for both longitudinal and transverse direction)

Figure 17.2.4-12 Structure Limiting Horizontal Displacement (Shear Keys)



No cross beam; seismically vulnerable due to lack of rigidity

Figure 17.2.4-13 Non-existence of Cross Beam at End Supports

17.2.5 Planning for Repair Works

In addition to the seismic retrofit plans, the following three repair work
- Replacement/installation of expansion joints
- Repainting of steel members
- Repair of connection/splice points of steel members
- Epoxy injection of deck slab

[Replacement/installation of expansion joints]
As for Abutment-A and Pier-1, the expansion joints should be replaced in order to repair the opened or closed gap between joint members. In case of the rest of substructures, expansion joints should be installed to improve the bridge function.

Abutment-A Pier-1 Pier-2 through 6 Abutment-B

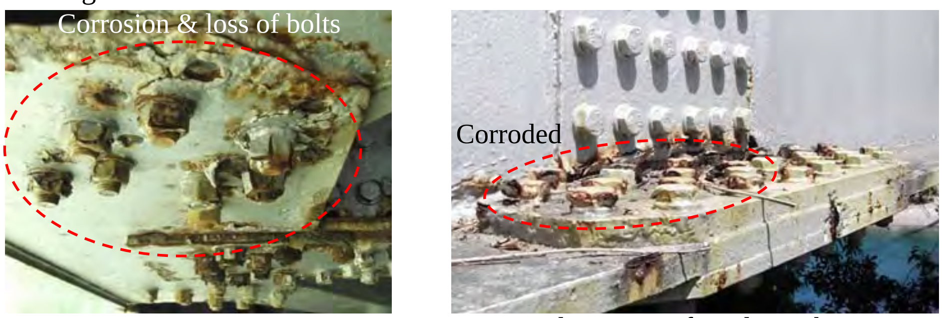
Figure 17.2.5-1 Current Condition of Existing Expansion Joints

[Repainting of steel members]
Repainting of existing steel members is recommended, especially for bottom flange of lower chord members which are heavily corroded. Besides the lower chord members, it's better to repaint other steel components for maintenance.

Lower chord member Lower chord member Arch rib

Figure 17.2.5-2 Current Condition of Existing Steel Members

[Repair of connection/splice points of steel members]
Repair of connection/splice points of steel members is recommended as a part of regular maintenance work. The condition of the connection is not critical. However, its better to repair them before their condition becomes worse because these connection points are one of the most important components of bridge structures. Deficiency of the connection points could cause fatal damage of whole entire bridge structure regardless of seismic issues.



Connection point of steel members Splice point of steel members

Figure 17.2.5-3 Current Condition of Connection/Splice Points of Existing Steel Members

[Epoxy injection of deck slab]
Epoxy injection of deck slab is recommended to repair cracking, Hanycomb, and water leaking of the existing deck slab. Additionally, repair by mortar covering is suggested as supplementary method.

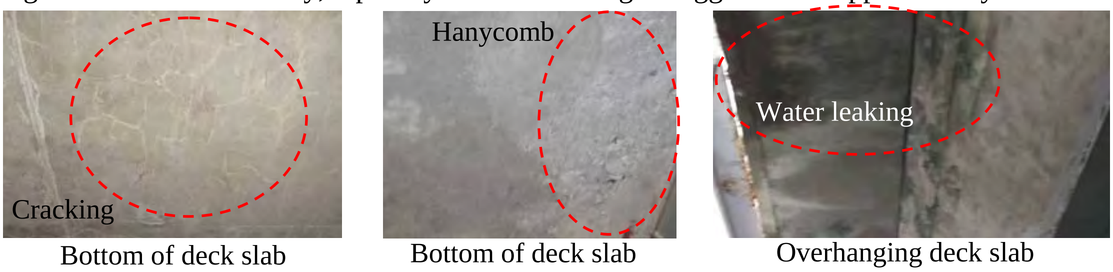
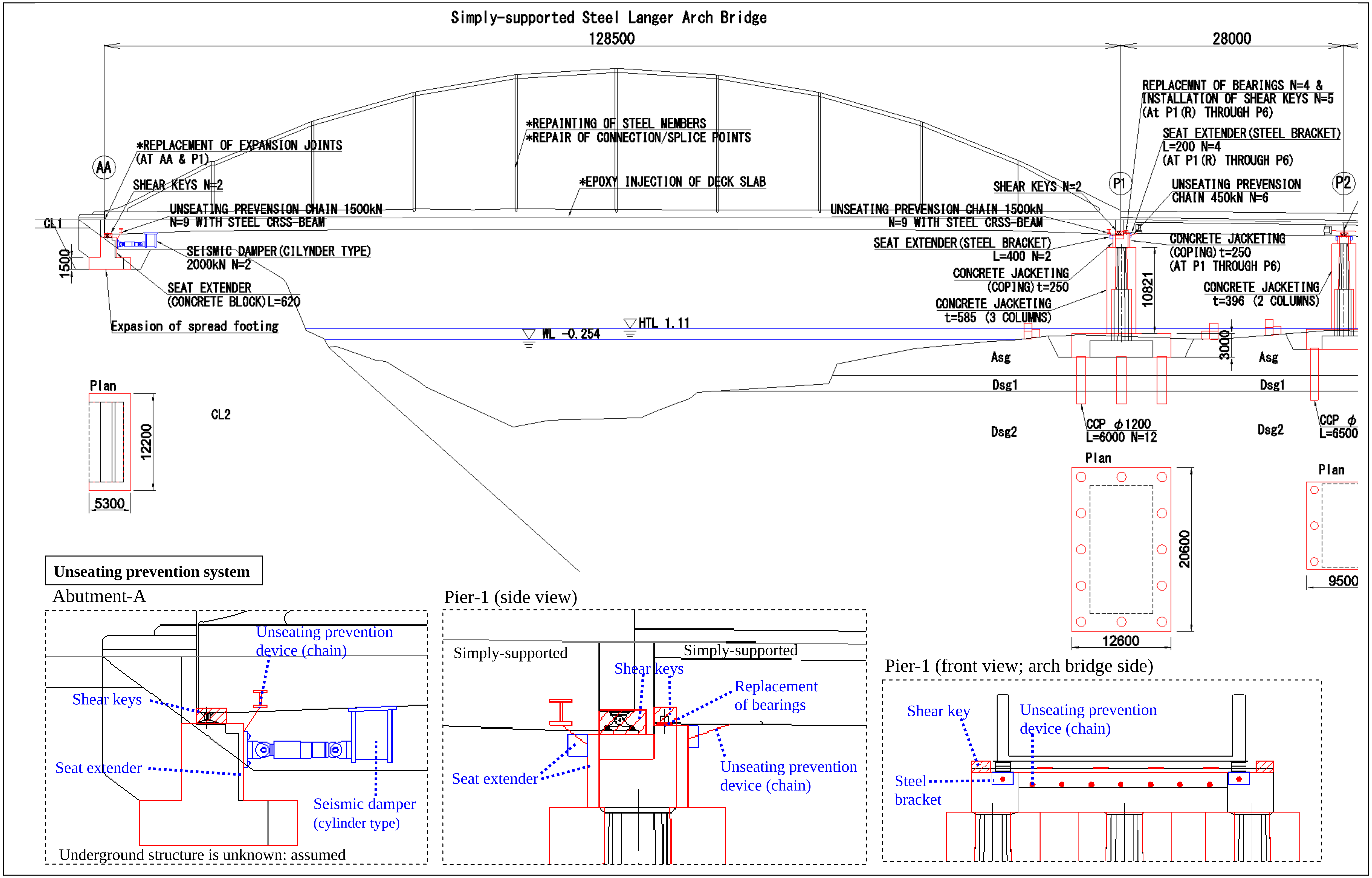
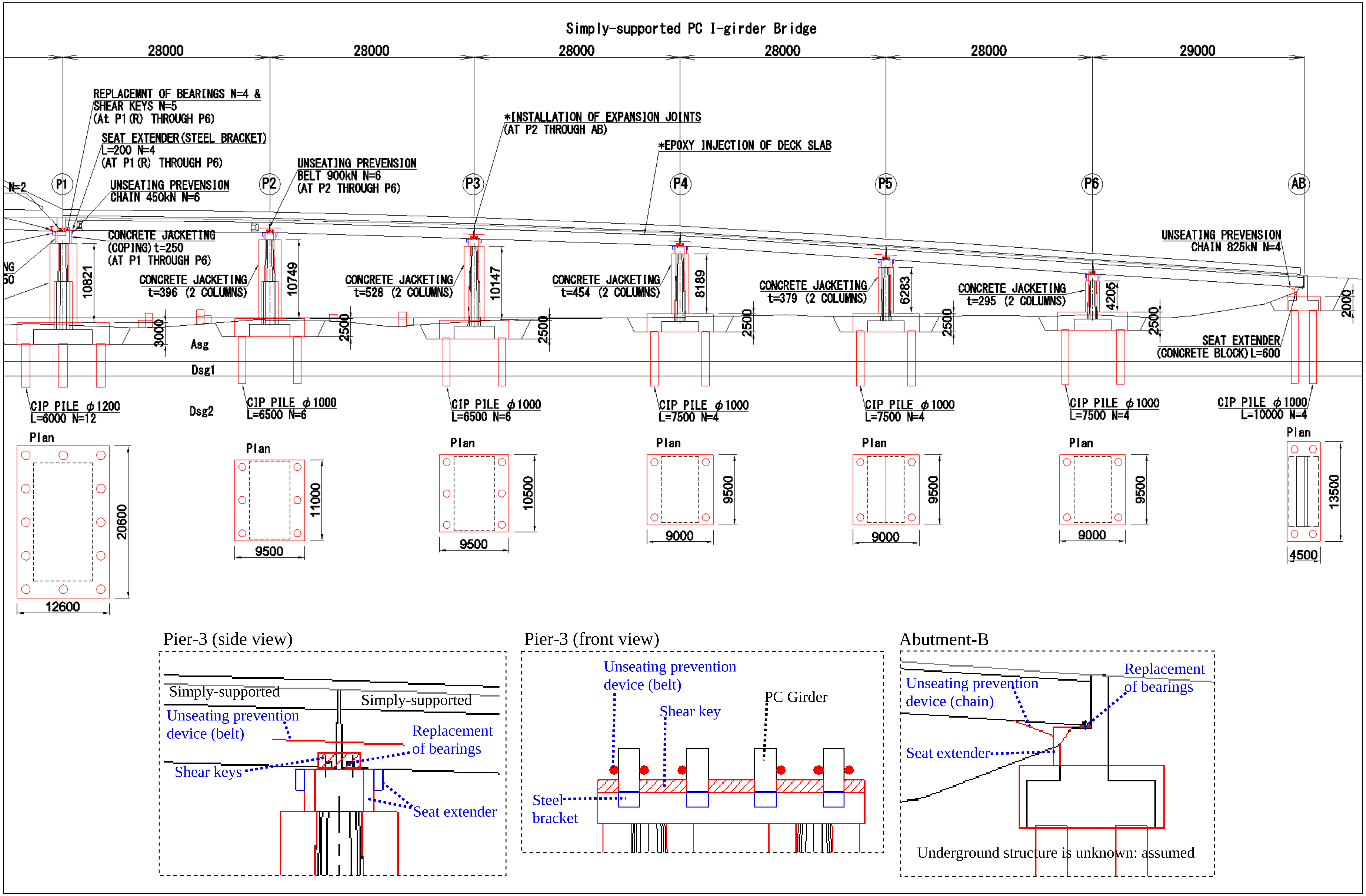


Figure 17.2.5-4 Current Condition of Existing Deck Slab

17.2.6 Summary of the Seismic Retrofit Planning & Repair Work



(General View & Structural Drawings are shown in Appendix-4)



(General View & Structural Drawings are shown in Appendix-4)

17.3 Outline Design of 1st Mandaue-Mactan Bridge

17.3.1 Structural Data of the Existing Bridge

(1) Outline of the Existing Bridge

1) Construction year: 1972

2) Total bridge length: 860m (topographic survey result)

(Span length)

- Simply-supported composite steel I-girder bridge: 37.2m
- 3-span continuous non-composite steel I-girder bridge: 50+50+50m
- 3-span continuous steel truss bridge: 112+144+112m

3) Applied specifications (superstructure)

(JRA: Japanese Road Association)

- The Specification for Steel Highway Bridge (except for live load methodology)
- The Specification for Welding Steel Highway Bridge
- Steel Highway Bridge of Composite Girder

(AASHTO: American Association of State Highway Officials)

- Standard Specification for Highway Bridges (only for live load methodology)
  Applied Live Load: H-20-S-16-44

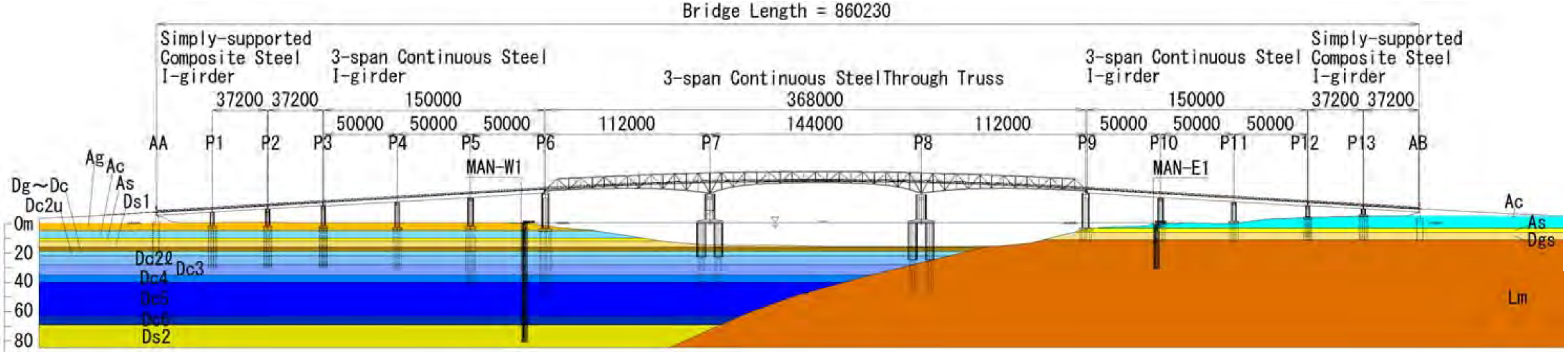
4) Steel Material (superstructure)

- SS41 (JIS G3101): corresponding to ASTM A7
- SM 50 (JIS G3106): corresponding to ASTM 242 or A441
- High strength bolt (JIS B1186): corresponding to ASTM A326

5) Reinforcing bars (superstructure): SD24 (JIS G3112): corresponding to ASTM A306

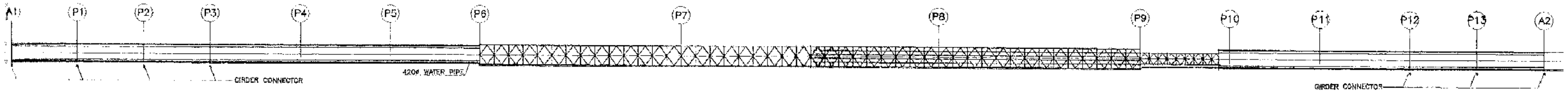
6) Concrete Compressive Strength (superstructure): $\sigma_{28} = 240kg/cm^2$ (Strength at 28 days)

(2) General View



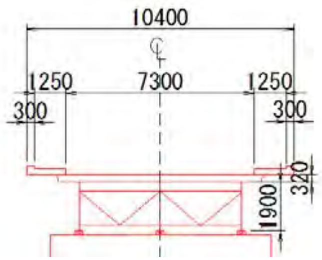
Underground structure is unknown: assumed.

Plan

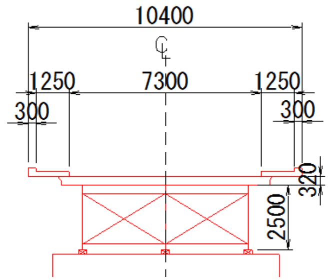


Cross-section

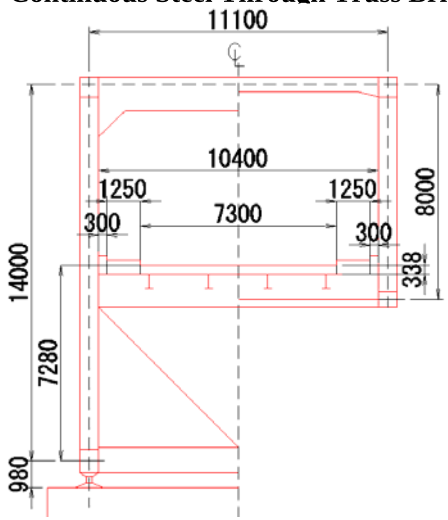
Simply-supported Composite Steel I-girder Bridge



Continuous Steel I-girder Bridge

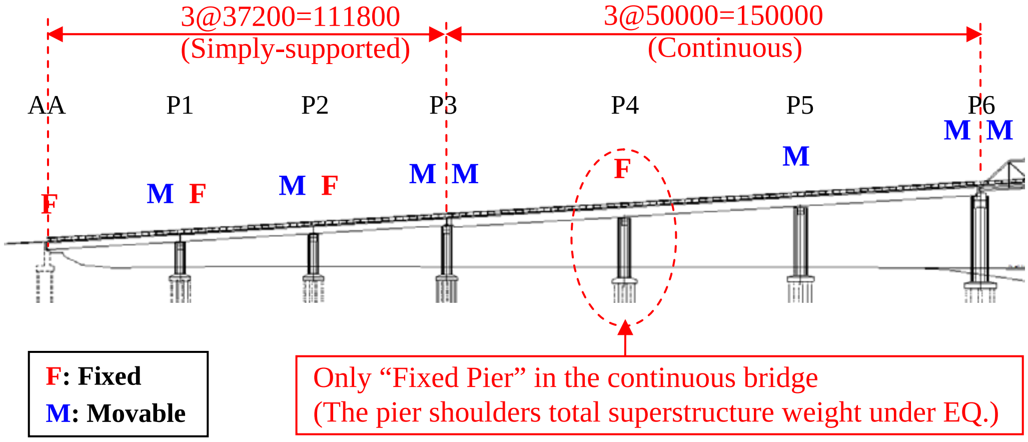
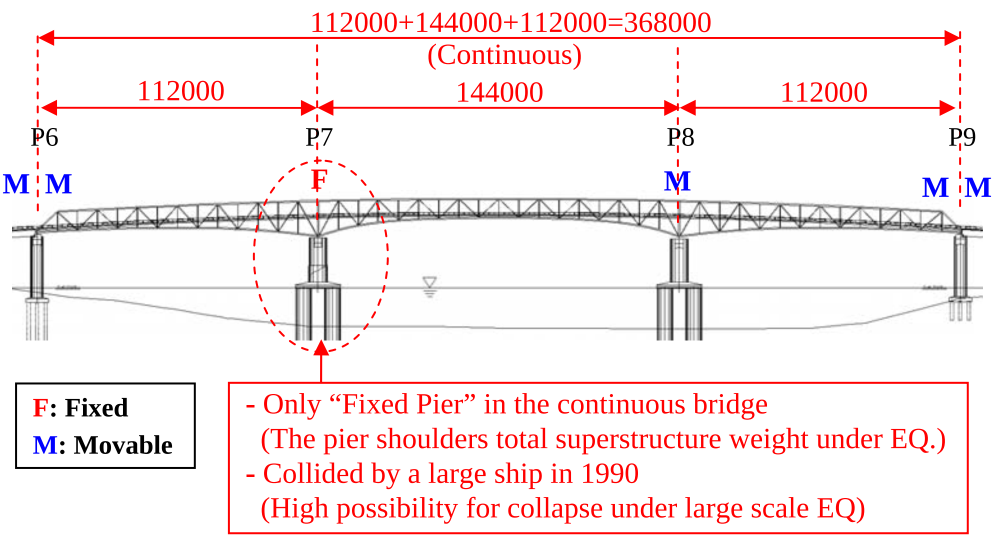
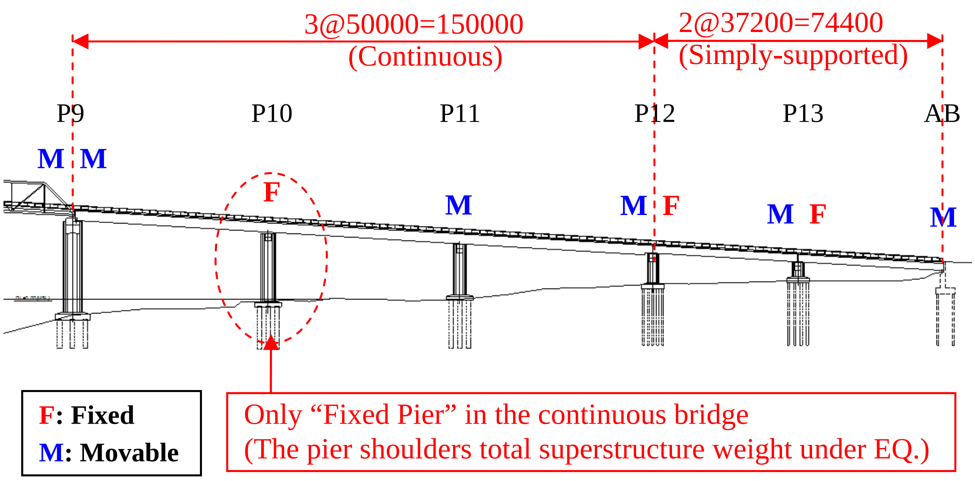


Continuous Steel Through Truss Bridge

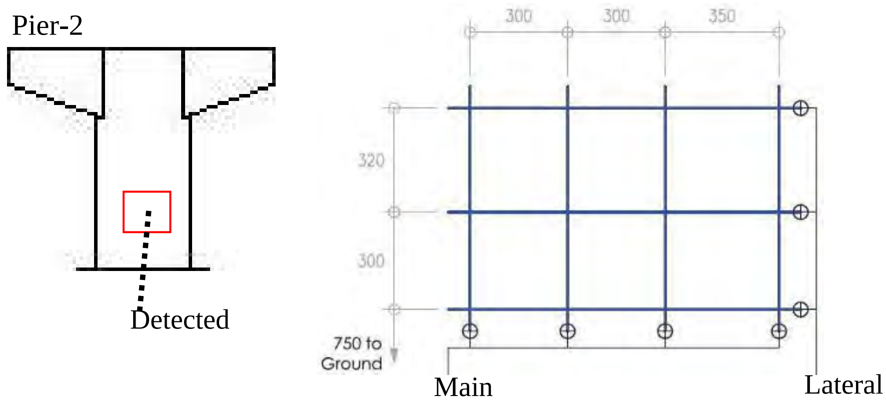


Source: Drawings prepared in previous projects
Note: - As-built drawing is not available.
- Modified based on the topographic survey result of this project

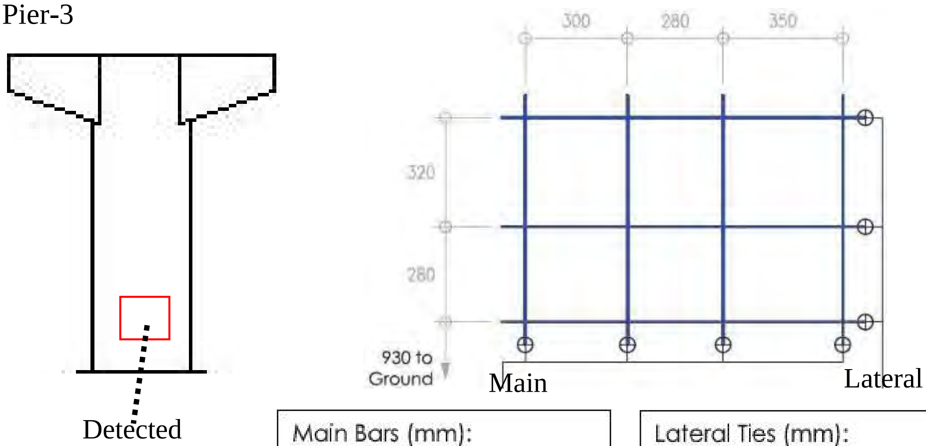
(3) Bridge/Span Length, Bridge Continuity and Bearing Restraint Conditions

Items	Contents
Span-1 to Span-6	
Span-7 to Span-9	
Span-10 to Span-14	

(4) Existing Pier Condition

<table>
<tr><th>Items</th><th>Contents</th></tr>
<tr><td>Concrete Strength</td><td>[Results of Concrete Strength Test[
1) Compressive Strength Test (P4) : 40.5 (MPa)
2) Schmidt Rebound Hammer Test
- P2: 38.6 (MPa)
- P3: 39.7 (MPa)
- P4: 40.5 (MPa)
Ave.: 39.6 (MPa)</td></tr>
<tr><td>Rebar Condition</td><td>[Results of Rebar Detection]
1) Pier-2


Main Bars (mm):
Ø = 2-28 mean (28 - 28)
C = 125 mean (85 -320)
No. of Readings = 8
Lateral Ties (mm):
Ø = 16 mean (16 - 16)
C = 74 mean (55 - 90)
No. of Readings = 7

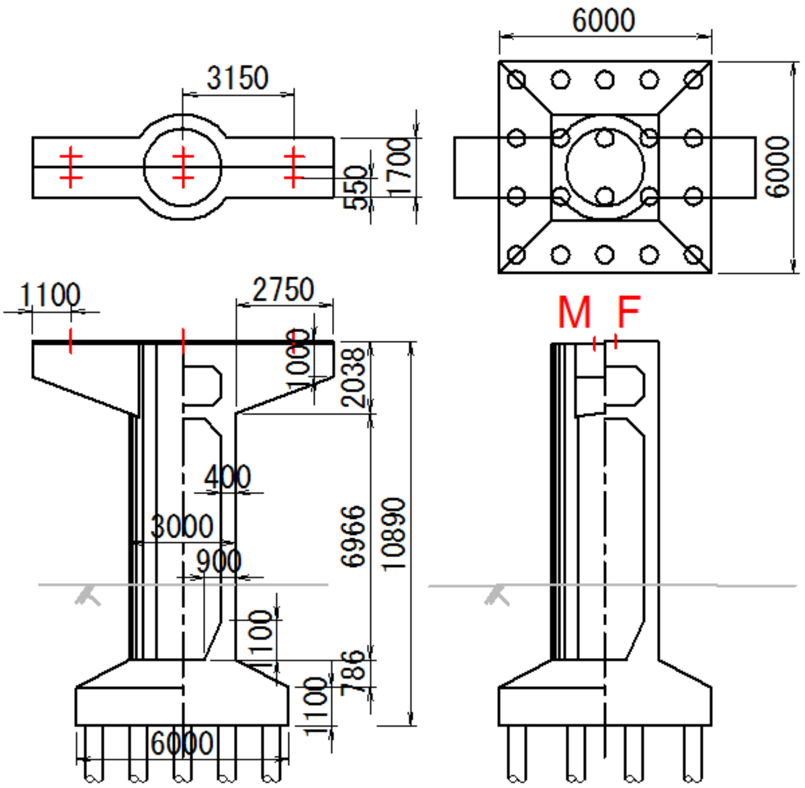
2) Pier-3


Main Bars (mm):
Ø = 2-28 mean (28 - 28)
C = 125 mean (85 -320)
No. of Readings = 8
Lateral Ties (mm):
Ø = 16 mean (16 - 16)
C = 74 mean (55 - 90)
No. of Readings = 7

Note: Existing rebars are ignored in the seismic retrofit design.: only for capacity check</td></tr>
</table>

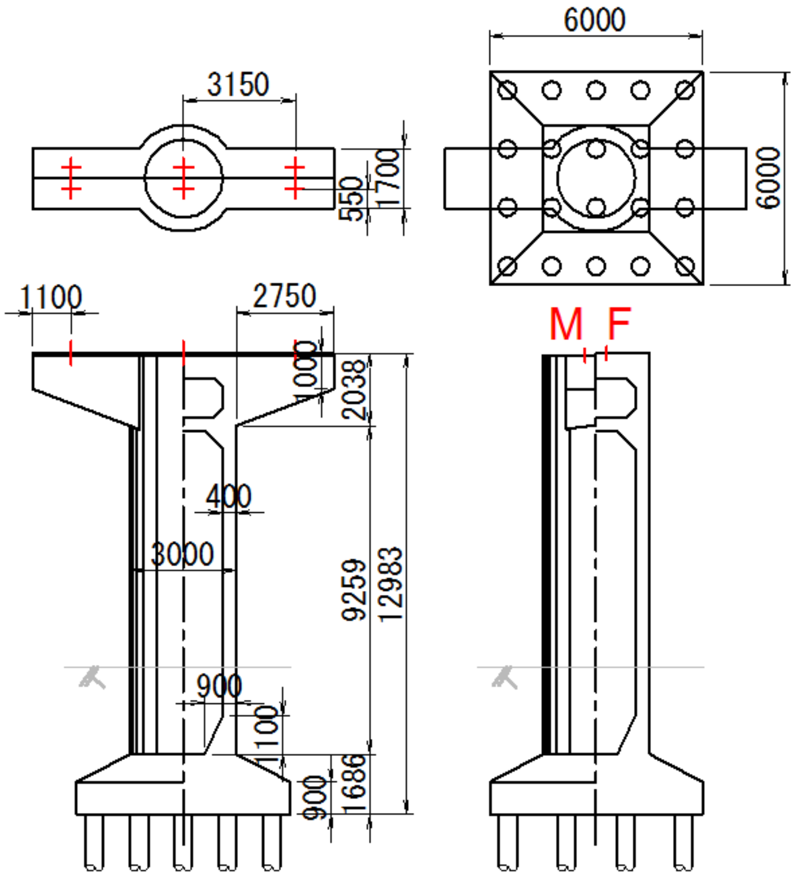
<table>
<tr><th>Items</th><th>Contents</th></tr>
<tr><td>Dimension</td><td>

P1



Pile:D500 (assumed)

P2



Pile:D500 (assumed)

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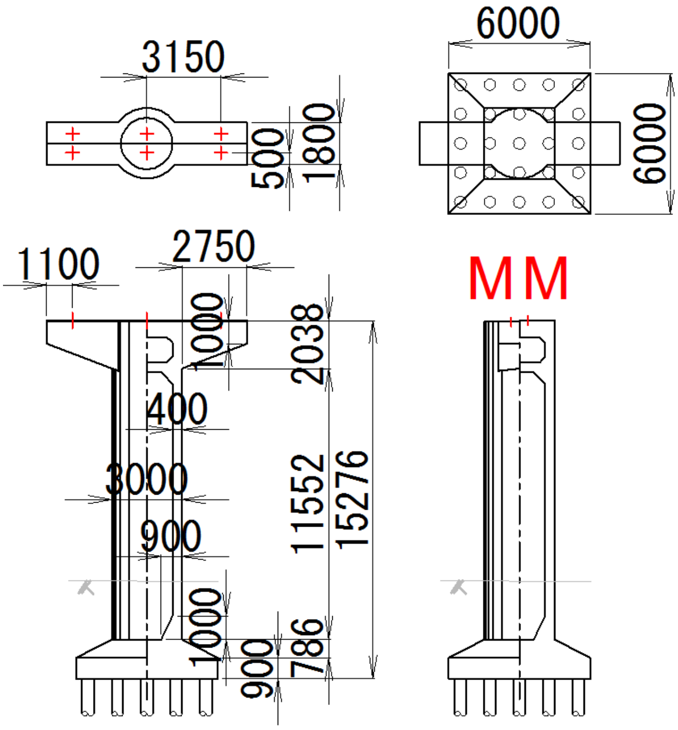
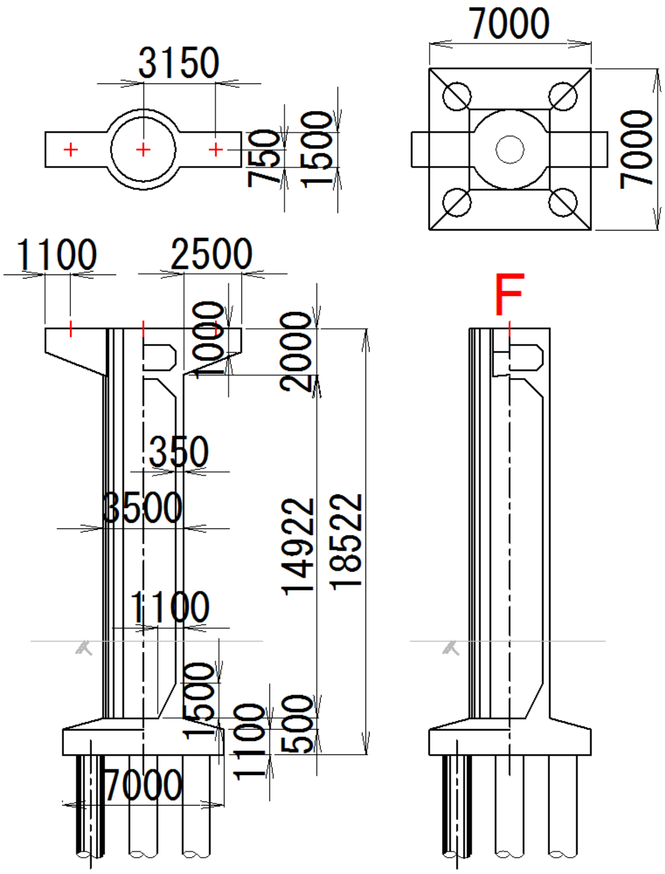


Pile:D500(assumed)

P4



Pile:D1200(assumed)

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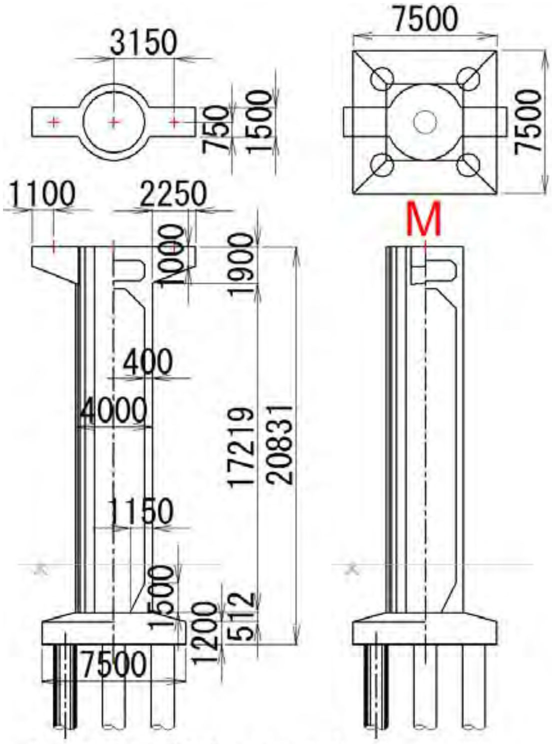
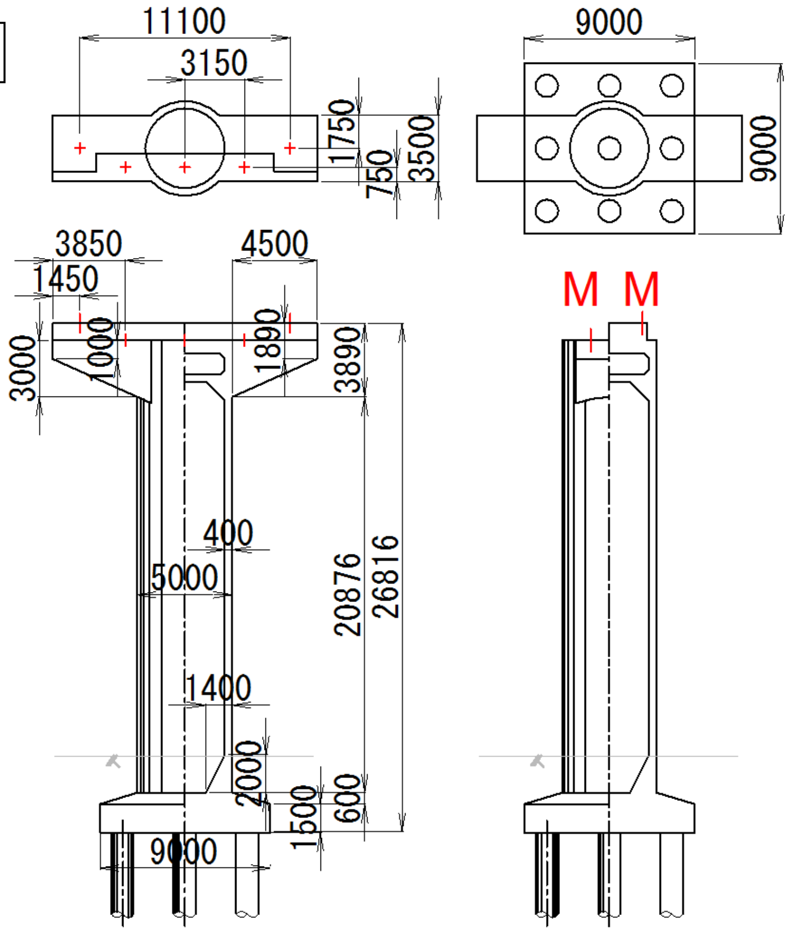
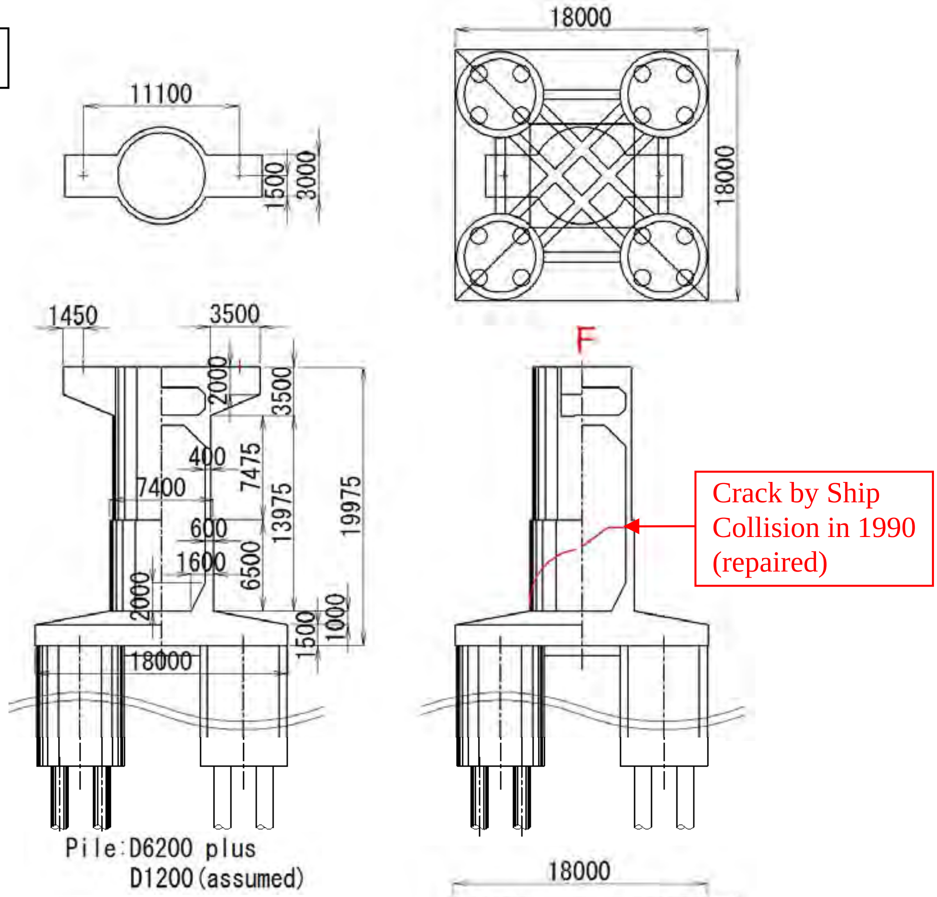
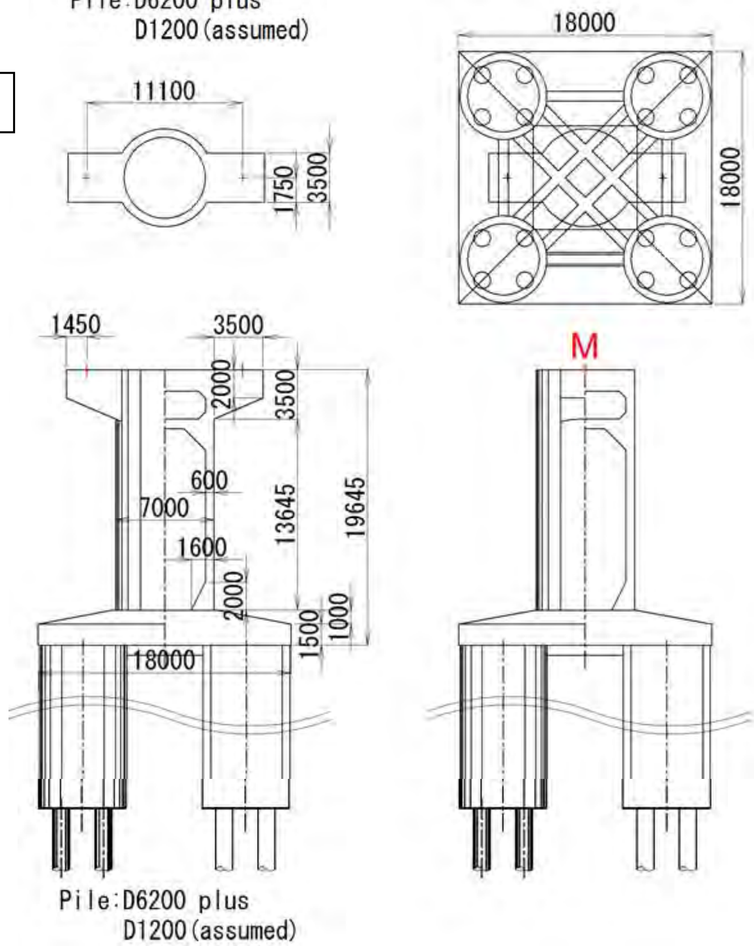
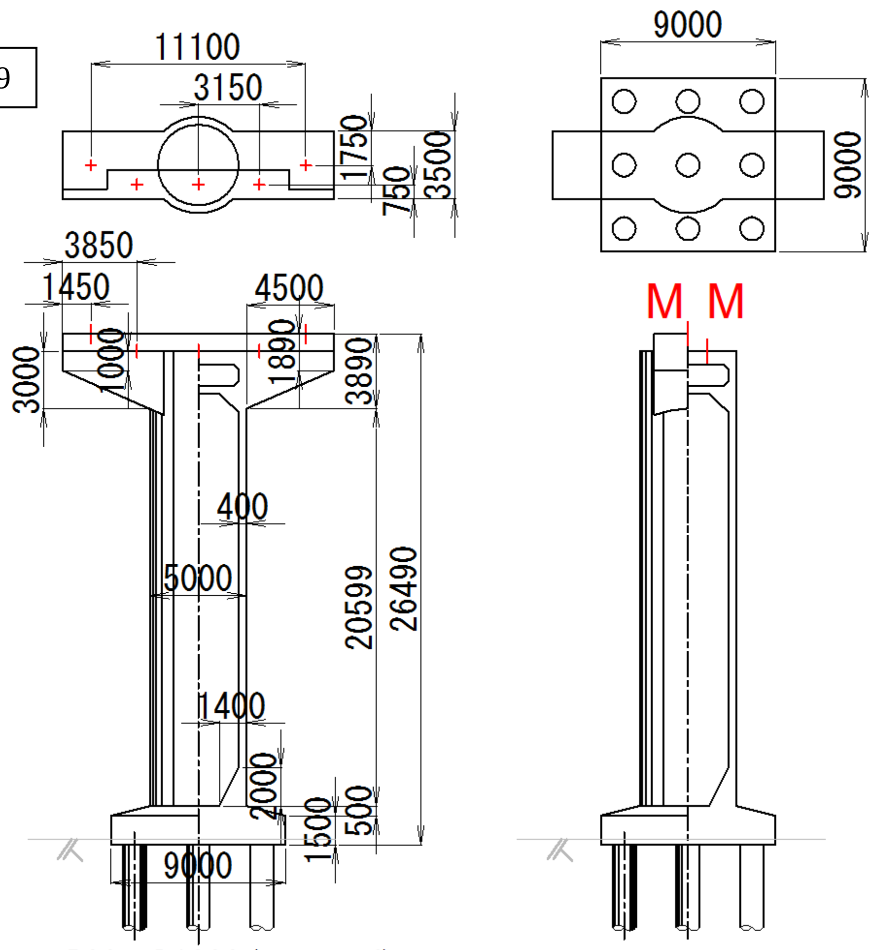
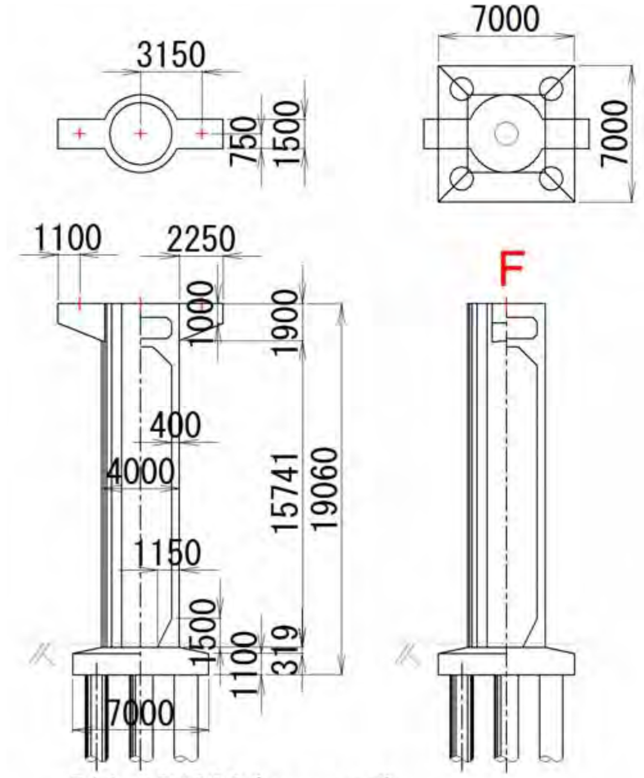
Dimension	P5  Pile:D1200(assumed) P6  Pile:D1200(assumed)

Items	Contents
Dimension	P7  P8 

Items	Contents
Dimension	P9  Pile:D1200(assumed) P10  Pile:D1200(assumed)

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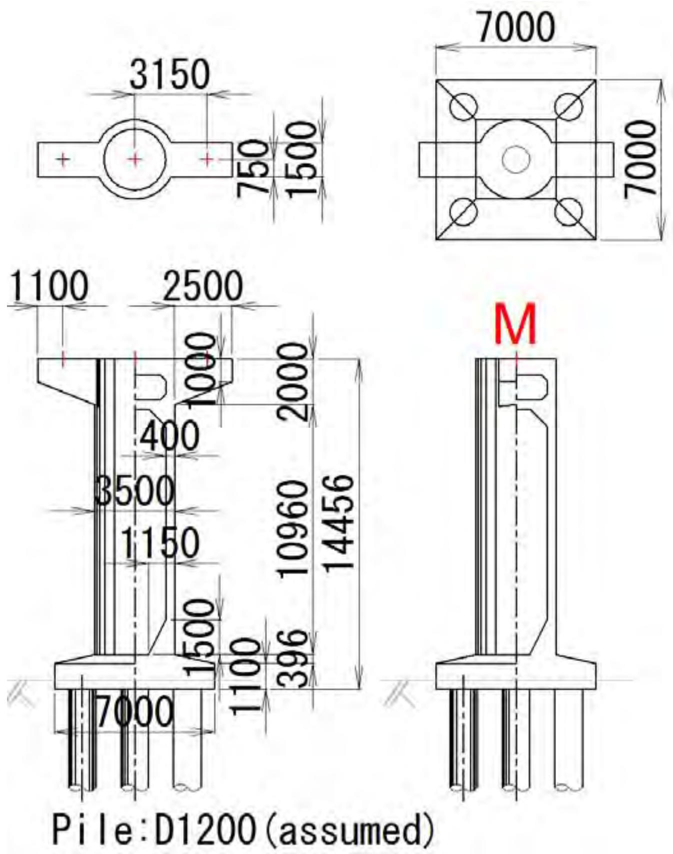
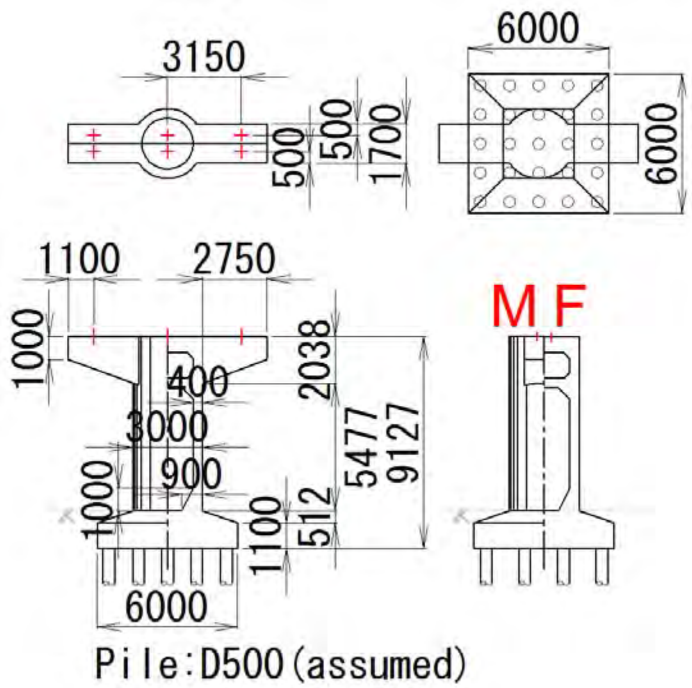
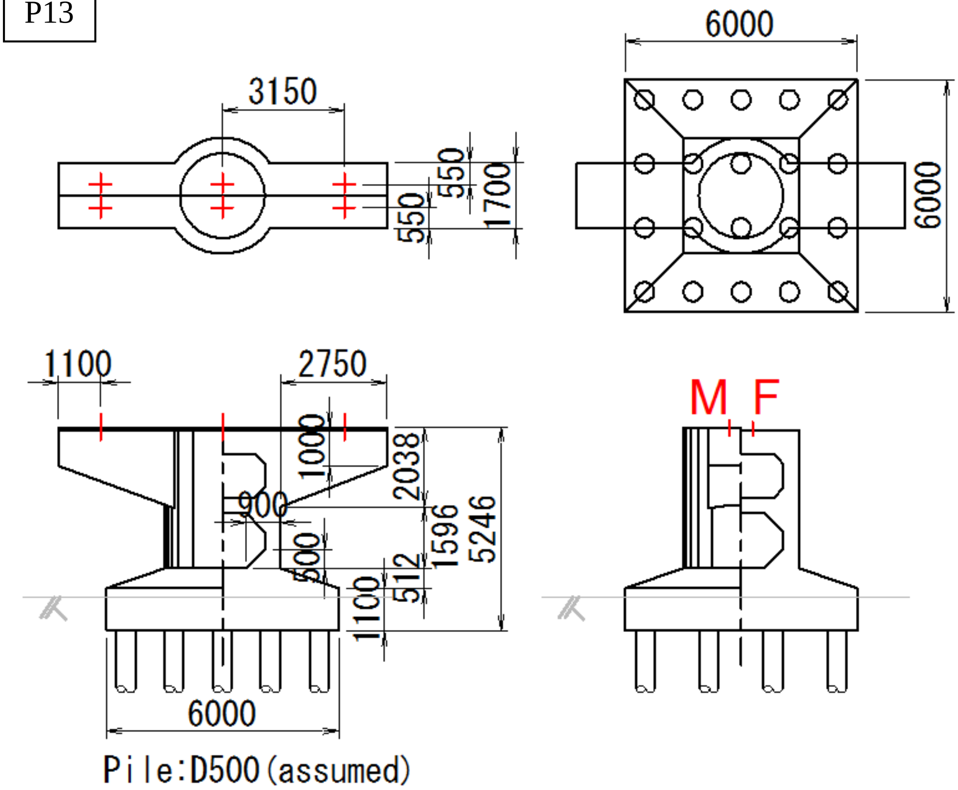
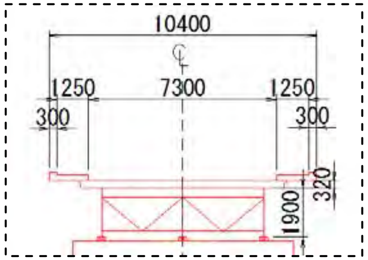
Dimension	P11  P12 

Items	Contents
Dimension	P13 

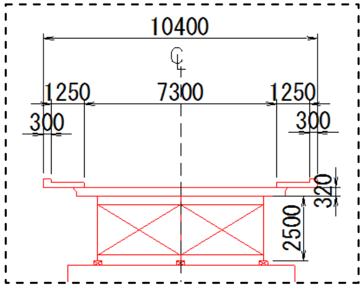
17.3.2 Design Conditions

(1) Design Loads

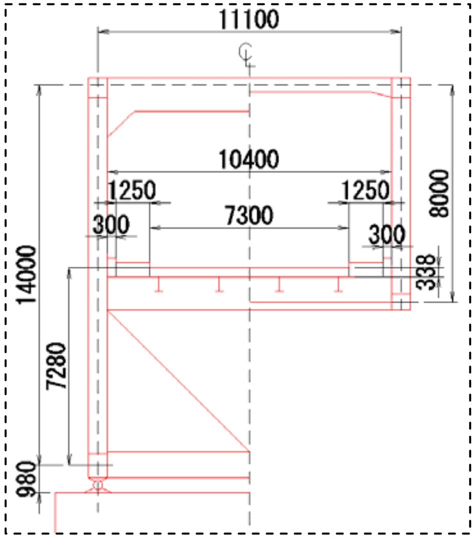
1 Simply-supported Composite Steel I-girder Bridge



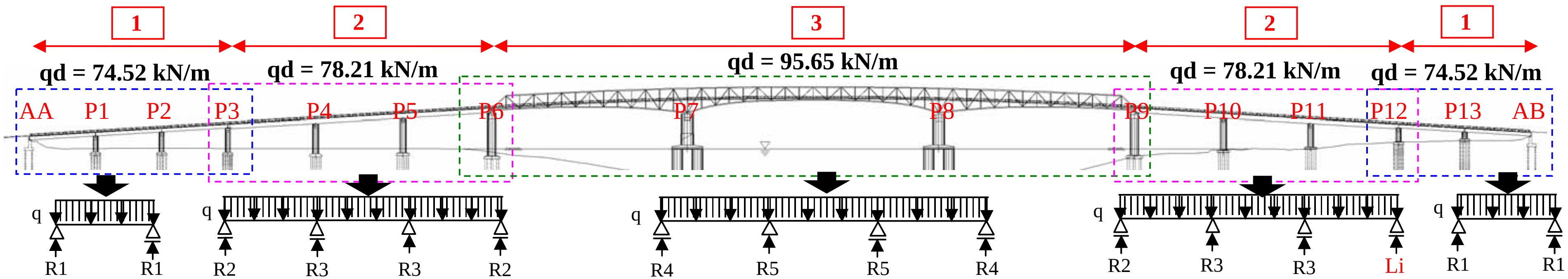
2 Continuous Steel I-girder Bridge



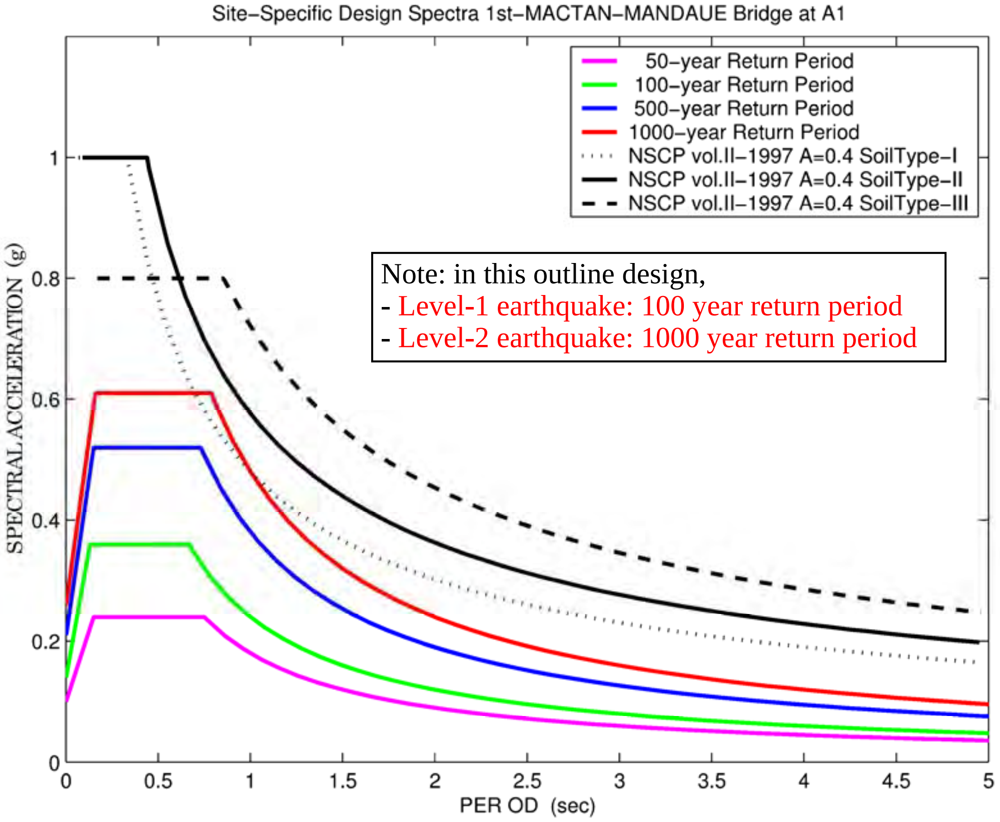
3 Continuous Steel Through Truss Bridge



Brige Type	Substructure			Sub-total (kN)		Total "Rd" (kN)		0.5Rl (kN)	Rd+0.5Rl (kN)
Simply-pupported steel I-girder bridge	Abut- A	-	F	1386	R1	1386	R1	750	2136
	Pier- 1	L	M	1386	R1	2772	R1 + R1	1000	3772
Simply-pupported steel I-girder bridge		R	F	1386	R1				
	Pier- 2	L	M	1386	R1	2772	R1 + R1	1000	3772
Simply-pupported steel I-girder bridge		R	F	1386	R1				
	Pier- 3	L	M	1386	R1	2931	R1 + R2	1100	4031
3-span continuous steel I-girder bridge		R	M	1545	R2				
	Pier- 4	-	F	4321	R3	4321	R3	1200	5521
	Pier- 5	-	M	4321	R3	4321	R3	1200	5521
	Pier- 6	L	M	1545	R2	4915	R2 + R4	1650	6565
3-span continuous steel truss bridge		R	M	3370	R4				
	Pier- 7	-	F	14230	R5	14230	R5	2300	16530
	Pier- 8	-	M	14230	R5	14230	R5	2300	16530
	Pier- 9	L	M	3370	R4	4915	R4 + R2	1650	6565
3-span continuous steel I-girder bridge		R	M	1545	R2				
	Pier- 10	-	F	4321	R3	4321	R3	1200	5521
	Pier- 11	-	M	4321	R3	4321	R3	1200	5521
	Pier- 12	L	M	1545	R2	2931	R2 + R1	1100	4031
Simply-pupported steel I-girder bridge		R	F	1386	R1				
	Pier- 13	L	M	1386	R1	2772	R1 + R1	1000	3772
Simply-pupported steel I-girder bridge		R	F	1386	R1				
	Abut- B	-	M	1386	R1	1386	R1	750	2136



The following figure summarizes "site-specific design spectrum of 50-, 100-, 500-, and 1000-year return period for 1st Mandaue-Mactan Bridge site" which were developed in this study.



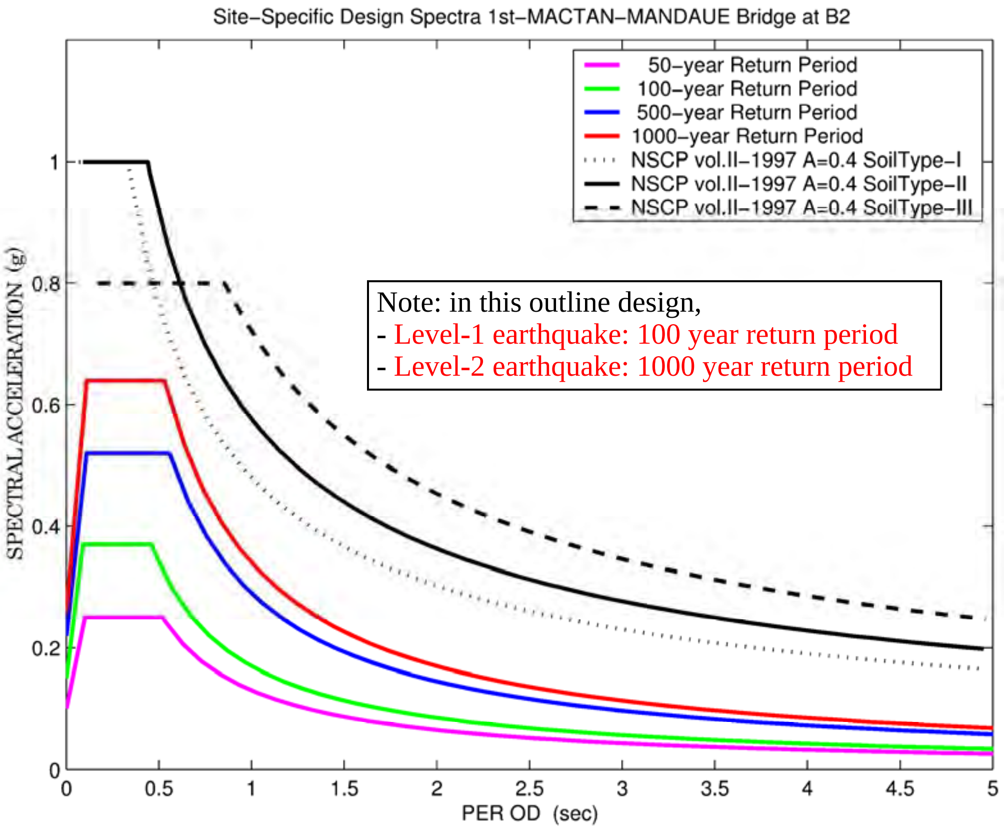


Figure 17.3.2-1 Site-Specific Design Spectrum of 50-, 100-, 500-, and 1000-Year return Periods for 1st Mandaue-Mactan Bridge Site

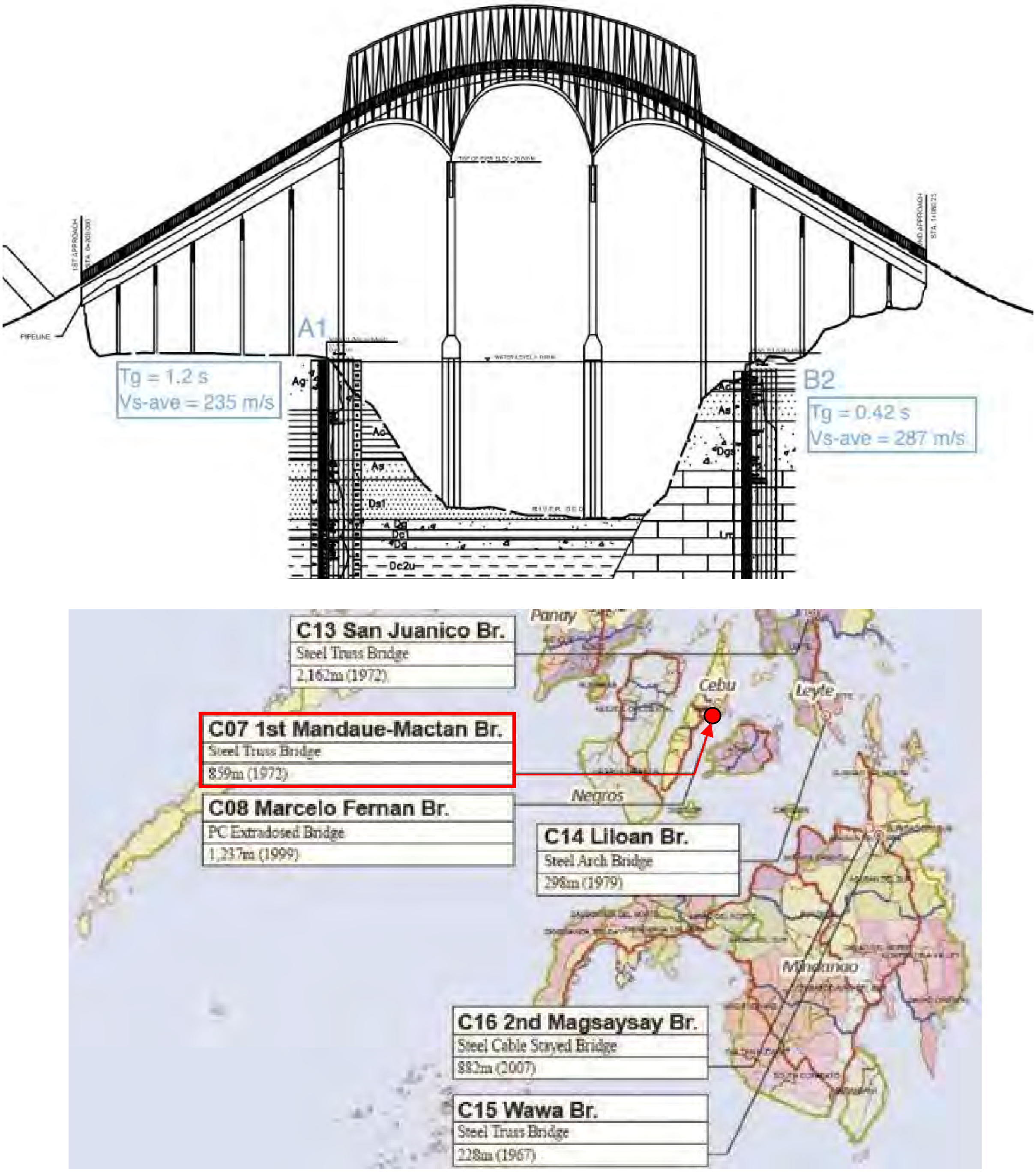


Figure 17.3.2-2 Site-Specific Design Spectrum of 50-, 100-, 500-, and 1000-Year Return Periods for 1st Mandaue-Mactan Bridge Site

The following table summarizes “the load distribution of existing bridge under earthquakes” and “application point of seismic inertial forces”.

Table 17.3.2-1 Load Distribution under EQ and Application Point of Seismic Inertial Forces

Substructure		Rd (kN)	Longitudinal Direction				Transeverse Direction			
			Restraint Condition	Wu (kN)		h (m)	Restraint Condition	Wu (kN)		h (m)
Abut- A	-	1386	F	2772	2772	-	F	-	-	-
Pier- 1	L	1386	M	0	2772	0	F	1386	2772	1.9
	R	1386	F	2772		0	F	1386		
Pier- 2	L	1386	M	0	2772	0	F	1386	2772	1.9
	R	1386	F	2772		0	F	1386		
Pier- 3	L	1386	M	0	0	0	F	1386	2931	2.5
	R	1545	M	0		0	F	1545		
Pier- 4	-	4321	F	11732	11732	0	F	4321	4321	2.5
Pier- 5	-	4321	M	0	0	0	F	4321	4321	2.5
Pier- 6	L	1545	M	0	0	0	F	1545	4915	1.7
	R	3370	M	0		0	F	3370		
Pier- 7	-	14230	F	35200	35200	0	F	14230	14230	7.9
Pier- 8	-	14230	M	0	0	0	F	14230	14230	7.9
Pier- 9	L	3370	M	0	0	0	F	3370	4915	1.7
	R	1545	M	0		0	F	1545		
Pier- 10	-	4321	F	11732	11732	0	F	4321	4321	2.5
Pier- 11	-	4321	M	0	0	0	F	4321	4321	2.5
Pier- 12	L	1545	M	0	2772	0	F	1545	2931	2.5
	R	1386	F	2772		0	F	1386		
Pier- 13	L	1386	M	0	2772	0	F	1386	2772	1.9
	R	1386	F	2772		0	F	1386		
Abut- B	-	1386	M	-	-	-	F	-	-	-

L: Left side bearing to the longitudinal direction
R: Right side bearing to longitudinal direction
h (m): Height from the top of the substructure body to the height of the superstructural inertia force
Wu (kN): Weight of the superstructure portion supported by the substructure body concerned

(2) Soil Conditions

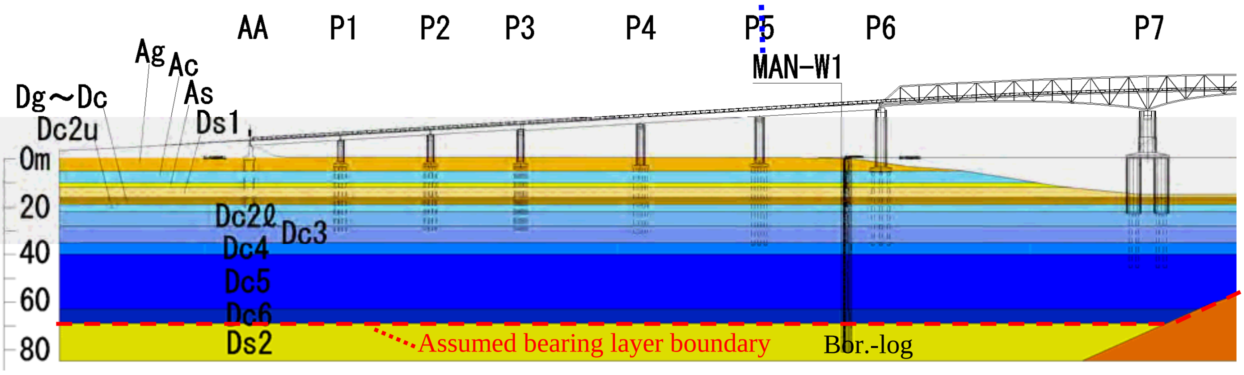
Soil condition of 1st Mandaue-Mactan Bridge is summarized as follows. The results of the liquefaction potential analysis are shown from the next page.

Design Soil Parameters for "1st Mandaue-Mactan Bridge" (MAN-E1 Site) → Soil Type: II

Layer Name	Soil Type	N	γt	C	Φ	α E_0		Vs	DE
						α = 4	α = 8		
		-	(kN/m^3)	(kN/m^2)	(°)	(kN/m^2)	(kN/m^2)	-	-
Ag	Gravel	23	18	0	37	64,400	128,800	228	-
Ac	Clay	7	15	44	0	19,600	39,200	191	-
As	Sand	7	17	0	29	19,600	39,200	153	-
Ds1	Sand	27	17	0	35	75,600	151,200	240	-
Dg1	Gravel	32	18	0	36	89,600	179,200	254	-
Dc1	Clay	25	18	156	0	70,000	140,000	292	-
Dg2	Gravel	35	18	0	36	98,000	196,000	262	-
Dc2u	Clay	23	18	144	0	64,400	128,800	284	-
Dc2ℓ	Clay	23	18	144	0	64,400	128,800	284	-
Dc3	Clay	13	18	81	0	36,400	72,800	235	-
Dc4	Clay	48	18	300	0	134,400	268,800	292	-
Dc5	Clay	16	18	100	0	44,800	179,200	252	-
Dc6	Clay	33	18	206	0	92,400	369,600	292	-
Ds2	Sand	50	19	0	41	140,000	280,000	295	-

α = 4 (Service state), 8 (Under earthquake)

Based on results of SPT & laboratory tests



Design Soil Parameters for "1st Mandaue-Mactan Bridge" (MAN-W1 Site) → Soil Type: I

Layer Name	Soil Type	N	γt	C	Φ	α E_0		Vs	DE
						α = 4	α = 8		
		-	(kN/m^3)	(kN/m^2)	(°)	(kN/m^2)	(kN/m^2)	(m/sec)	-
Ac	Clay	23	18	144	0	64,400	128,800	284	-
As	Sand	26	17	0	38	72,800	145,600	237	-
Dgs	Gravel	50	20	0	40	140,000	280,000	295	-
Lm	Rock	50	21	–	–	–	–	295	-

α = 4 (Service state), 8 (Under earthquake)

Based on results of SPT & laboratory tests

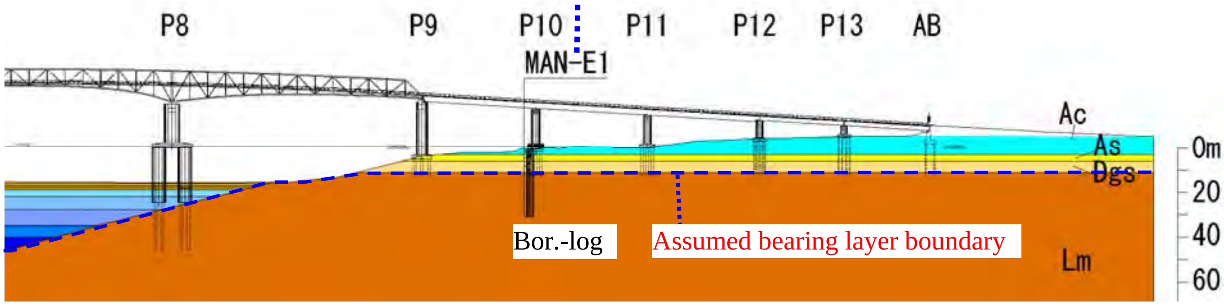
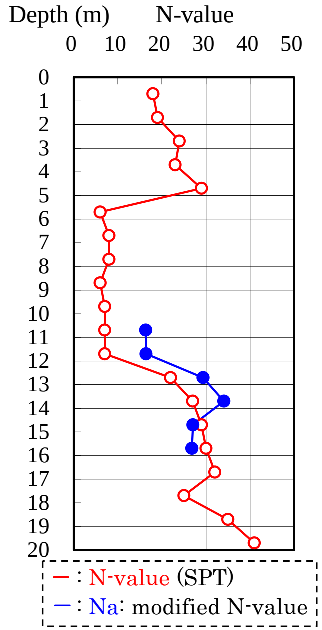


Table 17.3.2-2 Result of Liquefaction Potential Assessment (MAN-E1 side)

Water Lv.	0.00	(m)	As=Fpga*PGA	0.26	(g)

Depth X (m)	Layer symbol	N-value	Soil detail	γt2 (kN/m3)	γt1=γt2-1 (kN/m3)	D50 (mm)	FC (%)	σv (kN/m2)	σv' (kN/m2)	N1	C1	C2	Na
0.00	Ag				-	-	-	0.0	0.0		-	-	-
0.70		18	5 Non-liequefiable	19.0	-	-	-	13.3	6.3	40.1	-	-	-
1.70		19	5 Non-liequefiable	19.0	-	-	-	32.3	15.3	37.9	-	-	-
2.70		24	5 Non-liequefiable	19.0	-	-	-	51.3	24.3	43.3	-	-	-
3.70		23	5 Non-liequefiable	19.0	-	-	-	70.3	33.3	37.9	-	-	-
4.70		29	5 Non-liequefiable	19.0	-	-	-	89.3	42.3	43.9	-	-	-
5.70	Ac	6	5 Non-liequefiable	16.0	-	-	-	105.3	48.3	8.6	-	-	-
6.70		8	5 Non-liequefiable	16.0	-	-	-	121.3	54.3	10.9	-	-	-
7.70		8	5 Non-liequefiable	16.0	-	-	-	137.3	60.3	10.4	-	-	-
8.70		6	5 Non-liequefiable	16.0	-	-	-	153.3	66.3	7.5	-	-	-
9.70		7	5 Non-liequefiable	16.0	-	-	-	169.3	72.3	8.4	-	-	-
10.70	As	7	1 Alluvial (sand)	18.0	17.0	0.09	49.3	187.3	80.3	7.9	1.786	2.183	16.324
11.70		7	1 Alluvial (sand)	18.0	17.0	0.09	53.3	205.3	88.3	7.5	1.865	2.404	16.427
12.70		22	1 Alluvial (sand)	18.0	17.0	0.26	23.6	223.3	96.3	22.5	1.272	0.756	29.362
13.70		27	1 Alluvial (sand)	18.0	17.0	0.18	23.3	241.3	104.3	26.3	1.266	0.739	34.078
14.70		29	1 Alluvial (sand)	18.0	17.0	0.56	0.5	259.3	112.3	27.0	1.000	0.000	27.043
15.70		30	1 Alluvial (sand)	18.0	17.0	0.69	0.2	277.3	120.3	26.8	1.000	0.000	26.800
16.70	Dg	32	5 Non-liequefiable	19.0	-	-	-	296.3	129.3	27.3	-	-	-
17.70	Dc1	25	5 Non-liequefiable	19.0	-	-	-	315.3	138.3	20.4	-	-	-
18.70	Dg	35	5 Non-liequefiable	19.0	-	-	-	334.3	147.3	27.4	-	-	-
19.70	Dc2u	41	5 Non-liequefiable	19.0	-	-	-	353.3	149.1	31.8	-	-	-



Depth X (m)	Layer symbol	N-value	Soil detail	R	L	F_L
0.00	Ag			-	-	-
0.70		18	Non-liequefiable	-	-	-
1.70		19	Non-liequefiable	-	-	-
2.70		24	Non-liequefiable	-	-	-
3.70		23	Non-liequefiable	-	-	-
4.70		29	Non-liequefiable	-	-	-
5.70	Ac	6	Non-liequefiable	-	-	-
6.70		8	Non-liequefiable	-	-	-
7.70		8	Non-liequefiable	-	-	-
8.70		6	Non-liequefiable	-	-	-
9.70		7	Non-liequefiable	-	-	-
10.70	As	7	Alluvial (sand)	0.273	0.509	0.537
11.70		7	Alluvial (sand)	0.274	0.498	0.550
12.70		22	Alluvial (sand)	0.716	0.488	1.467
13.70		27	Alluvial (sand)	1.560	0.478	3.264
14.70		29	Alluvial (sand)	0.519	0.468	1.109
15.70		30	Alluvial (sand)	0.504	0.458	1.100
16.70	Dg	32	Non-liequefiable	-	-	-
17.70	Dc1	25	Non-liequefiable	-	-	-
18.70	Dg	35	Non-liequefiable	-	-	-
19.70	Dc2u	41	Non-liequefiable	-	-	-

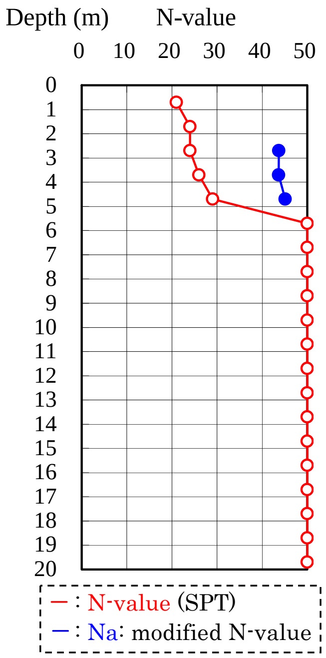
Depth (m)	Layer	R (Ave.)	FL (Ave.)	DE	
0.00 – 5.70	Ag	-	-	-	
5.70 – 10.70	Ac	-	-	-	
10.70 – 16.70	As	0.641	1.338	1	(FL>1; Not liquefiable)
16.70 – 17.70	Dg	-	-	-	
17.70 – 18.70	Dc1	-	-	-	
18.70 – 19.70	Dg	-	-	-	
19.70 – 20.70	Dc2u	-	-	-	

Liquefiable layer

Table 17.3.2-3 Result of Liquefaction Potential Assessment (MAN-W1 side)

Water Lv.	0.00	(m)	As=Fpga*PGA	0.26	(g)

Depth X (m)	Layer symbol	N-value	Soil detail	γt2 (kN/m3)	γt1=γt2-1 (kN/m3)	D50 (mm)	FC (%)	σv (kN/m2)	σv' (kN/m2)	N1	C1	C2	Na
0.00	Ac		5 Non-liequefiable	19.0	-	-	-	0.0	0.0		-	-	-
0.70		21	5 Non-liequefiable	19.0	-	-	-	13.3	6.3	46.8	-	-	-
1.70		24	5 Non-liequefiable	19.0	-	-	-	32.3	15.3	47.8	-	-	-
2.70	As	24	1 Alluvial (sand)	18.0	17.0	1.97	0.00	50.3	23.3	43.7	1.000	0.000	43.7
3.70		26	1 Alluvial (sand)	18.0	17.0	3.46	0.30	68.3	31.3	43.6	1.000	0.000	43.6
4.70		29	1 Alluvial (sand)	18.0	17.0	1.85	0.30	86.3	39.3	45.1	1.000	0.000	45.1
5.70	Dgs	50	5 Non-liequefiable	21.0	-	-	-	107.3	50.3	70.7	-	-	-
6.70		50	5 Non-liequefiable	21.0	-	-	-	128.3	61.3	64.7	-	-	-
7.70		50	5 Non-liequefiable	21.0	-	-	-	149.3	72.3	59.7	-	-	-
8.70		50	5 Non-liequefiable	21.0	-	-	-	170.3	83.3	55.4	-	-	-
9.70		50	5 Non-liequefiable	21.0	-	-	-	191.3	94.3	51.7	-	-	-
10.70	Lm	50	5 Non-liequefiable	22.0	-	-	-	213.3	106.3	48.2	-	-	-
11.70		50	5 Non-liequefiable	22.0	-	-	-	235.3	118.3	45.1	-	-	-
12.70		50	5 Non-liequefiable	22.0	-	-	-	257.3	130.3	42.4	-	-	-
13.70		50	5 Non-liequefiable	22.0	-	-	-	279.3	142.3	40.0	-	-	-
14.70		50	5 Non-liequefiable	22.0	-	-	-	301.3	154.3	37.9	-	-	-
15.70		50	5 Non-liequefiable	22.0	-	-	-	323.3	166.3	36.0	-	-	-
16.70		50	5 Non-liequefiable	22.0	-	-	-	345.3	178.3	34.2	-	-	-
17.70		50	5 Non-liequefiable	22.0	-	-	-	367.3	190.3	32.7	-	-	-
18.70		50	5 Non-liequefiable	22.0	-	-	-	389.3	202.3	31.2	-	-	-
19.70		50	5 Non-liequefiable	22.0	-	-	-	411.3	204.4	31.0	-	-	-



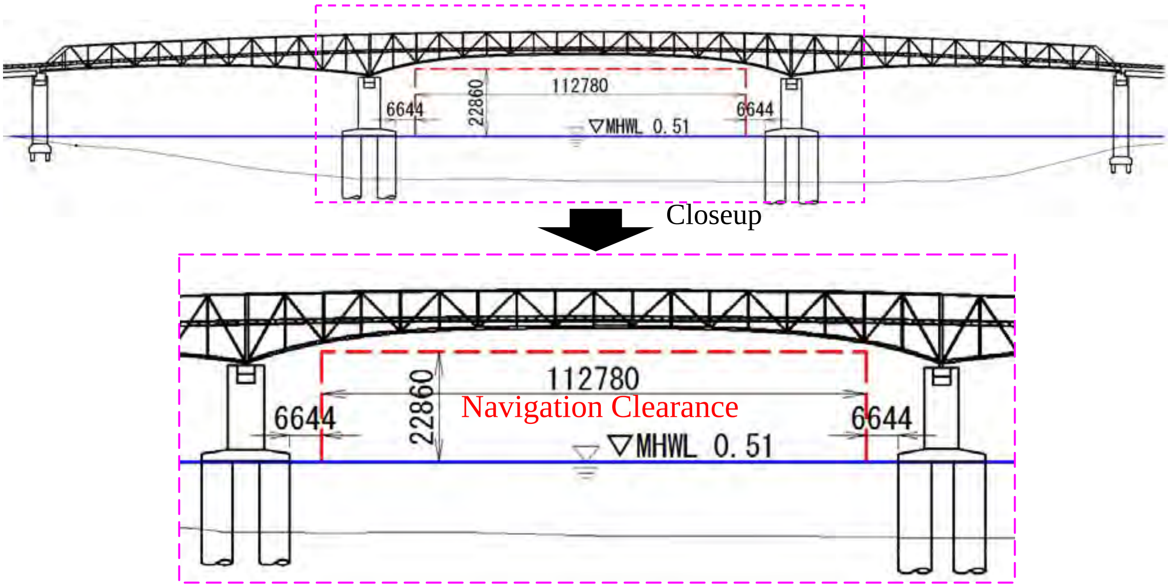
Depth X (m)	Layer symbol	N-value	Soil detail	R	L	F_L
0.00	Ac		Non-liequefiable	-	-	-
0.70		21	Non-liequefiable	-	-	-
1.70		24	Non-liequefiable	-	-	-
2.70	As	24	Alluvial (sand)	7.263	0.539	13.486
3.70		26	Alluvial (sand)	7.163	0.536	13.367
4.70		29	Alluvial (sand)	8.808	0.531	16.597
5.70	Dgs	50	Non-liequefiable	-	-	-
6.70		50	Non-liequefiable	-	-	-
7.70		50	Non-liequefiable	-	-	-
8.70		50	Non-liequefiable	-	-	-
9.70		50	Non-liequefiable	-	-	-
10.70	Lm	50	Non-liequefiable	-	-	-
11.70		50	Non-liequefiable	-	-	-
12.70		50	Non-liequefiable	-	-	-
13.70		50	Non-liequefiable	-	-	-
14.70		50	Non-liequefiable	-	-	-
15.70		50	Non-liequefiable	-	-	-
16.70		50	Non-liequefiable	-	-	-
17.70		50	Non-liequefiable	-	-	-
18.70		50	Non-liequefiable	-	-	-
19.70		50	Non-liequefiable	-	-	-

Depth (m)	Layer	R (Ave.)	FL (Ave.)	DE	
0.00 – 2.70	Ac	-	-	-	
2.70 – 5.70	As	7.744	14.483	1	(FL>1; Not liquefiable)
5.70 – 10.70	Dgs	-	-	-	
10.70 – 19.70	Lm	-	-	-	

Liquefiable layer

(3) Hydrological Condition

The Hydrological condition of 1st Mandaue-Mactan Bridge is as follows.
1. Mean high water level (MSWL) : 0.51m from mean sea level (MSL: 0m)
2. Navigation Clearance under the truss bridge
 - Vertical Clearance: 22.860m above Mean High Water Level (MHWL)
 - Horizontal Clearance: 112.78m: 6.644m from existing piers, Pier-7 & Pier-8



The above conditions are illustrated in the following figure.

Figure 17.3.2-3 Hydrological Condition of 1st Mandaue-Mactan Bridge

17.3.3 Seismic Capacity Verification of Existing Structures

(1) Summary of Seismic Capacity Verification

Seismic capacity verification of existing structures was conducted for Pier-4 and Pier-7, in accordance with BSDS provisions. In case of Pier-7, seismic capacity verification of foundation was conducted using provisions for columns because those piles are highly projected from the ground surface.
The following figure highlights the result of the seismic capacity verification of the existing structures. The detail of the verification is shown from the next page.

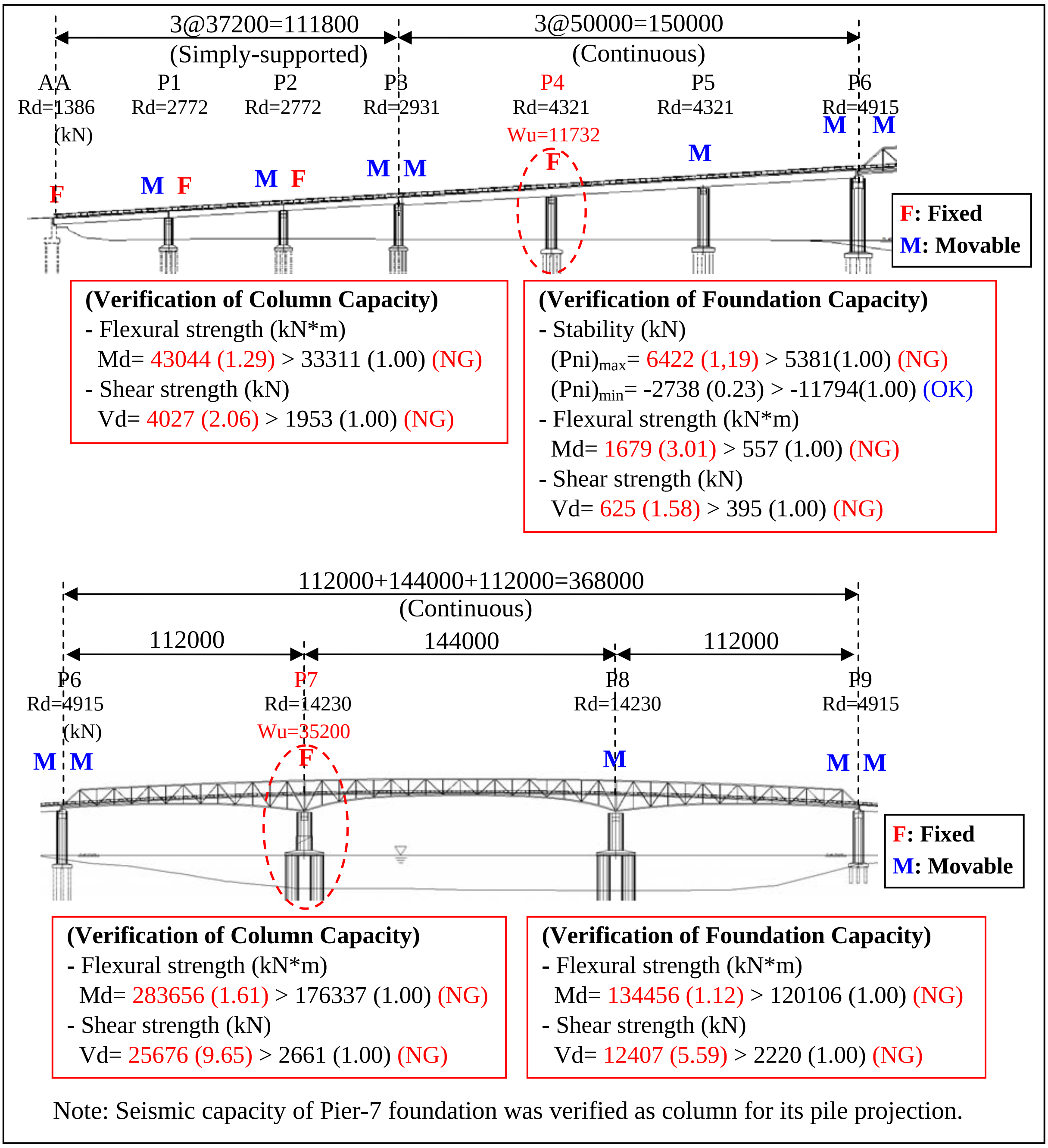
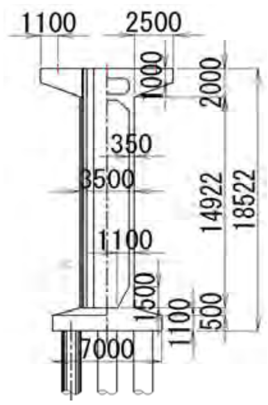
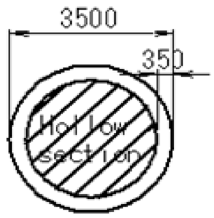


Figure 17.3.3-1 Summary of Seismic Capacity Verification

(2) Seismic Capacity Verification of Pier-4

1) Verification of Column Seismic Capacity





(Longitudinal reinforcement)
- diameter: 28 (mm)
- fy= 415 (N/mm2)
- No. of rebars: 68
- spacing: 150 (mm)
- concrete cover thickness: 125 (mm)

(Transverse reinforcement)
- diameter: 16 (mm)
- fy= 415 (N/mm2)
- spacing: 300 (mm)

Note: Pier-3 condition is applied.

- Bearing restraint condition: Fixed
- Reaction force: Rd+0.5Rl = 5,521 (kN)
- Weight of single column Wp = 1,810 (kN)
- Horizontal seismic coefficient: Csm= 0.29 (longitudinal dir.: T=1.64 (s))
  0.35 (transverse dir.T=1.36 (s))
- R-factor: 1.5 (importance: critical)
- Loads for capacity verification

Seismic Forces in Longitudinal dir.

	W	Csm	H=Wu*Csm	h	M=H*h
	(kN)	-	(kN)	(m)	(kN*m)
Superstructure	11,732	0.29	3,423	16.9	57,853
Pier	1,810	0.29	528	10.1	5,335
Sum		V_L=	3,951	M_L=	63,188

Seismic Forces in Transverse dir.

	W	Csm	H=Wu*Csm	h	M=H*h
	(kN)	-	(kN)	(m)	(kN*m)
Superstructure	5,521	0.35	1,948	19.4	37,783
Pier	1,810	0.35	639	10.1	6,450
Sum		V_L=	2,586	M_L=	44,233

Where
W (kN): Weight of the structures under consderation
H (kN): Horizontal seismic inertial force
h (m): Height from the botttom of column to the height of seismic inertial force
M (kN*m): Bending moment
V_L, V_T (kN): Shear force which acts at the bottom of pier columns/walls
M_L, M_T (kN): Bending moment which acts at the bottom of pier columns/walls

- Forces for verification (longitudinal dir.)

Nd= Rd+0.5Rl+Wp Nd: Design axial force for M-N interaction diagram
= 7,331 (kN)
Vd= $(Vmax^2+(0.3Vmin)^2)^{0.5}$ Vd: Shear force for capacity verification
= 4,027 (kN) (Vmax= 3,951 Vmin= 2,586)
Md= $(Mmax^2+(0.3Mmin)^2)^{0.5}/R$ Md: Bending moment for capacity verification
43,044 (kN*m) (Mmax= 63,188 Mmin= 44,233)

Note: - R-factor is applied to only bending moment.
- Load combination: consideration of 30% of perpendicular force

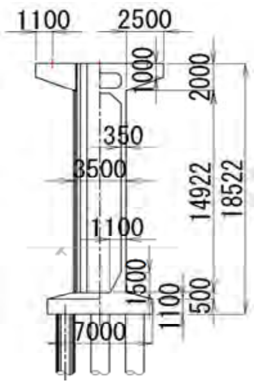
- Verification of "flexural strength"

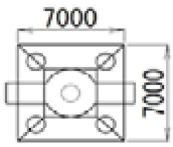
Md= 43,044 > 33,311 (=ϕ*Mn) (NG)
(1.29) (1.00)

- Verification of "shear strength"

Vd= 4,027 > 1,953 (=ϕ*Vn) (NG)
(2.06) (1.00)

2) Verification of Foundation Stability





(Assumed pile condition)
- Cast-in-place concrete pile foundation
- Diameter: 1200 (mm)
- Number of piles: 5
- Pile length: L=27 (m)
- Rebar conditon
  - Longitudinal: 12-D20 (ctc.300)
  - Transverse: D16 ctc.300 (mm)
  - Yeild strength: fy= 415 (N/mm2)

- Direction of seismic force: Longitudinal dir.
- Load condition

Rd+0.5Rl=	5,521	(kN)
Wu=	5,521	(kN)
Wp=	1,810	(kN)
As=	0.26	(g)
Mu=	37,012	(kN*m)

Seismic forces of footing

	Wf	0.5*As	Ff=Wf*(0.5*As)	hf/2	Mf=H*h
	(kN)	(g)	(kN)	(m)	(kN*m)
Footing	1,882	0.13	245	0.80	196

Where,
Wu (kN): Weight of superstructure under consideration
Wp (kN): Weight of pier column & coping
Wf (kN): Weight of pier footing
Ff (kN): Horizontal seismic inertial force
hf (m): height of footing
Mf (kN*m): Bending moment

- Forces for capacity verification

Nd= Rd+0.5Rl+Wp+Wf
= 9,213 (kN)
Vd= Mp/h+Ff (Mp= 48,116 Ff= 245)
= 3,092 (kN)
Md= Mp+Mf=1.3Mu+Mf
= 48312 (kN*m) (Mu= 37,012 Mf= 196)
Note: h= 16.9 (m) h: Height from the botttom of column to the point of seismic inertial force

- Capacity verification

1) Verification of "maximum axial load at the pile head"
$(Pni)_{max}$ = 6,422 > 5,381 (NG)
(1.19) (1.00)

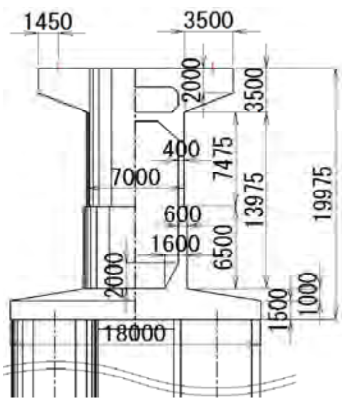
2) Verification of "maximum axial pull-out force at the pile head"
$(Pni)_{min}$ = -2,738 < -11,794 (OK)
(0.23) (1.00)

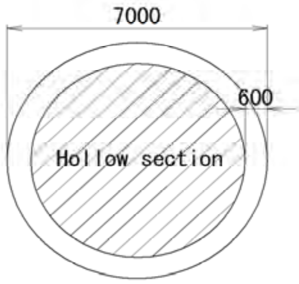
3) Verification of "flexural strength"
Md = 1,679 > 557 (NG)
(3.01) (1.00)

4) Verification of "shear strength" for longitudinal direction
Vd = 625 > 395 (NG)
(1.58) (1.00)

(3) Seismic Capacity Verification of Pier-7

1) Verification of Column Seismic Capacity





(Longitudinal reinforcement)
- diameter: 28 (mm)
- fy= 415 (N/mm2)
No. of rebars: 68
- spacing: 150 (mm)
- concrete cover thickness: 125 (mm)

(Transverse reinforcement)
- diameter: 16 (mm)
- fy= 415 (N/mm2)
- spacing: 300 (mm)

Note: Pier-3 condition is applied.

- Bearing restraint condition: Fixed
- Reaction force: Rd+0.5Rl = 16,530 (kN)
- Weight of single column Wp = 7,235 (kN)
- Horizontal seismic coefficient: Csm= 0.60 (longitudinal dir.: T=0.80 (s))
  0.61 (transverse dir.T=0.78 (s))
- R-factor: 1.5 (importance: critical)
- Loads for capacity verification

Seismic Forces in Longitudinal dir.

	W	Csm	H=Wu*Csm	h	M=H*h
	(kN)	-	(kN)	(m)	(kN*m)
Superstructure	35,200	0.60	21,048	17.5	368,338
Pier	7,235	0.60	4,326	11.4	49,318
Sum		V_L=	25,374	M_L=	417,656

Seismic Forces in Transverse dir.

	W	Csm	H=Wu*Csm	h	M=H*h
	(kN)	-	(kN)	(m)	(kN*m)
Superstructure	14,230	0.61	8,680	25.4	220,480
Pier	7,235	0.61	4,413	11.4	50,312
Sum		V_T=	13,094	M_T=	270,792

Where
W (kN): Weight of the structures under consderation
H (kN): Horizontal seismic inertial force
h (m): Height from the botttom of column to the height of seismic inertial force
M (kN*m): Bending moment
V_L, V_T (kN): Shear force which acts at the bottom of pier columns/walls
M_L, M_T (kN): Bending moment which acts at the bottom of pier columns/walls

- Forces for verification (longitudinal dir.)

Nd= Rd+0.5*Rl+Wp
= 21,465 (kN) — Nd: Design axial force for M-N interaction diagram

Vd= $(Vmax^2+(0.3Vmin)^2)^{0.5}$ — Vd: Shear force for capacity verification
= 25,676 (kN) — (Vmax= 25,374 Vmin= 13,094)

Md= $(Mmax^2+(0.3Mmin)^2)^{0.5}/R$ — Md: Bending moment for capacity verification
283,656 (kN*m) — (Mmax= 417,656 Mmin= 270,792)

Note: - R-factor is applied to only bending moment.
- Load combination: consideration of 30% of perpendicular force

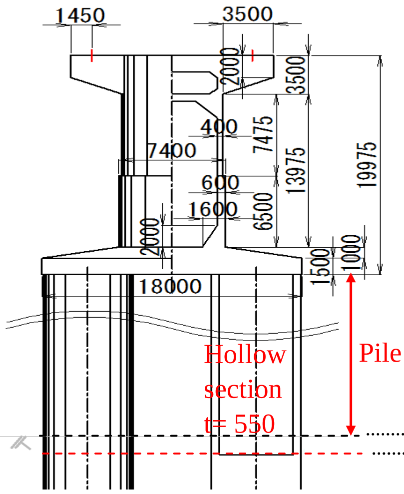
- Verification of "flexural strength"

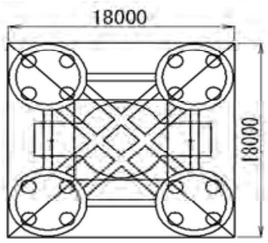
Md= 283,656 > 176,337 (=ϕ*Mn) (NG)
(1.61) (1.00)

- Verification of "shear strength"

Vd= 25,676 > 2,661 (=ϕ*Vn) (NG)
(9.65) (1.00)

2) Verification of Foundation Stability





- Cast-in-place concrete pile foundation (Hollow section)
- Diameter: 6200 (mm)
- Number of piles: 4
- Pile projection length: 14.8 (m)
- Rebar conditon
  - Longitudinal: 128-D25 (ctc.150)
  - Transverse: D16 ctc.300 (mm)
  - Yeild strength: fy= 415 (N/mm2)

- Direction of seismic force: Longitudinal dir.

- Load condition

Rd+0.5Rl=	16,530	(kN)	
Wp=	7,235	(kN)/column	
Wf=	15,763		
Csm=	0.60	(g)	
Mp= 1.3*Mu =	254,709	(kN*m)	(Mu= 195,930 (kN*m))
Vp= Mp/h =	14,555	(kN)	(h= 17.5 (m))
n=	4	; number of piles	

Seismic forces acting on single pile

	W	Csm	F	h	M
	(kN)	(g)	(kN)	(m)	(kN*m)
Column		Vp/n=	Vp/n= 3,639		Mp/n= 63,677
Footing	3,941	0.60	2,356	9.9	23,328
Single Pile	10,724	0.60	6,412	7.4	47,451
		Vd=	12,407	Md=	134,456

Where,
W (kN): Weight of structures
H (kN): Horizontal seismic inertial force
h (m): height of seismic inertial force
M (kN*m): Bending moment

- Forces for capacity verification

Nd= (Rd+0.5Rl+Wp+Wf)/n+Wpile (n= 4 ; number of piles)
= 20,606 (kN)
Vd= 12,407 (kN)
Md= 134,456 (kN*m)

- Verification of "flexural strength"

Md= 134,456 > 120,106 (=ϕ*Mn) (NG)
(1.12) (1.00)

- Verification of "shear strength"

Vd= 12,407 > 2,220 (=ϕ*Vn) (NG)
(5.59) (1.00)

17.3.4 Comparative Studies on Seismic Capacity Improvement Schemes

(1) Outline of Comparison Studies on Seismic Capacity Improvement Schemes

Selection of “Seismic Capacity Improvement Methods” was done in accordance with the following flowchart process.

Control of Seismic Inertial Force by Changing Bearing Restraint Conditions

Seismic Capacity Improvement Scheme for Pier Columns

Seismic Capacity Improvement Scheme for Pier Copings

Seismic Capacity Improvement Scheme for Foundations
(1) Improvement scheme for foundation of piers on land and piers in shallow water
(2) Improvement scheme for foundation of piers in deep water
(3) Improvement scheme for abutments

Planning for Unseating Prevention System
(1) Planning for replacement of bearings
(2) Planning for seat extender
(3) Planning for unseating prevention devices
(4) Planning for structural limiting (shear keys)

Planning for Repair Works
(not part of seismic capacity improvement)

Outline design

Figure 17.3.4-1 Outline of Comparison Studies on Seismic Capacity Improvement Schemes

(2) Control of Seismic Inertial Force by Changing Bearing Restraint Conditions

Old continuous bridges are likely to have only one pier with fixed bearings in longitudinal direction. The fixed piers are regarded as the weakest point of the bridge structures, for the fixed piers shoulder total superstructure weight under earthquakes. In this case, recombination of bearing restraint conditions should be considered with the application of seismic devices (ex. elastomeric bearing and seismic damper) in order to control the scale of seismic inertial forces on each substructure and save the only fixed substructures. The mechanism of seismic device application is shown below.

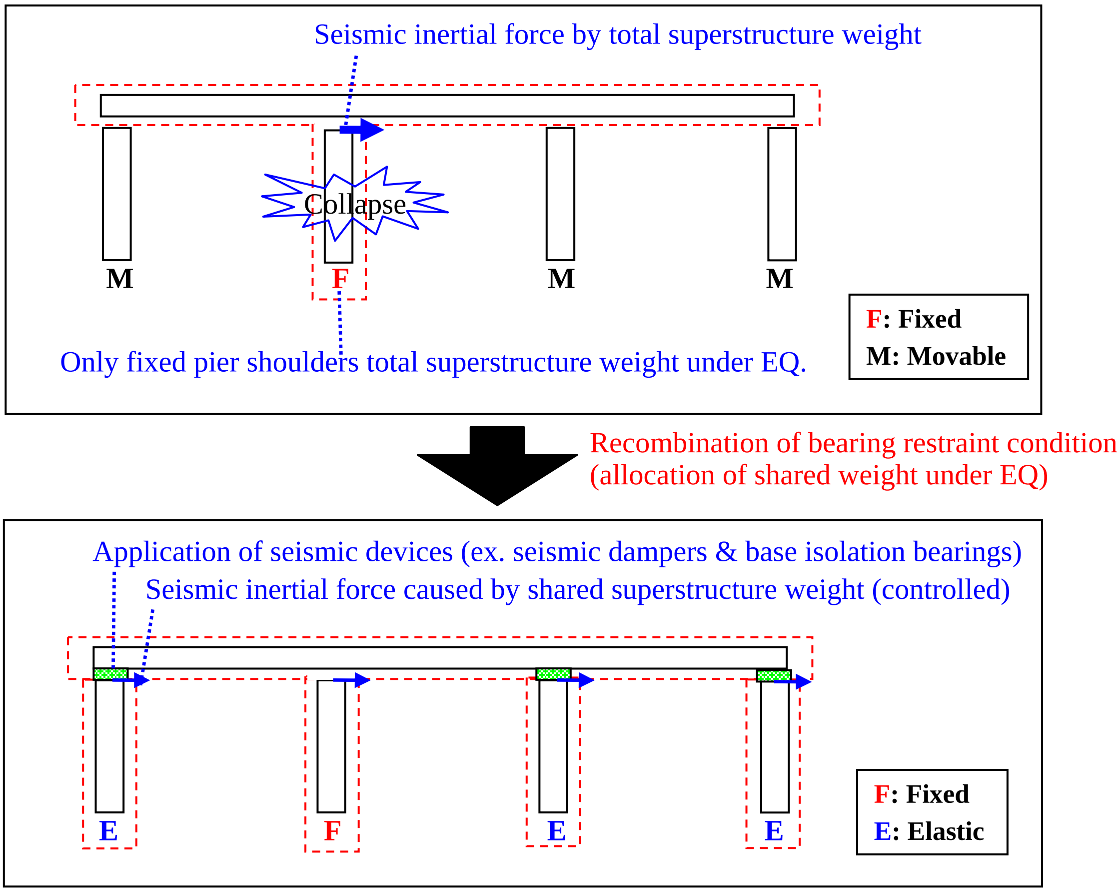


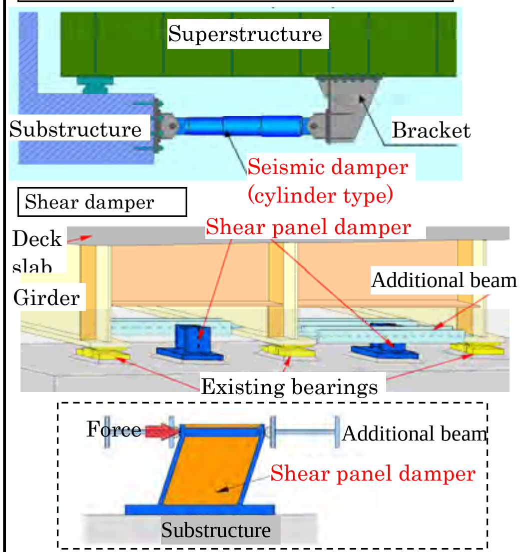
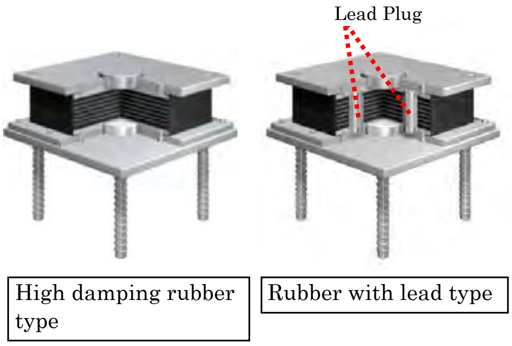
Figure 17.3.4-2 Control of Seismic Inertial Force by Application of Seismic Devices

In this study, the following two seismic devices are compared for the application.

- Alternative-1: Seismic damper (hydraulic cylinder type) & shear panel damper
- Alternative-2: Elastomeric bearing

As a result, "seismic damper & shear panel damper" are recommended to mitigate the seismic inertial force of fixed piers. Generally speaking, elastomeric bearings are more cost-effective than seismic dampers. However, they have problems in "structural characteristics" and "constraints of construction". Elastomeric bearings are recommended to be applied to newly-constructed bridges, considering its structural and constructive restrictive conditions. The detail of the comparison is shown in the following table.

Table 17.3.4-1 Comparison of Seismic Devices

	Seismic damper (hydraulic cylinder type) & Shear panel damper		Base isolation device	
Outline of improvement method	Seismic damper (hydraulic cylinder type)  (Source: Japan Bridge Association)		 (Source: Japan Bridge Association)	
	- To absorb seismic energy and control seismic inertial force on substructures		- To reduce seismic inertial force to substructures - To make natural period longer; base isolation - To avoid the “Sympathetic Vibration” of substructure and superstructure.	
Structural characteristic	- Easy to control the seismic inertial force on substructures - Possible to restrain/control the amout of structural movement under EQ - Possible to be used as unseating prevention device	A	- Difficult To control the seismic inertial force on substructures - New bearings are wider and taller than old ones: need of larger space for the installation - Impossible to restrain/control the amout of structural movement under EQ; superstructures and backwalls collide with each other under EQ	D
Constraints of construction	- Quick and easy installtation - No need of removal of existing structures for the installation - For shear panel dampers, need of installation of additional beams to be attached to esisting superstructure members	A	- Need of partial removal of existing pier coping to fit new bearings into the space; new bearing eight is higer than that of old one	C
Duration	Short	A	Typical priod of bearing installation	B
Recommend-ation	Recommended		Not applicable	

Recommendation for installation location of seismic dampers & shear panel dampers is shown in the next page. The location and number of the installation will be optimized during calculation process, studying the scale of seismic retrofit works for piers.

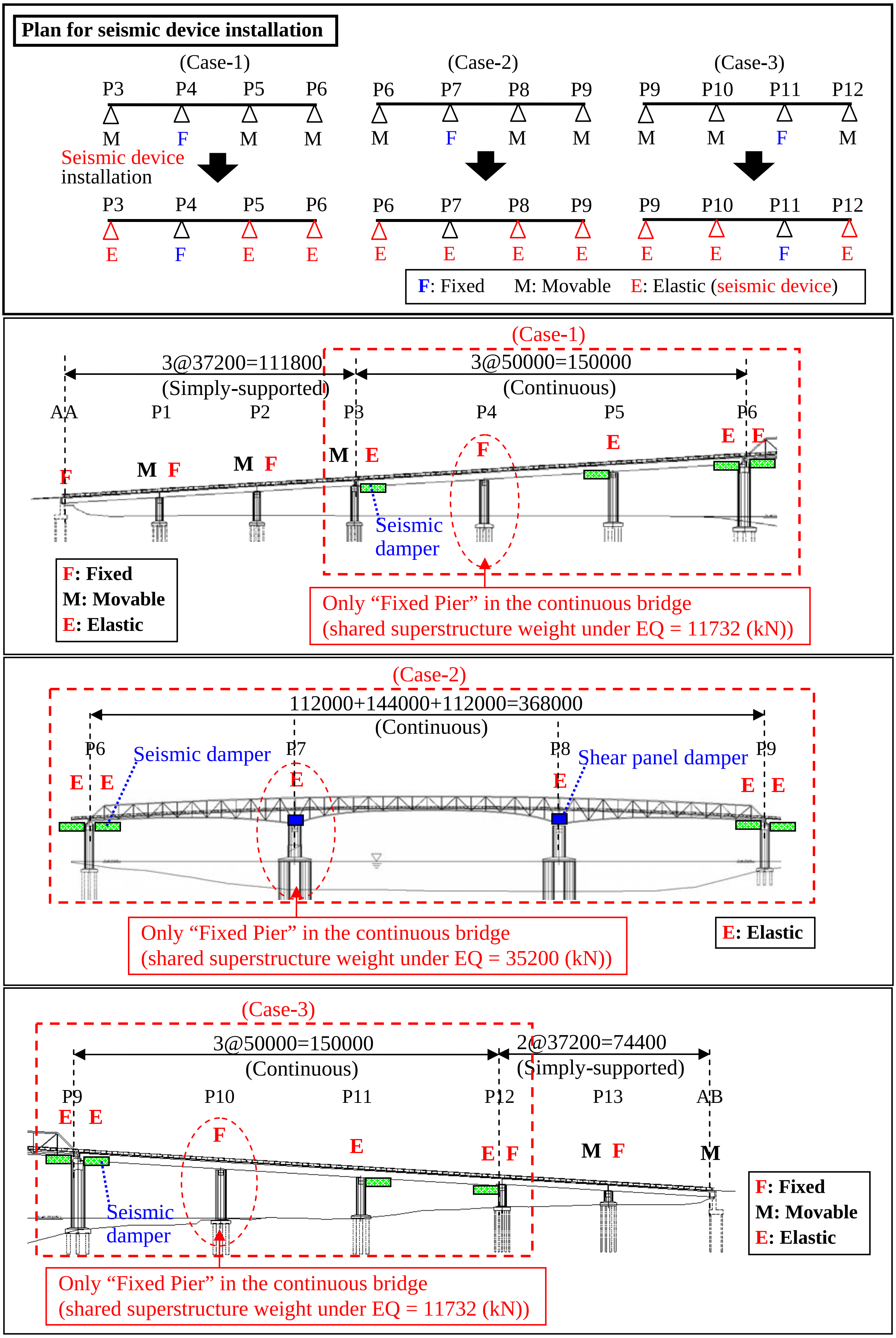


Figure 17.3.4-3 Recommendation for Location of Seismic Damper Installation

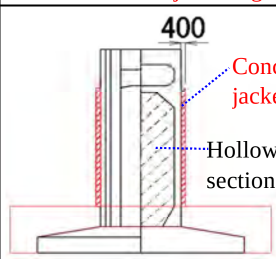
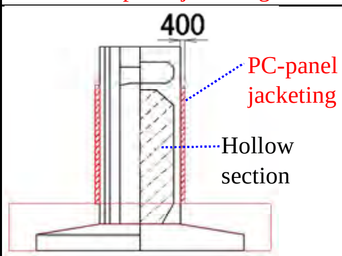
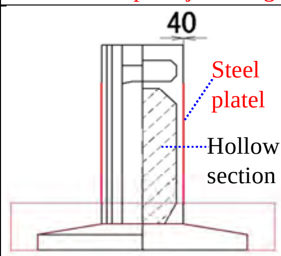
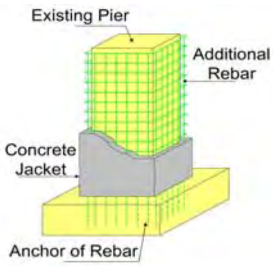
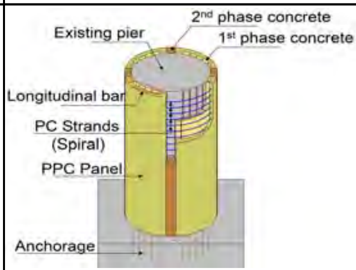
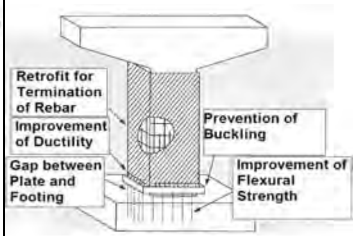
(3) Seismic Capacity Improvement Scheme for Pier Columns

The following three improvement schemes were compared for pier columns so as to improve flexural resistance, shear resistance.

- Alternative-1: Concrete jacketing
- Alternative-2: PC panel jacketing
- Alternative-3: Steel plate jacketing

As a result of evaluation, “Concrete jacketing” was selected for its cost-effectiveness and overall suitability for its implementation. The detail of the comparison is shown below.

Table 17.3.4-2 Comparison of Improvement Schemes for Pier Columns

	Concrete jacketing		PC panel jacketing		Steel plate jacketing	
Outline of improvement method	 Thickness: 400mm		 Thickness: 400mm		 Thickness: 40mm	
Detail image	 (Source: OCAJI)		 (Source: OCAJI)		 (Source: OCAJI)	
Cost*	1.00	A	4.17	C	Excluded	
Structural characteristic	- Improvement of flexural strengthand shear strength - Large impact to clearance/river flow - Effect of weight increase on existing foundation structures	B	- Improvement of flexural strengthand shear strength - High improvement of confinment by PC strands - Large impact to clearance/river flow - Effect of weight increase on foundation structures	A	- Improvement of flexural strengthand shear strength - Low impact to clearance/river fllow - Unable to develop required flexural strength	D
Durability / Maintenance	Typical durability of cast-in-place concrete structure: no need of maintenance	B	High durability by the application of precast panel: no need of maintenance	A	Need of constant maintenance to prevent corossions: every 30 years	C
Constraints of construction	- Need of cast-in-place concrete construction -Need of smoothing of existing column surface - Need of installation of shear connectors - Need of larger construction space	B	- No need of cast-in-place concrete construction: precast panel -Need of smoothing of existing column surface	A	- Need of installation of shear connectors - Need of splicing of steel plates -Need of smoothing of existing column surface - Need of painting for corossion protection	B
Duration	Long	B	Short	A	Long	B
Recommend-ation	Recommended					

Compared with “Pier-8 condition”.

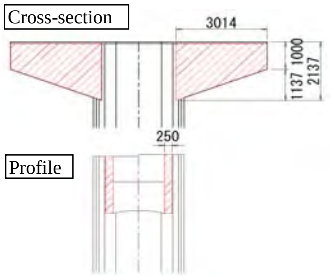
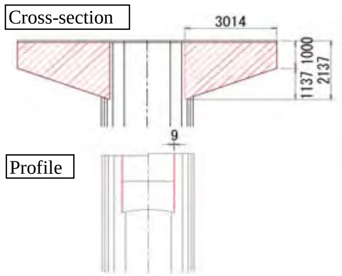
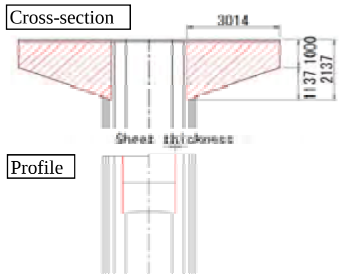
(4) Seismic Capacity Improvement Scheme for Pier Coping

The following three improvement methods were compared for pier copings so as to improve flexural resistance and shear resistance.

- Alternative-1: Concrete jacketing
- Alternative-2: Steel plate jacketing
- Alternative-3: Carbon fiber sheet jacketing

As a result of evaluation, "concrete jacketing" was selected for its structural advantage and overall suitability for its implementation. The detail of the comparison is shown below.

Table 17.3.4-3 Comparison of Improvement Schemes for Pier Copings

	Concrete jacketing		Steel plate jacketing		Carbon fiber sheet jacketing	
Outline of improvement method	 Thickness: 250mm		 Thickness: 9mm		 Sheet Tickness	
Structural characteristic	- Improvement of both flexural strength and shear strength - Able to extend seat width; 250mm - Effect of weight increase on foundation structures	A	- Improvement of shear strength - Unable to develop flexural strength	D	- Improvement of shear strength - Unable to develop enough flexural strength	D
Durability / Maintenance	No need of maintenance	A	Need of constant maintenance to prevent corossions: every 30 years	C	Need of constant maintenance for surface protection: every 30 years	C
Constraints of construction	- Need of cast-in-place concrete construction -Need of smoothing of existing column surface - Need of installation of shear connectors - Need of larger construction space	B	- Need of installation of shear connectors - Need of splicing of steel plates -Need of smoothing of existing column surface - Need of painting for corossion protection	B	- No need of installation of shear connectors: to be attached - Temprature and humidity must be checked for the construction quality - Need of placement of polymer cement mortar as surface protection	A
Duration	Long	B	Long	B	Short	A
Recommend-ation	Recommended					

Compared with "Pier-1 condition".

(5) Seismic Capacity Improvement Scheme for Foundations

1) Foundation Improvement Scheme for Piers on Land and Piers in Shallow Water

For improvement of existing pier foundation capacities, “additional piles for reinforcement” is recommended for piers on land and piers in shallow water. In case of 1st Mandaue-Mactan Bridge, construction types for the foundation retrofit work are categorized into the following three.

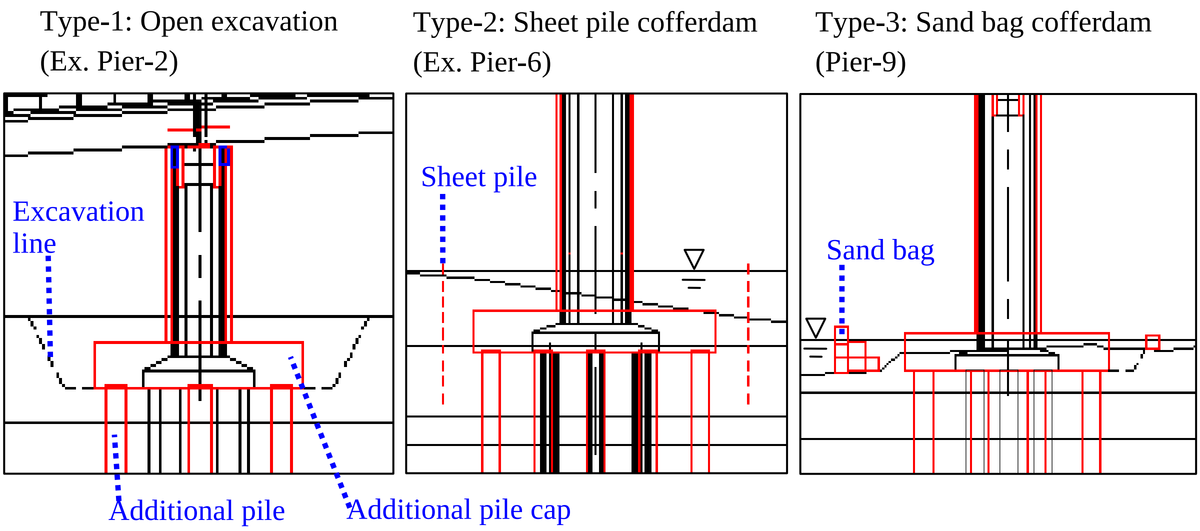


Figure 17.3.4-4 Construction Types for the Foundation Retrofit Work

Many pile-driving methods are available these days. However, in case of additional pile driving, applicability under lower vertical clearance must be considered in the selection of pile-driving method.

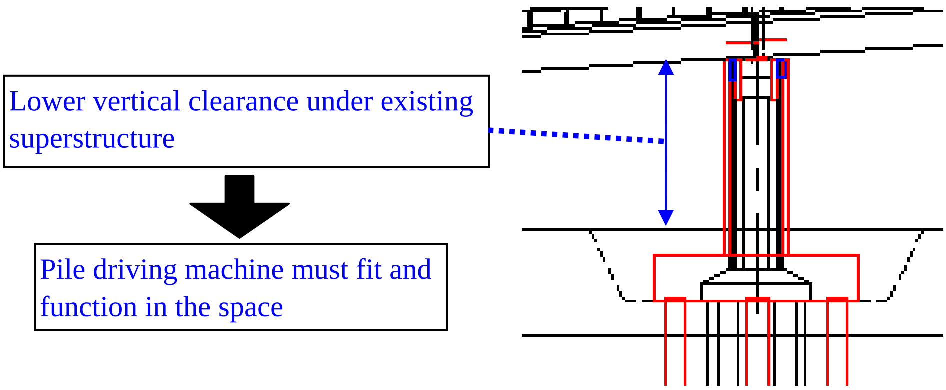


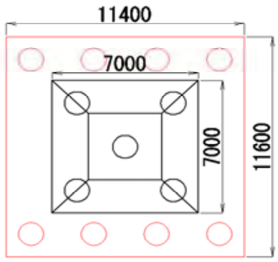
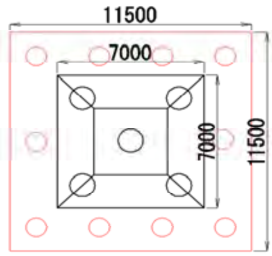
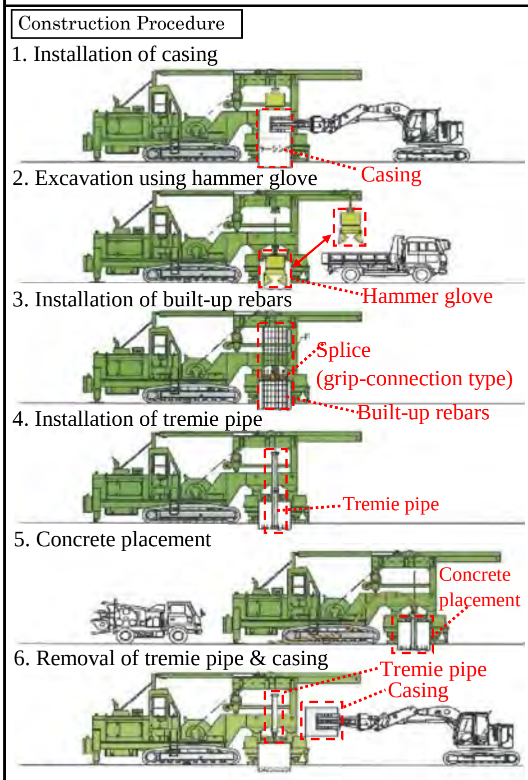
Figure 17.3.4-5 Restrictive condition for additional pile driving

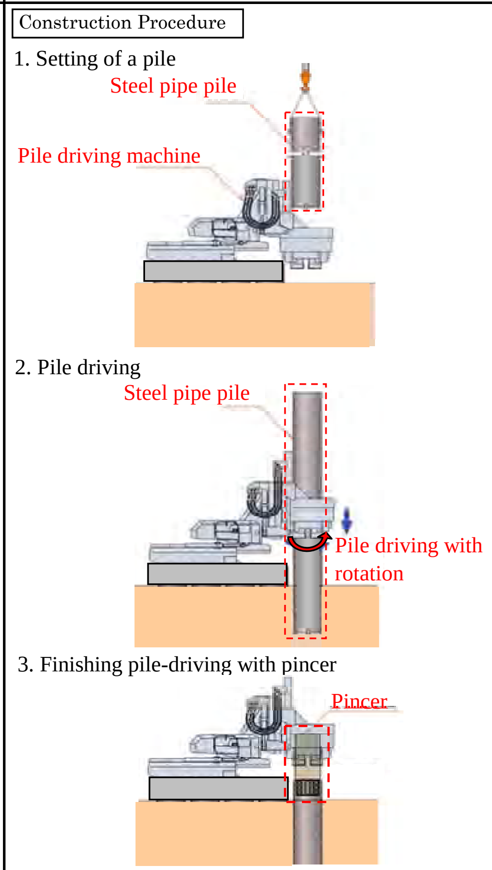
In consideration with the above restrictive condition, the following two applicable pile driving methods were compared.

- Alternative-1: Cast-in-place concrete pile foundation (revolving all casing method)
- Alternative-2: Steel pipe pile foundation (jacked pile method)

As a result of the comparison, “cast-in-place pile foundation (revolving all casing method)” is recommended for its suitability for the construction site which has rock layer and gravel layers. The detail of comparison study is shown in next page..

Table 17.3.4-4 Comparison of Improvement Schemes for Foundations (1)

<table>
<tr><td></td><td colspan="2">Cast-in-place concrete pile (CCP) foundation
(revolving all casing method)</td><td colspan="2">Steel pipe pile (SPP) foundation
(revolving type press-in method)</td></tr>
<tr><td rowspan="2">Outline of improvement method</td><td colspan="2">CCP Φ1200

</td><td colspan="2">SPP Φ1000

</td></tr>
<tr><td colspan="2">

(Source: NETIS plus)</td><td colspan="2">

(Source: NETIS plus)</td></tr>
<tr><td>Cost*</td><td>1.00</td><td>A</td><td>1.27</td><td>B</td></tr>
<tr><td>Structural characteristic</td><td>- Rigid concrete structure: valid against soft layer movement and liquefaction-induced lateral spreaading
- Cast-in-place concrete pile: Structural reliability depends on construction quality
- Maximum pile diameter: Φ1500</td><td>A</td><td>- Non-rigid structure: vulnerable to lateral forces such as liquefaction-induced lateral spreading
- Fabricated product: reliable structure
- Maximum pile diameter: Φ1500</td><td>B</td></tr>
<tr><td>Constraints of construction</td><td>- Minimum reuired vertical clearance: 5m
- Cast-in-place concrete pile: need of accurate field work
- Need of many rebar splices
- Able to penetrate solid substance such as rock</td><td>A</td><td>- Minimum reuired vertical clearance: 5-6m: it depends on the pile diameter size
- Need of many welding splices
- Able to penetrate solid substance such as rock
- Same machine is aplicable to SPSP installation.</td><td>A</td></tr>
<tr><td>Duration</td><td>Long</td><td>C</td><td>Short; twice as short as CIP pile foundation</td><td>A</td></tr>
<tr><td>Recommend-ation</td><td colspan="2">Recommended</td><td colspan="2"></td></tr>
</table>

Cost*: The cost is compared with “Pier-4 condition”.

2) Foundation Improvement Scheme for Piers in Deep Water

As illustrated below, in the selection of foundation improvement method for existing piers in deep water, the following two restrictive conditions must be considered.

a) Navigation width

Targets for the improvement, Pier-7 & Pier-8, are located in the vicinity of navigation clearance range. The additional structures must be outside the range. Also, obstacles such as temporary work platform should be minimized during construction.

b) Additional rigid structure for the solution of pile projection problem

The target pier foundations have a problem with lack of stiffness due to large pile projection length from riverbed surface. The improvement scheme must be selected focused on improvement of foundation stiffness around projected pile range.

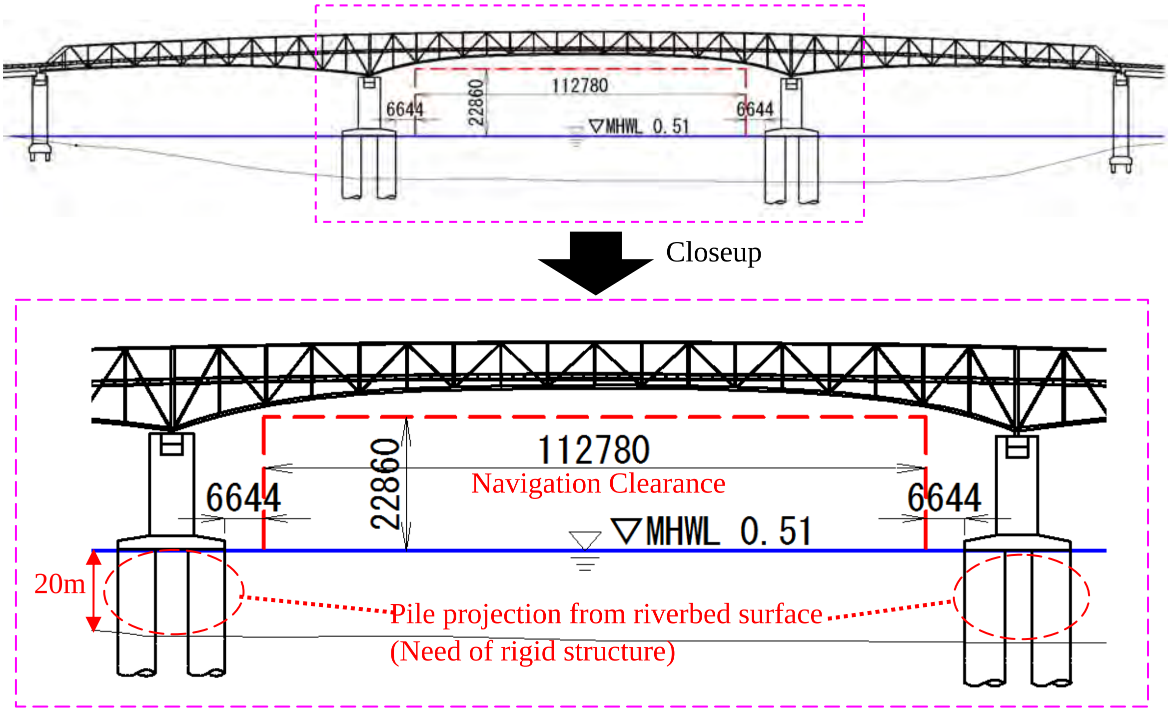


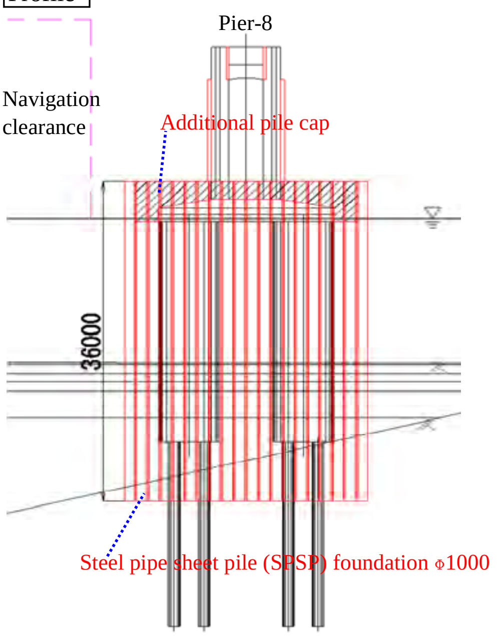
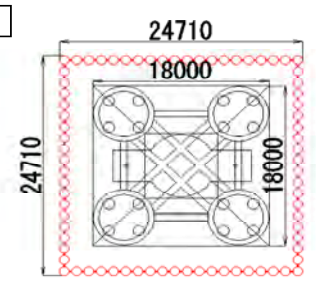
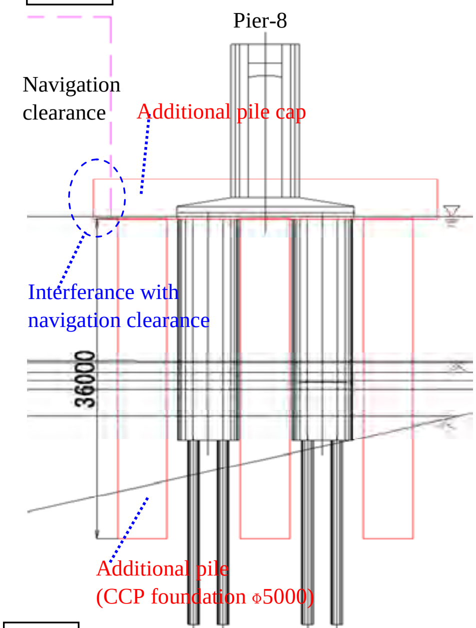
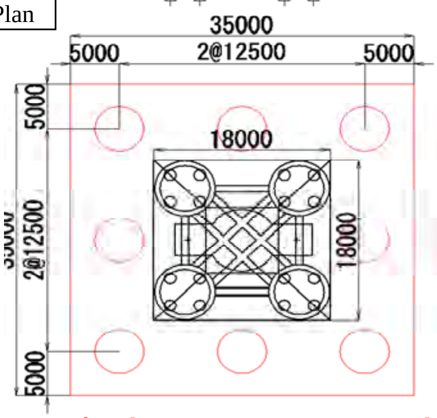
Figure 17.3.4-6 Restrictive Conditions for Selection of Foundation Improvement Method

In consideration with the above restrictive condition, the following two applicable pile-driving methods were compared.

- Alternative-1: Steel pipe sheet pile (SPSP) foundation
- Alternative-2: Multi-column foundation (Large diameter concrete pile foundation)

As a result of the comparison, “steel pipe sheet pile (SPSP) foundation” is recommended for its structural reliability. The detail of comparison study and construction procedure of “steel pipe sheet pile (SPSP) foundation” is shown from the next page.

Table 17.3.4-5 Comparison of Improvement Schemes for Foundations (2)

	Steel pipe sheet pile (SPSP) foundation		Multi-column foundation (Large diameter pile foundation)	
Outline of improvement method	Profile  Plan  Existing foundation structures are not considered in the design.		Profile  Plan  Existing foundation structures are not considered in the design.	
Structural characteristic	- Column type foundation structure: valid for "pile projection problem"; improvement of flexural resistance - Fabricated product: reliable structure - No effect on navigation clearance interfarance	A	- Large diameter cast-in-place concrete pile foundation: ϕ5000 - Permanent effect on navigation clearance interfarance - The structure can't develop enough flexural resistance against level-2 scale earthquakes	D
Constraints of construction	- Need of accurate splice work - Unable to penetrate solid substance such as rock: need of additional excavation machine - Less effect on navigation clearance interfarance even during construction by the application of "non-staging method".	A	- Cast-in-place concrete pile: need of accurate field work - Need of many rebar splices - Able to penetrate solid substance such as rock - No need of installation of sheet piles - Large effect on navigation clearance interfarance during construction	C
Duration	Long	C	Long	C
Recommend-ation	Recommended			

Compared with "Pier-8 condition".

Construction Procedure of SPSP Note: the example is the case of new pier construction.

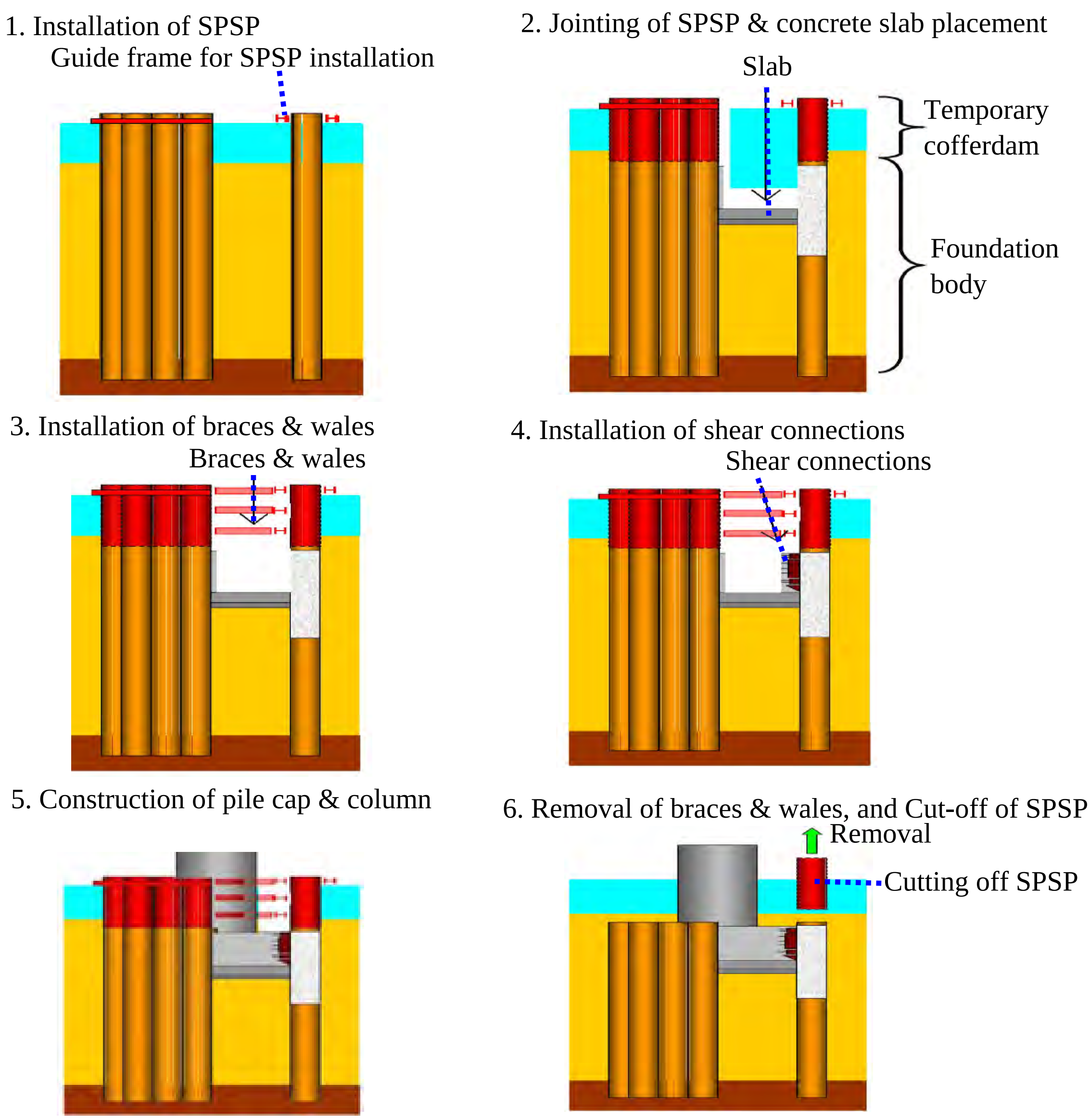


Figure 17.3.4-7 Construction Procedure of SPSP Foundation

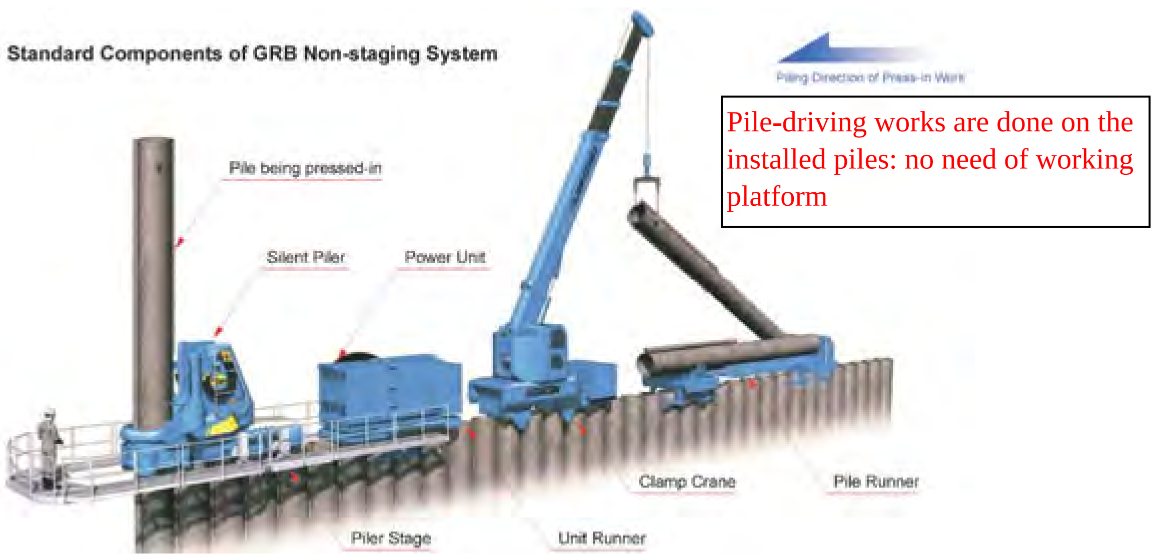


Figure 17.3.4-8 "None-stage method" for SPSP Foundation Installation

3) Seismic Capacity Improvement Scheme for Abutments

Structural information of existing abutments is unknown as shown in the following figure. Therefore, study on improvement schemes for abutments was conducted assuming the underground structures. The unknown structure must be revealed by certain survey during detail design stage.

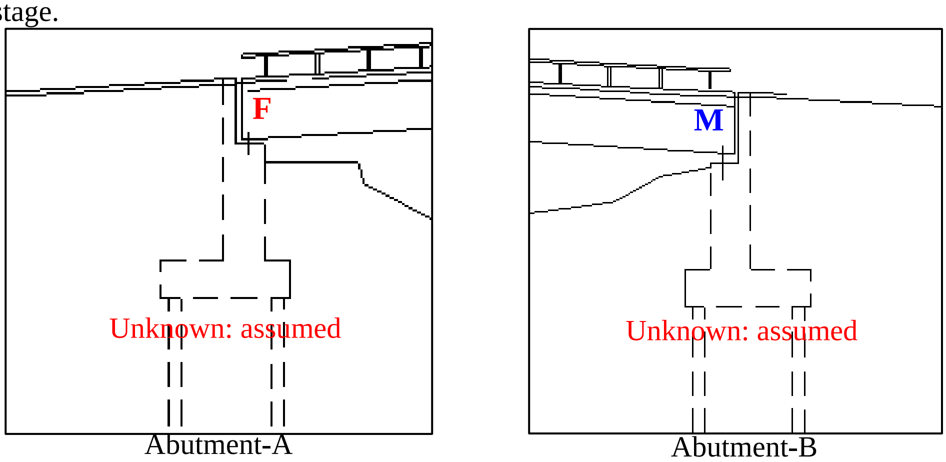


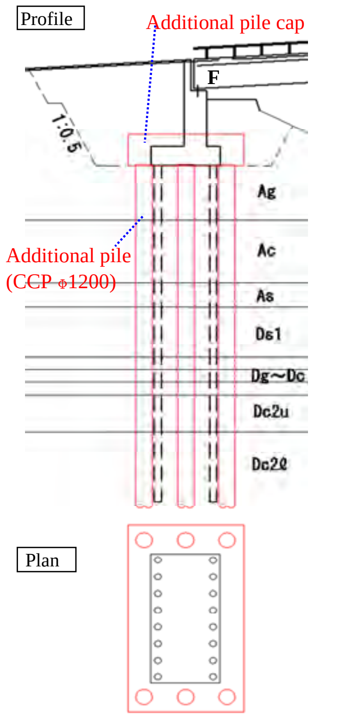
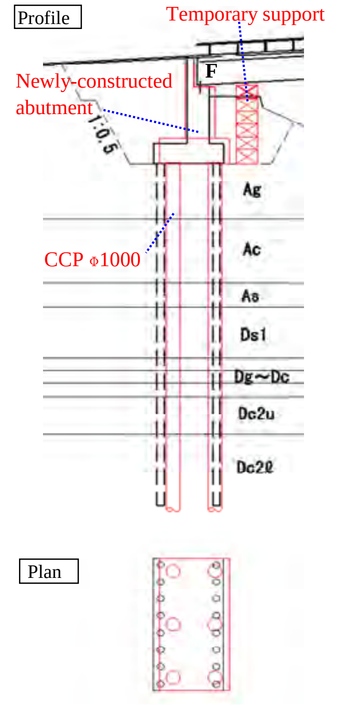
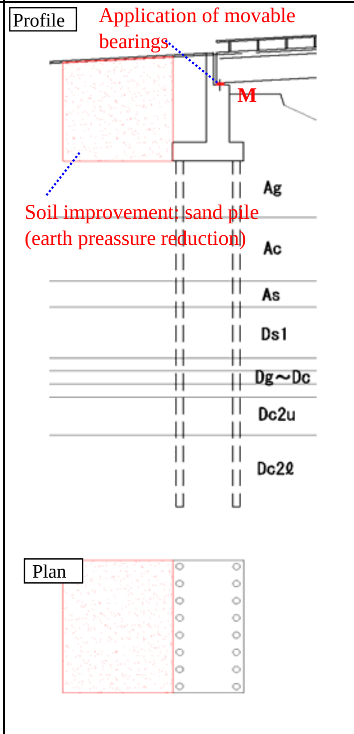
Figure 17.3.4-9 Assumed Existing Abutment Condition

In consideration with the structural characteristics of abutments, the following three improvement methods were compared.

- Alternative-1: Soil improvement work with application of movable bearings
- Alternative-2: Additional piles for reinforcement
- Alternative-3: Total reconstruction

As a result of the comparison, "additional piles for reinforcement" is recommended as abutment improvement method for the cost-effectiveness and overall suitability. The detail of comparative study is shown in the next page.

Table 17.3.4-6 Comparison of Improvement Schemes for Abutments

	Additional piles for reinforcement (Cast-in-place concrete pile)		Total reconstruction (Cast-in-place concrete pile)		Soil improvement with application of movable bearings	
Outline of improvement method						
Cost*	1.00	A	2.00	B	4.26	C
Structural characteristic	- Improved stability with additional piles - Application of cast-in-place pile foundation as friction pile foundation to minimize the pile length - The additional piles resist liquefaction-induced lateral spreading - Need of firm connection between existing pile cap and additional pile cap	B	- Total reconstruction of abutment - Application of cast-in-place pile foundation as friction pile foundation to minimize the pile length - The piles resist liquefaction-induced lateral spreading	A	- Soil improvement (sand pile) for earth preassure reduction - Soil improvement (chemical grouting) for liquefaction protection - Change of bearing restraint condition from "Fix" to "Movable": reduction of seismic inertia force	B
Constraints of construction	- Need of large-scale excavation - Need of pile driving under superstructure: small vertical clearance	B	- Need of large-scale excavation &demolishing of exiting structure - Installation of temporary support - Need of pile driving under superstructure: small vertical clearance - Need of lane-closure during construction - Risk of superstructure damage during construction	C	- No need of excavation & demolishing work - Need of a specific machine for chemical grouting work under existing structure - Need of lane-closure during sand pile development work	A
Necessary information	- Need of detail existing structure information for detail design	C	- No need of detail existing structure information for detail design	A	- Need of existing pile cap information for detail design of sand pile	B
Duration	Medium	B	Medium	B	Long	C
Recommend-ation	Recommended					

Cost*: The cost is compared with “Abutment-A condition”.

17.3.4.2Planning for Unseating Prevention System

1) Planning Procedure of Unseating Prevention System

Besides the strengthening of bridge structures, installation of unseating prevention system is very important in order to prevent superstructure-fall-down, which could happen in case that retrofitted structures are devastated by unexpectedly large-scale earthquakes. In this study, the planning of unseating prevention system was done in accordance with the following procedure.

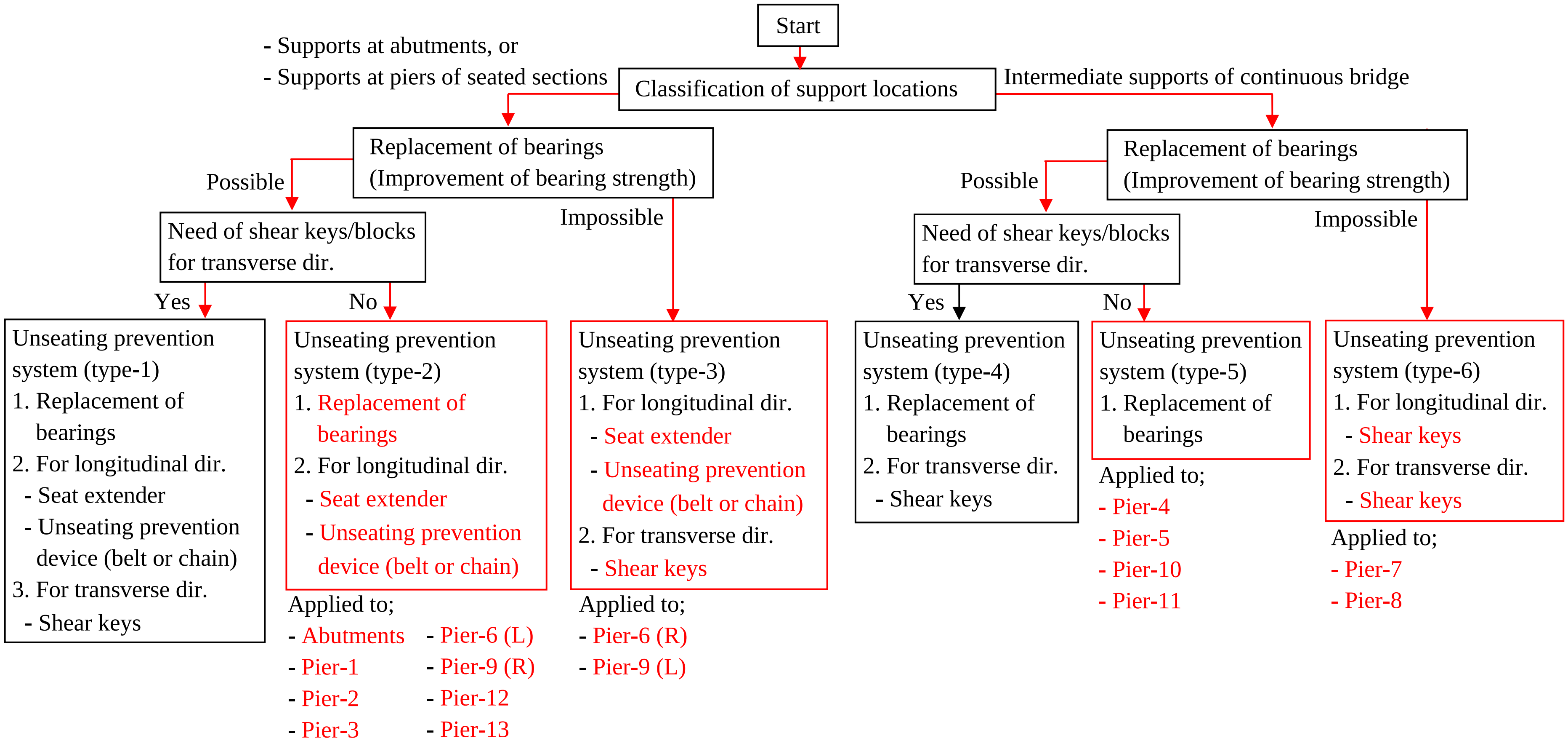


Figure 17.3.4-10 Basic Concept of Unseating Prevention System Planning

2) Planning for Replacement of Bearings

All the existing bearings will be replaced with steel bearings which can resist level-2-scale earthquakes and fit in the space between existing structures, except for bearings of continuous steel truss bridge whose superstructure is too heavy to jack up for the bearing replacement work.

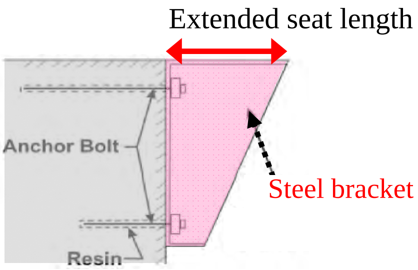
3) Planning for Seat Extender

Seat length of abutments and piers at seated sections must be checked with the following formula.
Required minimum seat length: $S_{EM} = 0.7 + 0.005L$ (m)

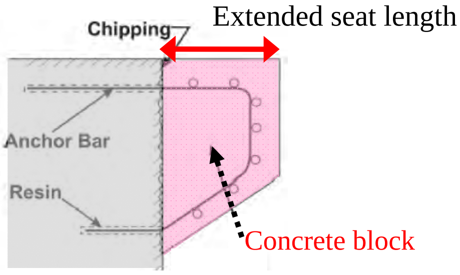
Where,

L: seat-length-related span length (m)

Seat extenders will be installed at abutments and piers at seated sections, where seat length doesn't satisfy the above requirement. In consideration of construction efficiency and quality, "steel bracket" was selected for piers which needs seat length extension. Concrete blocks will be installed at abutments where no scaffolding is required for the installation of seat extenders.



Steel bracket (applied to piers)
- Easy and quick installation
- Fabricated product; good quality



Concrete block (applied to abutments)
- Cast-in-place concrete structure;
  - Inefficient installation at higher locations
  - Structural reliability depends on construction quality

Figure 17.3.4-11 Concrete block and Steel Bracket

4) Planning for Unseating Prevention Devices

Unseating prevention devices will be installed at abutments and piers at seated sections. The device type was selected in accordance with the following rules.

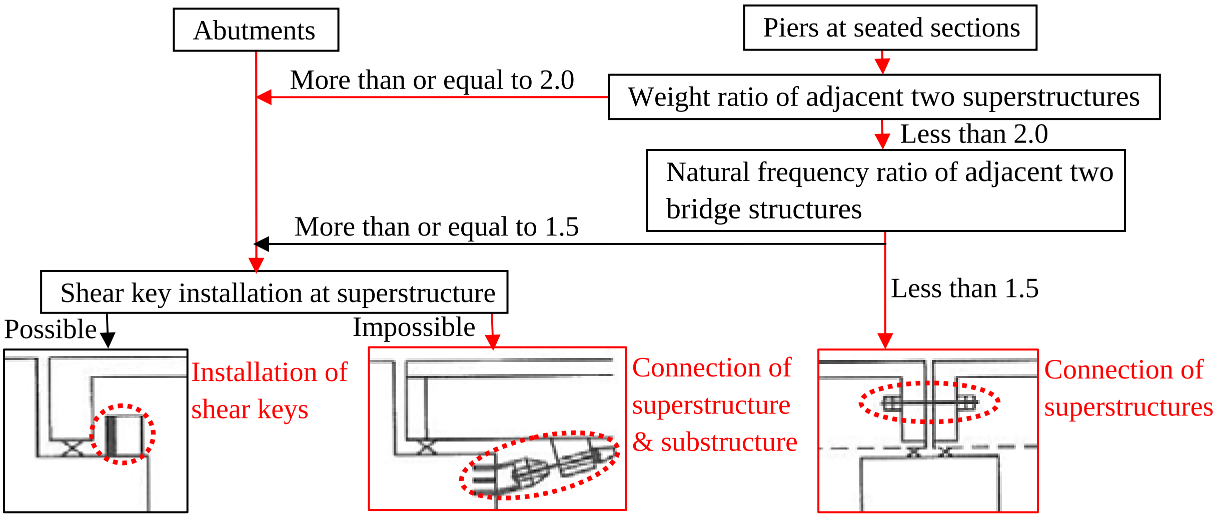
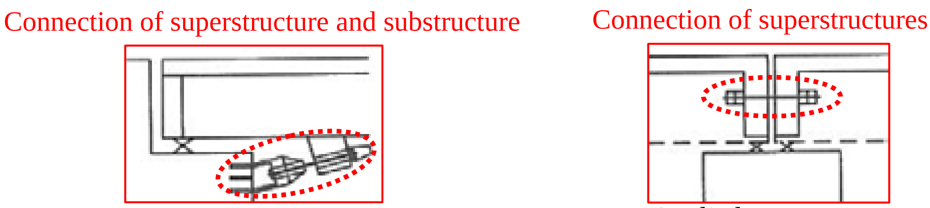


Figure 17.3.4-12 Selection of unseating prevention device type



Applied to;
- Abutments
- Pier-3
- Pier-6
- Pier-9
- Pier-12

Applied to;
- Pier-1
- Pier-2
- Pier-13

Note: no unseating prevention devices are needed at Pier-3, Pier-6, Pier-9, & Pier-12

Figure 17.3.4-13 Selection of Unseating Prevention Device type (continued)

5) Planning for Structure Limiting Horizontal Displacement (Shear Keys)

"Structure limiting horizontal displacement (shear keys)" must be installed for unseating prevention at piers where their bearings can't be replaced with new one which can resist large-scale earthquakes. In case of 1st Mandaue-Mactan Bridge, existing bearings of Pier-6 (R), Pier-7, Pier-8, and Pier-9 (L) can't be replaced because of heavy superstructure weight.

As for Pier-7 and Pier-8 at intermediate supports of continuous bridge, shear keys will be installed for both longitudinal and transverse direction.

As for Pier-6 (R) and Pier-9 (L) at seated sections, shear keys will be installed only for transverse direction because unseating prevention device will be installed for longitudinal direction instead.

The image of shear key installation is illustrated below.

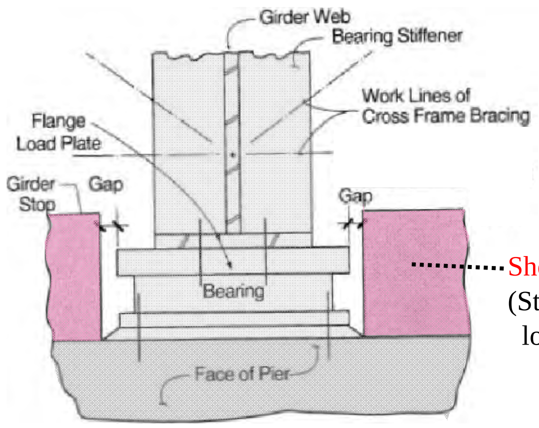


Figure 17.3.4-14 Structure Limiting Horizontal Displacement (Shear Keys)

17.3.5 Planning for Repair Works

In addition to the seismic retrofit plans, the following three repair work

- Replacement/installation of expansion joints
- Repainting of steel members
- Epoxy injection of deck slab

[Replacement/installation of expansion joints]

Apparently, existing expansion joints are in good condition. However, water leaking was confirmed at piers under expansion joints. It's better to replace them with new one for maintenance reason.

Abutment-B | Pier-9 | Pier-6 | Pier-1

Figure 17.3.5-1 Current Condition of Existing Expansion Joints

[Repainting of steel members]

Repainting of existing steel members is recommended, especially for primary and secondary members of I-girders which are heavily corroded. Besides the I-girders, it's better to repaint steel members of truss bridge for maintenance.

Secondary member of I-girder | Primary member of I-girder

Figure 17.3.5-2 Current Condition of Existing Steel Members

[Epoxy injection of deck slab]
Epoxy injection of deck slab is recommended to repair cracking, water leaking of the existing deck slab. Also, mortar covering is recommended to repair rebar exposure of the overhanging deck slab.

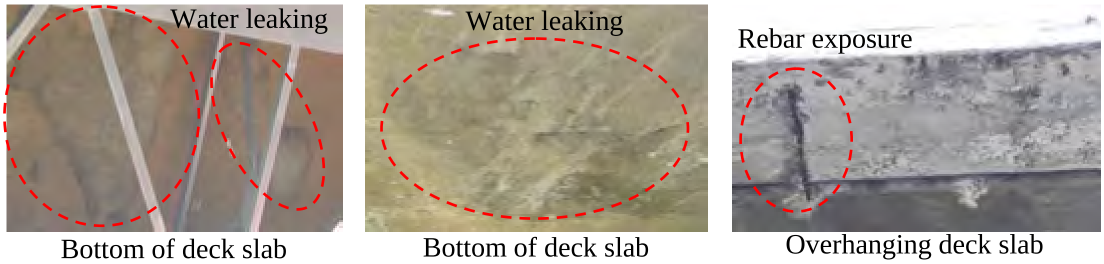
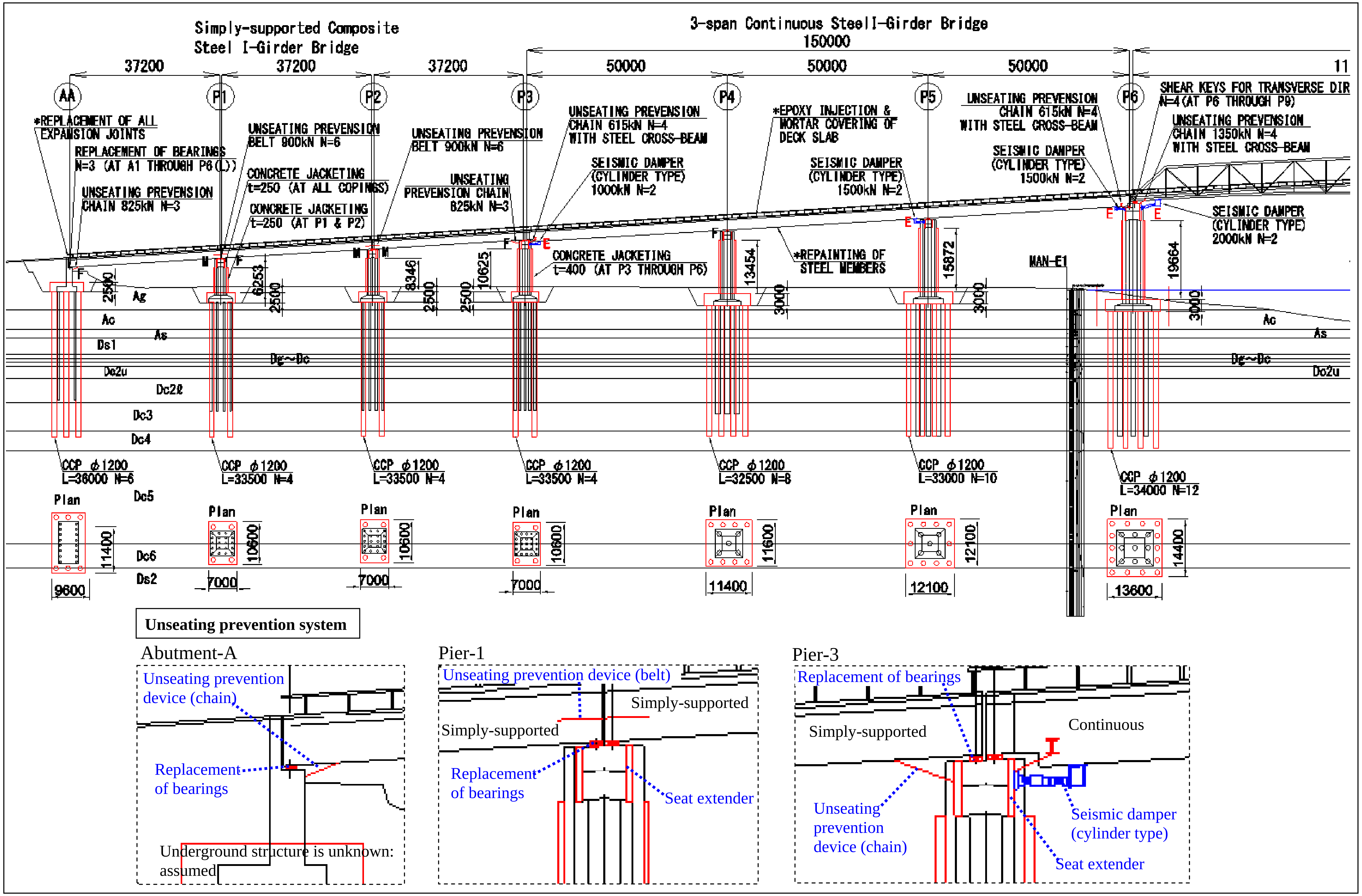
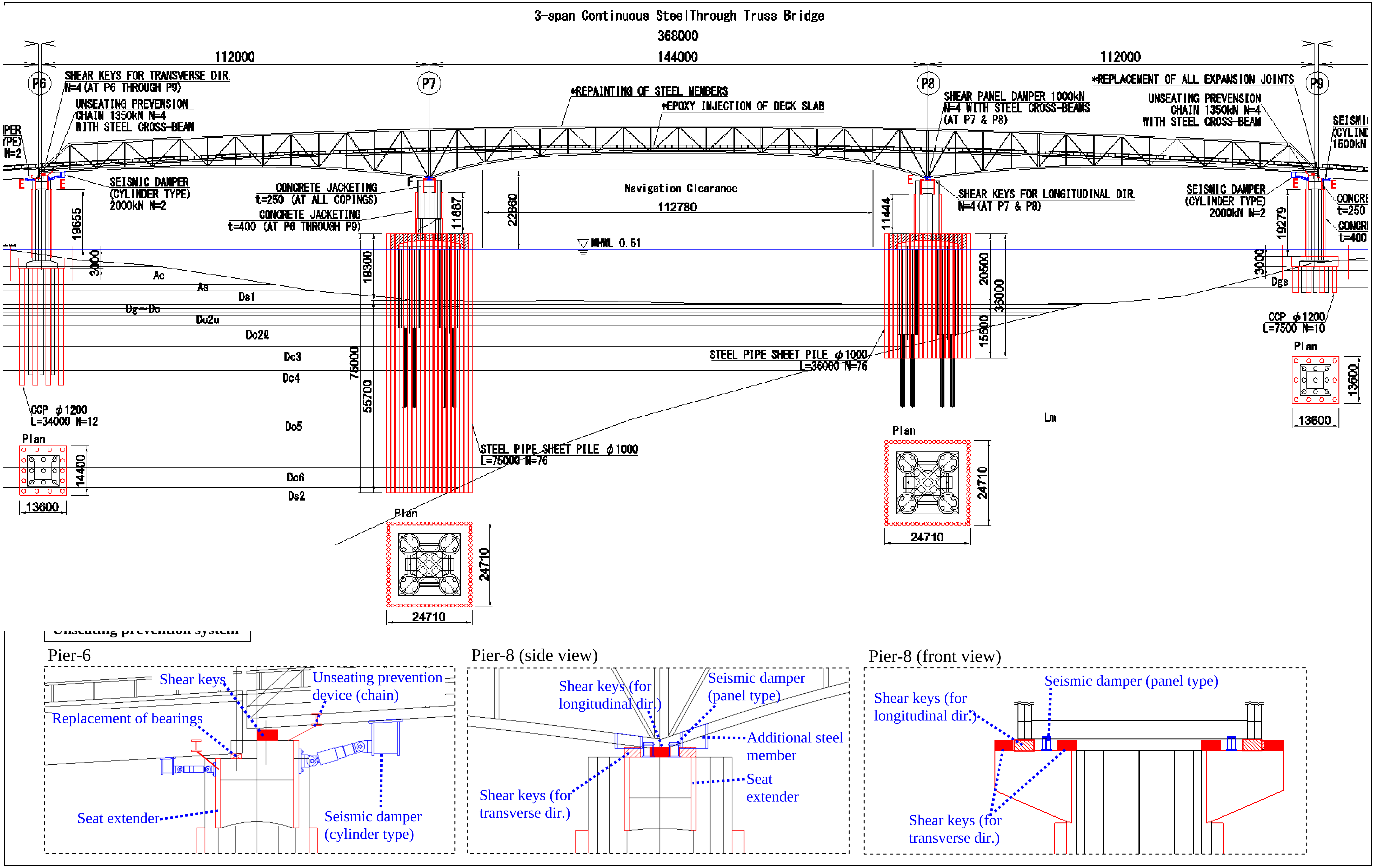


Figure 17.3.5-3 Current Condition of Existing Deck Slab

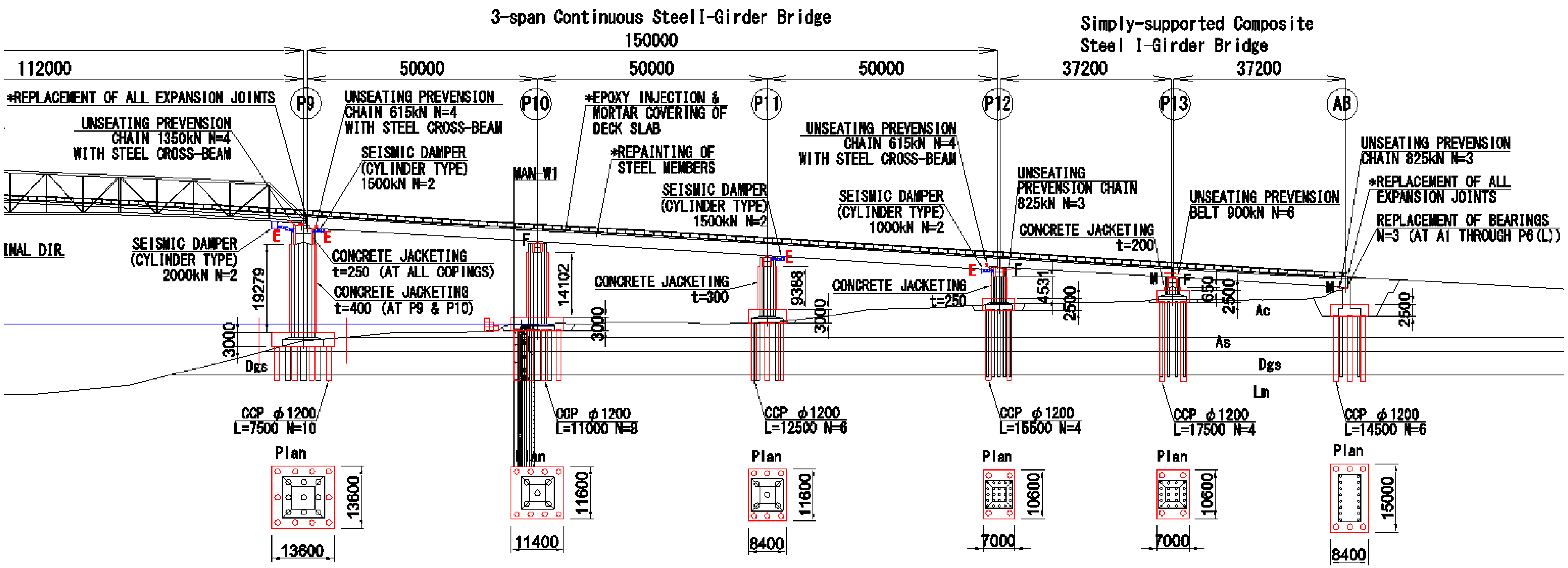
17.3.6 Summary of Proposed Seismic Retrofit Schemes & Repair Works



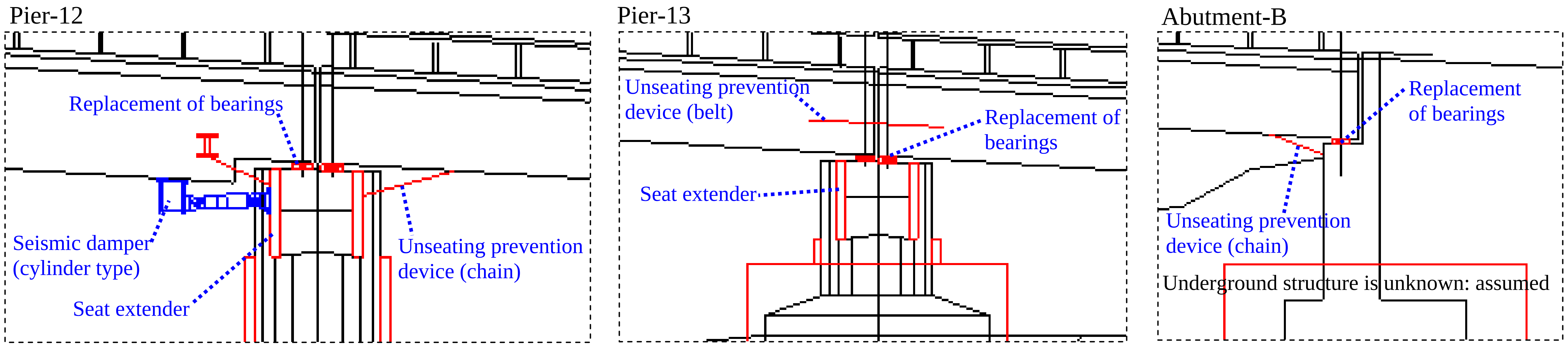
(General View & Structural Drawings are shown in Appendix-4)



(General View & Structural Drawings are shown in Appendix-4)



Unseating prevention system



(General View & Structural Drawings are shown in Appendix-4)

CHAPTER 18 CONSTRUCTION PLANNING AND COST ESTIMATE

18.1 General

18.1.1 Bridge type

The construction planning and the cost estimate were considered as the result of the outline design as shown in Table 18.1.1-1.

Table 18.1.1-1 The Recommended Structure Type of Selected Bridges

Bridge	Retrofit / Replace	Structure Type (Outline)
Lambingan	Replace	- Simple Supported Steel Deck Stiffened Lohse Bridge - Cast in Place Concrete Pile (CCP)
Guadalupe	<Outer> Replace	- 3-Span Continuous Steel Deck Box-Shape Girder - Steel Pipe Sheet Pile (SPSP) - CCP
	<Inner> Retrofit	- Reconstruction of Piers with SPSP - Ground Improvement of Abutments
1st Mandaue Mactan	Retrofit	- Concreting Jacket - Additional CCP - SPSP
Palanit	Replace	- 3-Span Continuous PC I-Shape Girder - Spread Foundation
Mawo	Replace	- 3-Span Continuous PC Fin Back - CCP
Lilo-an	Retrofit	- Concreting Jacket - Additional CCP
Wawa	Replace	- 3-Span Continuous Composite Steel Lattice Truss - CCP

18.2 Construction Planning

18.2.1 General

(1) Purpose of Construction Planning

The Purposes of construction planning are as follows:
- To study the construction method of the selected replace/retrofit plan;
- To study the traffic detour plan under minimum influence to the existing traffic; and
- To plan the temporary structure for cost estimation.

(2) Right of Way

The construction is to be conducted in the Right-of-Way, after the removal of squatter and facilities. As a result the meeting with the DPWH engineer in field survey, the Right-of-Way width is as shown in Table 18.2.1-1,

Table 18.2.1-1 The Width of Right of Way

Bridge	Width of Right of Way
Lambingan	Width of Bridge
Guadalupe	Width of Bridge
1st Mandaue Mactan	30m
Palanit	30m
Mawo	30m
Lilo-an	30m
Wawa	60m

(3) Procurement Planning

Most of the construction materials and equipment will be procured generally in the Philippines. On the other hand, steel materials and special equipment as shown in Table 18.2.1-2 are to be imported from other countries.

Table 18.2.1-2 List of Imported Items

Item		Remark
Steel Girder	Steel and Fabrication	- Import fabricated girder
	Erection	- Skilled Labor - Equipment for Slide and Block erection
PC structure	PC steel	- Material
	Casting	- Skilled Labor
CIP Pile (Under limited space)		- Equipment and Skilled Labor
Steel Pipe Sheet Pile		- Material ,Equipment and Skilled Labor
Bearing, Expansion, Unseating Prevention System		- Material (Installation cost includes the girder erection cost)

(4) Temporary Road

The temporary road for construction and detour traffic was planned utilizing the Right-of-Way. The drawing of temporary road is among the Outline Design Drawings.

18.2.2 Construction Planning of Lambingan Bridge

(1) Construction Site

The construction of Lambingan bridge will need a construction site for fragmentation of the existing girders and assembling of the new girders. The construction site of Pasig river improvement project near Lambingan bridge will be used for this construction. Since the aqueduct bridge crosses Pasig River at downstream side of Lambingan bridge, the assembled arch girder can be transported from downstream side. The assembly stage should be planned at the upstream side, and the site was planned near Makati-Mandaluyong bridge as shown in Figure 18.2.2-1.

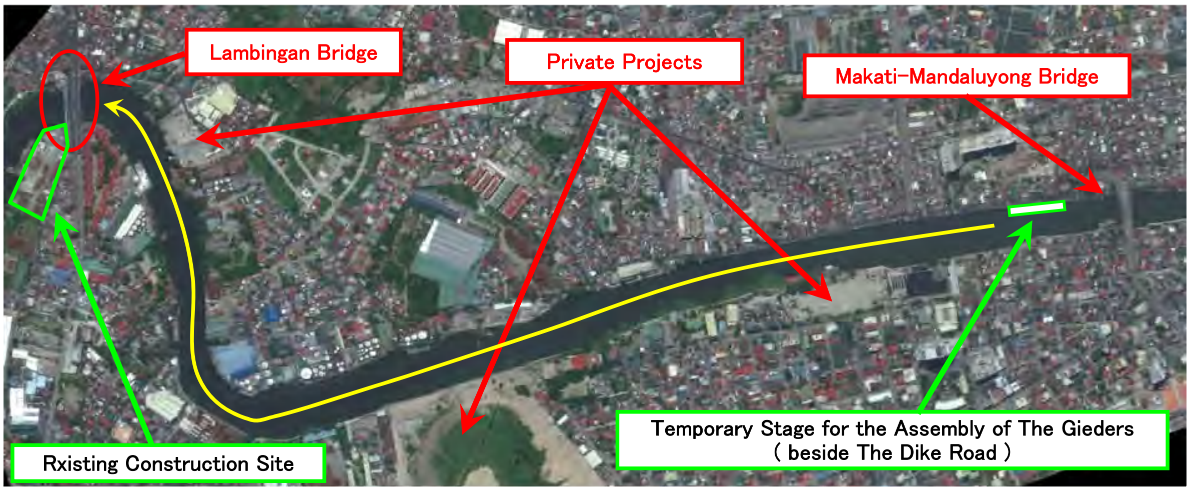
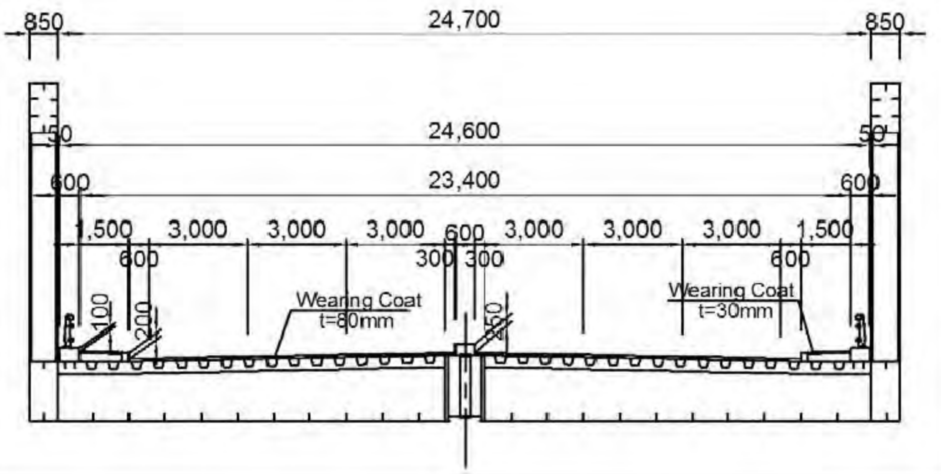


Figure 18.2.2-1 Location Site of Lambingan Bridge

(2) Traffic Detour Plan

As a result of outline design, superstructure type as shown in Figure 18.2.2-2 was recommended to be erected by stage construction.



(Simple Supported Steel Deck Stiffened Lohse Bridge)

Figure 18.2.2-2 Recommend superstructure Type of Lambingan Bridge

Based on the traffic count survey and traffic analysis of this project, lane control from six (6) lanes to two (2) lanes control will not make a queue at New Panaderos road around Lambingan bridge as shown in Table 18.2.2-1.

Table 18.2.2-1 Result of Traffic Analysis

Dir-1 from Sta. Ana to Sta. Mesa
Year
2018

Time	Volume (PCU)	Capacity (PCU)	Cap-Vol (PCU)	Queue Vehicle (PCU)	Queue Length (m)
6:00	837	2,156	-1,319	0	0
7:00	967	2,156	-1,189	0	0
8:00	1,100	2,156	-1,056	0	0
9:00	1,178	2,156	-978	0	0
10:00	1,026	2,156	-1,130	0	0
11:00	1,259	2,156	-897	0	0
12:00	949	2,156	-1,207	0	0
13:00	1,112	2,156	-1,044	0	0
14:00	1,390	2,156	-766	0	0
15:00	1,086	2,156	-1,070	0	0
16:00	1,361	2,156	-795	0	0
17:00	1,577	2,156	-579	0	0
18:00	1,546	2,156	-610	0	0
19:00	1,525	2,156	-631	0	0
20:00	1,330	2,156	-826	0	0
21:00	1,070	2,156	-1,086	0	0
22:00	1,097	2,156	-1,059	0	0
23:00	629	2,156	-1,527	0	0
0:00	486	2,156	-1,670	0	0
1:00	240	2,156	-1,916	0	0
2:00	247	2,156	-1,909	0	0
3:00	274	2,156	-1,882	0	0
4:00	406	2,156	-1,750	0	0
5:00	521	2,156	-1,635	0	0
Total	19,313				

Dir-2 from Sta. Mesa to Sta. Ana
Year
2018

Time	Volume (PCU)	Capacity (PCU)	Cap-Vol (PCU)	Queue Vehicle (PCU)	Queue Length (m)
6:00	1,589	2,156	-567	0	0
7:00	2,024	2,156	-132	0	0
8:00	1,951	2,156	-205	0	0
9:00	1,578	2,156	-578	0	0
10:00	1,339	2,156	-817	0	0
11:00	1,309	2,156	-847	0	0
12:00	1,117	2,156	-1,039	0	0
13:00	1,314	2,156	-842	0	0
14:00	1,249	2,156	-907	0	0
15:00	1,175	2,156	-981	0	0
16:00	1,215	2,156	-941	0	0
17:00	1,470	2,156	-686	0	0
18:00	954	2,156	-1,202	0	0
19:00	808	2,156	-1,348	0	0
20:00	781	2,156	-1,375	0	0
21:00	623	2,156	-1,533	0	0
22:00	525	2,156	-1,631	0	0
23:00	415	2,156	-1,741	0	0
0:00	451	2,156	-1,705	0	0
1:00	187	2,156	-1,969	0	0
2:00	214	2,156	-1,942	0	0
3:00	264	2,156	-1,892	0	0
4:00	321	2,156	-1,835	0	0
5:00	573	2,156	-1,583	0	0
Total	20,498				

* The left table is in case of 1 lane (northbound), the right is in case of 1 lane (southbound)

Incidentally, the number of traffic lane is four (4) beside the bridge and six (6) at bridge and there is no queue as shown in Figure 18.2.2-3.

Figure 18.2.2-3 Pictures of Field Survey

As the result of these studies, two (2) lanes will be utilized during construction.

(3) Navigation Width

Because of the Lambingan bridge is at the curve point of the Pasig river, Navigation will be kept as present condition at daytime.

(4) Erection Method

Based on the outline design, the superstructure can be erected without a temporary bent in the navigation area. The existing piers will be able used as bent, the erection method was recommended as shown in Figure 18.2.2-4.

- End side girders : Crane erection method
- Arch block : Block erection method with water course temporary closed at night

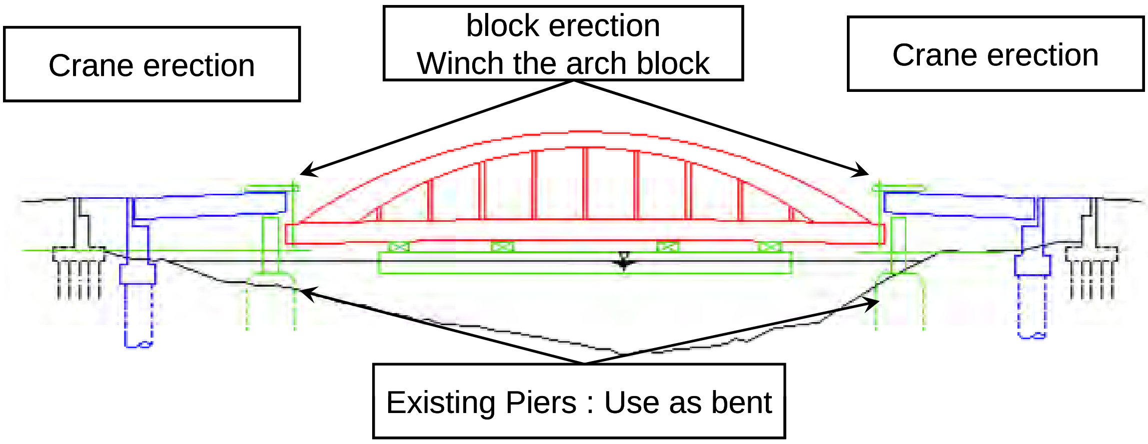


Figure 18.2.2-4 Erection Method of Lambingan Bridge

The stage erection was planned as shown in Figure 18.2.2-5.

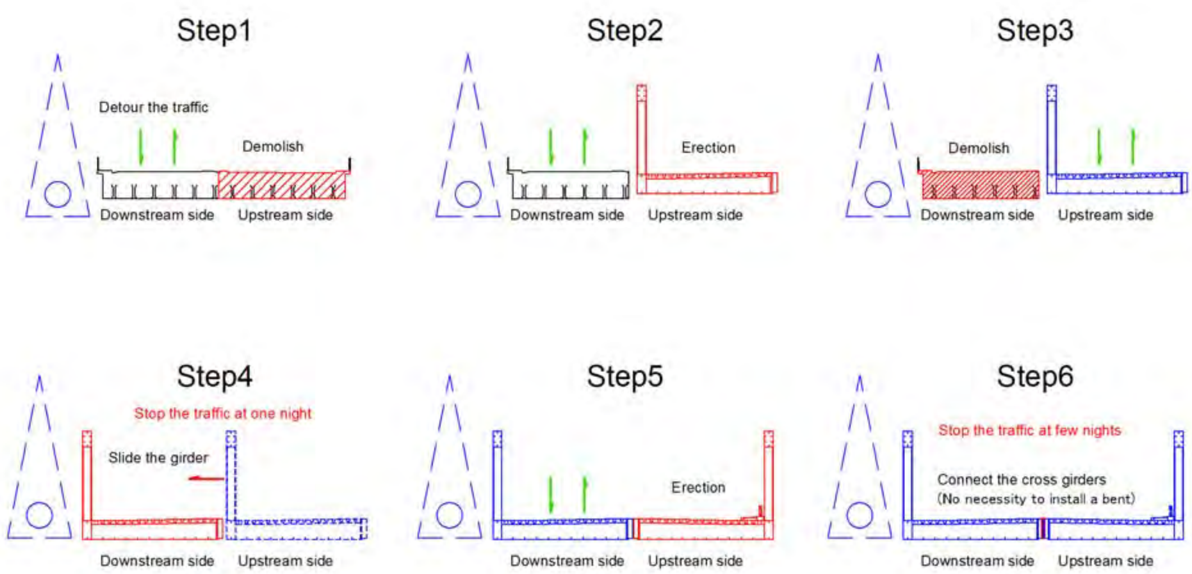


Figure 18.2.2-5 Erection steps of superstructure

- STEP 1
    - Traffic lane control from six (6) lanes to two (2) lanes at downstream side
    - Demolish the upstream side of existing girders

- STEP 2
    - Erect the new girder (downstream side) at upstream side

- STEP 3
    - Detour the traffic lane from downstream side to upstream side
    - Demolish the downstream side of existing girders

- STEP 4
    - Close the road at one night
    - Slide the erected girder from upstream side to downstream side
    - Open the road at downstream side

- STEP 5
    - Erect the upstream side new girder

- STEP 6
    - Close the road at a few nights
    - Connect the downstream side cross girders and upstream side cross girders
    - Open the road to traffic

(5) Construction Method under Limited Space

Result of erection method study, the new abutment should be constructed before the demolition of the existing girder. According to this order, the cast-in-place concrete pile should be constructed under the existing superstructure as shown in Figure 18.2.2-6.

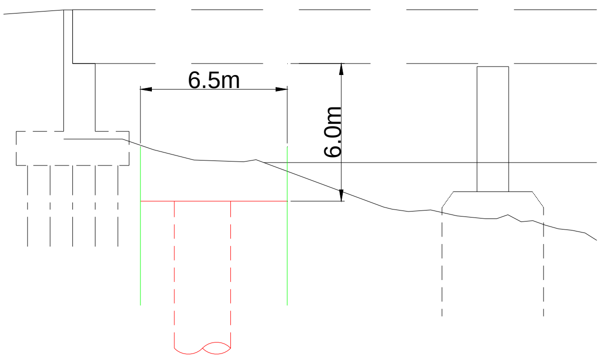


Figure 18.2.2-6 Construction Condition of Cast in Place Concrete Pile

The example of cast-in-place concrete pile installation method under limited space is as shown in Figure 18.2.2-7. This technology is possible to construct a D=2.5m pile.

(Equipment Height =1.8m, Equipment Weight = 4 ton, D = 0.8m~3.0m)

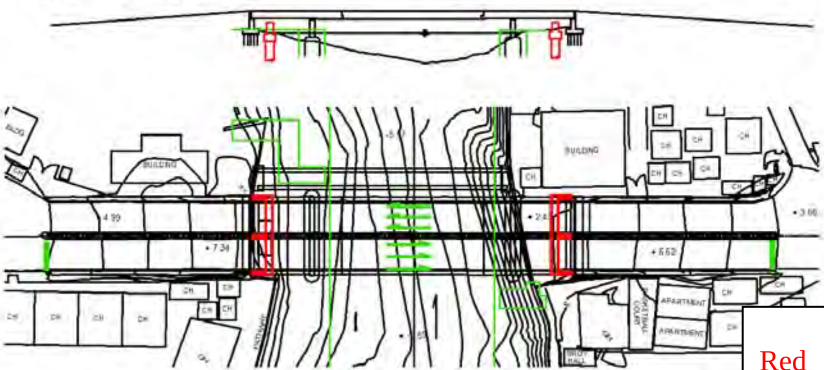
Figure 18.2.2-7 Example of Cast in Place Concrete Pile Method

(6) Construction Schedule

1) Construction Steps

The construction steps are shown in Figure 18.2.2-8~10.

STEP 0

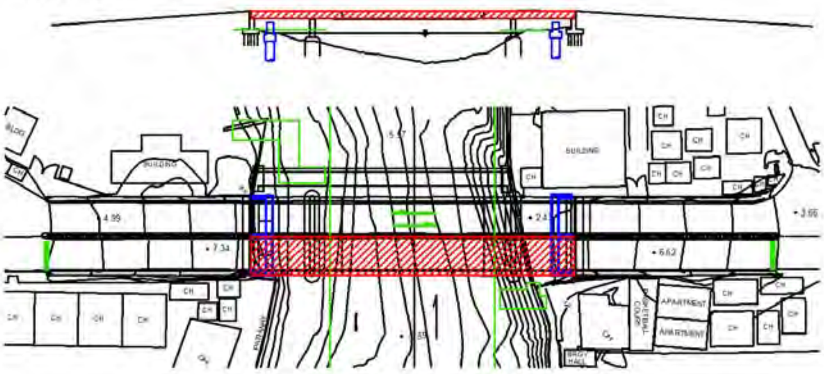


- Construct the new abutments

LEGEND

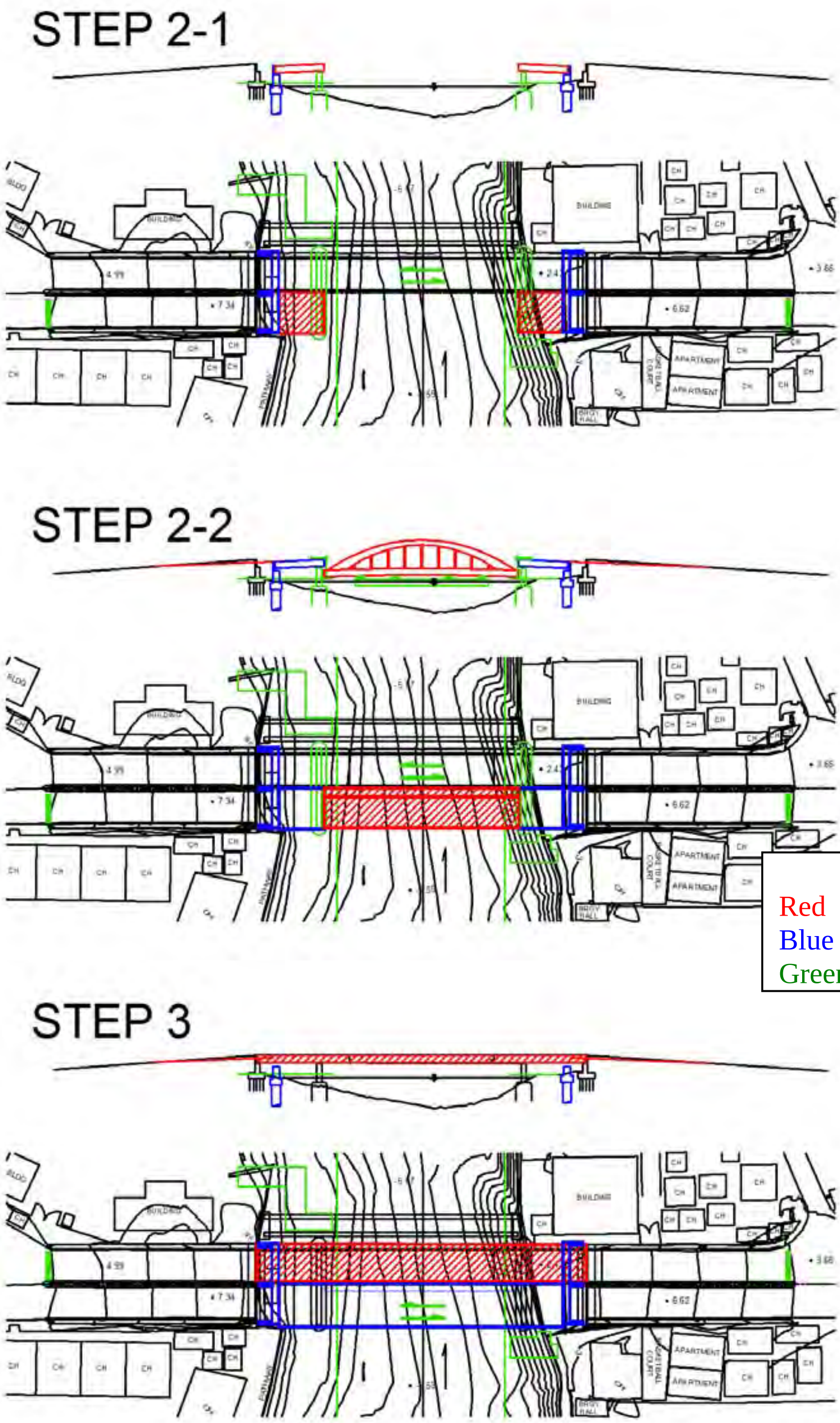
Red : Construction Work
Blue : Finished Work
Green : Temporary Work

STEP 1



- Detour the traffic from 6 lanes to 2 lanes (Downstream side)
- Demolish the existing Girders (Upstream side)

Figure 18.2.2-8 Construction Steps of Lambingan Bridge 1/3



- Erect the end side girders
  (Upstream side)
  (The downstream side
  girder will be used)

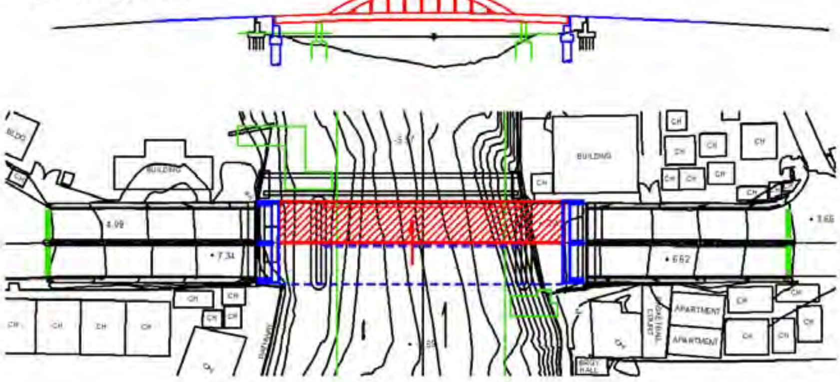
LEGEND

Red : Construction Work
Blue : Finished Work
Green : Temporary Work

- Detour the traffic to
  upstream side
- Demolish the existing
  superstructure
  (Downstream side)
- Improve the road
  (Downstream side)

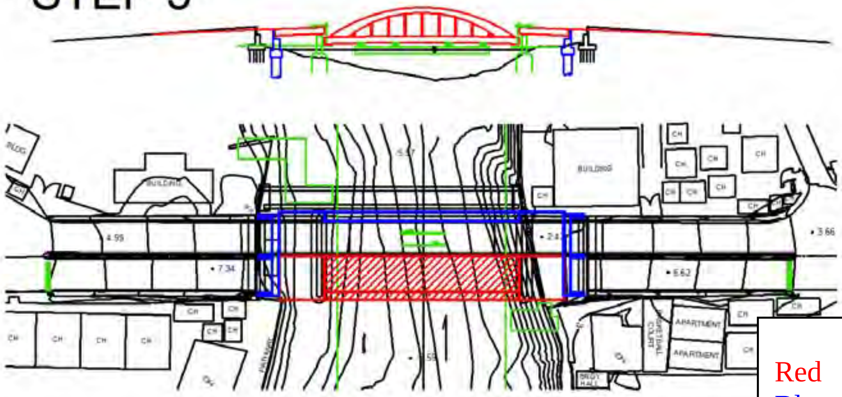
Figure 18.2.2-9 Construction Steps of Lambingan Bridge 2/3

STEP 4



- Stop the traffic at night
- Slide the erected girder from upstream side to downstream side

STEP 5

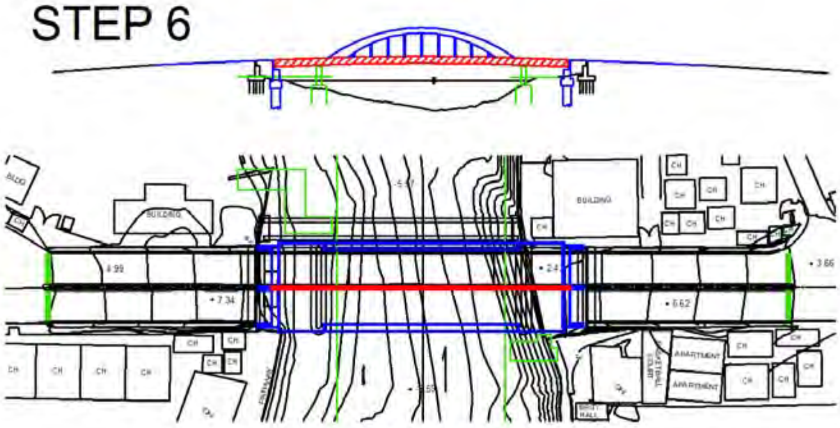


- Detour the traffic to downstream side
- Erect the girders (Upstream side)

LEGEND

Red : Construction Work
Blue : Finished Work
Green : Temporary Work

STEP 6



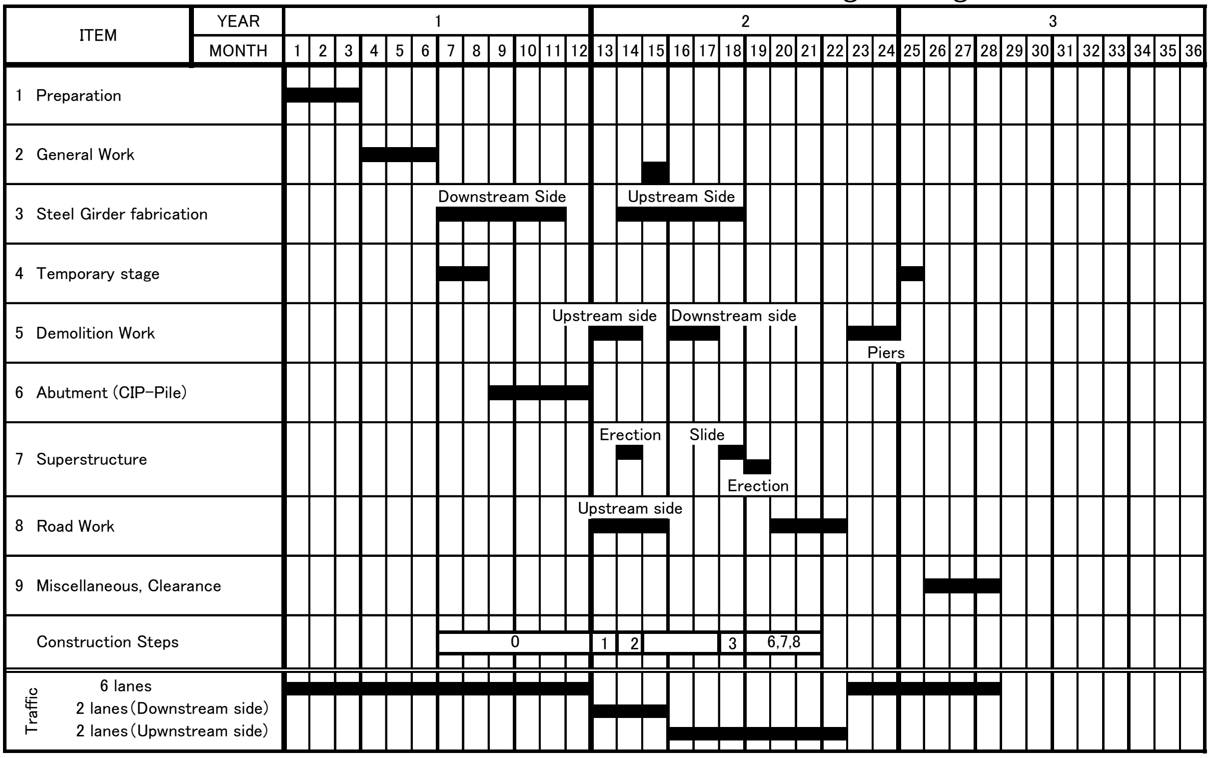
- Stop the traffic at night
- Connect the cross girders

Figure 18.2.2-10 Construction Steps of Lambingan Bridge 3/3

2) Construction Schedule

The construction schedule is shown in Table 18.2.2-2 based on the construction steps. Construction duration will be twenty eight (28) months and Detour duration will be ten (10) months.

Table 18.2.2-2 Construction Schedule of Lambingan Bridge



18.2.3 Construction Planning of Guadalupe Bridge

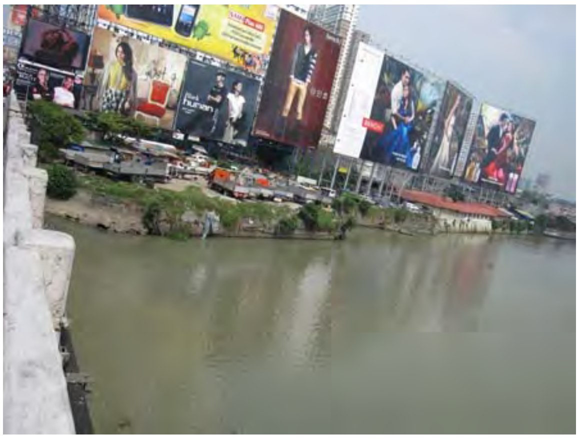
(1) Construction Site

The pictures of field survey are shown in Figure 18.2.3-1. The MMDA/Makati Park is not open to the public, but the house in the park was used as MMDA road maintenance base.

MMDA/Makati Park

MMDA/Makati Park

MMDA/Makati Park



Private Car Park

Figure 18.2.3-1 Pictures of Field Survey

The construction will need yards, such as assembling and erection of the new girders, fragmentation of the existing girders, material stock yard.

The construction yards were planned as shown in Figure 18.2.3-2.

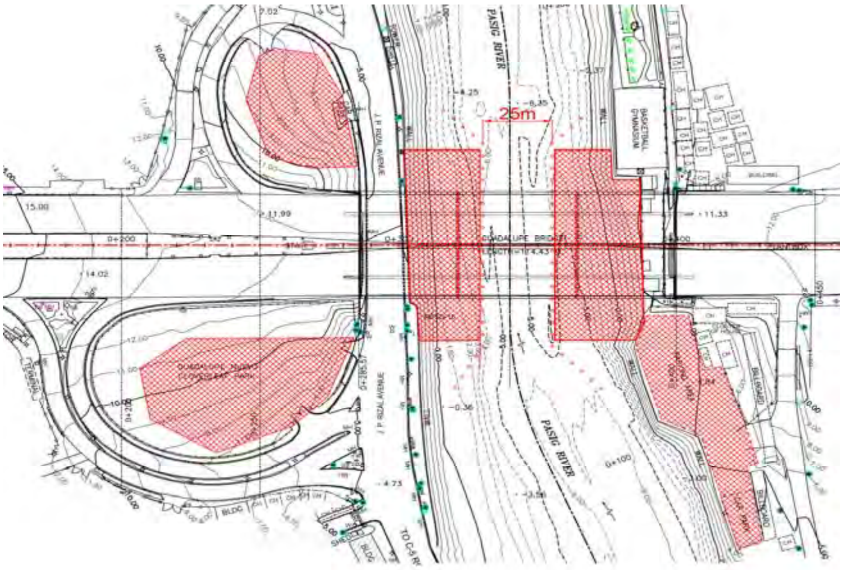
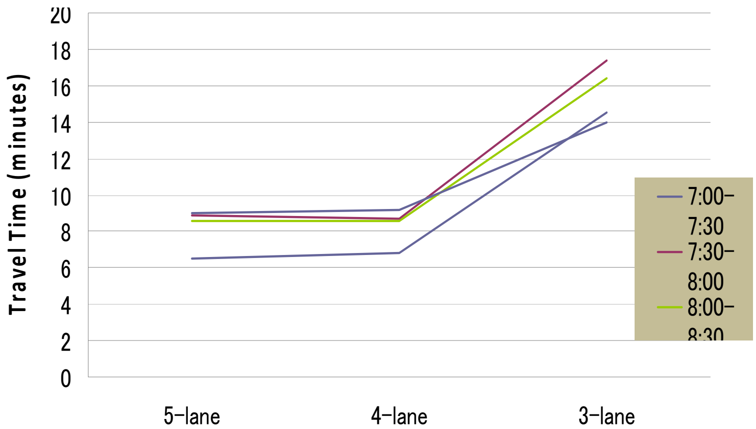


Figure 18.2.3-2 Construction Base and Site Location of the Guadalupe Bridge

(2) Traffic Detour Plan

Epifanio de los Santos Avenue (EDSA) is an arterial road that has 220,000 veh/day traffic volume and ten (10) lanes at Guadalupe bridge. During construction, influence of traffic congestion should be reduced.

The result of the traffic analysis, the travel time will not change from five (5) lanes to four (4) lanes at each direction as shown in Figure 18.2.3-3.



(From Ayala Ave. to Shaw Blvd. approx. 3.5km)

Figure 18.2.3-3 Travel Time in Case of Different Number of Traffic Lanes

The width of inner bridge has wide deck slab, and the deck slab will not be used for traffic lanes around the median. According to these information, EDSA detour will be planned as shown in Figure 18.2.3-5 and Figure 18.2.3-5.

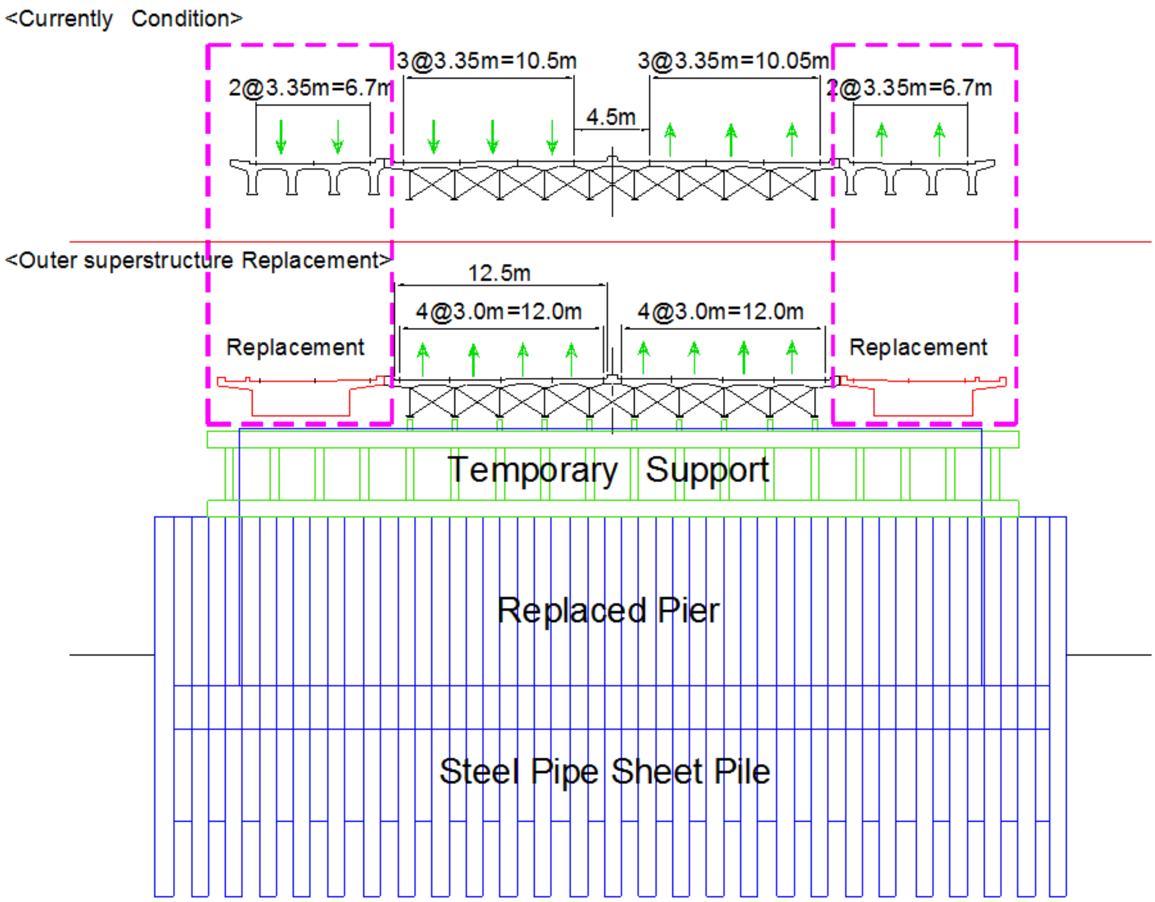
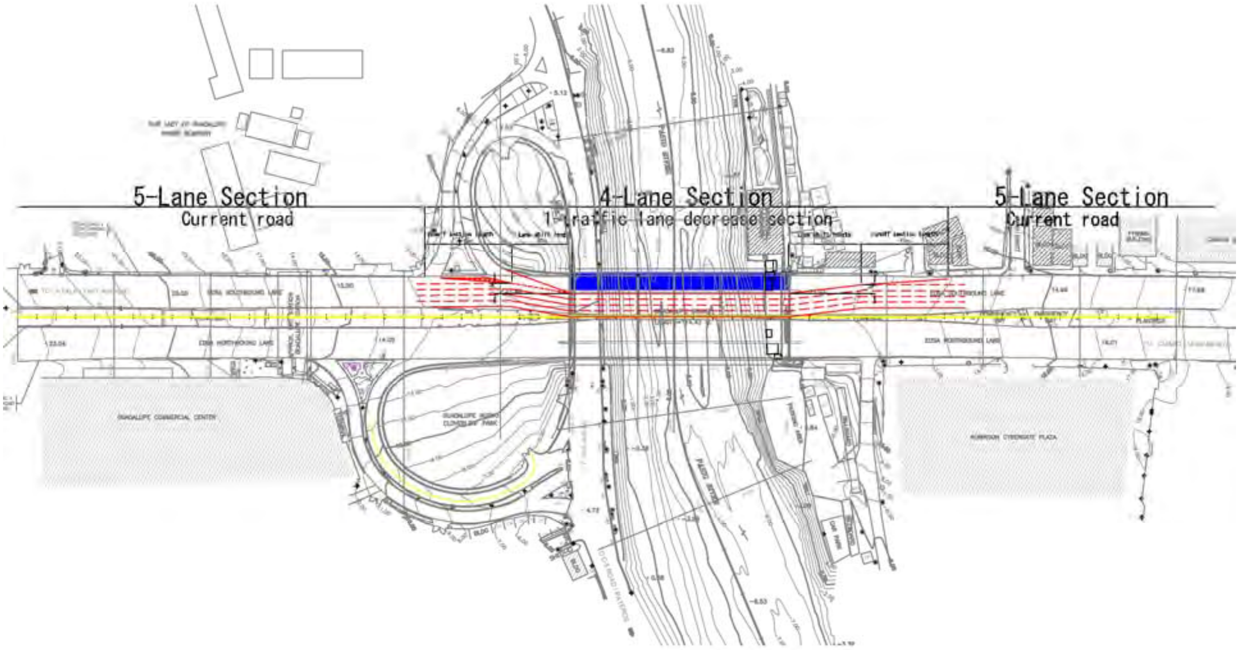
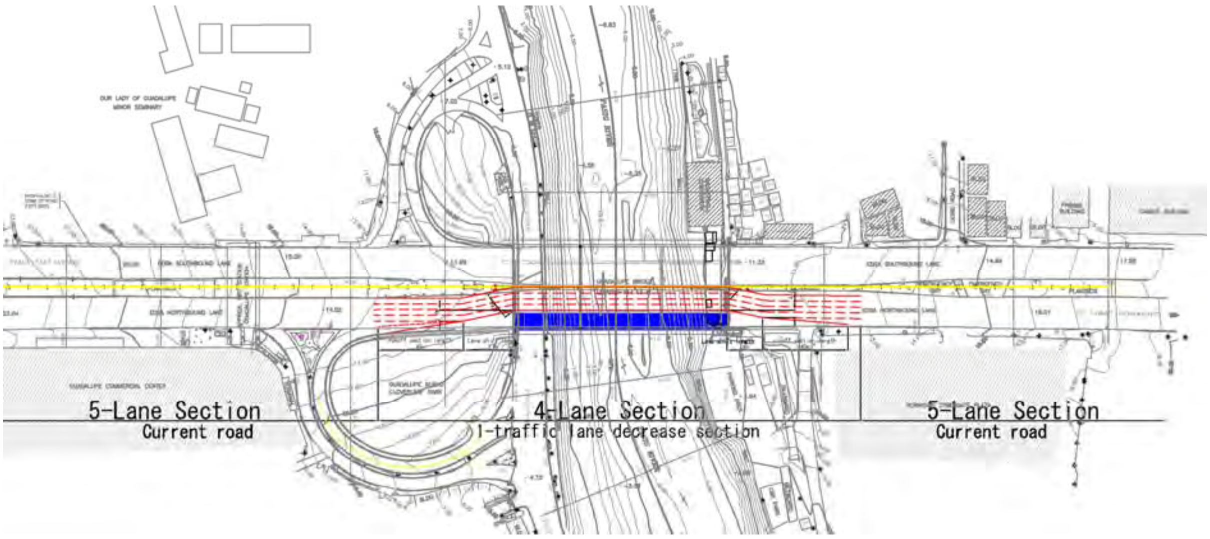


Figure 18.2.3-4 EDSA Detour Plan



Southbound



Northbound

Figure 18.2.3-5 EDSA Traffic Control Plan of Guadalupe Bridge

(3) Navigation Width

The navigation width at the existing bridges in Pasig River is as shown in Table 18.2.3-1. The temporary navigation width under construction was carried out 23m as the currently minimum navigation width and the straight line section of Pasig River.

Table 18.2.3-1 Navigation Width of Existing Bridges at the Pasig River

Existing Bridge	Navigation Width	
Delpan	47 m	
Jones	41 m	
McArthur	37 m	
LRT1	37 m	
Quzon	82 m	
Ayala (right)	55 m	
Ayala (Left)	56 m	
Nagtahan	54 m	
PASIG MWSS (water pipe)	23 m	
PNR (Railway)	23 m	Minimum
Pandacan	32 m	
Lambingan	56 m	
Makati-Mandaluyong	47 m	
Estrella-Pantaleon	52 m	
Guadalupe	38 m	
Sta Monica-Lawton	90 m	Under planning
C-5	43 m	

(4) Erection Method

The removal of existing girder and the erection of new girder will be recommended as block erection method with water course temporary closed only at night.

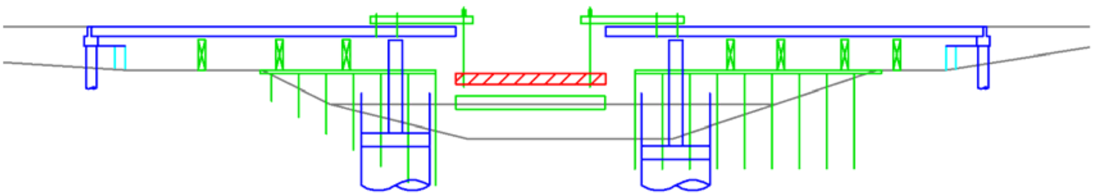


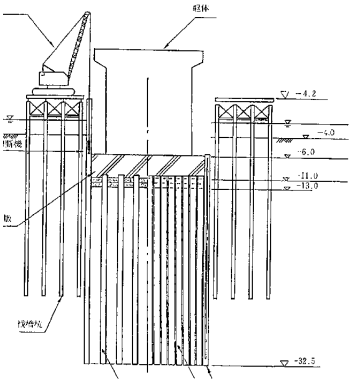
Figure 18.2.3-6 Erection Method of Center span of Guadalupe Bridge

(5) Construction Method under Limited Space

The comparison study of the construction method at the outline design, the several construction methods were introduced and recommended. The recommended construction method were studied focusing on construction planning, some of Japan special technology were introduced.

1) Pile Construction under Limited Space

The steel pile sheet pile (SPSP) was recommended by outline design under limited space. The SPSP can be constructed using the Press-in-Method under limited space as shown in Figure 18.2.3-7.



Ordinary Method Press in Method

Figure 18.2.3-7 Installation method of Steel Pipe Sheet Pile

2) Retrofitting under Temporary Support

The retrofitting method of inner pier was recommended for replacement under the existing superstructure. There was the achievement of replacement work under temporary support in Japan as shown in Figure 18.2.3-8. The Usage of this technology was recommended.

Shear Failure Temporary Support of superstructure

Figure 18.2.3-8 Pier Replacement Work with Temporary support

(6) Construction Schedule

1) Construction Steps of the Pier Replacement

The construction step of the pier replacement was planned as shown Figure 18.2.3-9.

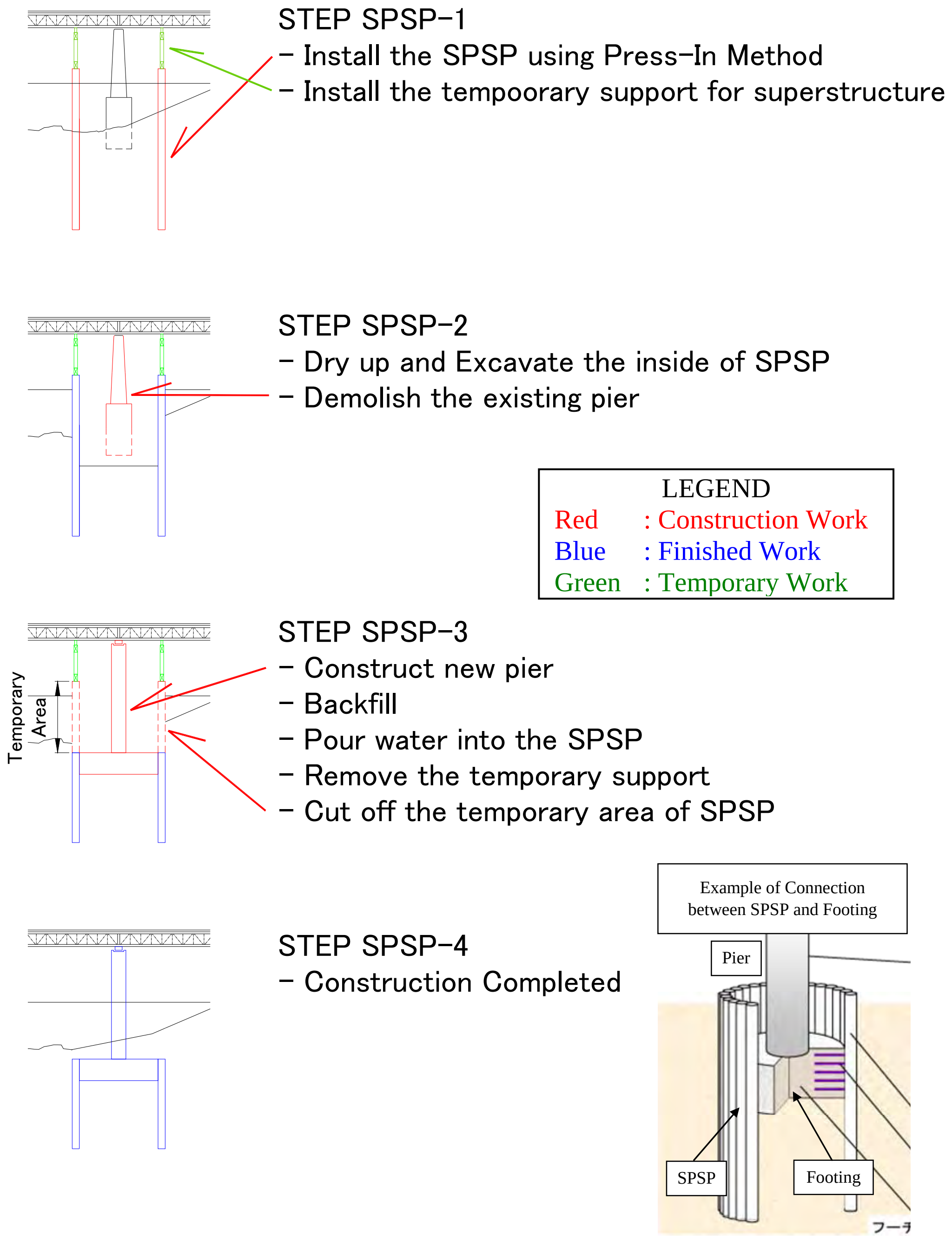


Figure 18.2.3-9 Construction Steps of Pier Replacement

The recommended methods used construction steps as follow;
- Steel Pipe Sheet Pile (SPSP) Foundation (Temporary area will be used as cofferdam)
- SPSP will be installed using Press-In Method
- The superstructure will be supported with the temporary support installed on SPSP

2) Construction Steps of the Outer Superstructure Replacement

According to the construction plan of pier replacement, the outer superstructure will be replaced after pier replacement, and EDSA traffic will be detour without increasing travel time. The construction step of the outer superstructure was planned as shown in Figure 18.2.3-10.

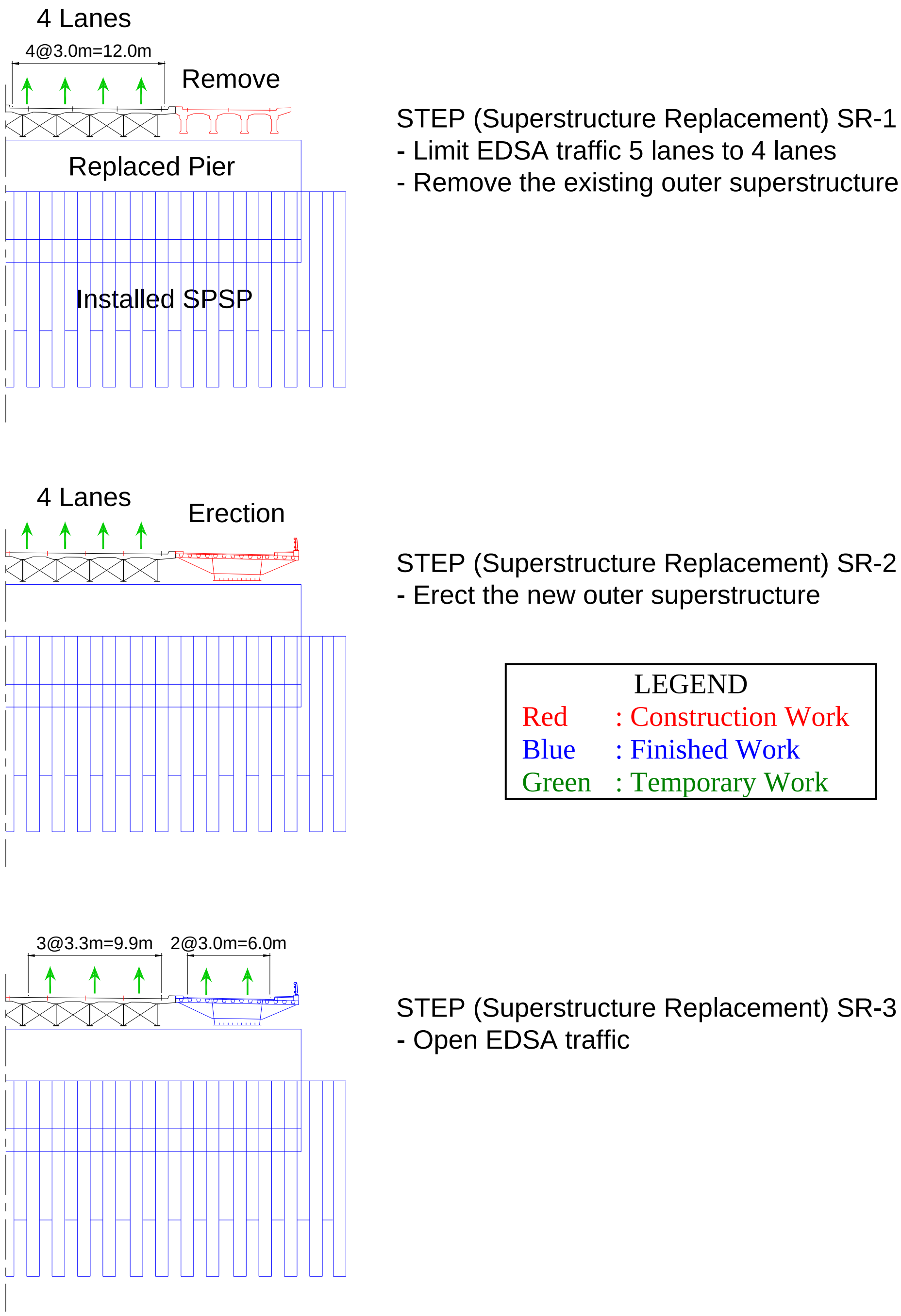


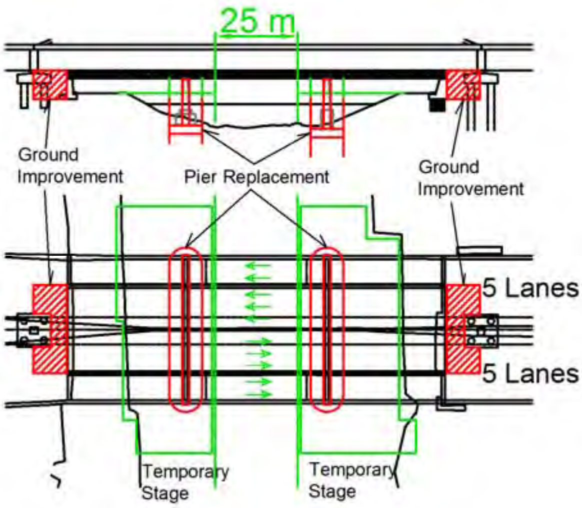
Figure 18.2.3-10 Construction Steps of Outer Superstructure

3) Construction Steps of the whole Construction

As a result of studies about construction planning, the entire construction steps was planned as shown in Figure 18.2.3-11

STEP 1

- Install the temporary stage in the river
- Press-in the steel pipe sheet pile (SPSP)
- Install the temporary support of superstructure on the SPSP
- Replace the Piers
- Improve ground for inner abutments



STEP 2

- Limit the EDSA traffic from 10 lanes to 8 lanes
- Construct the outer abutments
- Repalec the outer superstructures

LEGEND	
Red	: Construction Work
Blue	: Finished Work
Green	: Temporary Work

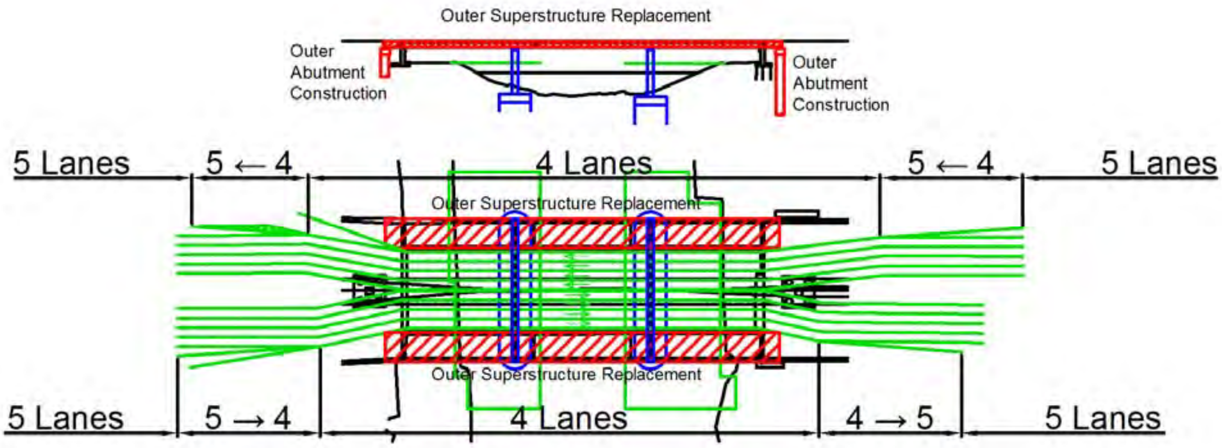
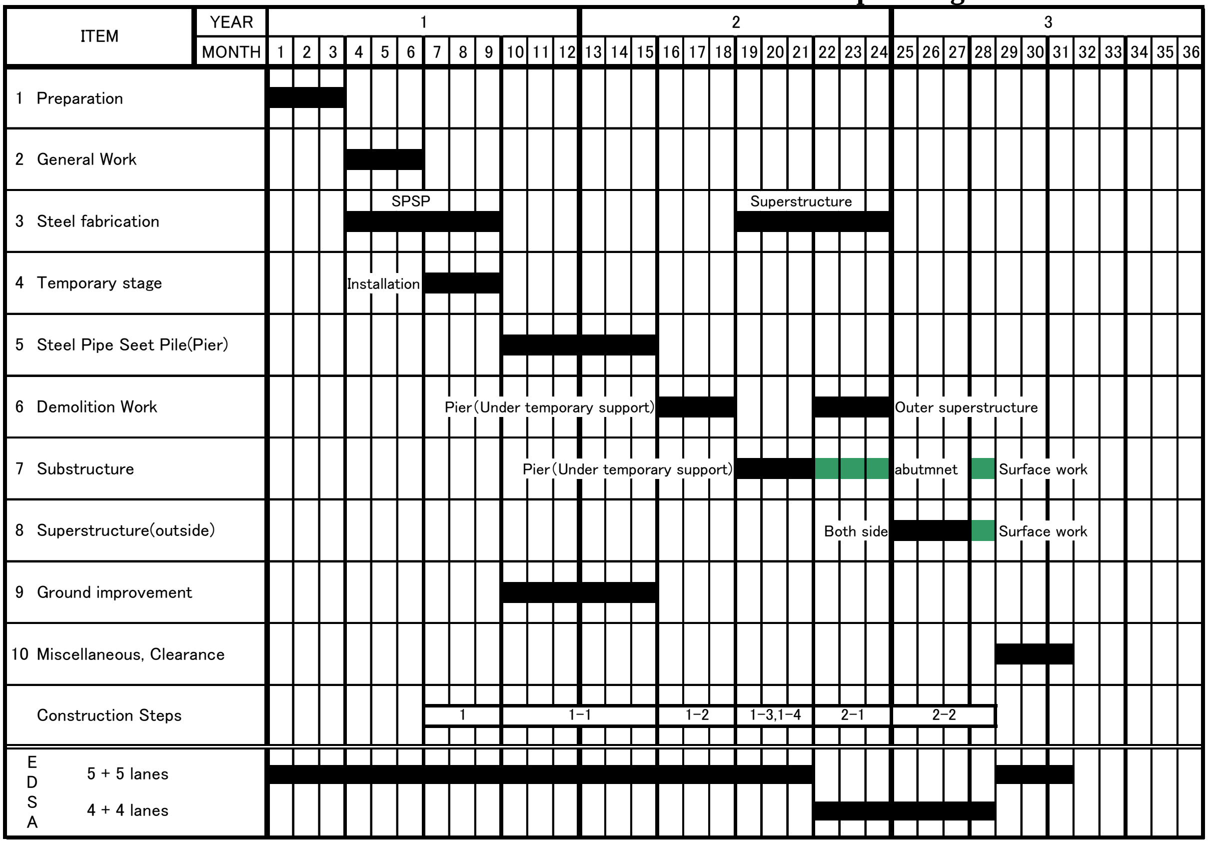


Figure 18.2.3-11 Construction Steps of the Guadalupe Bridge

4) Construction Schedule

The construction schedule is shown in Table 18.2.3-2 based on the construction steps. Construction duration will be thirty one (31) months and EDSA lane limit duration will be seven (7) months.

Table 18.2.3-2 Construction Schedule of Guadalupe Bridge



18.2.4 Construction Planning of 1st Mandaue Mactan Bridge

(1) Construction Site and Temporary Road

The concrete jacketing and the additional piles were recommended for the outline design of retrofit. The result of field survey is as shown figure 18.2.4-1~2, the construction will be done in the thirty meter (30m) right-of-way without land acquisition.

<Mactan Side> Side-Way

<Mactan Side> In the Right of WAY

<Mactan Side> Location of Sea Side

<Mactan Side> Location of the Piers in the Sea

Figure 18.2.4-1 Pictures of Field Survey <Mactan Side>

<Cebu Side> Cross Road

<Cebu Side>In the Right of WAY

<Cebu Side> Location of Sea Side

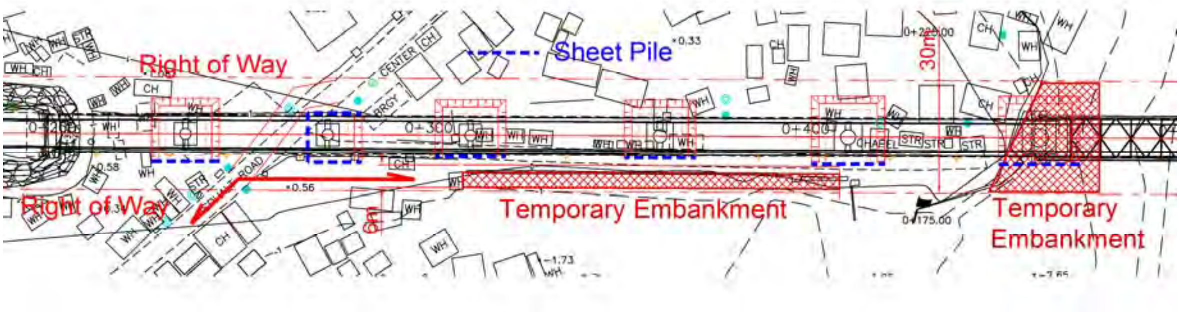
<Cebu Side> Location of the Pier in the Sea

<Cebu Side> Location of Right of Way

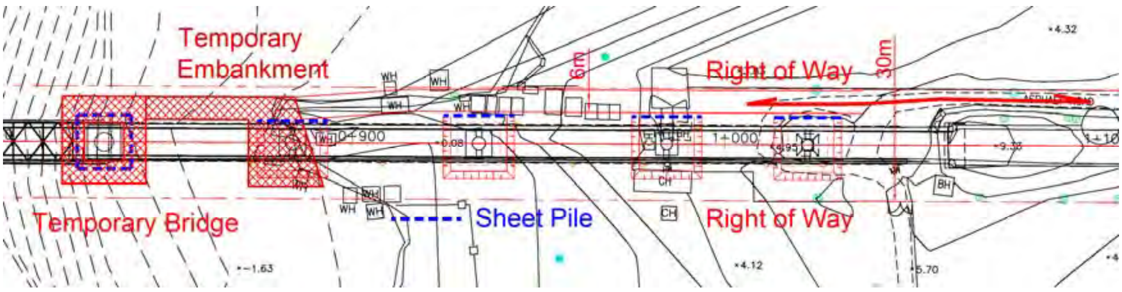
<Cebu Side> Location of the Piers in the Sea

Figure 18.2.4-2 Pictures of Field Survey <Cebu Side>

The retrofit construction will need heavy equipment such as pile and crane, the temporary road width was planned as six (6) meter in the Right –of-Way as shown Figure 18.2.4-3.



Cebu Side



Mactan Side

Figure 18.2.4-3 Basic Plan of Temporary Road of 1st Mandaue Mactan Bridge

(2) Navigation Width Control

The navigation width during construction will be the same as the current condition as in Figure 18.2.4-4.

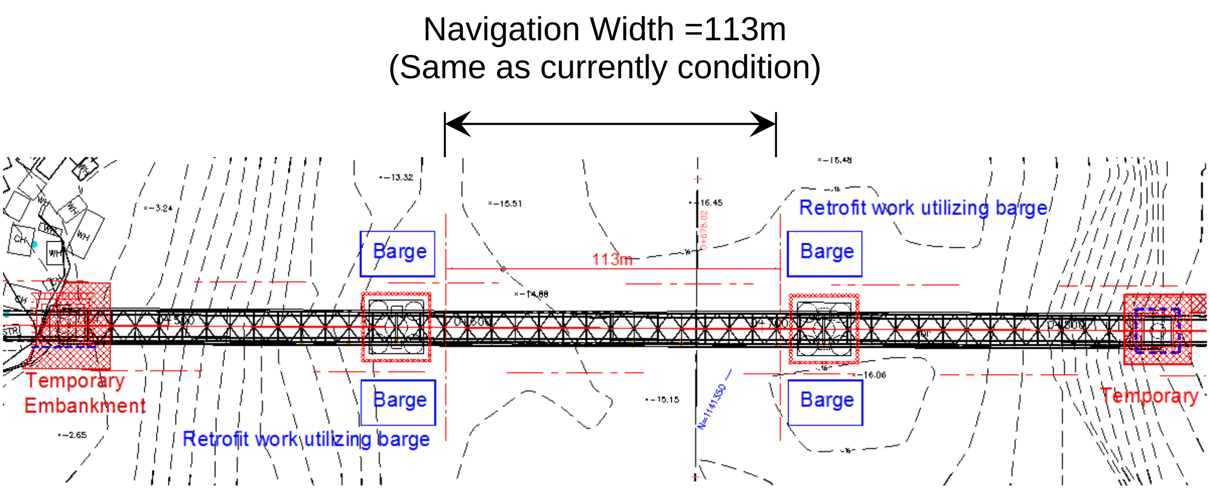


Figure 18.2.4-4 Navigation Width Control of 1st Mandaue Mactan Bridge

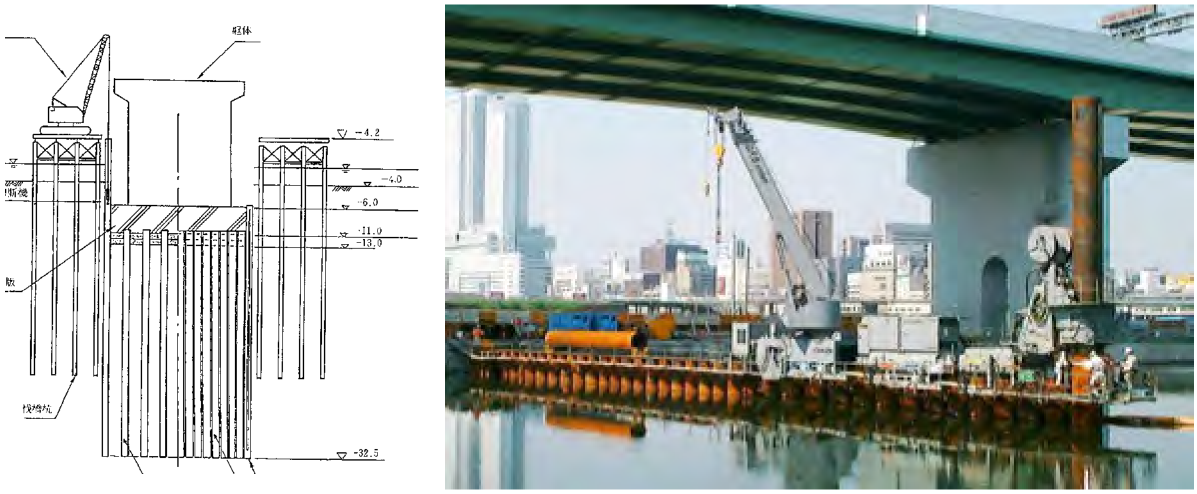
(3) Construction Method under Limited Space

The additional piles should be installed under the existing superstructure. There are some cast-in-place concrete pile methods as shown in Figure18.2.4-5. In case of additional piling, an All-Rotary-Casing-Method was recommended for neighboring construction of existing foundation.

All Rotary Casing (Minimum High = 5.5m) Reverse Circulation (Minimum High = 3.5m)

Figure 18.2.4-5 Construction Method of Cast in Place Concrete Pile under Limited Space

The steel pipe sheet pile (SPSP) for the piers in the sea, was recommended the Press in pile Method without temporary heavy-duty stage as shown in Figure 18.2.4-6.



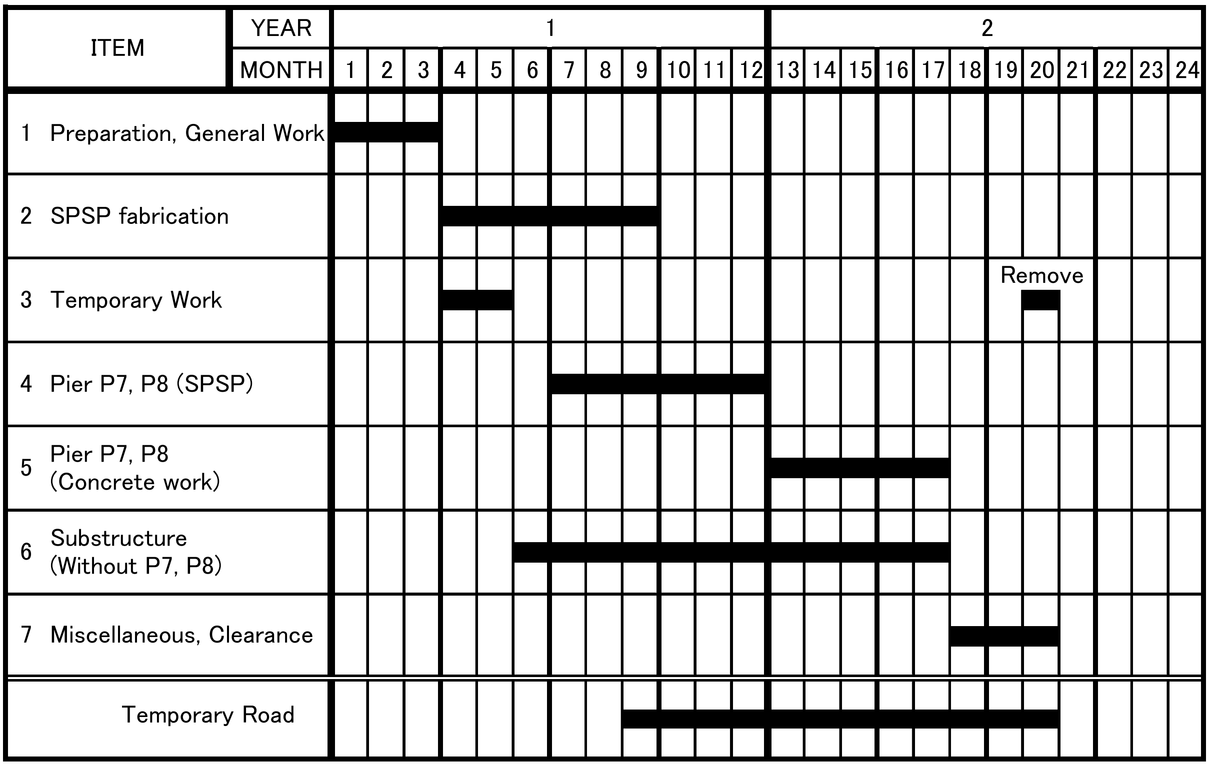
Ordinary Method Press in Method

Figure 18.2.4-6 Installation method of Steel Pipe Sheet Pile

(4) Construction Schedule

The construction schedule was planned as shown in Table 18.2.4-1. Construction duration will be twenty (20) months.

Table 18.2.4-1 Construction Schedule of 1st Mandaue Mactan Bridge



18.2.5 Construction Planning of Palanit Bridge

(1) Construction Site

The result of field survey as shown in Figure 18.2.5-1 and the detour road was planned in the thirty meter (30m) Right-of-Way

Basketball Court Ground Condition of The PC- Girder casting site

Figure 18.2.5-1 Pictures of Field Survey

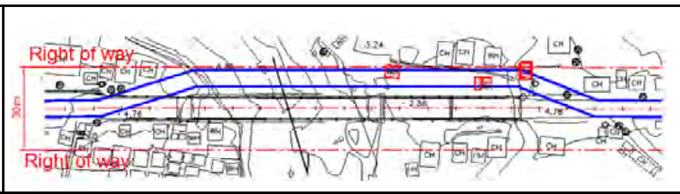
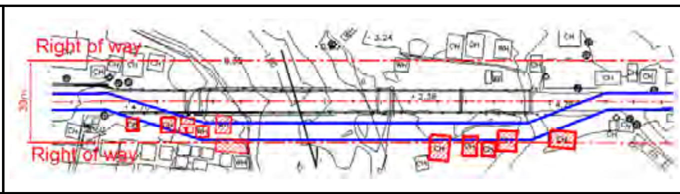
The construction site was planned as shown in Figure 18.2.5-2.
- The PC- Girder casting site: The existing plate-girder area
- The construction base: Temporary land acquisition (basketball court)

Figure 18.2.5-2 Site Location of Palanit Bridge

(2) Traffic Detour Plan and Temporary Road

As a result of the comparison study as shown Table 18.2.5-1, the detour to upstream side was recommended.

Table 18.2.5-1 Comparison Study of Detour Plan of Palanit Bridge

Alternative -1 : Upstream Side			
Plan			Recommended
No. of Affected Houses	- :4 houses (1 concrete house) should be removed	Positive	
Alternative -2 : Downstream Side			
Plan			
No. of Affected Houses	- 13 houses (7 concrete houses) should be removed	Negative	

(3) Erection Method

The recommended erection method was the crane erection from the detour road.

(4) Construction Schedule

The construction schedule was planned as shown in Table 18.2.5-2. Construction duration will be twenty (20) months.

Table 18.2.5-2 Construction Schedule of Palanit Bridges

ITEM	YEAR	1												2											
	MONTH	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
1 Preparation, General Work		■	■	■																					
2 Temporary Bridge & Embankment					Installation ■	■												Remove ■							
3 Demolition Work							■	■																	
4 Substructure									■	■	■	■													
5 Superstructure										Fabrication ■	■	■	■	■	■										
6 Embankment, Road Work																■	■	■							
7 Miscellaneous, Clearance																			■	■	■				
Detor (Temporary Road)										■	■	■	■	■	■	■	■	■							

18.2.6 Construction Planning of Mawo Bridge

(1) Construction Site

The result of field survey as shown in Figure 18.2.6-1, the detour road was planned in the thirty meter (30m) Right-of-Way

Park

Existing Side Way

Figure 18.2.6-1 Pictures of Field Survey

The construction site was planned as shown in Figure 18.2.6-2.
- Install the temporary embankment, bridge, stage in the river
- Install the sheet pile for construction and demolish piers
- Construction base: Temporary land acquisition (Park)

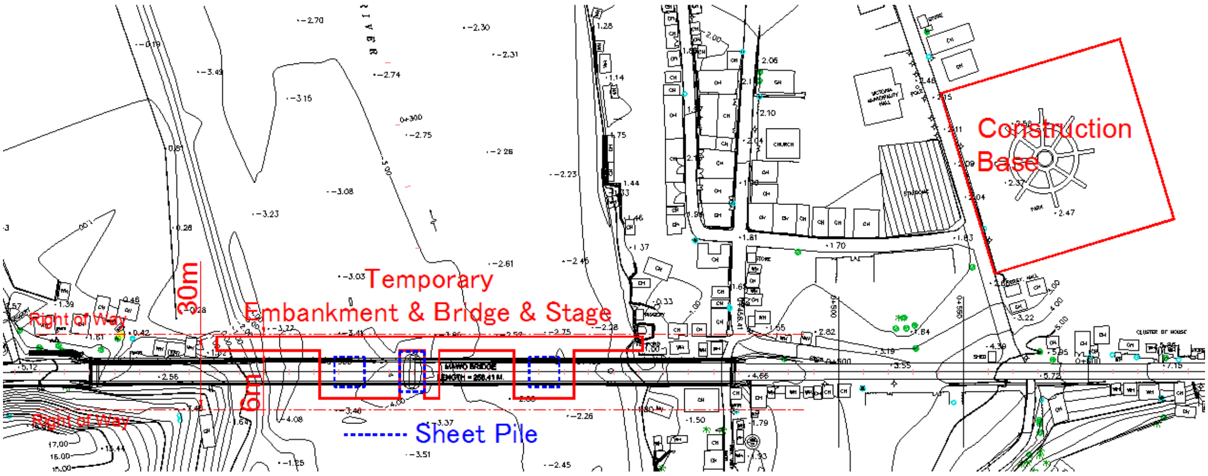


Figure 18.2.6-2 Site Location of Mawo Bridge

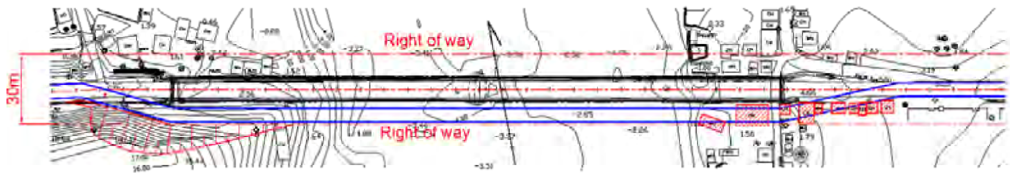
The transportation of heavy equipment and materials will utilize Mawo port as shown in Figure 18.2.6-3. Moreover, same for Palanit bridge, where this bridge is near.

Figure 18.2.6-3 Picture of Mawo Port (At Right side of Rivermouth)

(2) Traffic Detour Plan and Temporary Road

As a result of the comparison study as shown Table 18.2.6-1, the detour to downstream side was recommended.

Table 18.2.6-1 Comparison Study of Detour Plan of Mawo Bridge

Alternative -1 : Detour to Upstream Side			
Outline			
No. of Affected Houses	- 10 houses (6 concrete houses)	Negative	
Alternative -2 : Detour to Downstream Side			
Outline			Recommended
No. of Affected Houses	- 7 houses (1 concrete house)	Positive	

(3) Construction Method under Limited Space

The selected type of superstructure will be erected with cantilever and erection girder as shown in Figure 18.2.6-4.

(Erection Girder and Wagen)

(Low profile Wagen)

Figure 18.2.6-4 Construction Situation of PC Fin Back Bridge

There were some wagen of 1.5m clearance from bottom of girder to wagen floor in Japan. The use of wagen will be applicable under a limited vertical clearance.

(4) Construction Schedule

The construction schedule was planned as shown in Table 18.2.6-2. Construction duration will be twenty five (25) months.

Table 18.2.6-2 Construction Schedule of Mawo Bridge

ITEM	YEAR	1												2											
	MONTH	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
1 Preparation, General Work																									
2 Temporary Bridge & Stage					Installation																		Remove		
3 Demolition Work																									
4 Substructure																									
5 Superstructure																									
6 Embankment, Road Work																									
7 Miscellaneous, Clearance																									
Detour(Temporary Road)																									

* 2 sets of erection girder and 4 sets of wagen

18.2.7 Construction Planning of Lilo-an Bridge

(1) Construction Site

The result of field survey is shown in figure 18.2.7-1, the retrofit work will be done in the thirty meter (30m) Right-of-Way. On the other hand, retrofit work need heavy equipment like pile and crane, the construction area can access from existing road as shown Figure 18.2.7-2, and the transportation of heavy equipment and materials will utilize Lilo-an Ferry Port as shown in Figure 18.2.7-3.

Figure 18.2.7-1 Pictures of Field Survey

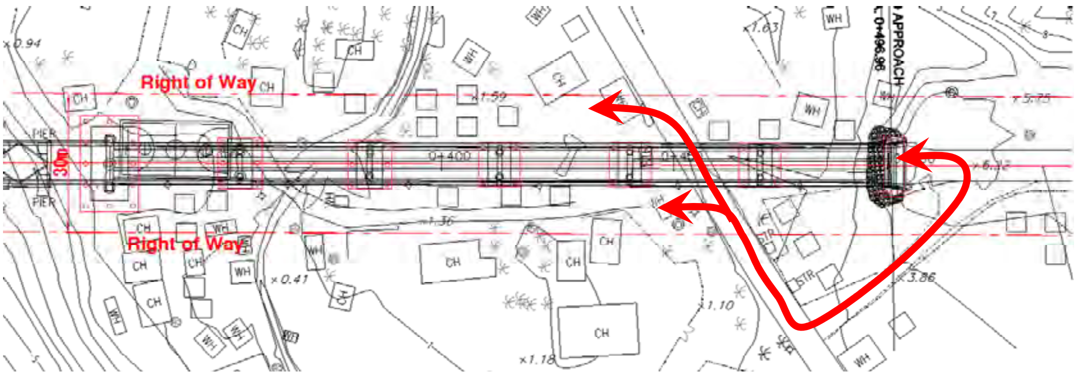


Figure 18.2.7-2 Site Location of the Lilo-an Bridge

Figure 18.2.7-3 Pictures of Lilo-an Port

(2) Construction Method under Limited Space

The additional piles should be constructed under the existing superstructure. There are some cast-in-place concrete pile methods in as shown in Figure18.2.7-4. In case of additional piling, an All-Rotary-Casing-Method was recommended for neighboring construction of existing foundation.

All Rotary Casing (Minimum High = 5.5m)　　Reverse Circulation (Minimum High = 3.5m)

Figure 18.2.7-4 Construction Method of Cast in Place Concrete Pile under Limited Space

(3) Construction Schedule

The construction schedule was planned as shown in Table 18.2.7-1. Construction duration will be fifteen (15) months.

Table 18.2.7-1 Construction Schedule of Lilo-an Bridge

ITEM	YEAR	1												2											
	MONTH	1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17	18	19	20	21	22	23	24
1 Preparation, General Work																									
2 Foundation																									
3 Substructure																									
4 Miscellaneous, Clearance																									

18.2.8 Construction Planning of Wawa Bridge

(1) Construction Site and Temporary Stage

The result of field survey is shown in figure 18.2.8-1, the construction will be done in the sixty meter (60m) right-of-way and river floor.

Figure 18.2.8-1 Pictures of Field Survey

The construction site was planned as shown in Figure 18.2.8-2.

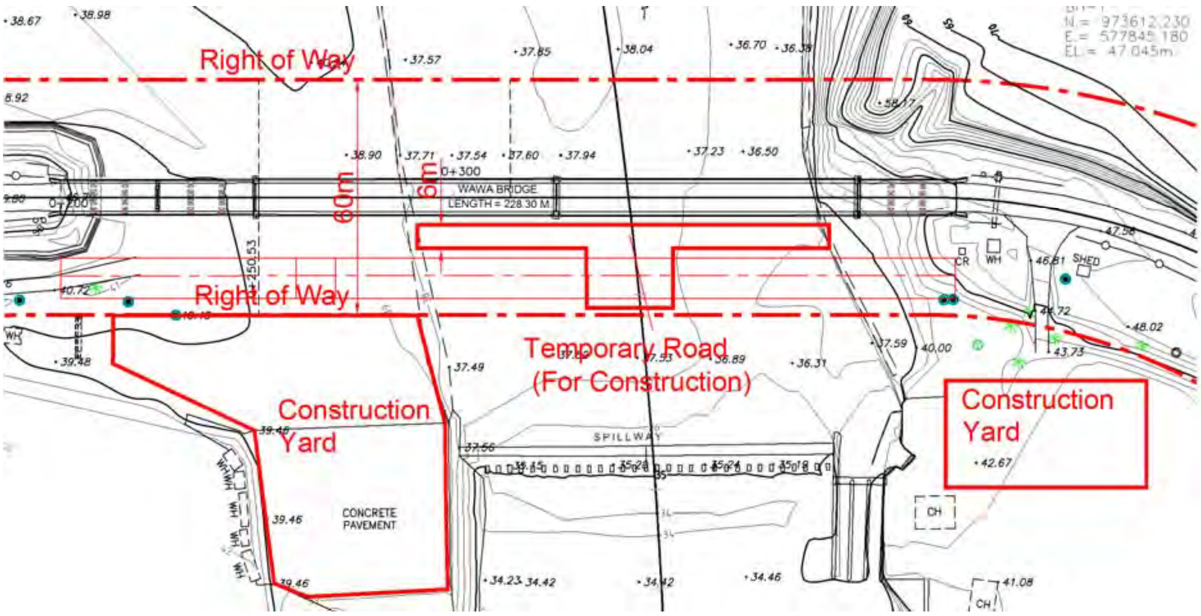


Figure 18.2.8-2 Site Location of the Wawa Bridge

(2) Construction Method

The 2nd Magsaysay bridge near Wawa bridge constructed to utilize some new technologies as follow;

- Steel Pipe Sheet Pile Foundation (Guadalupe Bridge, 1st Mandaue Mactan Bridge)
- Pc Precast Deck slab (Wawa Bridge)
- Anti-corrosion Steel (Wawa Bridge)

These technologies will be used in this project.

PC-Precast Deck slab

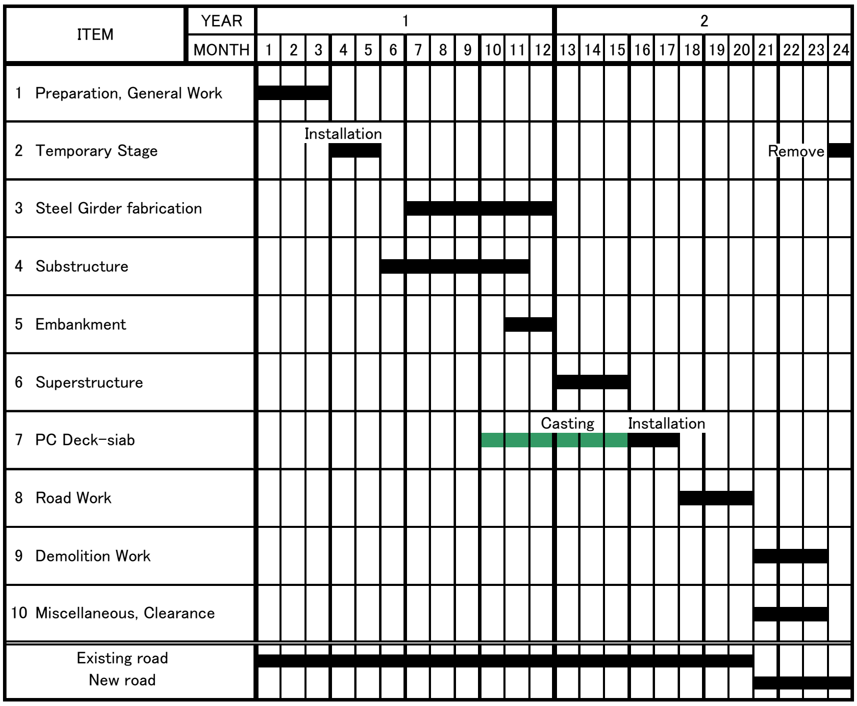
Anti-corrosion Steel

Figure 18.2.8-3 Pictures of Field Survey (2nd Magsaysay)

(3) Construction Schedule

The construction schedule was planned as shown in Table 18.2.8-1. Construction duration will be twenty four (24) months.

Table 18.2.8-1 Construction Schedule of Wawa Bridge



18.2.9 Construction Schedule of the Project

The construction schedule of the project was planned as shown in Table 18.2.9-1. Construction duration will be thirty two (32) months.

Table 18.2.9-1 Construction Schedule of the Project

Bridge	Construct Duration	1st Year				2nd Year				3rd Year			
		3	6	9	12	15	18	21	24	27	30	33	36
Lambingan	28 Months												
Guadalupe	31 Months												
1st Mandauc Mactan	20 Months												
Palanit	20 Months												
Mawo	24 Months												
Liloan	15 Months												
Wawa	24 Months												

This schedule was conducted as follows;
- Construction duration of the Project: Guadalupe 31 months
- Detour of Package B (Overpass of Pasig River) <Blue>: Lambingan -> Guadalupe

18.3 Cost Estimate

The project cost consisting of construction cost, land acquisition cost, compensation cost, consultancy service cost, administrative cost and tax were estimated.

18.3.1 General

(1) Basic Condition of Cost Estimate

1) Price Level

The cost estimates are updated on the price level as of August 2013.

2) Exchange Rate

Exchange rates are referred to the monthly average in August 2013 of Central Bank of the Philippines.

- 1.0 PHP = 2.222 JPY
- 1.0 USD = 97.229 JPY = 43.756 PHP.

3) Currency for Cost Estimation

The project cost component shall consist of foreign currency and local currency portions. Philippine Peso shall be used for both portions. The classifications of local and foreign portions are as given below.

1) Local Currency Portion
   - Labor Costs without foreign technical service
   - Cost of construction material and equipment lease locally procured
   - Administrative cost
   - Land acquisition and compensation cost
   - Tax
2) Foreign Currency Portion
   - Cost of construction materials, equipment and technical services procured from foreign countries

4) Reference Guidelines/Manuals

The cost estimates are referred to the following guidelines/manuals indicated below:

1) DPWH Department Order No. 72, Series of 2012 (Amendment to D.O. 29 Series of 2011 Re: Revised Guidelines on the Preparation of Approved Budget for the Contract)
2) DPWH Department Order No. 71, Series of 2012 (Guidelines for the Establishment of Construction Materials Price, Standard Labor and Equipment Rental Rates Data Base)

(2) Methodology of Cost Estimate

Costs for construction works are essentially estimated on the unit price basis. The construction cost consists of direct cost and indirect costs. The direct cost consists of equipment, material and labor costs. Indirect cost includes overhead expenses, preparation cost, administrative cost, contingencies, miscellaneous expenses, contractor's profit margin and tax.
The quantity of each direct cost items were calculated from the result of the out-line design.

1) Direct Cost

The unit prices of typical construction items were estimated by Approved Budget for the Contract (ABC). The items with imported material, equipment and technical service were estimated by quotation. The unit price of construction items were estimated shown in Appendix 4.

Result of the construction projects in the Philippines is shown in Table 18.3.1-2, the general work was estimated as 10% for package B and 5% for package C of the other estimated direct cost. The general work is as follow;

- Facilities for the engineer
- Facilities for the contractor and other
- Site Preliminaries

Table 18.3.1-1 General Work Ratio of the Past Project

Project		Ground Total	General work	Ratio
Manila	C-2/R-7 Interchange Project Under The Metro Manila Interchange Construction Project	685 M Php	62 M Php	9%
Suburb	Sixth Road Project ADB Loan No,1473-PHI, JEXIM United Loan (Mindanao Bridge Replace/Retrofit) 2005	173 M Php	5 M Php	3%
	Arterial Road Links Development Project, Phase III (San Juanico Bridge) 2005	965 M Php	35 M Php.	4%

2) Indirect Cost

a) Overhead Cost

The overhead cost was estimated by Approved Budget for the Contract (ABC).

Table 18.3.1-2 Overhead Ratio

ESTIMATED DIRECT COST (EDC)	INDIRECT COST % FOR OCM AND PROFIT		TOTAL INDIRECT COST % FOR OCM AND PROFIT
	OCM (% OF EDC)	PROFIT (% OF EDC)	
Up to P5Million	12	10	22
Above P5M up to P50M	9	8	17
Above P50M up to P150M	7	8	15
Above P150M	6	8	14

b) Consultancy Service Cost

Consultancy service cost was required for the detailed design including tender assistance and construction supervision.

c) Physical Contingency

Physical Contingency was estimated to be 5% of direct cost.

d) Administrative Cost

The administrative cost was estimated to be 3.0% of direct cost.

e) Land Acquisition

The land acquisition cost was estimated based on the zonal valuation by Bureau of Internal Revenue, or BIR

f) Compensation

The compensation of house relocation was estimated with a financial assistance of 25,000 Php/House.

g) Tax

VAT component shall be 12% of the sum of Estimated Direct Cost, Land acquisition cost, Compensation cost, Engineering service cost, Contingency cost and Administrative cost.

Customs duty will be 3% of the imported steel material price.

- Imported materials are steel sections and steel parts of bridge
- ASEAN Harmonized Tariff Nomenclature (AHTN) Book 2012
- HDG.No.73.08 Structures (excluding prefabricated buildings of heading 94.06) and parts of structures (for example, bridges and bridge-sections, lock gates, towers, lattice masts, roofs, roofing frame-works, doors and windows and their frames and thresholds for doors, shutters, balustrades, pillars and columns), of iron or steel; plates, rods, angles, shapes, sections, tubes and the like, prepared for use in structures, of iron or steel

3) Quantity

The quantity is a result of the outline design. Summary of quantity is shown in Appendix 4.

CHAPTER 19 TRAFFIC ANALYSIS AND ECONOMIC EVALUATION

19.1 Traffic Analysis

This chapter describes the traffic analysis and economic evaluation for the seven (7) bridges project. The purpose of the traffic analysis is to estimate traffic congestion during bridge improvement, and to prepare the base traffic data for benefit estimation of economic evaluation.
The procedure for the traffic analysis and economic evaluation is illustrated in Figure 19.1-1. The detailed procedure is described in the adequate section.

1. Traffic Analysis (19.2~19.4)

(a) Package B (19.2)
- ● Lambingan Bridge
- ● Guadalupe Bridge
- Present Traffic Assignment
- Future Traffic Assignment

↓

- To estimate Traffic Condition during construction

※ Using Traffic Microsimulator in Guadalupe Bridge Improvement (19.3)

- To evaluate designable traffic restriction.

(b) Package C (19.4)
- ● 1st Mandaue – Mactan Bridge
- ● Palanit Bridge
- ● Mawo Bridge
- ● Liloan Bridge
- ● Wawa Bridge
- Present Traffic Volume
- Growth Rate

↓

- Future Traffic Volume

↓

- To estimate Traffic Condition during construction.

2. Economic Evaluation (19.5)

(a) Package B
- Economic Cost
- Benefit (VOC, TTC)
  - ● Traffic Assignment
    (Do Case / Do Nothing Case)

(b) Package C
- Economic Cost
- Benefit (VOC, TTC)
  - ● Simplified Impact Analysis
    (Do Case / Do Nothing Case)

Figure 19.1-1 Procedure for Traffic Analysis and Economic Evaluation

Though Lambingan Bridge and Guadalupe Bridge will be reduced of the number of lanes during construction, the traffic will not be affected for the other five (5) bridges during construction due to temporally bridge or retrofits substructure only, shown in Table 19.1-1.

Table 19.1-1 Basic Traffic Restriction during Construction

No.	Bridge	Improvement	Present No. of lane	No. of lane during Construction	Remarks
1	Lambingan	Replacement	6(3+3)	2(1+1)	
2	Guadalupe	Replacement only outer bridge	10(5+5)	9(5+4)	
3	1st Mandaue-Mactan	Retrofit only substructure	2(1+1)	2(1+1)	Traffic will not be affected during construction because of preparation for temporary bridge
4	Palanit	Replacement	2(1+1)	2(1+1)	
5	Mawo	Replacement	2(1+1)	2(1+1)	
6	Liloan	Retrofit	2(1+1)	2(1+1)	
7	Wawa	Replacement	2(1+1)	2(1+1)	

In this study, how many lanes can be reduced without creating traffic congestion at each bridge will be verified.

19.2 Traffic Analysis of Package B

19.2.1 Traffic Assignment

In Metro Manila, traffic assignment to road network with bridge plan is made using JICA STRADA highway type assignment model.
Procedure for the present traffic assignment and future traffic assignment is presented in Figure 19.2.1-1. After obtaining the accuracy of present traffic assignment, future traffic assign is estimated. In estimating the future traffic volume, future road network was taken into account.

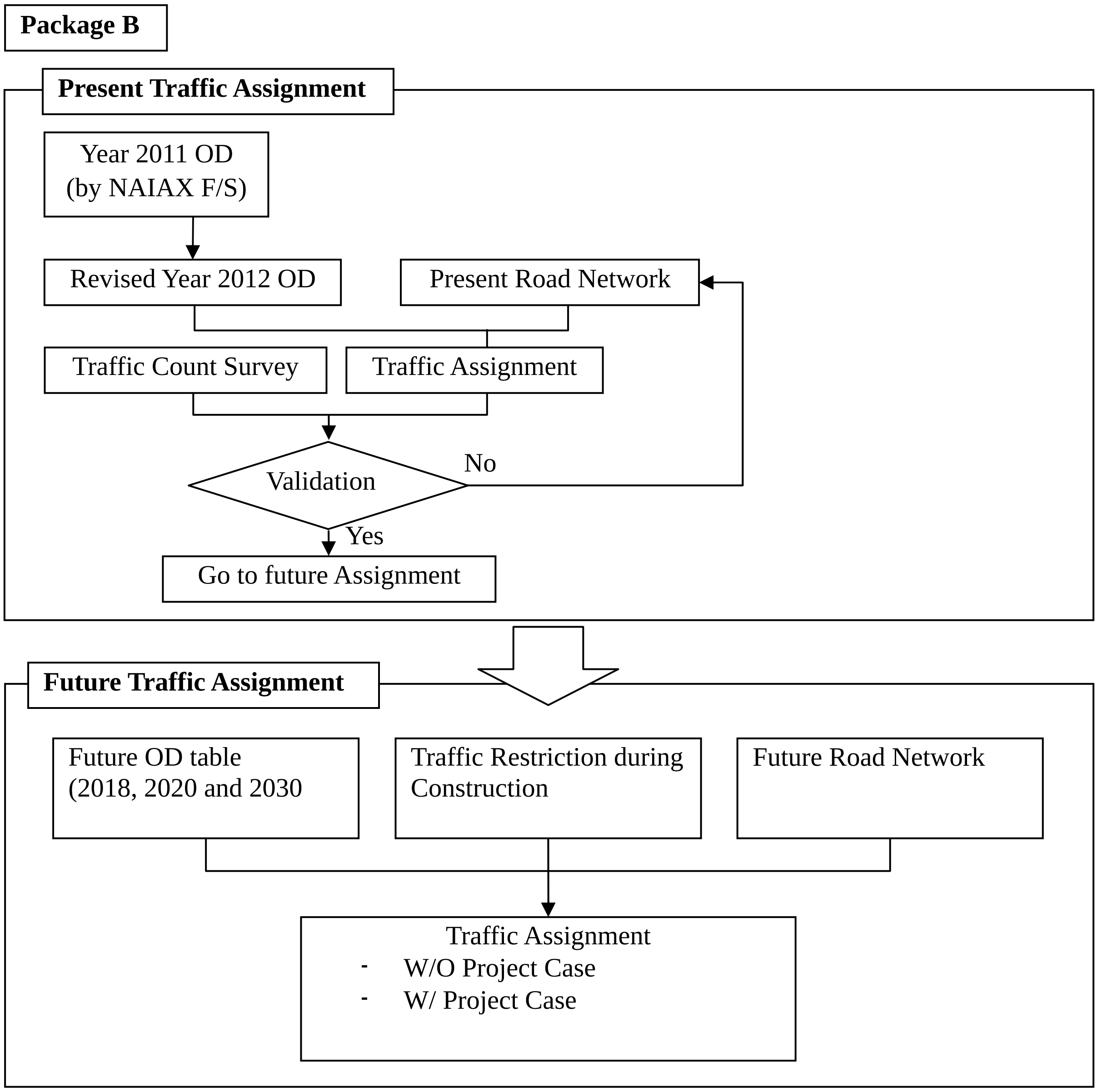


Figure 19.2.1-1 Procedure for Preparation of Present and Future Assignment

(1) Traffic Model Validation

The procedure of model validation entails two steps: first, the current OD matrix is assigned on an existing network. Second, the assigned traffic volume is compared with the result of the traffic count surveys at each corresponding location. This verification aims to check the accuracy of both the current OD matrix and an existing network model representing the existing transport situation.

Table 19.2.1-1 presents traffic volume generated from traffic assignment and observed traffic (traffic count survey). Figure 19.2.1-2 shows the result of comparison between the assigned traffic volume and observed traffic volume. This comparison between observed traffic count and assigned traffic flow at individual sites is done via the Mean Absolute Difference (MAD)[1] Ratio. For daily traffic counts, the value of the MAD ratio is 0.094 which is considered to reflect a good calibration. By all indicators, the assignment is acceptable level to replicate year 2012.

---

[1] MAD Ratio is defined by the following formula: MAD Ratio = $\sum\left|\frac{Count - assignment}{assignment}\right|/n$ where n is the number of observations.

Table 19.2.1-1 Comparison of Observed (Survey data) and Assigned Traffic Volume

Bridge Name	Observed Traffic (100 Veh./day)	Assigned Traffic (100 Veh./day)	Difference (100 Veh./day)	Rate
1.MARIKINA Bridge	400	451	-51	11%
2.MARCOS Bridge	791	712	79	-11%
3.GUADALUPE Bridge	2,009	2,047	-38	2%
4.C-5 Bridge	1,322	1,284	38	-3%
5.ESTRELLA PANTALEON Bridge	210	185	25	-14%
6.LAMBINGAN Bridge	209	193	16	-8%
7.MAKATI-MANDALUYON Bridge	311	271	40	-15%
8.PANDACAN Bridge	238	211	27	-13%
9.NAGTAHAN Bridge	753	803	-50	6%
10.AYALA Bridge	312	296	16	-5%
11.DELPAN Bridge	417	514	-97	19%
12.JONES Bridge	391	414	-23	6%
Total	**7,363**	**7,381**	**-18**	**0%**

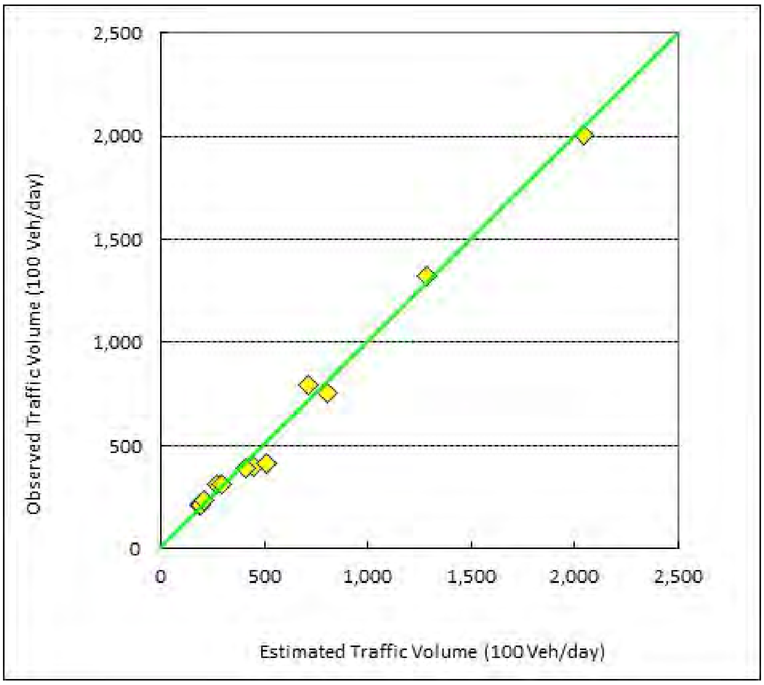


Figure 19.2.1-2 Comparison of Observed and Assigned Traffic Volume

(2) Future Traffic Assignment

1) Traffic Assignment Model

The traffic assignment to road network is made using STRADA highway-type incremental assignment model. The traffic assignment can be calculated by the following traffic assignment step. (See **Figure 19.2.1-3**)

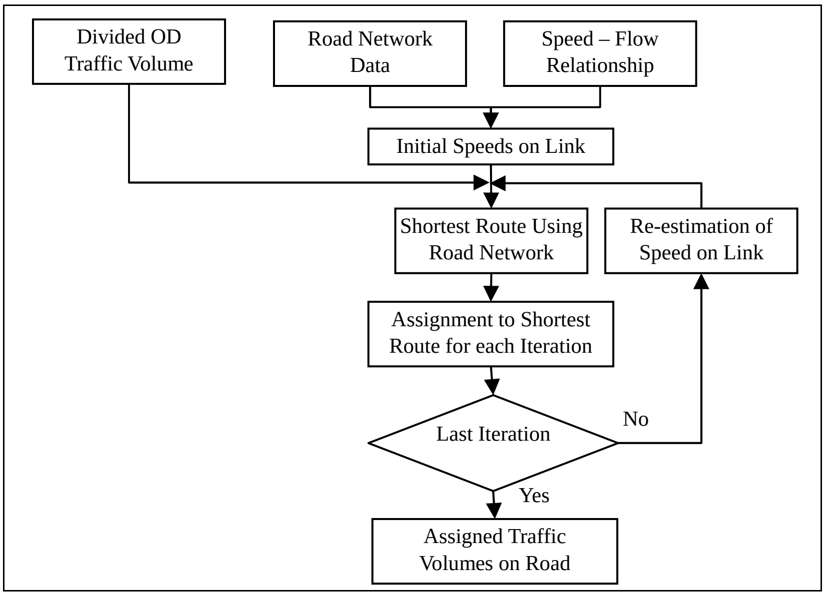


Figure 19.2.1-3 Traffic Assignment Method

2) Road Network Conditions

Based on the other road project, maturity in Metro Manila road network assumptions are prepared.

Open Year	**Road Project**
2018	NAIAX
2020	NLEX-SLEX Connector, C3 Missing Link, Lawton - Bridge
2030	C6

3) Result of Traffic Assignment

Table 19.2.1-23 shows the future traffic volume of bridge crossing Pasig River or Marikina River. Though Traffic volume of Guadalupe Bridge and Lambingan Bridge will increase in Year 2018, those Traffic Volume will drastically decrease in Year 2020 due to new construction of C3 Road, NS Connector Road and Sta. Monica – Lawton Bridge.

Table 19.2.1-2 Future Traffic Volume Crossing Pasig River / Marikina River

Unit: 100 vehicles per day

Bridge Name	Year2012	Year2018	Year2020	Year2030
1.MARIKINA Bridge	451	458	474	524
2.MARCOS Bridge	712	700	664	705
3.GUADALUPE Bridge (target bridge)	2,047	2,194	1,563	1,738
4.C-5 Bridge	1,284	1,349	1,169	1,315
5.ESTRELLA PANTALEON Bridge	185	171	175	344
6.LAMBINGAN Bridge (target bridge)	193	209	174	182
7.MAKATI-MANDALUYON Bridge	271	453	300	324
8.PANDACAN Bridge	211	221	137	144
9.NAGTAHAN Bridge	803	799	455	469
10.AYALA Bridge	296	408	254	236
11.DELPAN Bridge	514	625	493	533
12.JONES Bridge	414	419	289	351
13.C3 (Future)			528	600
14.NS-Connector (Future)			939	1,138
15.Sta. Monica-Lawton Bridge (Future)			770	1,059
16.C6 (Future)				765
Total	**7,381**	**8,006**	**8,384**	**10,427**

19.2.2 Analysis of Traffic Congestion during Bridge Improvement

(1) Guadalupe Bridge

Based on the assignment and survey results, traffic queue length during construction was estimated.

1) Target Year 2018 (construction year 2017 ~ 2018)

2) Traffic Capacity

$C_L = C_B \times \gamma_L \times \gamma_C \times \gamma_T$

C_L = Traffic Capacity per lane (pcu/h/lane)

C_B = Base Traffic Capacity (pcu/h/lane) = 2,200

$\gamma_L, \gamma_C, \gamma_T$: Adjustment Parameter

γ_L : Lane Width W_L = 3.25m or more than, $\gamma_L = 1.00$
= 3.00 m $\gamma_L = 0.94$

γ_C : Shoulder With W_C = 0.75 m or more than $\gamma_T = 1.00$
0.0 m $\gamma_T = 0.93$

γ_T : Track Occupancy Rate $\gamma_T = \dfrac{100}{\gamma_T = (100 - T) \times 1.7\text{-}T}$

T = 9% - γ_T 0.93

Case - 0 **5 lane** W_L = 3.25 m, W_C = 0.75 m, Truck 9%

$C_L = 2{,}200 \times 1.00 \times 1.00 \times 0.93 = 2{,}046$

$C_C = C_L \times 5 = 10{,}230$ pcu/h (per direction)

Case -1 **4 lane** W_L = 3.25 m, W_C = 0.75 m, Truck 9%

$C_L = 2{,}200 \times 1.00 \times 1.0 \times 0.93 = 2{,}046$

$C_C = C_L \times 4 = 8{,}184$ pcu/h (per direction)

Case -2 **3 lane** W_L = 3.25 m, W_C = 0.95 m, Truck 9%

$C_L = 2{,}200 \times 1.00 \times 1.00 \times 0.93 = 2{,}046$

$C_C = C_L \times 3 = 6{,}138$ pcu /h (per direction)

3) Volume vs. Capacity

Table 19.2.2-1 - Table 19.2.2-3 show the hourly volume vs. capacity during construction year. Though traffic queue will not occur in case-1(4-lane traffic restriction), long traffic queue may occur in case-2 (3-lanes). Estimated queue length in Case-2 is 26.5km at 5PM to northbound (from Buendia to Shaw Blvd.) and 3.5km at 7PM to southbound (from Shaw Blvd. to Buendia). At present, traffic of southbound is approximately 20,000 (vehicles/day) lower than that of northbound.

Table 19.2.2-1 Hourly Volume vs. Capacity in Guadalupe Bridge (1/3) (Case-0, No traffic restriction 5-lane)

Dir-1 from Buendia/Ayala to Shaw Blvd.
Year
2018

Time	Volume (PCU)	Capacity (PCU)	Cap-Vol (PCU)	Queue Vehicle (PCU)	Queue Length (km)
6:00	4,828	10,230	-5,402	0	0.0
7:00	6,751	10,230	-3,479	0	0.0
8:00	6,234	10,230	-3,996	0	0.0
9:00	7,668	10,230	-2,562	0	0.0
10:00	7,255	10,230	-2,975	0	0.0
11:00	6,724	10,230	-3,506	0	0.0
12:00	5,966	10,230	-4,264	0	0.0
13:00	7,551	10,230	-2,679	0	0.0
14:00	7,839	10,230	-2,391	0	0.0
15:00	8,101	10,230	-2,129	0	0.0
16:00	7,260	10,230	-2,970	0	0.0
17:00	7,539	10,230	-2,691	0	0.0
18:00	3,765	10,230	-6,465	0	0.0
19:00	4,761	10,230	-5,469	0	0.0
20:00	4,454	10,230	-5,776	0	0.0
21:00	4,335	10,230	-5,895	0	0.0
22:00	4,736	10,230	-5,494	0	0.0
23:00	6,274	10,230	-3,956	0	0.0
0:00	3,175	10,230	-7,055	0	0.0
1:00	2,457	10,230	-7,773	0	0.0
2:00	2,123	10,230	-8,107	0	0.0
3:00	1,702	10,230	-8,528	0	0.0
4:00	2,350	10,230	-7,880	0	0.0
5:00	2,669	10,230	-7,561	0	0.0
Total	126,516				

Dir-2 from Shaw Blvd. to Buendia/Ayala
Year
2018

Time	Volume (PCU)	Capacity (PCU)	Cap-Vol (PCU)	Queue Vehicle (PCU)	Queue Length (km)
6:00	6,690	10,230	-3,540	0	0.0
7:00	5,818	10,230	-4,412	0	0.0
8:00	5,011	10,230	-5,219	0	0.0
9:00	4,848	10,230	-5,382	0	0.0
10:00	4,811	10,230	-5,419	0	0.0
11:00	4,972	10,230	-5,258	0	0.0
12:00	4,968	10,230	-5,262	0	0.0
13:00	4,185	10,230	-6,045	0	0.0
14:00	4,921	10,230	-5,309	0	0.0
15:00	4,297	10,230	-5,933	0	0.0
16:00	5,013	10,230	-5,217	0	0.0
17:00	4,998	10,230	-5,232	0	0.0
18:00	5,755	10,230	-4,475	0	0.0
19:00	7,541	10,230	-2,689	0	0.0
20:00	4,957	10,230	-5,273	0	0.0
21:00	4,629	10,230	-5,601	0	0.0
22:00	4,325	10,230	-5,905	0	0.0
23:00	3,138	10,230	-7,092	0	0.0
0:00	2,818	10,230	-7,412	0	0.0
1:00	2,203	10,230	-8,027	0	0.0
2:00	1,643	10,230	-8,587	0	0.0
3:00	1,667	10,230	-8,563	0	0.0
4:00	2,545	10,230	-7,685	0	0.0
5:00	4,402	10,230	-5,828	0	0.0
Total	106,156				

Table 19.2.2-2 Hourly Volume vs. Capacity in Guadalupe Bridge (2/3) (Case-1, 4-lane)

Dir-1 from Buendia/Ayala to Shaw Blvd.
Year
2018

Time	Volume (PCU)	Capacity (PCU)	Cap-Vol (PCU)	Queue Vehicle (PCU)	Queue Length (km)
6:00	4,828	8,184	-3,356	0	0.0
7:00	6,751	8,184	-1,433	0	0.0
8:00	6,234	8,184	-1,950	0	0.0
9:00	7,668	8,184	-516	0	0.0
10:00	7,255	8,184	-929	0	0.0
11:00	6,724	8,184	-1,460	0	0.0
12:00	5,966	8,184	-2,218	0	0.0
13:00	7,551	8,184	-633	0	0.0
14:00	7,839	8,184	-345	0	0.0
15:00	8,101	8,184	-83	0	0.0
16:00	7,260	8,184	-924	0	0.0
17:00	7,539	8,184	-645	0	0.0
18:00	3,765	8,184	-4,419	0	0.0
19:00	4,761	8,184	-3,423	0	0.0
20:00	4,454	8,184	-3,730	0	0.0
21:00	4,335	8,184	-3,849	0	0.0
22:00	4,736	8,184	-3,448	0	0.0
23:00	6,274	8,184	-1,910	0	0.0
0:00	3,175	8,184	-5,009	0	0.0
1:00	2,457	8,184	-5,727	0	0.0
2:00	2,123	8,184	-6,061	0	0.0
3:00	1,702	8,184	-6,482	0	0.0
4:00	2,350	8,184	-5,834	0	0.0
5:00	2,669	8,184	-5,515	0	0.0
Total	126,516				

Dir-2 from Shaw Blvd. to Buendia/Ayala
Year
2018

Time	Volume (PCU)	Capacity (PCU)	Cap-Vol (PCU)	Queue Vehicle (PCU)	Queue Length (km)
6:00	6,690	8,184	-1,494	0	0.0
7:00	5,818	8,184	-2,366	0	0.0
8:00	5,011	8,184	-3,173	0	0.0
9:00	4,848	8,184	-3,336	0	0.0
10:00	4,811	8,184	-3,373	0	0.0
11:00	4,972	8,184	-3,212	0	0.0
12:00	4,968	8,184	-3,216	0	0.0
13:00	4,185	8,184	-3,999	0	0.0
14:00	4,921	8,184	-3,263	0	0.0
15:00	4,297	8,184	-3,887	0	0.0
16:00	5,013	8,184	-3,171	0	0.0
17:00	4,998	8,184	-3,186	0	0.0
18:00	5,755	8,184	-2,429	0	0.0
19:00	7,541	8,184	-643	0	0.0
20:00	4,957	8,184	-3,227	0	0.0
21:00	4,629	8,184	-3,555	0	0.0
22:00	4,325	8,184	-3,859	0	0.0
23:00	3,138	8,184	-5,046	0	0.0
0:00	2,818	8,184	-5,366	0	0.0
1:00	2,203	8,184	-5,981	0	0.0
2:00	1,643	8,184	-6,541	0	0.0
3:00	1,667	8,184	-6,517	0	0.0
4:00	2,545	8,184	-5,639	0	0.0
5:00	4,402	8,184	-3,782	0	0.0
Total	106,156				

Table 19.2.2-3 Hourly Volume vs. Capacity in Guadalupe Bridge (3/3) (Case-2, 3-lane)

Dir-1 from Buendia/Ayala to Shaw Blvd.

Year 2018

Time	Volume (PCU)	Capacity (PCU)	Cap-Vol (PCU)	Queue Vehicle (PCU)	Queue Length (km)
6:00	4,828	6,138	-1,310	0	0.0
7:00	6,751	6,138	613	613	**1.4**
8:00	6,234	6,138	96	709	**1.7**
9:00	7,668	6,138	1,530	2,239	**5.2**
10:00	7,255	6,138	1,117	3,356	**7.8**
11:00	6,724	6,138	586	3,942	**9.2**
12:00	5,966	6,138	-172	3,770	**8.8**
13:00	7,551	6,138	1,413	5,184	**12.1**
14:00	7,839	6,138	1,701	6,884	**16.1**
15:00	8,101	6,138	1,963	8,848	**20.6**
16:00	7,260	6,138	1,122	9,970	**23.3**
17:00	7,539	6,138	1,401	11,371	**26.5**
18:00	3,765	6,138	-2,373	8,998	**21.0**
19:00	4,761	6,138	-1,377	7,621	**17.8**
20:00	4,454	6,138	-1,684	5,936	**13.9**
21:00	4,335	6,138	-1,803	4,133	**9.6**
22:00	4,736	6,138	-1,402	2,731	**6.4**
23:00	6,274	6,138	136	2,867	**6.7**
0:00	3,175	6,138	-2,963	0	0.0
1:00	2,457	6,138	-3,681	0	0.0
2:00	2,123	6,138	-4,015	0	0.0
3:00	1,702	6,138	-4,436	0	0.0
4:00	2,350	6,138	-3,788	0	0.0
5:00	2,669	6,138	-3,469	0	0.0
Total	126,516				

Dir-2 from Shaw Blvd. to Buendia/Ayala

Year 2018

Time	Volume (PCU)	Capacity (PCU)	Cap-Vol (PCU)	Queue Vehicle (PCU)	Queue Length (km)
6:00	6,690	6,138	552	552	**1.3**
7:00	5,818	6,138	-320	232	**0.5**
8:00	5,011	6,138	-1,127	0	0.0
9:00	4,848	6,138	-1,290	0	0.0
10:00	4,811	6,138	-1,327	0	0.0
11:00	4,972	6,138	-1,166	0	0.0
12:00	4,968	6,138	-1,170	0	0.0
13:00	4,185	6,138	-1,953	0	0.0
14:00	4,921	6,138	-1,217	0	0.0
15:00	4,297	6,138	-1,841	0	0.0
16:00	5,013	6,138	-1,125	0	0.0
17:00	4,998	6,138	-1,140	0	0.0
18:00	5,755	6,138	-383	0	0.0
19:00	7,541	6,138	1,403	1,403	**3.3**
20:00	4,957	6,138	-1,181	223	**0.5**
21:00	4,629	6,138	-1,509	0	0.0
22:00	4,325	6,138	-1,813	0	0.0
23:00	3,138	6,138	-3,000	0	0.0
0:00	2,818	6,138	-3,320	0	0.0
1:00	2,203	6,138	-3,935	0	0.0
2:00	1,643	6,138	-4,495	0	0.0
3:00	1,667	6,138	-4,471	0	0.0
4:00	2,545	6,138	-3,593	0	0.0
5:00	4,402	6,138	-1,736	0	0.0
Total	106,156				

As this traffic analysis was only point of Guadalupe Bridge, traffic simulation was conducted. The traffic simulation result is described in next section.

(2) Lambingan Bridge

As same method used for Guadalupe Bridge, the traffic queue length during construction was estimated.

1) Target Year: 2018 (Construction Year 2017 ~ 2018)

2) Traffic Capacity

$$C_L = C_B \times \gamma_L \times \gamma_C \times \gamma_T$$

C_L= Traffic Capacity per lane (pcu/h/lane)

C_B = Base Traffic Capacity (pcu/h/lane) = 2,200

$\gamma_{L,}$ $\gamma_{C,}$ $\gamma_{T,}$; Adjustment Parameter

$\gamma_{L:}$ Lane

Case – 0 **3 lane** W_L = 3.25, W_C = 0.75 m, Truck 3%

C_L = 2,200 × 1.00 × 1.00 × 0.98 = 2,156

$C_C = C_L \times 3$ = ~ 6,468 pcu/h (per direction)

Case – 1 **2 lane** W_L = 3.25, W_C = 0.75 Truck 3%

C_L = 2,156

$C_C = C_L \times 2$ = 4,312 pcu/h (per direction)

Case – 2 **1 lane** W_L = 3.25, W_C = 0.75 Truck 3%

C_L = 2,156

C_C = 2,156 pcu/h (per direction)

3) Volume vs. Capacity

Table 19.2.2-4 - Table 19.2.2-6 show the hourly volume vs. capacity during the construction year.

The entire hourly volume is lower than capacity in Case-1 and Case-2. At Lambingan Bridge, it is possible for a 2-lane traffic restriction (one lane each direction only) for all the day in year 2018 based on this result. Since morning peak hour’s (7:00-8:00) volume from Sta. Mesa to Sta. Ana is higher than usual, it is recommended that implementation of traffic restriction should be carefully studied.

Table 19.2.2-4 Hourly Volume vs. Capacity in Lambingan Bridge (1/3) (Case-0, No traffic restriction 3-lane)

Dir-1 from Sta. Ana to Sta. Mesa
Year
2018

Time	Volume (PCU)	Capacity (PCU)	Cap-Vol (PCU)	Queue Vehicle (PCU)	Queue Length (m)
6:00	837	6,468	-5,631	0	0
7:00	967	6,468	-5,501	0	0
8:00	1,100	6,468	-5,368	0	0
9:00	1,178	6,468	-5,290	0	0
10:00	1,026	6,468	-5,442	0	0
11:00	1,259	6,468	-5,209	0	0
12:00	949	6,468	-5,519	0	0
13:00	1,112	6,468	-5,356	0	0
14:00	1,390	6,468	-5,078	0	0
15:00	1,086	6,468	-5,382	0	0
16:00	1,361	6,468	-5,107	0	0
17:00	1,577	6,468	-4,891	0	0
18:00	1,546	6,468	-4,922	0	0
19:00	1,525	6,468	-4,943	0	0
20:00	1,330	6,468	-5,138	0	0
21:00	1,070	6,468	-5,398	0	0
22:00	1,097	6,468	-5,371	0	0
23:00	629	6,468	-5,839	0	0
0:00	486	6,468	-5,982	0	0
1:00	240	6,468	-6,228	0	0
2:00	247	6,468	-6,221	0	0
3:00	274	6,468	-6,194	0	0
4:00	406	6,468	-6,062	0	0
5:00	521	6,468	-5,947	0	0
Total	19,313				

Dir-2 from Sta. Mesa to Sta. Ana
Year
2018

Time	Volume (PCU)	Capacity (PCU)	Cap-Vol (PCU)	Queue Vehicle (PCU)	Queue Length (m)
6:00	1,589	6,468	-4,879	0	0
7:00	2,024	6,468	-4,444	0	0
8:00	1,951	6,468	-4,517	0	0
9:00	1,578	6,468	-4,890	0	0
10:00	1,339	6,468	-5,129	0	0
11:00	1,309	6,468	-5,159	0	0
12:00	1,117	6,468	-5,351	0	0
13:00	1,314	6,468	-5,154	0	0
14:00	1,249	6,468	-5,219	0	0
15:00	1,175	6,468	-5,293	0	0
16:00	1,215	6,468	-5,253	0	0
17:00	1,470	6,468	-4,998	0	0
18:00	954	6,468	-5,514	0	0
19:00	808	6,468	-5,660	0	0
20:00	781	6,468	-5,687	0	0
21:00	623	6,468	-5,845	0	0
22:00	525	6,468	-5,943	0	0
23:00	415	6,468	-6,053	0	0
0:00	451	6,468	-6,017	0	0
1:00	187	6,468	-6,281	0	0
2:00	214	6,468	-6,254	0	0
3:00	264	6,468	-6,204	0	0
4:00	321	6,468	-6,147	0	0
5:00	573	6,468	-5,895	0	0
Total	20,498				

Table 19.2.2-5 Hourly Volume vs. Capacity in Lambingan Bridge (2/3) (Case-1, 2-lane)

Dir-1 from Sta. Ana to Sta. Mesa
Year
2018

Time	Volume (PCU)	Capacity (PCU)	Cap-Vol (PCU)	Queue Vehicle (PCU)	Queue Length (m)
6:00	775	4,312	-3,537	0	0
7:00	895	4,312	-3,418	0	0
8:00	1,018	4,312	-3,294	0	0
9:00	1,091	4,312	-3,222	0	0
10:00	950	4,312	-3,363	0	0
11:00	1,165	4,312	-3,147	0	0
12:00	878	4,312	-3,434	0	0
13:00	1,029	4,312	-3,283	0	0
14:00	1,286	4,312	-3,026	0	0
15:00	1,005	4,312	-3,307	0	0
16:00	1,260	4,312	-3,053	0	0
17:00	1,459	4,312	-2,853	0	0
18:00	1,431	4,312	-2,882	0	0
19:00	1,412	4,312	-2,901	0	0
20:00	1,231	4,312	-3,081	0	0
21:00	991	4,312	-3,322	0	0
22:00	1,016	4,312	-3,297	0	0
23:00	582	4,312	-3,730	0	0
0:00	450	4,312	-3,862	0	0
1:00	222	4,312	-4,090	0	0
2:00	229	4,312	-4,084	0	0
3:00	254	4,312	-4,058	0	0
4:00	376	4,312	-3,937	0	0
5:00	483	4,312	-3,830	0	0
Total	17,873				

Dir-2 from Sta. Mesa to Sta. Ana
Year
2018

Time	Volume (PCU)	Capacity (PCU)	Cap-Vol (PCU)	Queue Vehicle (PCU)	Queue Length (m)
6:00	1,471	4,312	-2,842	0	0
7:00	1,874	4,312	-2,439	0	0
8:00	1,806	4,312	-2,507	0	0
9:00	1,461	4,312	-2,852	0	0
10:00	1,240	4,312	-3,073	0	0
11:00	1,212	4,312	-3,101	0	0
12:00	1,034	4,312	-3,279	0	0
13:00	1,216	4,312	-3,096	0	0
14:00	1,156	4,312	-3,157	0	0
15:00	1,088	4,312	-3,225	0	0
16:00	1,125	4,312	-3,188	0	0
17:00	1,360	4,312	-2,952	0	0
18:00	883	4,312	-3,429	0	0
19:00	748	4,312	-3,564	0	0
20:00	723	4,312	-3,589	0	0
21:00	577	4,312	-3,735	0	0
22:00	486	4,312	-3,827	0	0
23:00	384	4,312	-3,928	0	0
0:00	418	4,312	-3,895	0	0
1:00	173	4,312	-4,139	0	0
2:00	199	4,312	-4,114	0	0
3:00	244	4,312	-4,068	0	0
4:00	297	4,312	-4,015	0	0
5:00	530	4,312	-3,782	0	0
Total	18,969				

Table 19.2.2-6 Hourly Volume vs. Capacity in Lambingan Bridge (3/3) (Case-2, 1-lane)

Dir-1 from Sta. Ana to Sta. Mesa

Year 2018

Time	Volume (PCU)	Capacity (PCU)	Cap-Vol (PCU)	Queue Vehicle (PCU)	Queue Length (m)
6:00	837	2,156	-1,319	0	0
7:00	967	2,156	-1,189	0	0
8:00	1,100	2,156	-1,056	0	0
9:00	1,178	2,156	-978	0	0
10:00	1,026	2,156	-1,130	0	0
11:00	1,259	2,156	-897	0	0
12:00	949	2,156	-1,207	0	0
13:00	1,112	2,156	-1,044	0	0
14:00	1,390	2,156	-766	0	0
15:00	1,086	2,156	-1,070	0	0
16:00	1,361	2,156	-795	0	0
17:00	1,577	2,156	-579	0	0
18:00	1,546	2,156	-610	0	0
19:00	1,525	2,156	-631	0	0
20:00	1,330	2,156	-826	0	0
21:00	1,070	2,156	-1,086	0	0
22:00	1,097	2,156	-1,059	0	0
23:00	629	2,156	-1,527	0	0
0:00	486	2,156	-1,670	0	0
1:00	240	2,156	-1,916	0	0
2:00	247	2,156	-1,909	0	0
3:00	274	2,156	-1,882	0	0
4:00	406	2,156	-1,750	0	0
5:00	521	2,156	-1,635	0	0
Total	19,313				

Dir-2 from Sta. Mesa to Sta. Ana

Year 2018

Time	Volume (PCU)	Capacity (PCU)	Cap-Vol (PCU)	Queue Vehicle (PCU)	Queue Length (m)
6:00	1,589	2,156	-567	0	0
7:00	2,024	2,156	-132	0	0
8:00	1,951	2,156	-205	0	0
9:00	1,578	2,156	-578	0	0
10:00	1,339	2,156	-817	0	0
11:00	1,309	2,156	-847	0	0
12:00	1,117	2,156	-1,039	0	0
13:00	1,314	2,156	-842	0	0
14:00	1,249	2,156	-907	0	0
15:00	1,175	2,156	-981	0	0
16:00	1,215	2,156	-941	0	0
17:00	1,470	2,156	-686	0	0
18:00	954	2,156	-1,202	0	0
19:00	808	2,156	-1,348	0	0
20:00	781	2,156	-1,375	0	0
21:00	623	2,156	-1,533	0	0
22:00	525	2,156	-1,631	0	0
23:00	415	2,156	-1,741	0	0
0:00	451	2,156	-1,705	0	0
1:00	187	2,156	-1,969	0	0
2:00	214	2,156	-1,942	0	0
3:00	264	2,156	-1,892	0	0
4:00	321	2,156	-1,835	0	0
5:00	573	2,156	-1,583	0	0
Total	20,498				

19.3 Traffic Influence Analysis during Rehabilitation Works at Guadalupe Bridge

19.3.1 Background

Guadalupe Bridge is a bridge along EDSA about 200,000 vehicles passes through per day. It is selected as one of the bridges that need to be repaired to be earthquake resistant. This is to make sure that the bridge would be used as the safety route or to prevent the bridge from a possible collapse. Thus, prompt attention is needed.

19.3.2 Purpose

In the present condition, heavy traffic congestion occurs in the morning peak and the evening peak. In case of the reduction of the number of lanes during Rehabilitation Works, the influence of traffic congestion needs to be analyzed.

This section analyzes the influence of traffic condition during Guadalupe Bridge Rehabilitation Works with the use of a traffic microscopic simulation.

19.3.3 Present Traffic Condition at Guadalupe Bridge

The present traffic congestion condition at Guadalupe Bridge is as follows.

(1) Northbound (Bound to Quezon City)

1) Morning Peak

a) Present traffic condition

Traffic lane is decreased by takingen up the entire lane with some longtime parking queuing of jeepnieys right at the mouth of the Guadalupe Bridge. The vehicles which go passes through Guadalupe Bridge can use only 3 lanes instead of 5 lanes as shown in Figure 19.3.3-1. The volume of the traffic here counts is about 5,000~6,000 vehicles per hour as shown in Table 19.3.3-1.

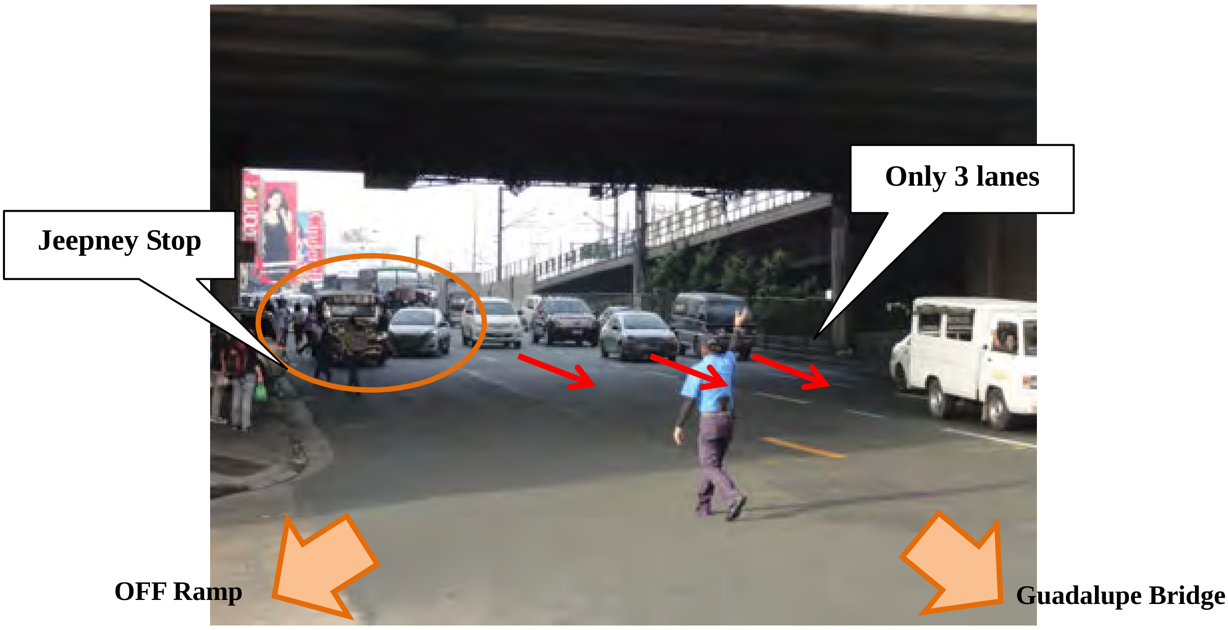


Figure 19.3.3-1 Traffic Condition at MRT Line-3 Guadalupe Station

Table 19.3.3-1 Traffic Volume (Bound to Guadalupe Bridge)

Time Period		1. Motorcycle / Tricycle	2. Car / Taxi / Pick-up / Van	3. Jeepney	4. Large Bus	5. 2-Axle Truck	6. 3-Axle Truck	7. Truck trailer	TOTAL
6:00	7:00	488	5,297	5	412	96	2	1	6,301
7:00	8:00	561	3,828	6	413	107	3	0	4,918
8:00	9:00	434	4,137	1	293	82	8	0	4,955

Date of Survey: 2013.4.11

b) Situation of Traffic Lane Operation at Guadalupe Bridge

The situation of traffic lane operation at Guadalupe Bridge is shown as follows.

- The traffic volume at Guadalupe Bridge is about 7,000 vehicles per hour in the morning peak (Table 19.3.3-2).
- The vehicles which pass through from Guadalupe MRT Line-3 Station to Guadalupe Bridge uses the inner 3 lanes (Figure 19.3.3-2).
- The traffic volume at ON Ramp is at a maximum of about 2,000 vehicles per hour during morning peak. And it merges into the main road from ON Ramp using 2 lanes (Table 19.3.3-3).

Guadalupe Station On Ramp

Figure 19.3.3-2 Traffic Condition at Guadalupe Bridge

Table 19.3.3-2 Traffic Volume (Guadalupe Bridge)

Time Period		1. Motorcycle / Tricycle	2. Car / Taxi / Pick-up / Van	3. Jeepney	4. Large Bus	5. 2-Axle Truck	6. 3-Axle Truck	7. Truck trailer	TOTAL
6:00	7:00	686	5,850	5	417	102	9	2	7,071
7:00	8:00	825	5,016	6	418	115	5	0	6,385
8:00	9:00	665	5,788	1	298	101	13	0	6,866

Date of Survey: 2013.4.11

Table 19.3.3-3 Traffic Volume (On Ramp)

Time Period		1. Motorcycle / Tricycle	2. Car / Taxi / Pick-up / Van	3. Jeepney	4. Large Bus	5. 2-Axle Truck	6. 3-Axle Truck	7. Truck trailer	TOTAL
6:00	7:00	198	553	0	5	6	7	1	770
7:00	8:00	264	1,188	0	5	8	2	0	1,467
8:00	9:00	231	1,651	0	5	19	5	0	1,911

Date of Survey: 2013.4.11

2) Evening Peak

a) Present traffic condition

During the evening peak, the traffic congestion occurring at the bottleneck point after the Guadalupe Bridge is extending across Guadalupe Bridge as shown in Figure 19.3.3-3.

However, the traffic volume of vehicles which passes through Guadalupe Bridge decreases sharply after the peak time as shown in Table 19.3.3-4.

Figure 19.3.3-3 Traffic Congestion at Guadalupe Bridge

Table 19.3.3-4 Traffic Volume (Bound to Guadalupe Bridge)

Time Period		1. Motorcycle / Tricycle	2. Car / Taxi / Pick-up / Van	3. Jeepney	4. Large Bus	5. 2-Axle Truck	6. 3-Axle Truck	7. Truck trailer	TOTAL
16:00	17:00	386	5,960	0	355	123	7	0	6,831
17:00	18:00	328	3,883	3	351	128	3	0	4,696
18:00	19:00	415	1,938	1	483	35	3	3	2,878
19:00	20:00	381	1,561	0	385	48	11	0	2,386
20:00	21:00	398	2,113	0	516	53	2	1	3,083
21:00	22:00	402	2,154	3	485	46	1	0	3,091

The traffic volume decrease sharply.

Date of Survey: 2013.4.11

(2) Southbound (Bound to Makati)

1) Morning Peak

a) Present traffic condition

The traffic lane is decreased by longtime parallel queuing of buses at the bus stop in Figure 19.3.3-4.

Bound to Guadalupe Bridge

Figure 19.3.3-4 Bus Stop at Guadalupe Bridge

The number of lanes decreases from 5 to 4 at Kalayaan Flyover as shown in Figure 19.3.3-5. The traffic congestion bound to Makati occurs from this point and is extended across Guadalupe Bridge as shown in Figure 19.3.3-6. As a result, the traffic volume at Guadalupe Bridge decreases sharply between 8:00 and 9:00 as shown in Table 19.3.3-5.

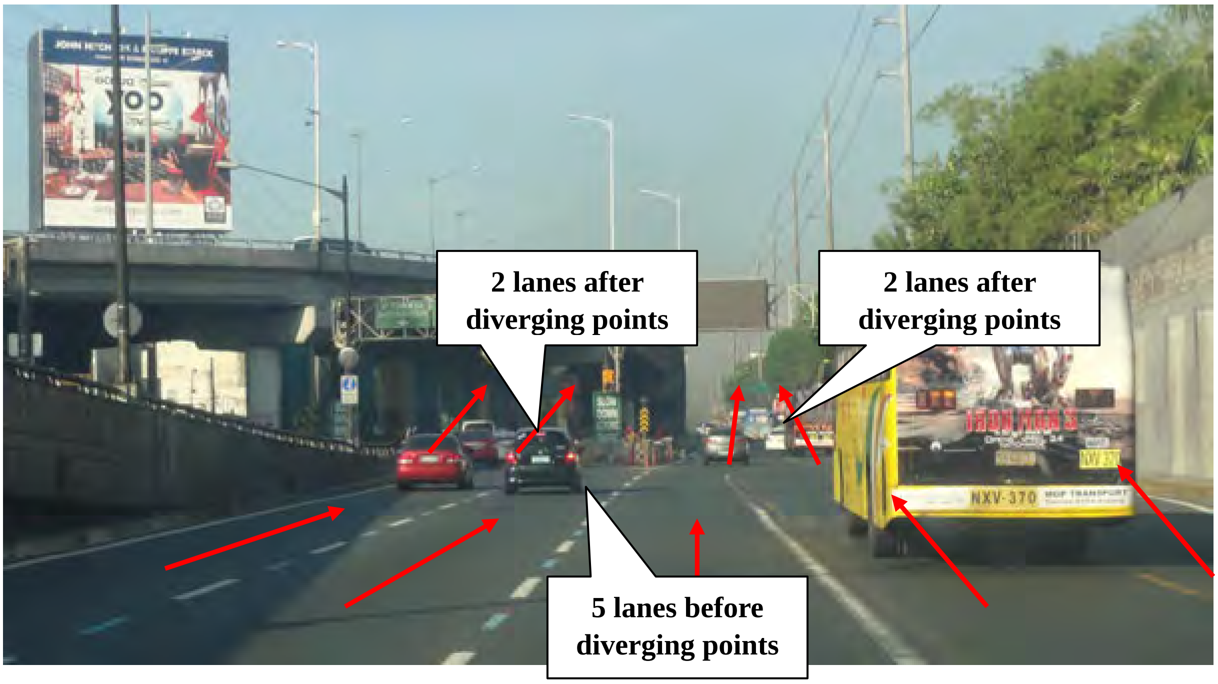


Figure 19.3.3-5 Bottleneck at Kalayaan Fly Over

Table 19.3.3-5 Traffic Volume (Guadalupe Bridge)

Time Period		1. Motorcycle / Tricycle	2. Car / Taxi / Pick-up / Van	3. Jeepney	4. Large Bus	5. 2-Axle Truck	6. 3-Axle Truck	7. Truck trailer	TOTAL
6:00	7:00	903	4,724	3	497	88	4	0	6,219
7:00	8:00	761	4,950	3	404	45	2	0	6,165
8:00	9:00	614	3,575	0	336	48	4	0	4,577
9:00	10:00	530	3,667	4	361	63	1	0	4,626
10:00	11:00	517	3,745	0	354	98	0	0	4,714

The traffic volumes decrease sharply.

Date of Survey: 2013.4.11

Figure 19.3.3-6 Traffic condition at Guadalupe Bridge

2) Evening Peak

a) Present traffic condition

The present traffic congestion occurs from Kalayaan Flyover and is extended to Guadalupe Bridge as shown in Figure 19.3.3-7. The traffic volume of Guadalupe Bridge decreases between 18:00 and 19:00 as shown in Table 19.3.3-6.

Figure 19.3.3-7 Traffic Condition at Guadalupe Bridge

Table 19.3.3-6 Traffic Volume (Guadalupe Bridge)

Time Period		1. Motorcycle / Tricycle	2. Car / Taxi / Pick-up / Van	3. Jeepney	4. Large Bus	5. 2-Axle Truck	6. 3-Axle Truck	7. Truck trailer	TOTAL
16:00	17:00	539	4,944	6	494	90	7	0	6,080
17:00	18:00	505	4,076	6	379	47	5	0	5,018
18:00	19:00	597	3,766	7	487	34	3	0	4,894

The traffic volumes decrease sharply.

(3) Travel Speed Survey

The travel speed survey was conducted at about 4 km including Guadalupe Bridge. This survey utilized a GPS.

Overview of the travel speed survey is as follows.

1) Overview

a) Survey method

The vehicle which carried GPS obtained latitude, longitude and time.

b) Survey day and time

■Survey Day
2013.4.16(Tue)

■Survey time
Survey data was obtained once or twice per hour in the morning peak and in the evening peak.

c) Data collection interval

A data collection interval is 1 second.

2) Result of the Travel Speed Survey

Result of the travel speed survey is shown as follows (Figure 19.3.3-8).

a) Northbound (Bound to Quezon City)

- Travel speed decreases from Guadalupe Bridge in the morning peak.
- Travel speed decreases at the point past Guadalupe Bridge in the evening peak.

b) Southbound (Bound to Makati City)

- Travel speed decreases before and after Guadalupe Bridge in the morning peak and in the evening peak.
- Travel speed decreases at Kalayaan Flyover and this influence is extended up to Guadalupe Bridge.

Result of Travel Speed Survey

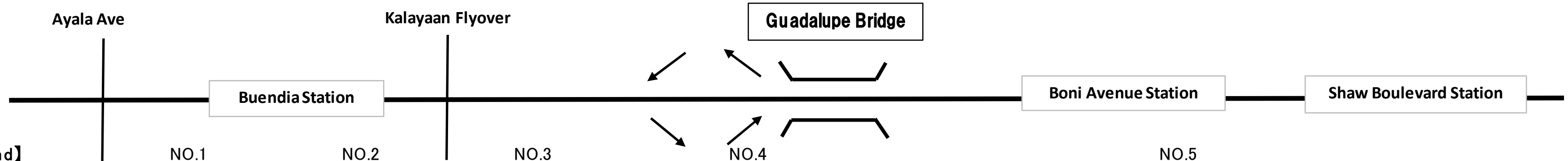
Bound to Makati City

Low speed

Low speed

【Southbound】

		NO.6	No.5	NO.4	NO.3	No.2	NO.1
PM_peak	18:00～19:00	17.1	7.9	9.0	10.3	17.9	50.6
		13.5	12.1	32.7	28.7	18.9	39.9
	17:00～18:00	9.9	23.0	24.5	36.6	13.2	32.3
AM_peak	8:00～9:00	18.6	9.7	5.3	7.4	5.3	13.3
		16.9	10.0	7.6	15.0	10.8	38.9
	7:00～8:00	16.4	17.1	42.9	41.8	11.7	13.5
	6:00～7:00	38.0	26.7	21.4	40.6	13.6	41.0
		28.5	35.0	40.4	38.5	15.8	33.7



【Northbound】

		NO.1	NO.2	NO.3	NO.4	NO.5
AM_peak	6:00～7:00	37.5	29.1	41.2	43.0	42.7
		39.6	29.5	40.9	45.0	38.0
	7:00～8:00	33.9	20.7	35.6	39.4	43.6
		33.4	18.7	14.9	43.0	39.5
	8:00～9:00	20.9	18.1	13.6	52.5	44.5
PM_peak	17:00～18:00	13.5	17.2	20.8	39.4	18.2
		14.4	17.0	12.2	16.6	12.5
	18:00～19:00	6.2	10.3	5.9	5.4	10.3

【km/h】

Bound to Quezon City

Low speed

Low speed

0～20km/h
20～30km/h
30km/h～

Figure 19.3.3-8 Result of Travel Speed Survey

19.3.4 Reappearance of the Traffic Condition around Guadalupe Bridge

The traffic condition around Guadalupe Bridge was created based on the result of the present traffic survey by using microscopic traffic simulation.

(1) Overview of VISSIM

The microscopic traffic simulation software used is versatile VISSIM which can evaluate traffic congestion and travel time.

1) Sale Agency

- PTV Planung Transport Verkehr AG（Germany）

2) Feature

- VISSIM is used in more than 80 countries in the world.
- Basic Car-Following Model was developed at Technical University of Karlsruhe
- This simulation model can simulate different traffic conditions that are globally acceptable.

3) Driving Behavior

The VISSIM simulates the traffic flow by moving "driver-vehicle-units" through the network. Every driver with his specific behavior or characteristics is assigned to a specific vehicle. As a consequence, the driving behavior corresponds to the technical capabilities of his vehicle. Attributes characterizing each driver-vehicle unit can be discriminated into three categories:

- Technical specification of the vehicle,:
- Length
- Maximum speed
- Potential acceleration
- Behavior of driver-vehicle units,:
- Psycho-physical sensitivity thresholds of the driver (ability to estimate, aggressiveness)
- Acceleration based on current speed and driver's desired speed
- Interdependence of driver-vehicle units,:
- Reference to leading and following vehicles on own and adjacent travel lanes
- Reference to current link and next intersection
- Reference to next traffic signal

(2) Construction of Simulation Model

The construction of simulation model is shown as follows.

1) Target Area

Target area of the microscopic traffic simulation is shown in Figure 19.3.4-1.

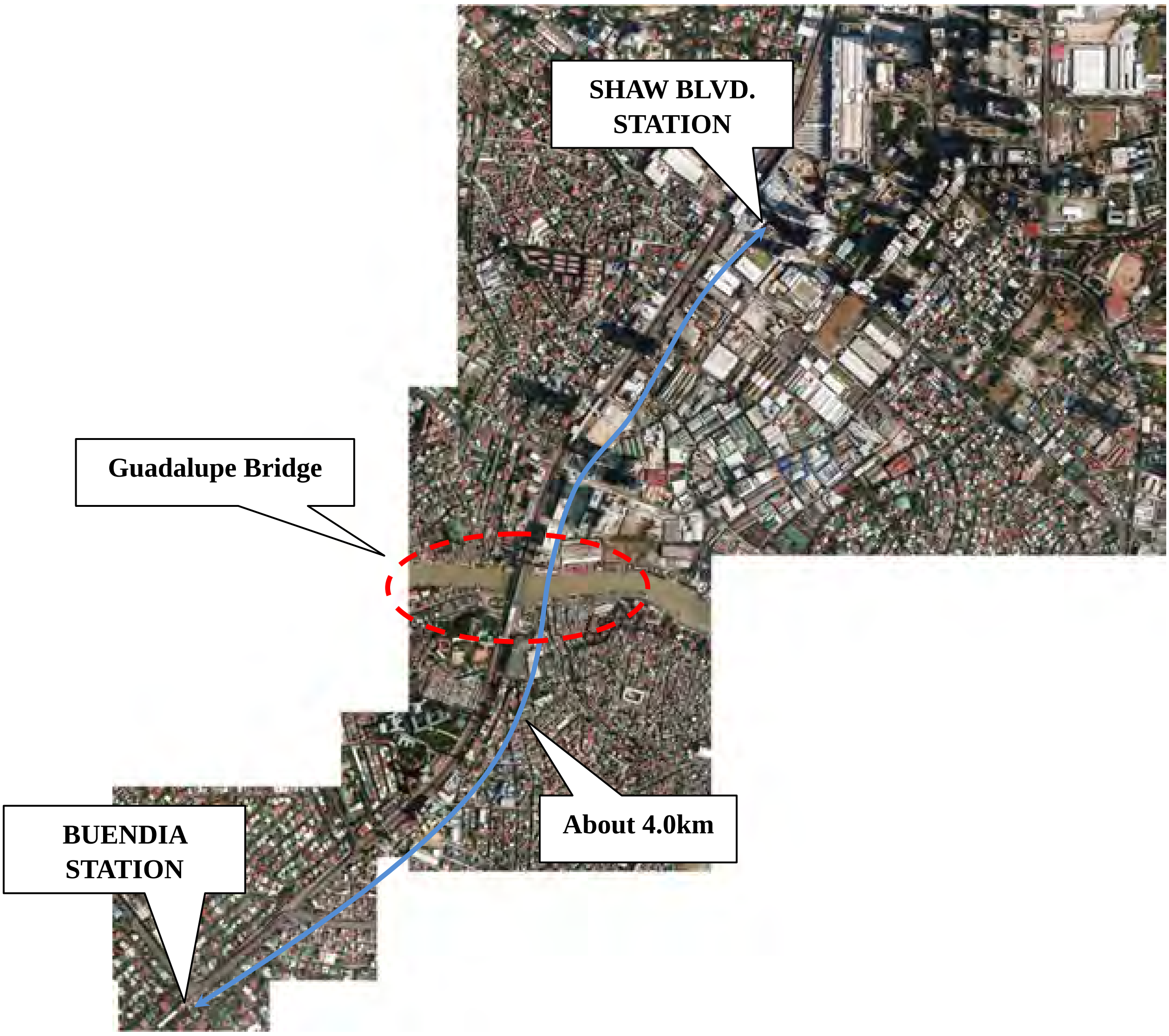


Figure 19.3.4-1 Target Area of Microscopic Traffic Simulation

2) Time Period

The simulation about the traffic condition was conducted in the morning peak and in the evening peak.

- Morning peak : 7:00~9:00
- Evening peak : 17:00~19:00

3) Input Data

a) Geometric structure

- The number of lanes and a lane width according to the present traffic condition.
- A priority rule, a stop position, etc. in a ramp merging section.

b) Traffic volume

- Five types of vehicle classification (Passenger car, Heavy truck, Bus, Motor Cycle, Jeepney).
- Average speed for every vehicle type (For example, Passenger car was input 60km/h).
- The simulated traffic volume was validated by the result of the traffic count survey at Guadalupe Bridge.

c) Others

- A bus stop and a jeepney stop were included to the present traffic condition.

4) Output Data

- Traffic volume
- Average speed
- Average travel time

(3) Verification of the Simulation Model

Verification of the simulation model was validated by the traffic volume and the average speed.

1) Morning Peak

a) Traffic volume

Traffic volume was validated by the comparison of the result of the traffic count survey and a microscopic traffic simulation (Figure 19.3.4-2).

The multiple correlation coefficient is 0.9 or more as shown in Figure 19.3.4-3. Result of the simulation is the same as the present traffic condition.

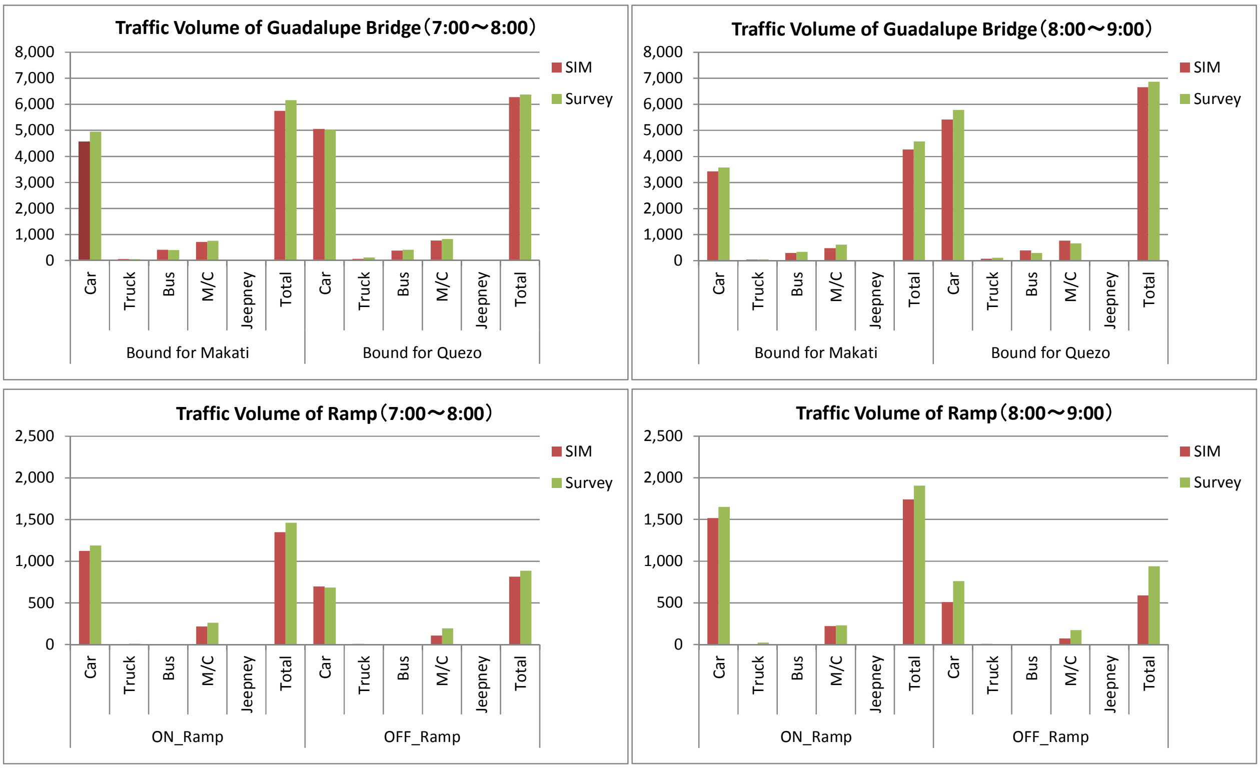


Figure 19.3.4-2 Comparison of Traffic Volume (Morning Peak)

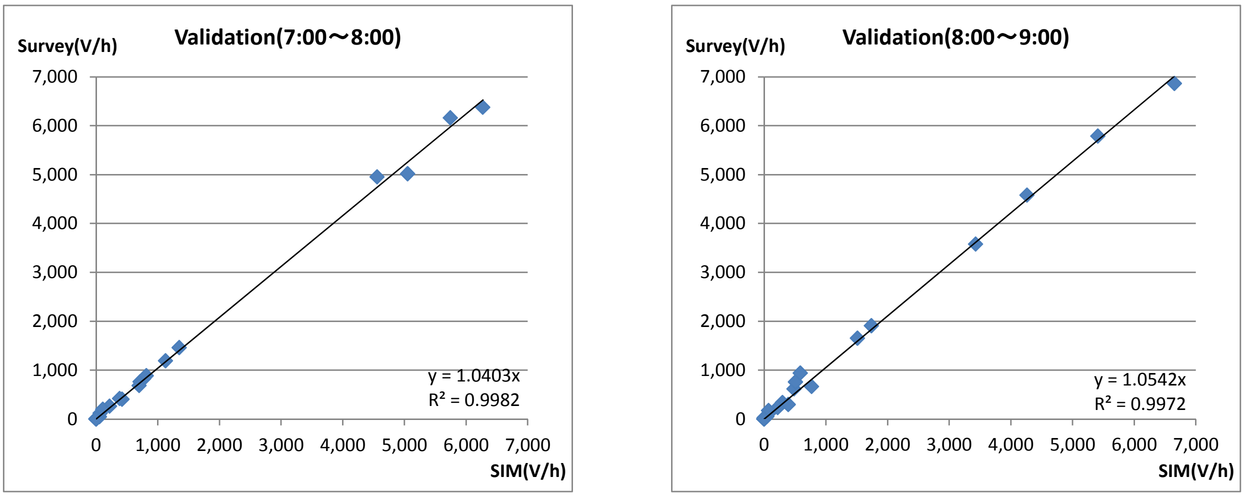
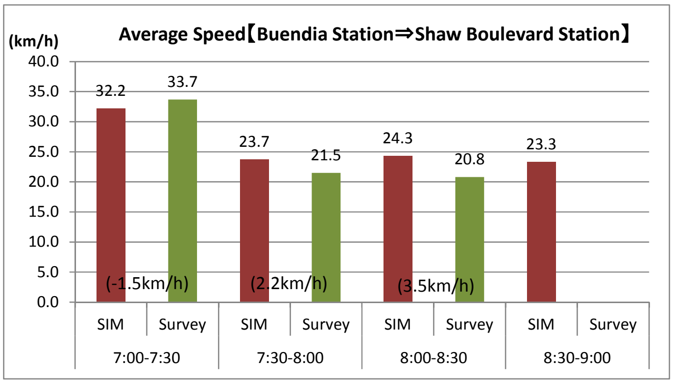


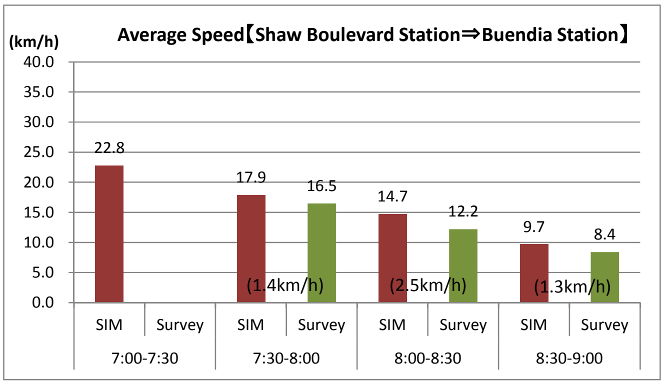
Figure 19.3.4-3 Verification of the Simulation Model (Traffic Volume during Morning Peak)

b) Average speed-1

- The average speed from Buendia Station to Shaw Boulevard Station was validated by the comparison of the result of the GPS survey and a microscopic traffic simulation.
- The average speed difference is less than 5km/h as shown in Figure 19.3.4-4. Result of the simulation is the same as the present traffic condition.



【Northbound (Bound to Quezon City)】



【Southbound (Bound to Makati City)】

Figure 19.3.4-4 Comparison of the Travel Speed (Average speed-1, Morning Peak)

c) Average speed-2

Detailed result of the GPS survey and the simulation is shown as follows (Figure 19.3.4-5).

(I) Northbound (Bound to Quezon City)

- Traffic congestion occurs before Guadalupe Bridge by longtime queuing of buses and jeepneys between 7:00 and 9:00.
- Result of the simulation is the same as the present traffic condition.

(II) Southbound (Bound to Makati City)

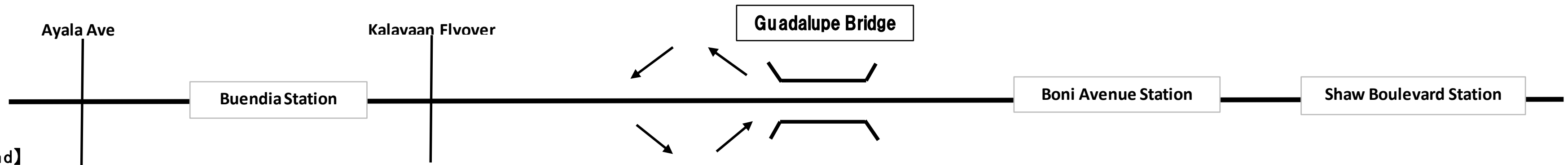
- Partial traffic congestion occurs between 7:00 and 8:00 at the bus stop of Boni Avenue Station and the influence at Kalayaan Flyover.
- There is no traffic at Guadalupe Bridge between 7:00 and 8:00.
- There is traffic congestion from Kalayaan Flyover extended to Guadalupe Bridge between 8:00 and 9:00.
- Result of the simulation is the same as the present traffic condition.

The comparative result of survey and simulation(AM peak)

Bound to Makati City

【Southbound】

SIM	8:30-9:00	7.0		7.2	7.9	9.4	9.1
	8:00-8:30	11.3		12.7	15.4	16.9	16.8
	7:30-8:00	13.0		20.9	41.9	19.5	20.7
	7:00-7:30	19.7		43.3	46.6	20.0	21.6
Survey	8:00～9:00	18.6	9.7	5.3	7.4	5.3	13.3
		16.9	10.0	7.6	15.0	10.8	38.9
	7:00～8:00	16.4	17.1	42.9	41.8	11.7	13.5



【Northbound】

Survey	7:00～8:00	33.9	20.7	35.6	39.4	43.6
		33.4	18.7	14.9	43.0	39.5
	8:00～9:00	20.9	18.1	13.6	52.5	44.5
SIM	7:00-7:30	27.1		22.3	45.6	45.5
	7:30-8:00	15.4		17.8	45.6	45.3
	8:00-8:30	15.4		18.2	44.7	44.9
	8:30-9:00	14.9		17.6	43.8	44.6

【km/h】

Bound to Quezon City

0～20km/h
20～30km/h
30km/h～

Figure 19.3.4-5 Comparison of the Travel Speed (Average speed-2, Morning Peak)

2) Evening Peak

a) Traffic volume

Traffic volume was validated by the comparison of the result of the traffic count survey and a microscopic traffic simulation (Figure 19.3.4-6).

The multiple correlation coefficient is 0.9 or more as shown in Figure 19.3.4-7. The result of the simulation is the same as the present traffic condition.

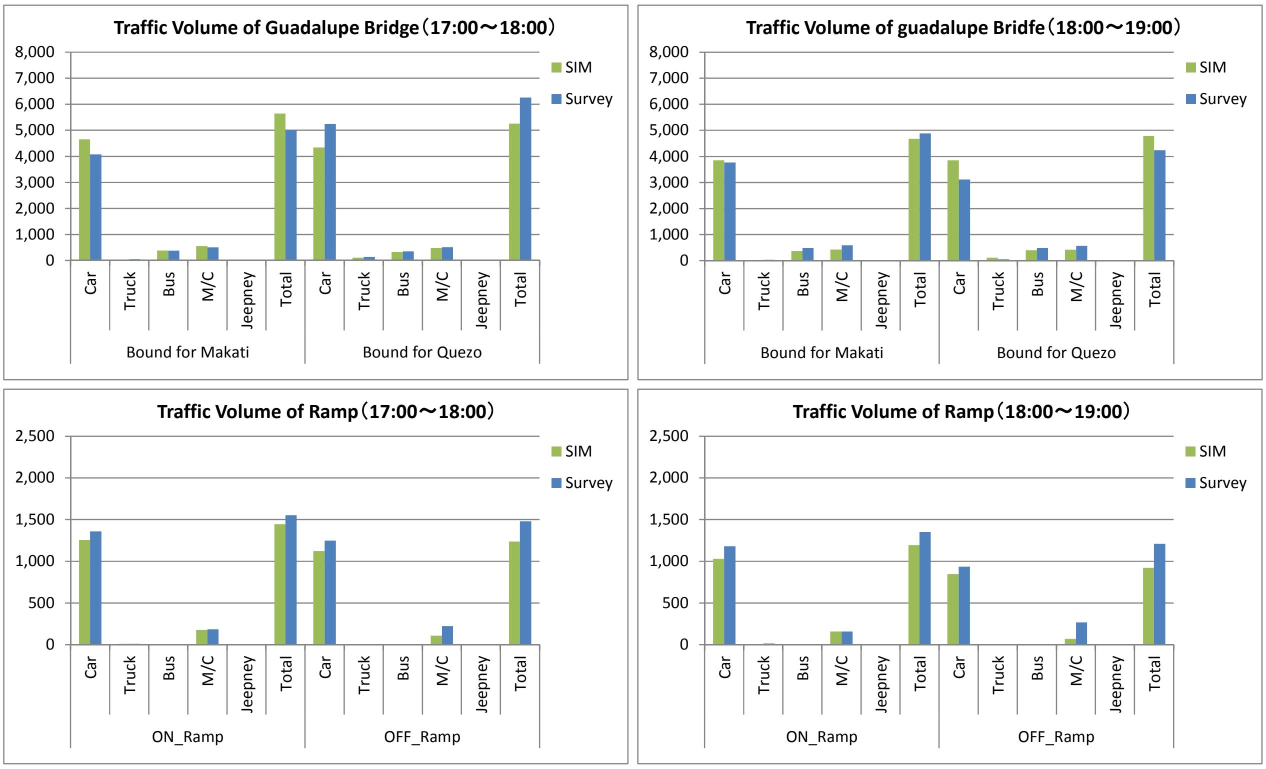


Figure 19.3.4-6 Comparison of Traffic Volume (Evening Peak)

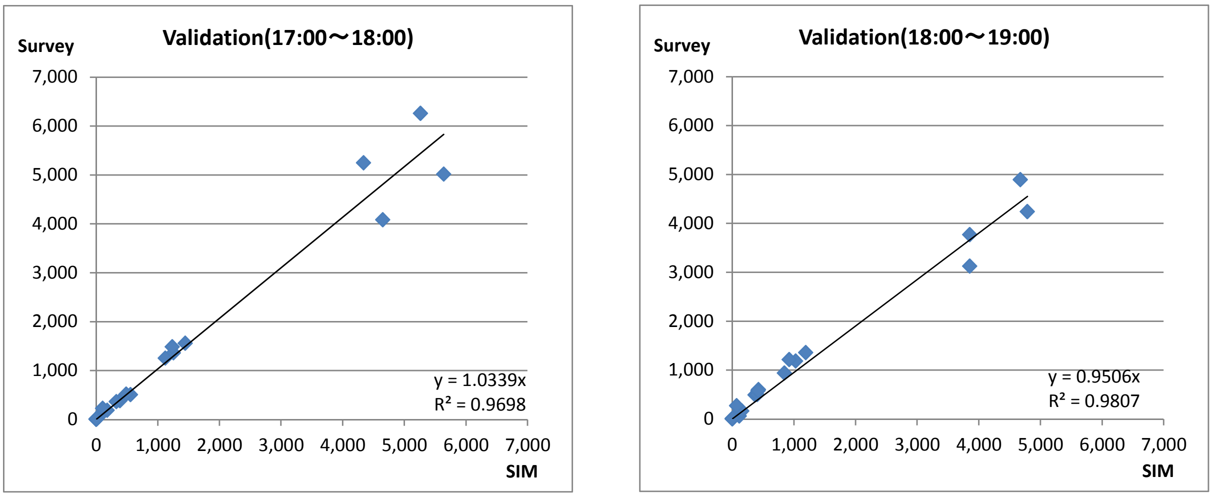
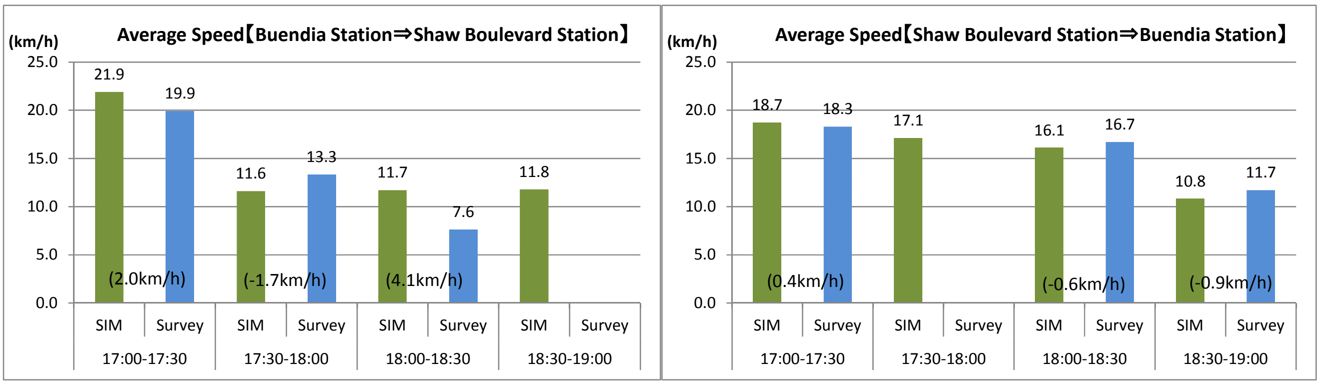


Figure 19.3.4-7 Verification of the Simulation model (Traffic Volume, Evening Peak)

b) Average speed-1

Average speed from Buendia Station to Shaw Boulevard Station was validated by the comparison of the result of the GPS survey and a microscopic traffic simulation.

The average speed difference is less than 5km/h as shown in Figure 19.3.4-8. The result of the simulation is the same as the present traffic condition.



【Northbound (Bound to Quezon City)】 【Southbound (Bound to Makati City)】

Figure 19.3.4-8 Comparison of the Travel Speed (Average Speed-1, Evening)

c) Average speed-2

Detailed result of the GPS survey and the simulation is as follows (Figure 19.3.4-9).

(I) Northbound (Bound to Quezon City)

- Traffic congestion occurs by longtime queuing of buses and jeepneys before Guadalupe Bridge between 17:00 and 18:00.
- Traffic congestion occurs from the bottleneck point after Guadalupe Bridge between 17:00 and 18:00.
- Traffic congestion which occurs from the bottleneck point after Guadalupe Bridge was extended to Guadalupe Bridge between 18:00 and 19:00.
- Result of the simulation is the same as the present traffic condition.

(II) Southbound (Bound to Makati City)

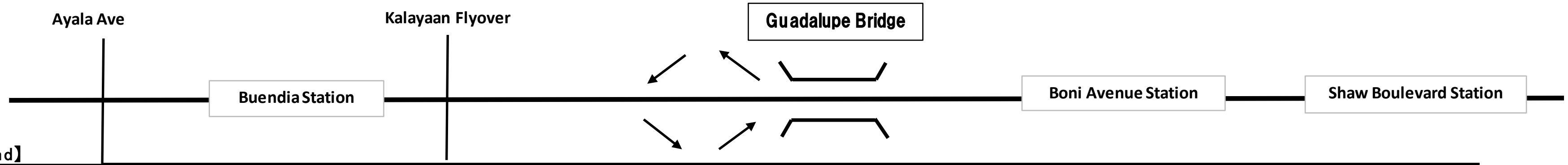
- Partial traffic congestion occurs at a bus stop at Boni Avenue Station and the influence of Kalayaan Flyover between 17:00 and 18:00.
- There is no traffic congestion at Guadalupe Bridge between 17:00 and 18:00.
- There is traffic congestion from Kalayaan Flyover extended to Guadalupe Bridge between 18:00 and 19:00.
- Result of the simulation is the same as the present traffic condition.

The comparative result of survey and simulation(PM peak)

Bound to Makati City ←

【Southbound】

SIM	18:30-19:00	7.2		8.1	9.7	11.4	11.0
	18:00-18:30	11.3		20.3	22.6	18.3	17.6
	17:30-18:00	11.8		46.5	35.3	20.8	23.2
	17:00-17:30	13.8		47.2	44.1	19.7	18.7
Survey	18:00～19:00	17.1	7.9	9.0	10.3	17.9	50.6
		13.5	12.1	32.7	28.7	18.9	39.9
	17:00～18:00	9.9	23.0	24.5	36.6	13.2	32.3



【Northbound】

Survey	17:00～18:00	13.5	17.2	20.8	39.4	18.2
		14.4	17.0	12.2	16.6	12.5
	18:00～19:00	6.2	10.3	5.9	5.4	10.3
SIM	17:00-17:30	20.8		19.5	23.4	20.1
	17:30-18:00	9.5		10.8	13.6	11.3
	18:00-18:30	9.4		11.1	13.7	11.5
	18:30-19:00	9.9		11.4	14.1	11.2

【km/h】

→ Bound to Quezon City

0～20km/h
20～30km/h
30km/h～

Figure 19.3.4-9 Comparison of the Travel Speed (Average speed-2)

19.3.5 Influence of the Lane Reduction

(1) Analysis Flow

The analysis method of the influence of lane reduction was conducted as follows (Figure 19.3.5-1).

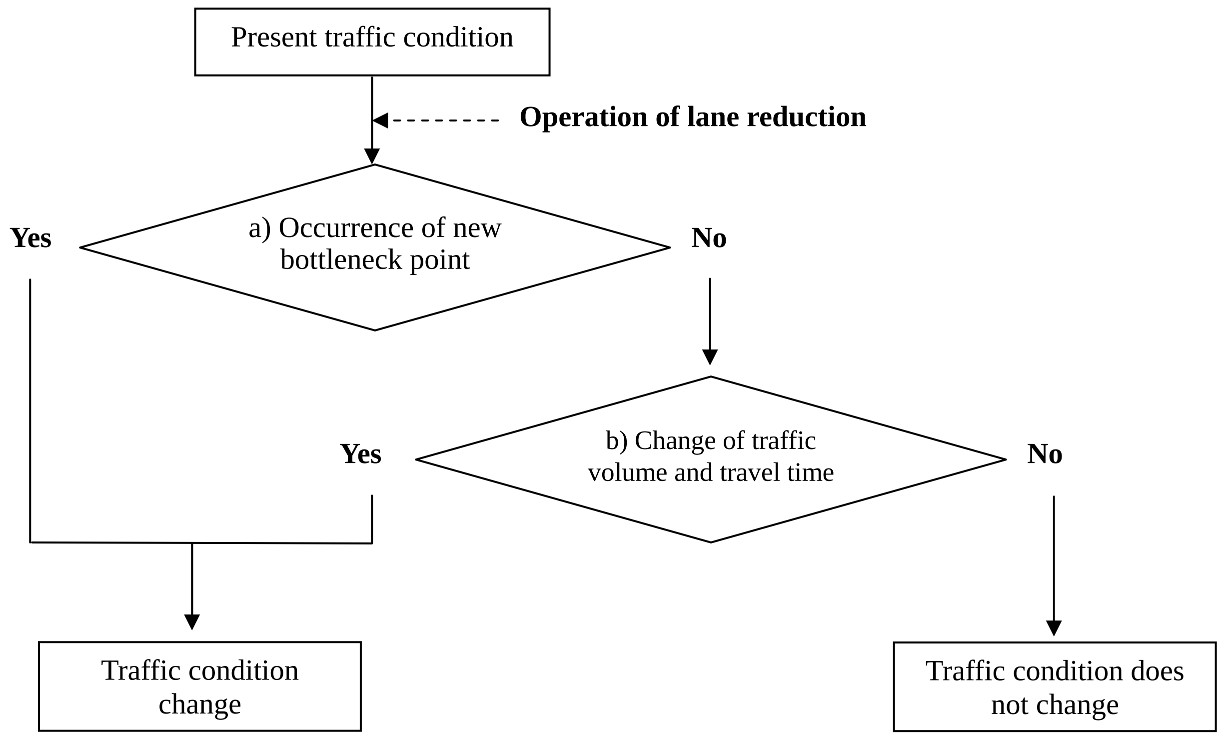


Figure 19.3.5-1 Flow of Analysis

(2) Geometric Structure

1) 4-Lanes

a) Northbound (Bound to Quezon City)

【5-lanes (Present traffic condition)】

【4-lanes】

Figure 19.3.5-2 Geometric Structure of 4-Lanes

b) Southbound (Bound to Makati City)

【5-lanes (Present traffic condition)】

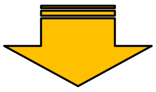
【4-lanes】

Figure 19.3.5-3 Geometric Structure of 4-Lanes

2) 3-Lanes

a) Northbound (Bound to Quezon City)

【5-lanes (Present traffic condition)】



【3-lanes】

Figure 19.3.5-4 Geometric Structure of 3-Lanes

b) Southbound (Bound to Makati City)

【5-lanes (Present traffic condition)】

【3-lanes】

Figure 19.3.5-5 Geometric Structure of 3-Lanes

(3) Traffic Condition during Rehabilitation Works (Morning peak)

1) Bottleneck Point

The result of analysis is shown as follows (Figure 19.3.5-6).

a) 4-lanes

(I) Northbound (Bound to Quezon City)

- In case of the reduction of the number of lanes from 5 to 4, the bottleneck point would be before the bridge which is the same as the present traffic condition.

(II) Southbound (Bound to Makati City)

- In case of the reduction of the number of lanes from 5 to 4, low speed occurred before Guadalupe Bridge.
- However, the bottleneck point was before the bridge, the same as the present traffic condition.

b) 3-lanes

(I) Northbound (Bound to Quezon City)

- In case of the reduction of the number of lanes from 5 to 3, the bottleneck point would be before the bridge which is the same as the present traffic condition.

(II) Southbound (Bound to Makati City)

- In case of the reduction of the number of lanes from 5 to 3, the bottleneck point would change from Kalayaan Flyover to Guadalupe Bridge.
- The traffic condition of 3-lanes would be changed from the present traffic condition.

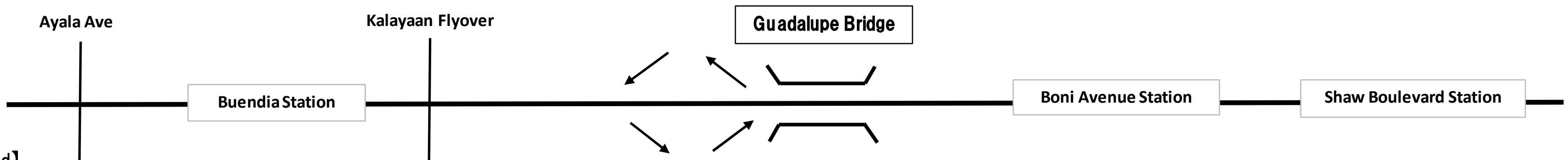
Bottleneck point

Average Speed Comparison in case of No. of lanes(AM peak)

Bound to Makati City

【Southbound】

SIM (3lane)	8:30-9:00	49.3	41.9	22.2	12.3	11.9
	8:00-8:30	50.9	40.8	20.4	11.4	11.4
	7:30-8:00	49.2	40.0	21.2	11.8	11.4
	7:00-7:30	47.5	40.2	21.0	11.8	11.7
SIM (4lane)	8:30-9:00	6.9	9.5	9.6	9.8	8.9
	8:00-8:30	11.3	18.7	17.8	18.3	17.0
	7:30-8:00	14.9	34.9	27.2	21.5	21.1
	7:00-7:30	21.1	38.1	26.2	22.0	21.6
SIM (5lane)	8:30-9:00	7.0	7.2	7.9	9.4	9.1
	8:00-8:30	11.3	12.7	15.4	16.9	16.8
	7:30-8:00	13.0	20.9	41.9	19.5	20.7
	7:00-7:30	19.7	43.3	46.6	20.0	21.6



【Northbound】

SIM (5lane)	7:00-7:30	27.1	22.3	45.6	45.5
	7:30-8:00	15.4	17.8	45.6	45.3
	8:00-8:30	15.4	18.2	44.7	44.9
	8:30-9:00	14.9	17.6	43.8	44.6
SIM (4lane)	7:00-7:30	25.7	21.4	39.1	47.2
	7:30-8:00	15.6	18.2	39.6	48.2
	8:00-8:30	15.6	18.1	38.2	48.0
	8:30-9:00	14.7	17.2	38.1	46.5
SIM (3lane)	7:00-7:30	6.9	8.3	43.0	52.4
	7:30-8:00	6.9	7.8	41.4	52.4
	8:00-8:30	7.7	8.6	40.8	51.7
	8:30-9:00	9.0	9.8	40.5	51.9

【km/h】

Bound to Quezon City

Bottleneck point

0～20km/h
20～30km/h
30km/h～

Figure 19.3.5-6 Average Speed Comparison in Case of No. of Lanes (Guadalupe Bridge, Morning Peak)

2) Comparison of Travel Time and Traffic Volume

The travel time and traffic volume of the same case as the present bottleneck point is shown as follows.

a) 4-lanes

(I) Northbound (Bound to Quezon City)

- The travel time and the traffic volume are the same as the present traffic condition as shown in Figure 19.3.5-7.

(II) Southbound (Bound to Makati City)

- The travel time and the traffic volume are the same as the present traffic condition as shown in Figure 19.3.5-8.

b) 3lanes

(I) Northbound (Bound to Quezon City)

- In case of the reduction of the number of lanes from 5 to 3, the travel time increased sharply and also the traffic volume of Guadalupe Bridge decreased sharply as shown in Figure 19.3.5-7.

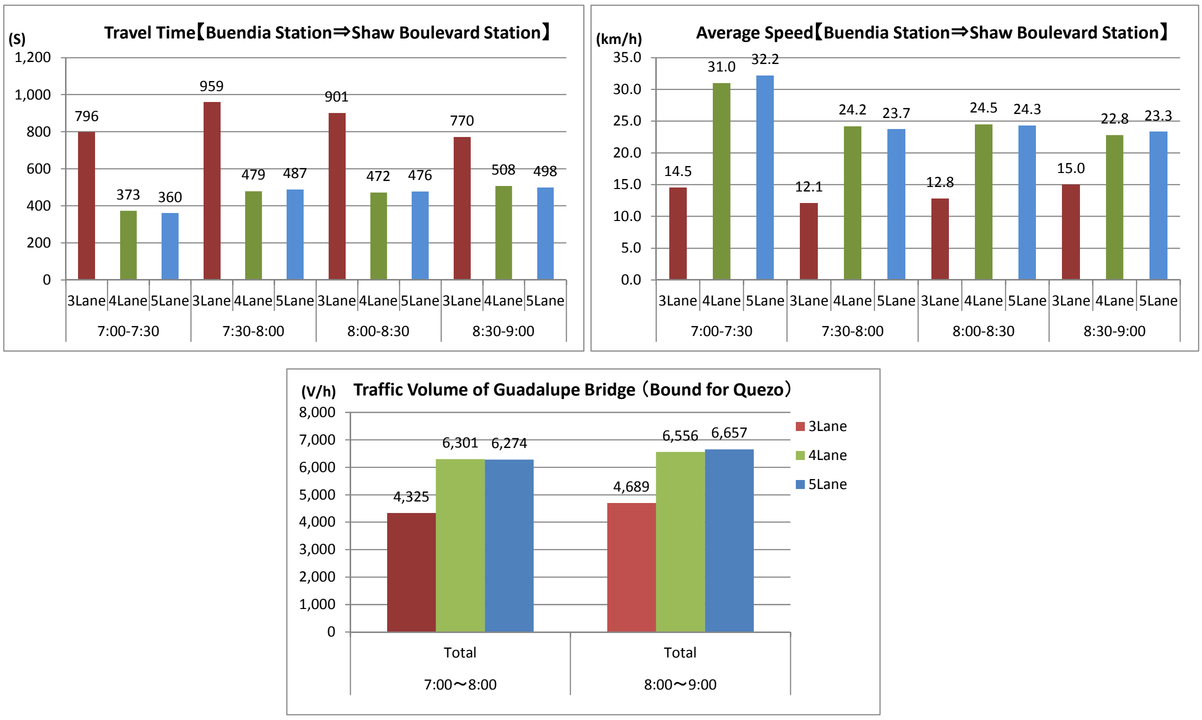


Figure 19.3.5-7 Traffic Condition Comparison in Case of No. of Lanes-Guadalupe Bridge (Northbound (Bound to Quezon City))

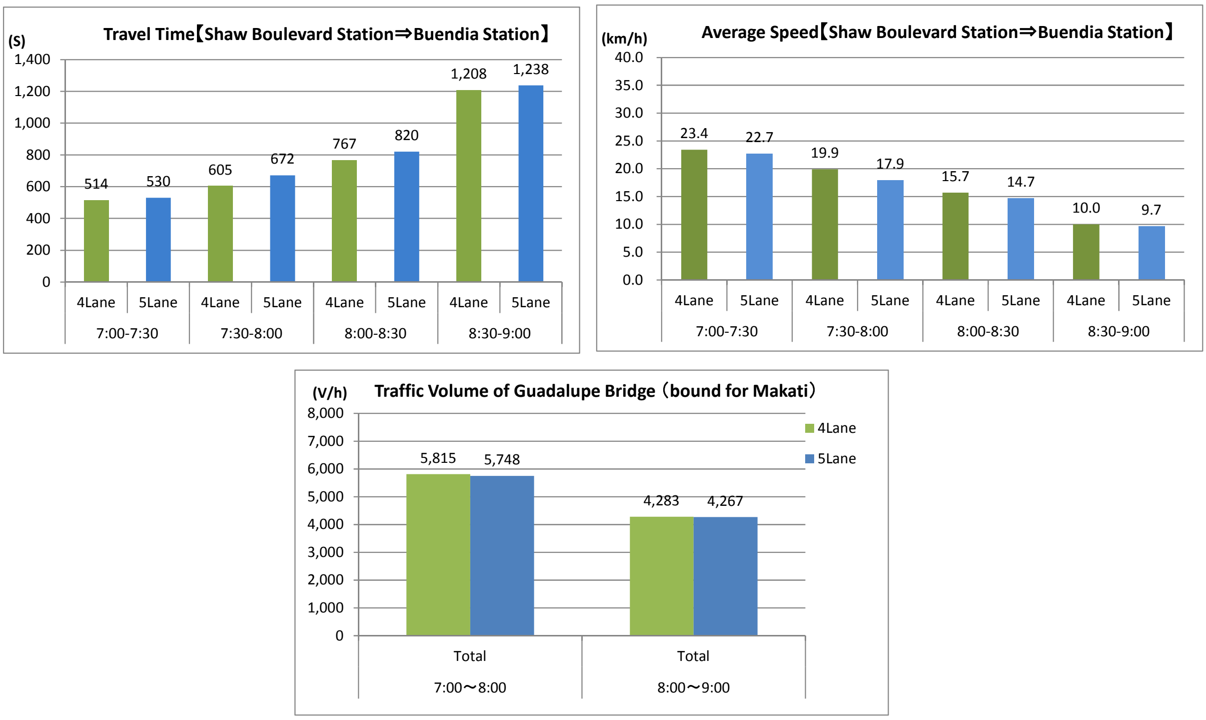


Figure 19.3.5-8 Traffic Condition Comparison in Case of No. of Lanes-Guadalupe Bridge (Southbound (Bound to Makati City))

3) Obtained Result and Consideration

a) Northbound (Bound to Quezon City)

- In case of the reduction of the number of lanes from 5 to 4; the bottleneck point, the travel time and the traffic volume of Guadalupe Bridge are the same as the present traffic condition.
- In case of the reduction of the number of lanes from 5 to 3; the bottleneck point is the same as the present traffic condition. However, travel time and traffic volume are changed.
- The result of analysis, in case of the reduction of the number of lanes from 5 to 4, the traffic condition is the same as the present traffic condition. However, in case of the reduction of the number of lanes from 5 to 3, the traffic condition would change from the present traffic condition.

b) Southbound (Bound to Makati City)

- In case of the reduction of the number of lanes from 5 to 4; the bottleneck point, the travel time and the traffic volume of Guadalupe Bridge are the same as the present traffic condition.
- In case of the reduction of the number of lanes from 5 to 3; the bottleneck point would change from Kalayaan Flyover to Guadalupe Bridge.
- The result of analysis, in case of the reduction of the number of lanes from 5 to 4, the traffic condition is the same as the present traffic condition. However, in the case of the reduction of the number of lanes from 5 to 3, the traffic condition would change from the present traffic condition.

4) Traffic Condition of 3-Lanes

In case of the reduction of the number of lanes from 5 to 3, the traffic condition would change from the present traffic condition. The traffic volume and the traffic congestion of the traffic condition of 3-lanes are shown as follows.

a) Northbound (Bound to Quezon City)

【Traffic volume 】

Figure 19.3.5-9 shows change of traffic volume.

- Traffic volume reduced by 2,000(Veh/hr)【7:00-8:00】
- Traffic volume reduced by 2,000(Veh/hr)【8:00-9:00】

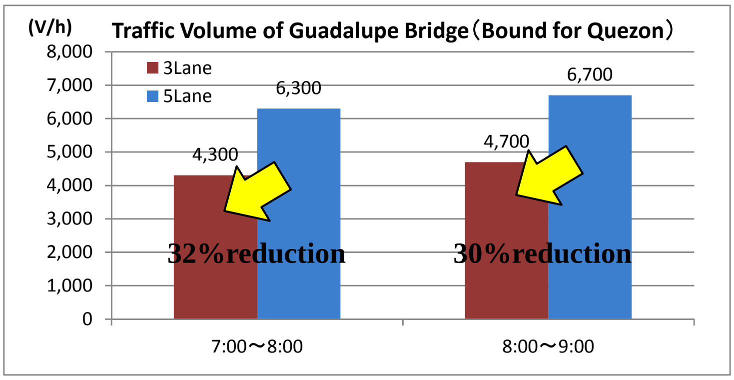


Figure 19.3.5-9 Traffic Volume at Guadalupe Bridge in Case of 3-Lanes (Northbound (Bound to Quezon City))

【Queue length】

- Bottleneck point is the same as the present condition
- The increase of queue length【7:00-8:00】:2,000(Veh/hr)×7.5(m)÷5(m)=3,000m
- The increase of queue length【8:00-9:00】:2,000(Veh/hr)×7.5(m)÷5(m)=3,000m

※The number of lanes:5-lanes

※Average headway:7.5m

【Obtained Result and Consideration】

In case of the reduction of the number of lanes from 5 to 3, the traffic volume of Guadalupe Bridge decreased by about 30% in the morning peak of 2 hours.

As a result, traffic congestion was extended to 6 km in the morning peak that is 2 hours as compared with the present traffic condition.

b) Southbound (Bound to Makati City)

【Traffic volume 】

Figure 19.3.5-10 shows change of traffic volume.

Traffic volume reduced by 1,300(Veh/hr)【7:00-8:00】

- Traffic volume is the same as the present condition【8:00-9:00】

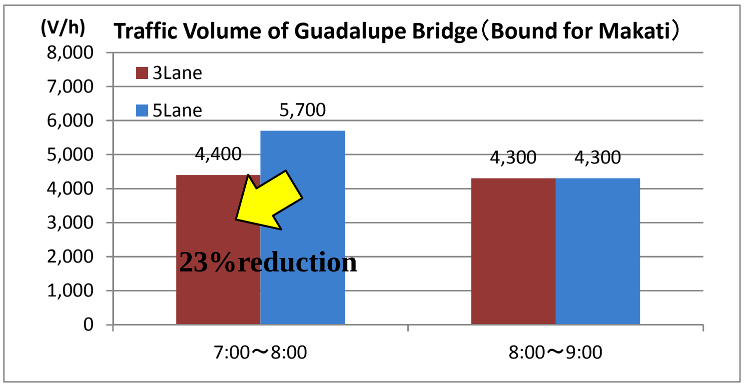


Figure 19.3.5-10 Traffic Volume of Guadalupe Bridge in Case of 3-Lanes (Southbound (Bound to Makati City))

【Queue length】

Bottleneck point would change from Kalayaan Flyover to Guadalupe Bridge.

There would be an increase of queue length【7:00-8:00】:1,300(Veh/hr)×7.5(m)÷5(m)=1,950m

※The number of lanes:5-lanes

※Average headway:7.5m

【Obtained Result and Consideration】

<u>In case of the reduction of the number of lanes from 5 to 3,</u> the bottleneck point would change from Kalayaan Flyover to Guadalupe Bridge. The traffic volume at Guadalupe Bridge would be reduced by 20% between 7:00 and 8:00. It is the same as the present traffic condition between 8:00 and 9:00.

The reason for the difference in the change of traffic volume is the influence of the present traffic congestion. The present traffic congestion is not extended to Guadalupe Bridge between 7:00 and 8:00. However, it is extended to Guadalupe Bridge between 8:00 and 9:00. Therefore, the traffic volume of Guadalupe Bridge decreases in the present traffic condition between 8:00 and 9:00. However, the traffic volume of Guadalupe Bridge in 3-lanes did not change in the morning peak of 2 hours.

As a result, the traffic congestion was extended 2 km in the morning peak by 2 hours as compared with the present traffic condition.

(4) Traffic Condition during Rehabilitation Works (Evening peak)

1) Bottleneck Point

The result of analysis is shown as follows (Figure 19.3.5-11).

a) 4-lanes

(I) Northbound (Bound to Quezon City)

- In case of the reduction of the number of lanes from 5 to 4, the bottleneck point is before the bridge, the same as the present traffic condition.

(II) Southbound (Bound to Makati City)

- In case of the reduction of the number of lanes from 5 to 4, low speed occurred before Guadalupe Bridge.
- However, the bottleneck point is still before the bridge, the same as the present traffic condition.

b) 3-lanes

(I) Northbound (Bound to Quezon City)

- In case of the reduction of the number of lanes from 5 to 3, the bottleneck point would change from the point before Guadalupe Bridge to Guadalupe Bridge.
- The traffic condition of 3-lanes would change from the present traffic condition.

(II) Southbound (Bound to Makati City)

- In case of the reduction of the number of lanes from 5 to 3, the bottleneck point would change from Kalayaan Flyover to Guadalupe Bridge.
- The traffic condition of 3-lanes would change from the present traffic condition.

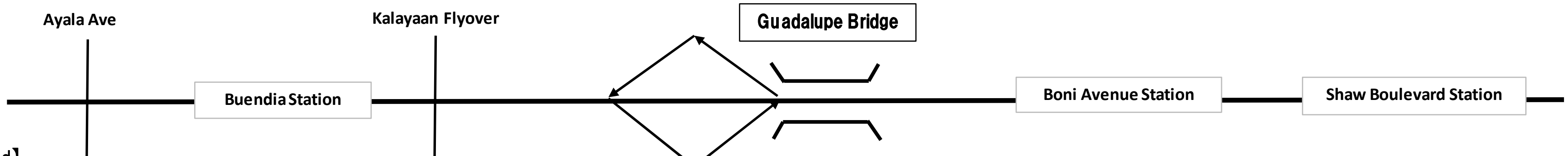
Bottleneck points

Average Speed Comparison in case of No. of lanes(PM peak)

Bound to Makati City

【Southbound】

SIM (3lane)	18:30-19:00	30.4	36.9	21.6	11.9	11.5
	18:00-18:30	51.5	36.5	20.7	11.7	11.5
	17:30-18:00	52.6	35.3	20.9	11.6	11.2
	17:00-17:30	52.3	38.1	20.6	11.7	11.6
SIM (4lane)	18:30-19:00	7.7	12.5	12.5	12.2	11.1
	18:00-18:30	14.3	35.3	20.6	19.1	16.8
	17:30-18:00	12.0	30.3	18.7	20.7	22.1
	17:00-17:30	11.6	24.7	19.6	20.6	21.6
SIM (5lane)	18:30-19:00	7.2	8.1	9.7	11.4	11.0
	18:00-18:30	11.3	20.3	22.6	18.3	17.6
	17:30-18:00	11.8	46.5	35.3	20.8	23.2
	17:00-17:30	13.8	47.2	44.1	19.7	18.7



【Northbound】

SIM (5lane)	17:00-17:30	20.8	19.5	23.4	20.1
	17:30-18:00	9.5	10.8	13.6	11.3
	18:00-18:30	9.4	11.1	13.7	11.5
	18:30-19:00	9.9	11.4	14.1	11.2
SIM (4lane)	17:00-17:30	22.6	22.0	30.2	26.1
	17:30-18:00	8.9	10.5	14.7	12.0
	18:00-18:30	8.7	10.3	14.6	12.0
	18:30-19:00	9.4	10.7	15.3	12.1
SIM (3lane)	17:00-17:30	8.6	9.7	40.8	51.9
	17:30-18:00	7.6	8.5	41.4	51.3
	18:00-18:30	8.7	9.3	41.6	53.1
	18:30-19:00	11.0	11.2	40.8	53.1

【km/h】

Bound to Quezon City

Bottleneck points

0～20km/h
20～30km/h
30km/h～

Figure 19.3.5-11 Average Speed Comparison in Case of No. of Lanes (Guadalupe Bridge, Evening Peak)

2) Comparison of Travel Time and Traffic Volume

Travel time and the traffic volume of the same case as the present bottleneck point is shown as follows.

a) 4-lanes

(I) Northbound (Bound to Quezon City)

- The travel time and the traffic volume are the same as the present traffic condition as shown in Figure 19.3.5-12.

(II) Southbound (Bound to Makati City)

- The travel time and the traffic volume are the same as the present traffic condition as shown in Figure 19.3.5-13.

Figure 19.3.5-12 Traffic Condition Comparison in Case of No. of Lanes-Guadalupe Bridge (Northbound (Bound to Quezon City))

Figure 19.3.5-13 Traffic Condition Comparison in Case of No. of Lanes-Guadalupe Bridge (Southbound (Bound to Makati City))

3) Obtained Result and Consideration

a) Northbound (Bound to Quezon City)

- In case of the reduction of the number of lanes from 5 to 4; the bottleneck point, the travel time and the traffic volume of Guadalupe Bridge are the same as the present traffic condition.
- In case of the reduction of the number of lanes from 5 to 3; the bottleneck point would change from the point before Guadalupe Bridge to Guadalupe Bridge.
- The result of analysis, in case of the reduction of the number of lanes from 5 to 4; the traffic condition is the same as the present traffic condition. However, in case of the reduction of the number of lanes from 5 to 3, the traffic condition would change from the present traffic condition.

b) Southbound (Bound to Makati City)

- In case of the reduction of the number of lanes from 5 to 4; the bottleneck point, the travel time and the traffic volume of Guadalupe Bridge are the same as the present traffic condition.
- In case of the reduction of the number of lanes from 5 to 3, the bottleneck point would change from Kalayaan Flyover to Guadalupe Bridge.
- The result of analysis, in case of the reduction of the number of lanes from 5 to 4, the traffic condition is the same as the present traffic condition. However, in the case of the reduction of the number of lanes from 5 to 3, the traffic condition would change from the present traffic condition.

4) Traffic Condition of 3-Lanes

In the case of the reduction of the number of lanes from 5 to 3; the traffic condition would change from the present traffic condition. The traffic volume and traffic congestion of the traffic condition of 3-lanes are as follows.

a) Northbound (Bound to Quezon City)

【Traffic volume 】

Figure 19.3.5-14 shows the change of traffic volume.

- Traffic volume reduced by 700(Veh/hr)【17:00-18:00】
- Traffic volume reduced by 500(Veh/hr)【18:00-19:00】

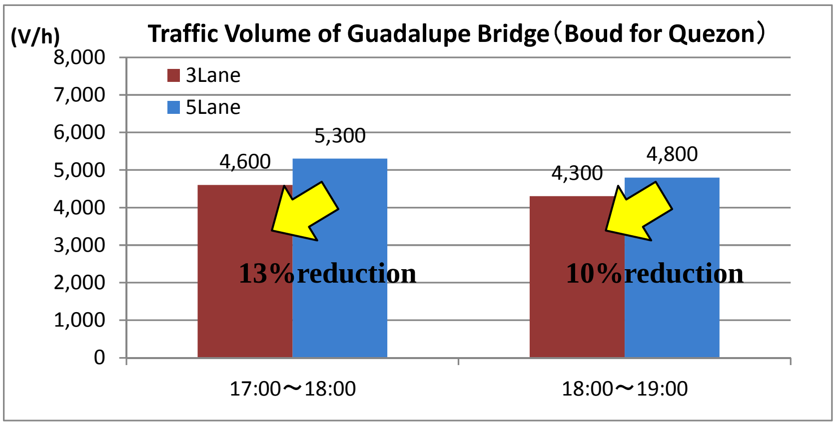


Figure 19.3.5-14 Traffic Volume of Guadalupe Bridge in Case of 3-Lanes (Northbound (Bound to Quezon City))

【Queue length】

- Bottleneck point would change from the point before Guadalupe Bridge to Guadalupe Bridge.
- The increase of queue length【7:00-8:00】:700(Veh/hr)×7.5(m)÷5(m)=1,050m
- The increase of queue length【7:00-8:00】:500(Veh/hr)×7.5(m)÷5(m)=750m

※The number of lanes:5-lanes

※Average headway:7.5m

【Obtained Result and Consideration】

I<u>n case of the reduction of the number of lanes from 5 to 3</u>; the bottleneck point would change from the point before Guadalupe Bridge to Guadalupe Bridge. The traffic volume of Guadalupe Bridge would be reduced by 10% between 17:00 and 19:00.

As a result, traffic congestion is extended by 2 km in the evening peak for 2 hours as compared with the present traffic condition.

b) Southbound (Bound to Makati City)

【Traffic volume 】

Figure 19.3.5-15 shows the change of traffic volume.

- Traffic volume is reduced by 1,300(Veh/hr)【17:00-18:00】
- Traffic volume is reduced by 300(Veh/hr)【18:00-19:00】

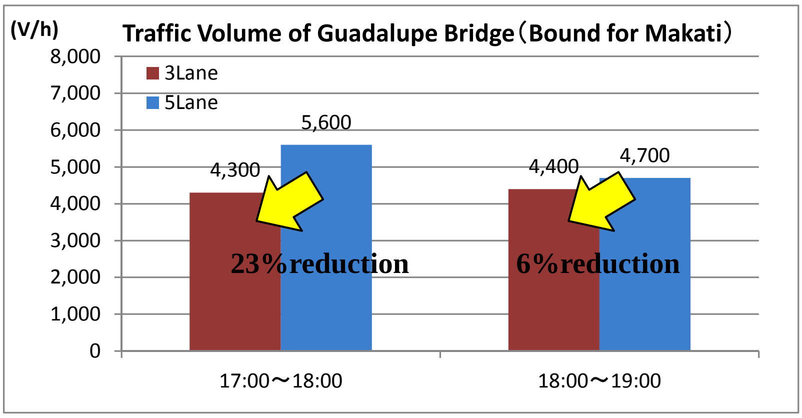


Figure 19.3.5-15 Traffic Volume of Guadalupe Bridge in Case of 3-Lanes (Southbound (Bound to Makati City))

【Queue length】

- Bottleneck point would change from Kalayaan Flyover to Guadalupe Bridge.
- The increase of queue length【17:00-18:00】:1,300(Veh/hr)×7.5(m)÷5(m)=1,950m
- The increase of queue length【18:00-19:00】:300(Veh/hr)×7.5(m)÷5(m)=450m

※The number of lanes:5-lanes

※Average headway:7.5m

【Obtained Result and Consideration】

In case of the reduction of the number of lanes from 5 to 3; the bottleneck point would change from Kalayaan Flyover to Guadalupe Bridge. The traffic volume of Guadalupe Bridge would decrease by 23% between 17:00 and 18:00 and would decrease by 6 % between 18:00 and 19:00.

The reason of the difference in change of traffic volume is the influence of the present traffic congestion. The present traffic congestion would not be extended to Guadalupe Bridge between 17:00 and 18:00. However, it would be extended to Guadalupe Bridge between 18:00 and 19:00. Therefore, the traffic volume of Guadalupe Bridge would decrease from the present traffic condition between 18:00 and 19:00. The traffic volume of Guadalupe Bridge in 3-lanes did not change in the evening peak for 2 hours.

As a result, traffic congestion is extended by 2.5 km in the evening peak for 2 hours as compared with the present traffic condition.

19.3.6 Result of the Traffic Analysis of Guadalupe Bridge

(1) Northbound (Bound to Quezon City)

■Morning peak

No. of lane	Traffic condition at Guadalupe Bridge			
5-lanes (Present condition)	Bottleneck point		Guadalupe Bridge	
	Traffic volume	7:00-8:00	6,300veh/hr	
		8:00-9:00	6,700veh/hr	
4-lanes	Bottleneck point		Guadalupe Bridge	・ Traffic condition is the same as the present traffic condition.
	Traffic volume	7:00-8:00	6,300veh/hr	
		8:00-9:00	6,600veh/hr	
3-lanes	Bottleneck point		Guadalupe Bridge	・ Traffic capacity is reduced by about 30%. ・ Traffic congestion is extended by 6 km.
	Traffic volume	7:00-8:00	4,300veh/hr	
		8:00-9:00	4,700veh/hr	

■Evening peak

No. of lane	Traffic condition at Guadalupe Bridge			
5-lanes (Present condition)	Bottleneck point		The point before Guadalupe Bridge	
	Traffic volume	17:00-18:00	5,300veh/hr	
		18:00-19:00	4,800veh/hr	
4-lanes	Bottleneck point		The point before Guadalupe Bridge	・ Traffic condition is the same as the present traffic condition.
	Traffic volume	17:00-18:00	5,300veh/hr	
		18:00-19:00	4,800veh/hr	
3-lanes	Bottleneck point		Guadalupe Bridge	・ Bottleneck point is changed. ・ Traffic capacity is reduced by about 10%. ・ Traffic congestion is extended by 2 km.
	Traffic volume	17:00-18:00	4,600veh/hr	
		18:00-19:00	4,300veh/hr	

(2) Southbound (Bound to Makati City)

■Morning peak

No. of lane	Traffic condition at Guadalupe Bridge			
5-lanes (Present condition)	Bottleneck point		Kalayaan Flyover	
	Traffic volume	7:00-8:00	5,700veh/hr	
		8:00-9:00	4,300veh/hr	
4-lanes	Bottleneck point		Kalayaan Flyover	・Traffic condition is the same as the present traffic condition.
	Traffic volume	7:00-8:00	5,800veh/hr	
		8:00-9:00	4,300veh/hr	
3-lanes	Bottleneck point		Guadalupe Bridge	・Bottleneck point is changed. ・Traffic capacity is reduced by about 20%. ・Traffic congestion is extended by 2 km.
	Traffic volume	7:00-8:00	4,400veh/hr	
		8:00-9:00	4,300veh/hr	

■Evening peak

No. of lane	Traffic condition at Guadalupe Bridge			
5-lanes (Present condition)	Bottleneck point		The point before Guadalupe Bridge	
	Traffic volume	17:00-18:00	5,600veh/hr	
		18:00-19:00	4,700veh/hr	
4-lanes	Bottleneck point		The point before Guadalupe Bridge	・Traffic condition is the same as the present traffic condition.
	Traffic volume	17:00-18:00	5,600veh/hr	
		18:00-19:00	4,800veh/hr	
3-lanes	Bottleneck point		Guadalupe Bridge	・Bottleneck point is changed ・Traffic capacity is reduced by about 6-23%. ・Traffic congestion is extended by 2.5 km.
	Traffic volume	17:00-18:00	4,300veh/hr	
		18:00-19:00	4,400veh/hr	

19.4 Traffic Analysis of Package C

19.4.1 Analysis of Traffic Congestion during Bridge Improvement

(1) Assumption

As mentioned in section 19.1, there will be no traffic restriction during construction outside Metro Manila. But it may be used as some two- way traffic alternating along a single lane in some work duration. The traffic analysis was done in case of a two-way traffic alternating along a single lane.

1) Traffic Growth Rate

The traffic growth rate used was the 2011 Atlas data of DPWH. This growth rate was estimated from the past traffic count result. Based on the growth rate, the traffic volume during construction in year 2018 was estimated as seen in Table 19.4.1-1.

Table 19.4.1-1 2011 DPWH Traffic Growth Rate

unit: %

Bridge Name	Province	MC/ Tricycle	Car	Jeepney	L-Bus	Truck
1st Mactan Br.	Cebu	2.28	2.70	2.28	2.28	2.17
Palanit Br.	N. Samar	2.21	2.51	2.21	2.21	2.15
Mawo Br.						
Liloan Br.	S.Leyte	2.25	2.61	2.25	2.25	2.18
Wawo Br.	Butuan	2.06	2.27	2.06	2.06	1.99

Note: MC: Motor Cycle
Source: DPWH ATLAS 2011

2) Traffic Restriction during Construction Stage

The existing number of lanes is two lanes. Although there will be no traffic restriction during construction, the traffic analysis was done basically for a two-way alternating traffic along a single lane except at 1st Mactan Bridge, shown in Table 19.4.1-2

Table 19.4.1-2 Assumed Traffic Restriction during Construction

Bridge Name	Existing No. of Lane	Traffic Analysis case of traffic restriction	Remarks
1st Mactan Br.	2	No Traffic Restriction	Sub-structure retrofitting only, traffic may not be affected.
Palanit Br.	2	Two-way traffic alternating along a single lane	
Mawo Br.	2		
Liloan Br.	2		
Wawo Br.	2		

Sourc: JICA Study Team

3) Traffic Capacity during Construction Stage

Traffic capacity during construction stage was assumed as follows.

- Traffic Capacity is 700 vehicles /hour for two-way traffic alternating along a single lane (Source: Road Construction Capacity of Tokyo Metropolitan Police, Japan)
- Traffic Capacity is converted as PCU: 840 PCU/hr (It is assumed that large vehicle occupancy rate is 20% in Tokyo. 700*1.2 = 840 PCU/hr.)
- Actually the capacity of a two-way traffic alternating along a single lane depends on the length. For these bridges lengths are not so long, thus, it is assumed to be as the same capacity.

(2) Traffic Restriction during Construction Stage

Figure 19.4.1-1 - Figure 19.4.1-4 show the hourly volume and hourly capacity.

- The entire hourly volume along these four (4) bridges are lower than capacity, thus, traffic congestion may not occur.
- Since morning and evening peak’s volume along Mawo Bridge and Wawo Bridge will be nearing capacity, it is recommended to avoid two- way alternating traffic along a single lane during peak hours for Mawo and Wawo Bridges.

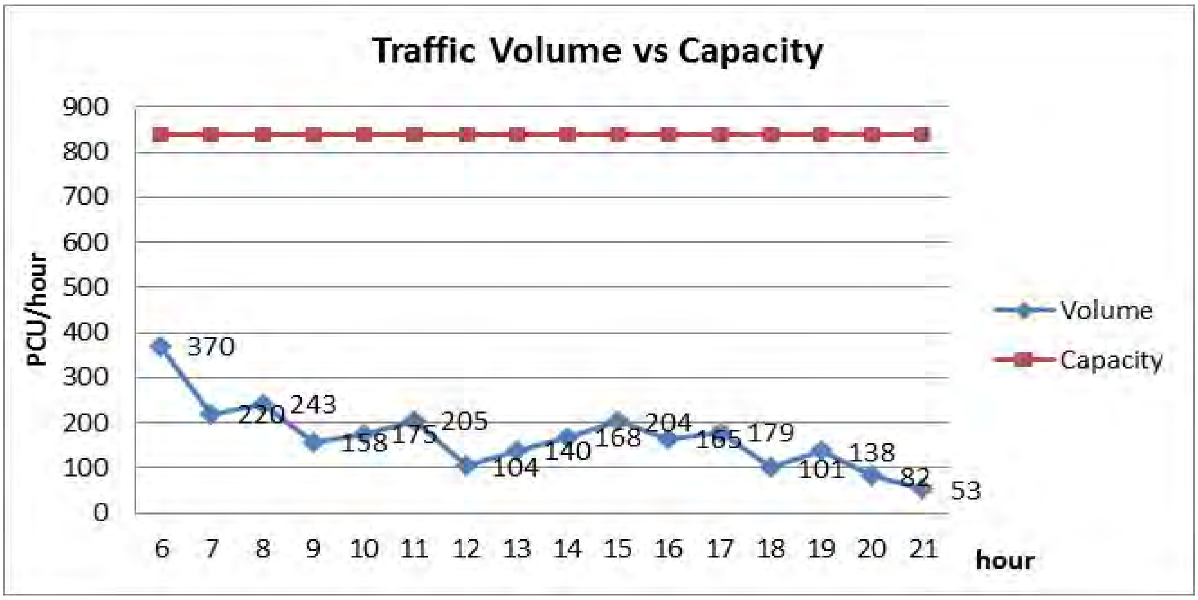


Figure 19.4.1-1 Hourly Traffic Vlume vs.Capacity during Traffic Restriction at Palanit Bridge (Y2018)

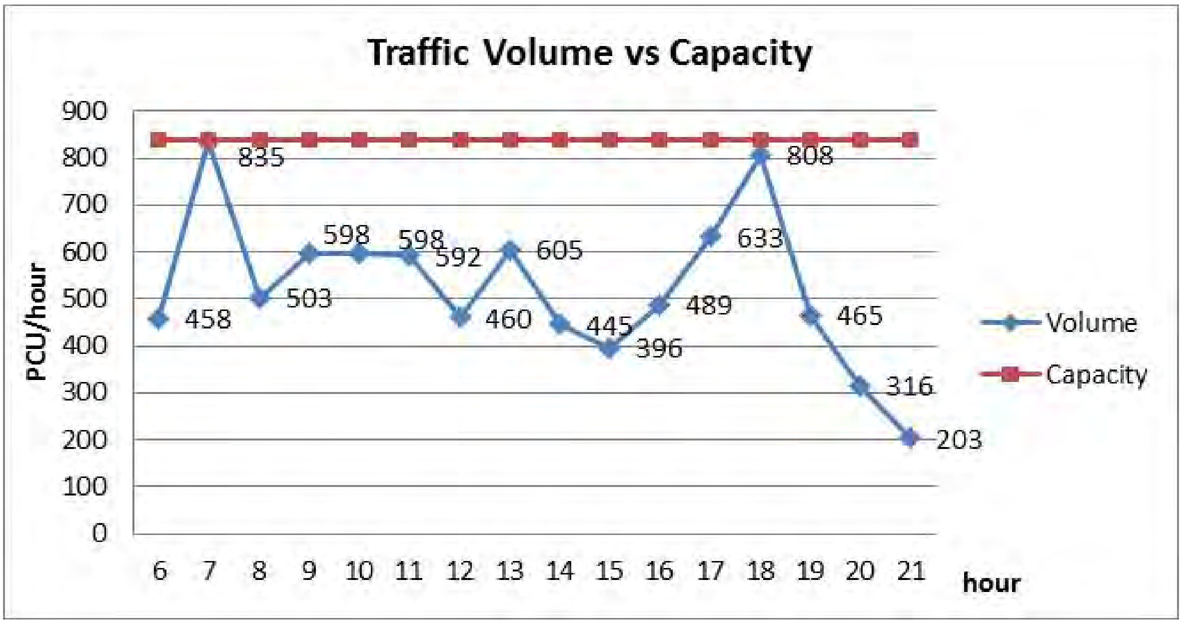


Figure 19.4.1-2 Hourly traffic volume vs. capacity during traffic restriction at Mawo Bridge (Y2018)

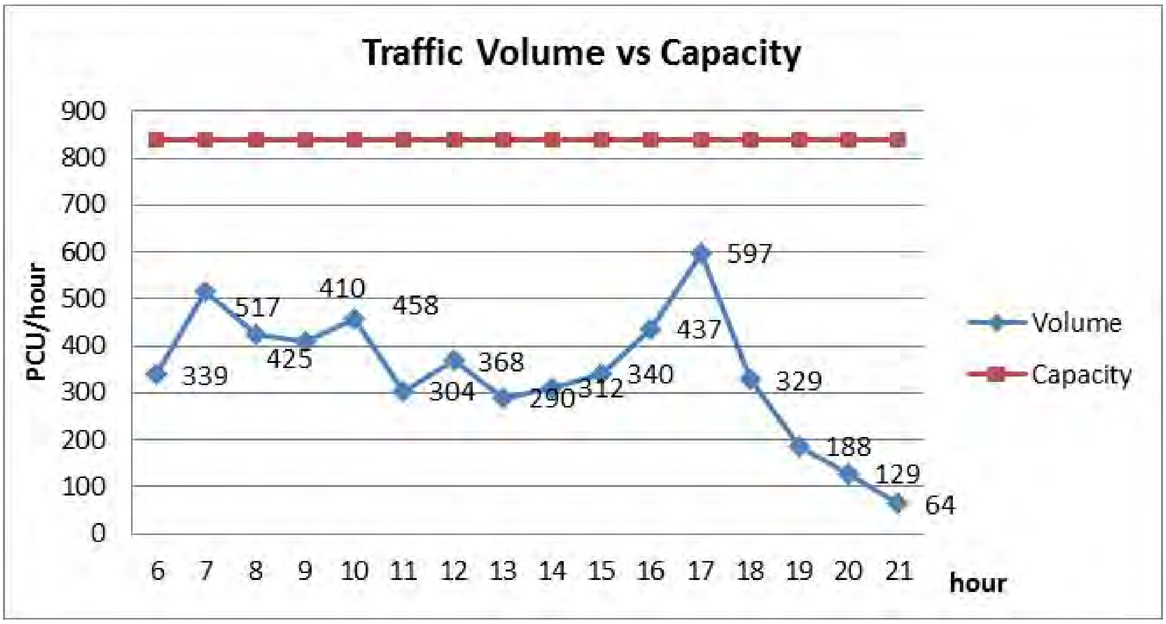


Figure 19.4.1-3 Hourly Traffic Volume vs. Capacity during Traffic Restriction at Liloan Bridge (Y2018)

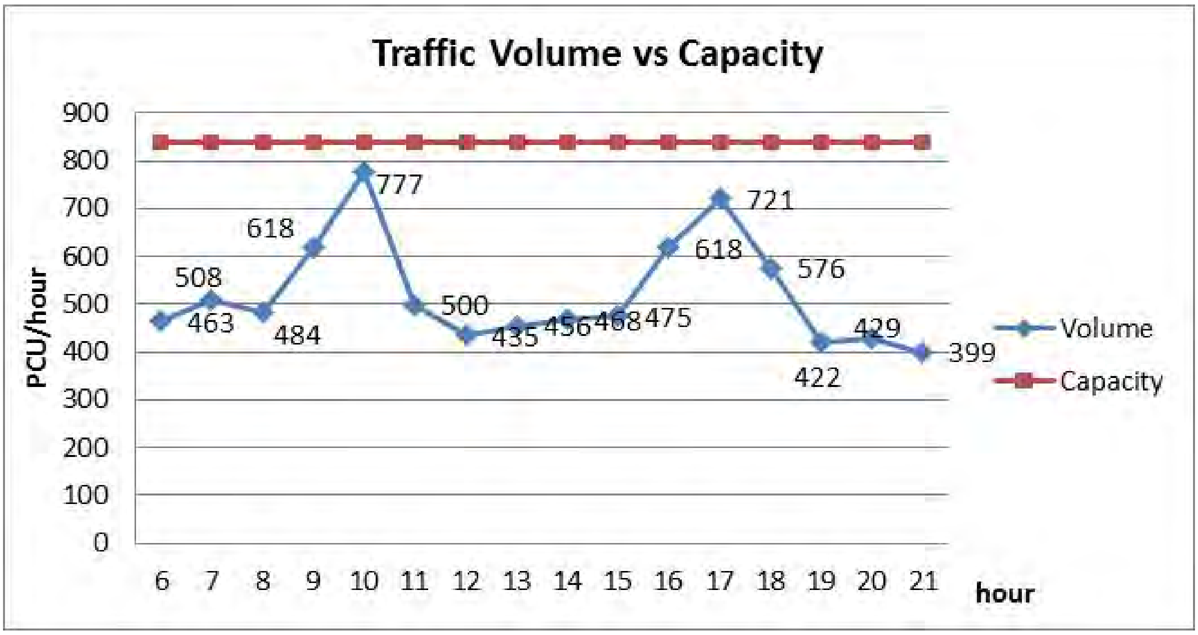


Figure 19.4.1-4 Hourly Traffic Volume vs. Capacity during Traffic Restriction at Wawa Bridge (Y2018)

19.5 ECONOMIC EVALUATION

19.5.1 General

The economic evaluation of the bridge improvement project is carried out by comparing the economic cost of the project with the economic benefit that will be brought about by the bridge replacement/retrofit.
The following three indexes are used to assess the project viability:

- Internal Rate of Return (IRR)
- Net Present Value (NPV)
- Benefit Cost Ratio (B/C Ratio)

19.5.2 Basic Assumption and Condition

In general, the economic analysis method for new road construction is established and formulated, but for improvement especially for bridges has not been established, it is still under academic study.
Based on the "with case" and "without case" for bridge improvement, project cost and benefit are estimated as shown in Table 19.5.2-1.
The project benefits are evaluated as the reduction costs which are the costs in case of 'without case'. Note that "without case" is estimated under the scenario which will happen when the bridge will not be replaced or retrofitted in the future.

Table 19.5.2-1 Basic Concepts of Cost and Benefit

	With Case	Without Case
Scenario	To conduct improvement of bridge - To extend the life of bridge - To withstand a large sale earthquake	Not to conduct improvement of bridge - To become unusable when the bridge has reached its life - To fall down if a large earthquake occurs.
Cost	Work cost for replacement or retrofit	-
		Bridge life Scenario - Work cost for re-construction - Detour cost due to traffic closure in re-construction period
		Huge earthquake Scenario - Work cost for re-construction - Detour cost due to traffic closure during re-construction period
Benefit	Reduction Cost of Without Case	

Source: JICA Study Team

Based on the concepts, the characteristics of Cost and Benefit appearance is shown below.
It is clear that the benefits appear only when the events occurs under the scenarios due to bridge life and large earthquake.

- Benefits from bridge's life will appear will appear at "each year" because there is a probability of bridge yearly deterioration.
- Benefits from large earthquake will appear at "each year" because there is a probability of earthquake occurrence.

(1) Implementation Schedule

The project is proposed to be implemented for the following schedule:
2015~2016 : Detailed design
2016~2017 : Procurement of contractors
2017~2020 : Replacement/retrofit of bridges
2020~2021 : Opening to traffic (Opening year depends on the bridge construction)

Construction period of bridges are variable depending on the length of bridge, location and construction method. Construction schedule was shown in Table 18.2.9-1

(2) Project Life

Economic life of the project is set as 30 years (2013-2042), although the physical bridges are much longer. Economic viability of bridge shall be verified with the period of 30 years.

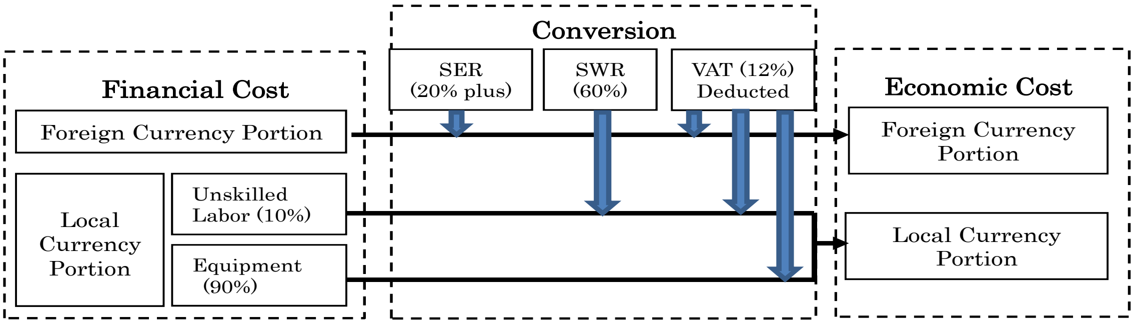
(3) Discount Rate

The rate of the capital opportunity cost is estimated at 15%. This rate is generally used as the discount rate for the evaluation of infrastructure projects in the Philippines.

19.5.3 Economic Cost

Financial cost need to be converted into that of the economic cost when conducting an economic evaluation, and the way of conversion from financial cost to economic cost is described below and illustrated by the following chart.

- The Shadow Exchange Rate (SER) which is 20% higher than the official rate is used to convert the items of foreign currency portion from dollar into Peso.
- The Shadow Wage Rate (SWR) which is 60% of current wage rate is used to convert the unskilled worker cost (10% of the local currency portion) into economic price.
- The value of VAT (12%) is deducted from all the cost items.



Source: JICA Study Team

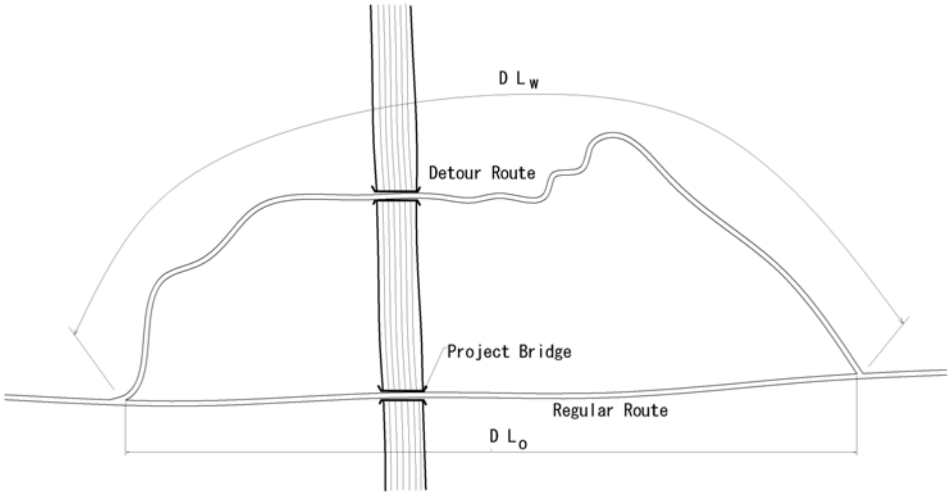
Figure 19.5.3-1 Process of Converting the Initial Cost from Financial to Economic Value

19.5.4 Benefits

This section shows the benefit estimation method for two scenarios, those that are caused by bridge deterioration and by huge earthquake.

(1) Scenario Caused by Bridge Life (deterioration)

Substantial part of benefits derived from replacement or retrofitting of bridges are saving of Vehicle Operating Cost (VOC) and Travel Time Cost (TTC) of passing vehicles by reducing the probability of bridge collapse. As shown in the following sketch, if a bridge will collapse, vehicles crossing the bridge will be obliged to take another bridge (or sea transport) located along detour route, which are normally with longer travel distance and/or inferior surface condition.



Probability of bridge collapse depends on the condition of bridge and replacement or retrofit.

Traffic impact of collapse for Lambingan Bridge, Guadalupe Bridge and 1st Mactan Bridge was estimated based on the traffic assignment.
Other four (4) bridges are assumed with the following condition based on the present transport network. Table 19.5.4-1 shows that the travel length and travel time of regular route and detour route based on present transport condition.

Table 19.5.4-1 Estimation of Travel Time and Length for Regular Route and Detour Route

Bridge Name	Regular Route	Detour Route	Remarks
Lambingan	-	-	Traffic impact analysis was done by traffic assignment.
Guadalupe	-	-	
1st Mandaue-Mactan	-	-	
Palanit	Travel length; 75m Travel Time; 0.1 min.	Travel length; 200m Travel Time; 60.9min. (incl. ferry waiting time, loading and unloading time)	No alternate Route, must use water transport
Mawo	Travel length; 260m Travel Time;0.3 min.	Travel length; 800m Travel Time; 82.4 min. (incl. ferry waiting time, loading and unloading time)	No alternate Route, must use water transport, no visible ferry terminal
Liloan	Travel length;130m Travel Time;0.2min.	Travel length;400m Travel Time; 61.2min. (incl. ferry waiting time, loading and unloading time)	No alternate Route, must use water transport
Wawa	Travel length;2,700m Travel Time;2.35 min.	Travel length;9,000m Travel Time; 27 min. (assumed 20km/h as detour route)	via Magsaysay Viaduct

Source: JICA Study Team

Construction of new bridge will reduce the probability of bridge collapse and entailing bridge un-service which requires vehicle to take detour road. Differences in vehicle operating costs between detour road and regular road is considered as benefit.

1) Probability Model of Bridge Un-serviceability

Annual probability of bridge un-serviceability is assumed to follow a normal distribution with the following parameters.

$F(x) = N\,[m.\sigma^2] = N\,[50, 16.7^2]$*

$$= \frac{1}{\sqrt{2\pi\sigma^2}} \text{ x } EXP\left[\frac{-(x-m)^2}{2\sigma^2}\right]$$

where, F(x) = Probability of bridge un-serviceability at year x

m = Average bridge life (50 years)

σ = Standard deviation (16.7 years)

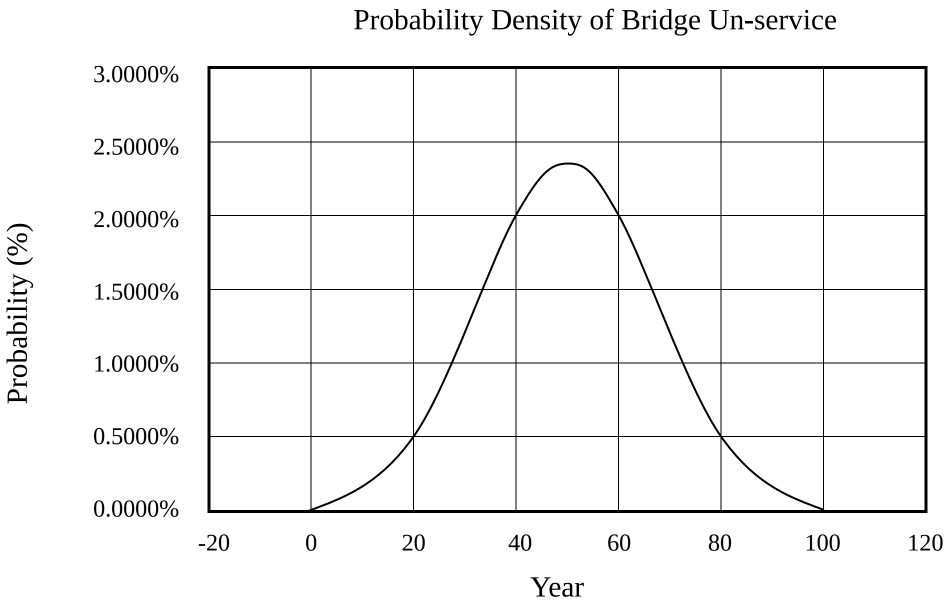


Figure 19.5.4-1 Probability Density of Bridge Un-Service[1]

[1] The Study on the Maintenance and Rehabilitation of Bridges in Malaysia, 1992 (JICA)

2) Bridge Age

Residual life of bridges differs by structure type, traffic volume, geography, present structural conditions and other factors even calendar age is same. Since bridges identified for reconstruction in this study are in an advanced stage of dilapidation, their physical lives are assumed to be elapsed. Hence, age for bridges proposed for reconstruction in this study is set as 50 years.

The probability densities for bridge age of 50 and newly constructed bridge are shown in Table 19.5.4-2 in comparison with original probability densities.

Table 19.5.4-2 Probability Density of Bridges

Sq	Year	Original Probability Density	Replacement Bridge (with project)	Retrofit Bridge (with project)	Bridge age 50[2] (without project)
51	2020	2.3846%	0.0270%	2.536%	4.8862%
52	2021	2.3718%	0.0323%	2.574%	4.8599%
53	2022	2.3506%	0.0384%	2.607%	4.8166%
54	2023	2.3213%	0.0455%	2.634%	4.7565%
55	2024	2.2842%	0.0538%	2.655%	4.6804%
56	2025	2.2396%	0.0633%	2.671%	4.5890%
57	2026	2.1880%	0.0743%	2.680%	4.4833%
58	2027	2.1299%	0.0868%	2.683%	4.3643%
59	2028	2.0660%	0.1011%	2.680%	4.2333%
60	2029	1.9968%	0.1173%	2.671%	4.0915%
61	2030	1.9230%	0.1356%	2.656%	3.9403%
62	2031	1.8453%	0.1563%	2.635%	3.7812%
63	2032	1.7644%	0.1794%	2.608%	3.6154%
64	2033	1.6811%	0.2052%	2.575%	3.4446%
65	2034	1.5959%	0.2339%	2.537%	3.2701%
66	2035	1.5096%	0.2657%	2.494%	3.0933%
67	2036	1.4229%	0.3007%	2.446%	2.9156%
68	2037	1.3364%	0.3391%	2.398%	2.7383%
69	2038	1.2506%	0.3810%	2.350%	2.5625%
70	2039	1.1661%	0.4265%	2.302%	2.3895%
71	2040	1.0835%	0.4758%	2.206%	2.2201%
72	2041	1.0031%	0.5289%	2.045%	2.0554%
73	2042	0.9254%	0.5858%	1.890%	1.8961%
Total		42.4697%	4.8538%	57.5330%	83.6833%

Source: JBIC SAPROF for Eastern Bangladesh Bridge Improvement Project

3) Un-service Duration of Bridges

In order to estimate the un-service duration of bridges, the number of months required for bridge construction was assumed as a function of bridge length as follows:[3]

$$\mathrm{Log}(M) = 0.5721 \log(L) + 0.043$$

Where;

M = Standard number of months required for bridge construction
L = Bridge length (m)

Based on the above formula, reconstruction month is estimated as the purpose of economic analysis shown in Table 19.5.4-3.

2 JBIC SAPROF for Eastern Bangladesh Bridge Improvement Project
3 JBIC SAPROF for Eastern Bangladesh Bridge Improvement Project

Table 19.5.4-3 Assumed Un-service Duration of Bridges

Bridge name	Length	Reconstruction months
Lambingan	144m	19
Guadalupe	98m	15
1st Mandaue- Mactan	859m	52
Palanit	123m	18
Mawo	259m	26
Liloan	298m	29
Wawa	228m	25

Source: Calculated by the JICA Study Team

4) Benefit Calculations

VOC and TTC savings from replacement of permanent bridge is calculated from the following formulae:

$$B_{xc} = \sum [f_o(x)-f_w(x)]*d*AADT_{xi}*(DL_o*VOC_{oi}-DL_w*VOC_{wi}) + [f_o(x)-f_w(x)]*C$$

$$B_{xt} = \sum [f_o(x)-f_w(x)]*d*AADT_x i*(OL_o/V_{Oi} - DL_w/V_{wi})*TTC_i$$

where:

B_{xc} = VOC savings at year x
B_{xt} = TTC savings at year x
$f_o(x)$ = Probability of bridge unusable in year x for without project case
$f_w(x)$ = Probability of bridge unusable in year x for with project case
d = Number of days required for bridge reconstruction
$AADT_{xi}$ = Average Annual Daily Traffic of vehicle type i in year x
DL_o = Length of detour route (km)
DL_w = Length of regular route (km)
VOC_{oi} = Vehicle operating cost of vehicle type i along detour route (peso/km)
VOC_{wi} = Vehicle operating cost of vehicle type i along regular route (peso/km)
TTC_i = Travel time cost of vehicle type i (peso/h)
C = Bridge reconstruction cost
V_{oi} = Vehicle operating speed of vehicle type i along detour route (km/h)
V_{wi} = Vehicle operating speed of vehicle type i along regular route (km/h)

5) Benefit Measurement

<u>Unit Vehicle Operating Cost (VOC)</u>

Benefit derived from road and bridge project is mainly accrued from savings in Vehicle Operating Cost (VOC) that consists of cost of operation and maintenance of each vehicle category such as fuel and lubrication cost, oil consumption cost, tire cost, repair/maintenance cost and depreciation cost.

The Department of Public Works and Highways (DPWH) has been periodically updating VOC data in order to use as input to the HDM Model for the appraisal of highway development and maintenance projects. There are the detailed data of VOC in 2008 (see Table 19.5.4-4), therefore, these data are revised and updated in accordance with the consumer price indices (average CPI 3.6%) . They are summarized in Table 19.5.4-5.

Table 19.5.4-4 Unit VOC by Vehicle Type in September 2008

(Pesos per veh-km)

Speed (km/h)	1 Motor-Tricycle	2 Car	3 Jeepney	4 Goods Utility	5 Small Bus	6 Large Bus	7 Rigid Truck 2ax	8 Rigid Truck 3ax	9 Semi-Trailer 4ax	10 Semi-Trailer 5ax
20	3.32	12.33	9.54	10.85	23.81	33.37	23.17	37.71	41.40	43.79
30	2.78	10.51	8.09	9.06	20.31	28.11	20.02	32.50	36.37	38.73
40	2.43	9.19	7.13	7.83	17.78	24.40	17.89	29.06	33.26	35.63
50	2.32	8.53	6.75	7.31	16.53	22.66	17.01	27.86	32.46	34.86
60	2.35	8.22	6.72	7.18	15.96	22.00	16.76	27.85	32.79	35.13
70	2.46	8.14	6.91	7.32	15.79	22.04	16.83	28.51	33.55	35.78
80	2.48	8.21	7.24	7.61	15.83	22.55	17.06	29.45	34.52	36.69
90	2.48	8.37	7.63	7.97	15.95	22.57	17.35	29.45	35.58	37.73
100	2.48	8.58	8.00	8.32	16.10	22.57	17.51	29.45	36.04	38.19
110	2.48	8.78	8.30	8.59	16.22	22.57	17.51	29.45	36.04	38.19
120	2.48	8.83	8.52	8.78	16.30	22.57	17.51	29.45	36.04	38.19

Source: DPWH

Table 19.5.4-5 Unit VOC by Vehicle Type in 2013

(Pesos per veh-km)

Speed (km)	Motorcycle /Tricycle	Passenger Car	Jeepney	Bus	Truck
20	3.00	14.92	11.54	40.36	35.66
30	2.56	12.71	9.79	34.01	30.95
40	2.24	11.12	8.62	29.52	27.85
50	2.08	10.32	8.17	27.40	26.72
60	2.00	9.94	8.13	26.61	26.60
70	1.98	9.84	8.36	26.66	26.96
80	2.00	9.93	8.76	27.27	27.56
90	2.04	10.13	9.23	27.31	27.99

Source: DPWH, JICA Study Team

Based on traffic assignment results for Guadalupe Bridge, Lambingan Bridge and 1st Mactan Bridge, the VOC saving for the whole road network will be calculated based on the product of the estimated traffic volumes and unit VOC. Other four (4) bridges will be calculated based on the product of the traffic volume and unit VOC (assumed speed of the regular route is 60kph, detour route is 20kph).

<u>Unit Travel Time Cost (TTC)</u>

The Travel Time Cost (TTC) is normally calculated based on the average labor productivity in the Philippines. The basic costs for TTC by type of passenger were obtained also from the DPWH. The values are 2013 price level. In the derivation of the TTC, the average income, employment and the gross national product were used as the basis to calculate for the working time and non-working time per person-hour for representative vehicle type and then estimate for the passenger time cost per person.
The unit TTC cost by type of vehicles in year 2013 which were updated based on the consumer price indices (Average CPI 3.6%), is shown in Table 19.5.4-6-Table 19.5.4-7**.**

Table 19.5.4-6 Unit Travel Time Cost in 2008

Peso/min/veh.

1. Motorcycle/ Tricycle	2. Passenger Car	3. Jeepney	4. Goods Utility	5. Small Bus	6. Large Bus	7. Rigid Truck 2axle	8. Rigid Truck 3axle	9. Rigid Truck 4axle	10. Rigid Truck 5axle
1.37	6.81	7.44	2.57	12.69	27.82	1.02	1.46	2.10	2.10

Source: DPWH

Table 19.5.4-7 Unit Travel Time Cost in 2013

Peso/min/veh.

Vehicle Type	2013
Motorcycle/Tricycle	1.66
Passenger Car	8.23
Jeepney	9.00
Bus	33.65
Truck	1.57

Source: JICA Study Team

(2) Scenario Caused by Large Earthquake

Scenario caused by “Earthquake” is assumed that the bridge will collapse if an earthquake occurs. Therefore, it is not predictable; it only has the probability for each year.
If the **Earthquake’s Probability of Occurrence** is assumed to be **30years**, the benefit in each year will take the **1/30 of benefit** when the earthquake occurs at the year.

1) Occurrence Probability of Earthquake in the Philippine

The intensity scale or PGA (Peak Ground Acceleration) was used as a measure of damageability. Table 19.5.4-8 shows the PEIS (Philippine Earthquake Intensity Scale) and equivalent PGA range. Though bridge collapse is expected at PEIS VIII according to the PEIS description, **PEIS –VII** was applied as the threshold of bridge collapse because these selected bridges were already damaged.

Table 19.5.4-8 PHILVOLCS Earthquake Intensity Scale

PEIS	Description	PGA (g values)
PEIS-I. Scarcely Perceptible	• Perceptible to people only under favorable circumstances. • Delicately-balanced objects are disturbed slightly. Still water in containers oscillates slightly.	0.0005
PEIS-II. Slightly Felt	• Felt by few individuals at rest indoors. • Hanging objects swing slightly. Still water in containers oscillates noticeably.	0.0009
PEIS-III. Weak	• Felt by many people indoors specially in upper floors of buildings. Vibration is felt like the passing of a light truck. Dizziness and nausea are experienced by some people. • Hanging objects swing moderately. Still water in containers oscillates moderately.	0.0011
PEIS-IV. Moderately Strong	• Felt generally by people indoors and some people outdoors. Light sleepers are awakened. Vibration is felt like the passing of a heavy truck. • Hanging objects swing considerably. Dinner plates, glasses, windows and doors rattle. Floors and walls of wood-framed building creak. Standing motor cars may rock slightly. • Water in containers oscillates strongly. Rumbling sounds may sometimes be heard.	0.0050

PEIS	Description	PGA (g values)
PEIS-V. Strong	• Generally felt by most people indoors and outdoors many sleeping people awakened. Some are frightened; some run outdoors. Strong shaking and rocking are felt throughout the building. • Hanging objects swing violently. Dining utensils clatter and clink; some are broken. Small light and unstable objects may fall or overturn. Liquids spill from filled open containers. Standing vehicles rock noticeably. Shaking of leaves and twigs of trees is noticeable.	0.0100
PEIS-VI. Very Strong	• Many people are frightened; many run outdoors, some people lose their balance. Motorists feel like driving with flat tires. • Heavy objects and furniture move or may be shifted. Small church bells may ring. Wall plaster may crack. Very old or poorly built houses and man-made structures are slightly damaged. Though well-built structures are not affected. Limited rock falls and rolling boulders occur in hilly to mountainous areas and escarpments. Trees are noticeably shaken.	0.1200
PEIS-VII. Destructive	• Most people are frightened and run outdoors. People find it difficult to stand in upper floors. • Heavy objects and furniture overturn or topple. Big church bells may ring. Old or poorly built structures suffer considerable damage. Some well-built structures are slightly damaged. Some crocks may appear on dikes, fishponds, road surfaces, or concrete hollow block walls. Limited liquefaction, literal spreading and landslides are observed. Trees are shaken strongly. (Liquefaction is a process by which loose saturated sand loses strength during an earthquake. And behaves like liquid.)	0.2100
PEIS-VIII. Very Destructive	• People are panicky. People find it difficult to stand even outdoors. • Many well-buildings are considerably damaged. Concrete dikes and foundations of bridges are destroyed by ground setting or topping. Railway tracks are bent or broken. • Tombstones may be displaced. Twisted or overturned. Utility posts, towers and monuments may tilt or topple. Water and sewer pipes may be bent, twisted or broken. Liquefaction and literal spreading causes man-made structures to sink, tilt or topple. Numerous landslides and rock falls occur in mountainous and hilly areas. Boulders are thrown out from their positions particularly near the epicenter. Fissures and fault rupture may be observed. Trees are violently shaken. Water splashes of slops over dikes or banks of rivers.	0.3600-0.5300
PEIS-IX. Devastating	• People are forcibly thrown to the ground. Many cry and shake with fear. • Most buildings are totally damaged. Bridges and elevated concrete structures are toppled or destroyed. • Numerous utility posts, towers and monuments are titled, toppled or broken. Water and sewer pipes are bent, twisted or broken. Landslides and liquefaction with lateral spreading and sand boils are widespread. The ground is distorted into undulations. Trees are shaken very violently with some toppled or broken. Boulders are commonly thrown out. River water splashes violently or slops over dikes and banks.	0.71100-0.8600
PEIS-X. Completely Devastating	• Practically all man-made structures are destroyed. Massive landslides and liquefaction, large scale subsidence and uplifting of landforms. And many ground fissures are observed. Changes in river courses and destructive seiche in lakes occur. Many trees are toppled, broken or uprooted.	>1.1500

Source: Philippine Institute of Volcanology and Seismology

The JICA Study Team computed the return period in years that PGA value is exceeding as shown in Table 19.5.4-9

Table 19.5.4-9 Return Period of PGA Value

	Lambingan and Guadalupe Bridge	1st Mandaue Mactan Bridge	Palanit and Mawo Bridge	Liloan Bridge	Wawa Bridge
PEIS-VI(0.12g)	22yrs	97yrs	9yrs	30yrs	19yrs
PEIS-VII(0.21g)	**83yrs**	**550yrs**	**43yrs**	**142yrs**	**78yrs**
PEIS-VIII(0.36g)	412yrs	4,249yrs	260yrs	959yrs	438yrs
PEIS-IX(0.6g)	2,658yrs	43,461yrs	1,900yrs	8,651yrs	3,116yrs
PEIS-X(0.9g)	15,747yrs	396,840yrs	12,011yrs	67,921yrs	19,233yrs

Source: Calculated by the JICA Study Team

2) Benefit Calculations

VOC and TTC savings from replacement of permanent bridge considering the earthquake occurrence is calculated from the following formula:

$$B_{xc} = P*d*AADT_{xi}*(DL_o*VOC_{oi}-DL_w*VOC_{wi}) + P*C$$

$$B_{xt} = P*d*AADT_{x}i*(OL_o/V_{Oi} - DL_w/V_{wi})*TTC_i$$

where:

P = Probability of Earthquake's Occurrence
B_{xc} = VOC savings at year x
B_{xt} = TTC savings at year x
d = Number of days required for bridge reconstruction
$AADT_{xi}$ = Average Annual Daily Traffic of vehicle type i in year x
DL_o = Length of detour route (km)
DL_w = Length of regular route (km)
VOC_{oi} = Vehicle operating cost of vehicle type i along detour route (peso/km)
VOC_{wi} = Vehicle operating cost of vehicle type i along regular route (peso/km)
TTC_i = Travel time cost of vehicle type i (peso/h)
C = Bridge reconstruction cost
V_{oi} = Vehicle operating speed of vehicle type i along detour route (km/h)
V_{wi} = Vehicle operating speed of vehicle type i along regular route (km/h)

19.5.5 Result of Economic Evaluation

Results of economic evaluation by bridges are shown in Table 19.5.5-1. All bridges were evaluated as economically feasible.

Table 19.5.5-1 Results of Economic Evaluation by Bridges

Bridge	EIRR	B/C	NPV (Million Peso @i=15%)
Lambingan	26.5%	1.84	
Guadalupe	26.8%	2.08	
1st Mandaue- Mactan	20.3%	1.42	
Palanit	19.1%	1.27	
Mawo	16.1%	1.06	
Liloan	19.8%	1.27	
Wawa	15.4%	1.02	
Projects (all seven bridges)	**22.8%**	**1.59**	

Source: JICA Study Team

19.5.6 Project Sensibility

The Project Sensitivity to the identified risks is shown in Table 19.5.6-1

Table 19.5.6-1 Project Sensitivity

	Base	Cost plus 10%	Cost plus 20%
Base	22.8%	21.0%	19.5%
Benefit less 10%	20.8%	19.2%	17.7%
Benefit less 20%	18.8%	17.2%	15.9%

Source: JICA Study Team

Results show that the project is able to hurdle the minimum acceptable criteria of EIRR that is15%. Even if cost goes up and/or benefit goes down as shown in the following condition, the minimum criteria of 15% EIRR would still meet.

- Cost plus 59%
- Benefit less 47%
- Cost plus 22% and Benefit less 22%

CHAPTER 20 NATURAL AND SOCIAL ENVIRONMENT ASSESSMENT

20.1 Environmental and Social Consideration

20.1.1 Legal Framework

(1) National and Local Environmental Assessment Laws, Regulation and Standard

The proposed Project will be governed by the existing environmental assessment laws and regulations specifically under the Philippine Environmental Impact Statement System (PEIS). It will also be guided by the DENR/EMB policies and other local environmental and social instruments specifically for projects experiencing adverse impacts within the direct impact area (DIA). Table 20.1.1-1 shows the national and local environmental assessment laws, regulations and standards applicable for the proposed Project. Table 20.1.1-2 also shows other related environmental laws and regulations that might be applicable once the Project starts its construction depending on the scale of construction activities that will be implemented.

Table 20.1.1-1 National and Local Environmental Assessment Laws, Regulations and Standards

Number	Title/Description
PD 1151	Philippine Environmental Policy
PD 1586	Establishing an Environmental Impact Statement System including other Environmental Management related Measures and for other purposes
DAO 2009-15	Implementation of EIS-Information System, CNC Automated Processing System, GIS Maps of Environmentally Critical Areas
DAO 2003-30	Implementing Rules and Regulations (IRR) for the Philippine Environmental Impact Statement (EIS) System
DAO 2000-37	Addendum to Article VIII Section 1.0 of DAO 96-37 re: Standard Costs and Fees for Various Services of the EMB relative to the Implementation of the Philippine EIS System
DAO2000-05	Revising DAO No. 94-11, supplementing the DAO No. 96-37 and providing for Programmatic Compliance Procedures within the EIS System
DAO 1999-37	Implementing Rules and Regulations for the Operationalization of the Environmental Revolving Fund under PD 1586
DENR MC 2010-14	Standardization of Requirements and Enhancement of Public participation in the Streamlined Implementation of the Philippine EIS System
DENR MC 2007-08	Simplifying the Requirements for Environmental Compliance Certificate or certificate of Non-Coverage Applications
DENR MC 2008-08	Clarification of the Role of LGUs in the Philippine EIS System in relation to MC 2007-08
EMB MC 2011-005	Incorporating Disaster Risk Reduction (DRR) and Climate Change Adaptation (CCA) concerns in the Philippine EIS System
EMB MC 2010-004	Guidelines for use of Screening and Environmentally Critical Area (ECA) Map Systems
EMB MC 2010-002	Clarification to DENR MC 2010-14 and other EIS System Policy Issuances
EMB MC 2007-002	Revised Procedural Manual for DENR Administrative Order No. 30, Series of 2003 (DAO 03-30)
EMB MC 2007-001	Environmental Impact Assessment (EIA) Review Manual

Table 20.1.1-2 Other National and Local Environmental Laws, Regulations and Standards

Number	Title/Description
AIR QUALITY	
RA 8749	Philippine Clean Air Act of 1999
DAO 2000-82	Integrated Air Quality Improvement Framework – Air Quality Control Action Plan
DAO 2000-81	Implementing Rules and Regulations for RA 8749
DAO 1998-46	1998 Revised Implementing Rules and Regulations for the Prevention, Control and Abatement of Air Pollution from Motor Vehicles
DAO 1993-14	Air Quality Standards of the Philippines
WATER QUALITY	
RA 9275	The Philippine Clean Water Act of 2004
DAO 2003-27	Amending DAO 26, DAO 29 and DAO 2000-81 among others on the Preparation and Submission of Self-Monitoring Report (SMR)
DAO 1990-35	Revised Effluent Standards of 1990
DAO 1990-34	Revised Water Usage and Classification – Water Quality Criteria
MC 2009-014	Strict Implementation of the 50 meters Buffer Zone
MC 2003-008	Procedural and Reference Manual for DAO 2003-27
HAZARDOUS WASTE MANAGEMENT	
RA 6969	Toxic Substances and Hazardous and Nuclear Wastes Control Act of 1990
DAO 2004-36	Procedural Manual for DAO 1992-29
DAO 1992-29	Implementing Rules and Regulations for RA 6969
SOLID WASTE MANAGEMENT	
	RA 9003 Ecological Solid Waste Management Act of 2000

Legend:
PD – Presidential Decree
RA – Republic Act
DAO – Department Administrative Order
MC – Memorandum Circular
Source: EMB-EIA website

(2) JICA Environmental and Social Screening Requirement

Determination of environmental and social impacts is one of the studies included in the seismic improvement prioritization of the proposed Project. The process is based on the “JICA Guidelines for Environmental and Social Considerations – April 2010”. The following activities and policies must be considered for the evaluation of environmental and social aspects:

- Collection and analysis of data and information
- Scoping
- Prediction of environmental and social impacts of works on selected bridges
- Consideration alternatives
- Consideration of mitigation method
- Consideration of Environmental Management and Monitoring Plan
- Support for Stakeholders’ Meeting
- Support for preparing a draft Resettlement Action Plan

An environmental screening checklist was also used as guidelines for the conduct of the environmental and social survey.

(3) Philippine Environmental impact Assessment System for Road and Bridge Project

The Philippine EIS, under Presidential Decree (PD) No. 1586, is a key planning tool and a decision-making guide for any major project to ensure a rational balance between socio-economic development and environmental protection for the benefit of present and future generations. It involves assessing the direct and indirect impacts of the Project to its surrounding physical and human environment, and requiring the incorporation of appropriate enhancement and mitigating measures for environmental protection throughout the different development phases of a Project.

The DENR Administrative Order No. 30 Series of 2003 (DAO 03-30), otherwise known as the Implementing Rules and Regulations of PD No. 1586, presents the requirements, document reporting outline, screening and evaluation procedures, and other provisions regarding the issuance of an ECC for new and existing projects.

Depending on the scope of the rehabilitation and retrofitting works, the Proponent may be required to secure an ECC from the DENR-EMB prior to start of works. The project screening matrix determines the type of document that the Proponent must prepare. The requirement is specifically stated in Annex 2-1b Item No. 77-C.4.a (bridges and viaducts projects) of DAO 03-30. An IEE Report is required for bridges or viaducts with length of > 80m but < 10km. The outline and modules for this IEE report are based on Annex 2-15 of DAO 03-30.

However, DENR-EMB may further require additional information or studies aside from what is indicated in the IEE Report. Under special circumstances, a full Environmental Impact Study (EIS) may be required for the project. This requirement shall be made after thorough assessment by DENR-EMB on the impact of the proposed project. Such circumstances may include but not limited to the following:

- Potential project impact on critical protected areas, historical or cultural properties
- Presence of indigenous people/communities in the direct impact area
- Construction activities, such as drilling of foundation piles, significant expansion of ROW and road approaches, may have massive and irreversible impacts on the present environmental and social conditions.

Figure 20.1.1-1 shows a simplified flowchart for the ECC application and review processes.

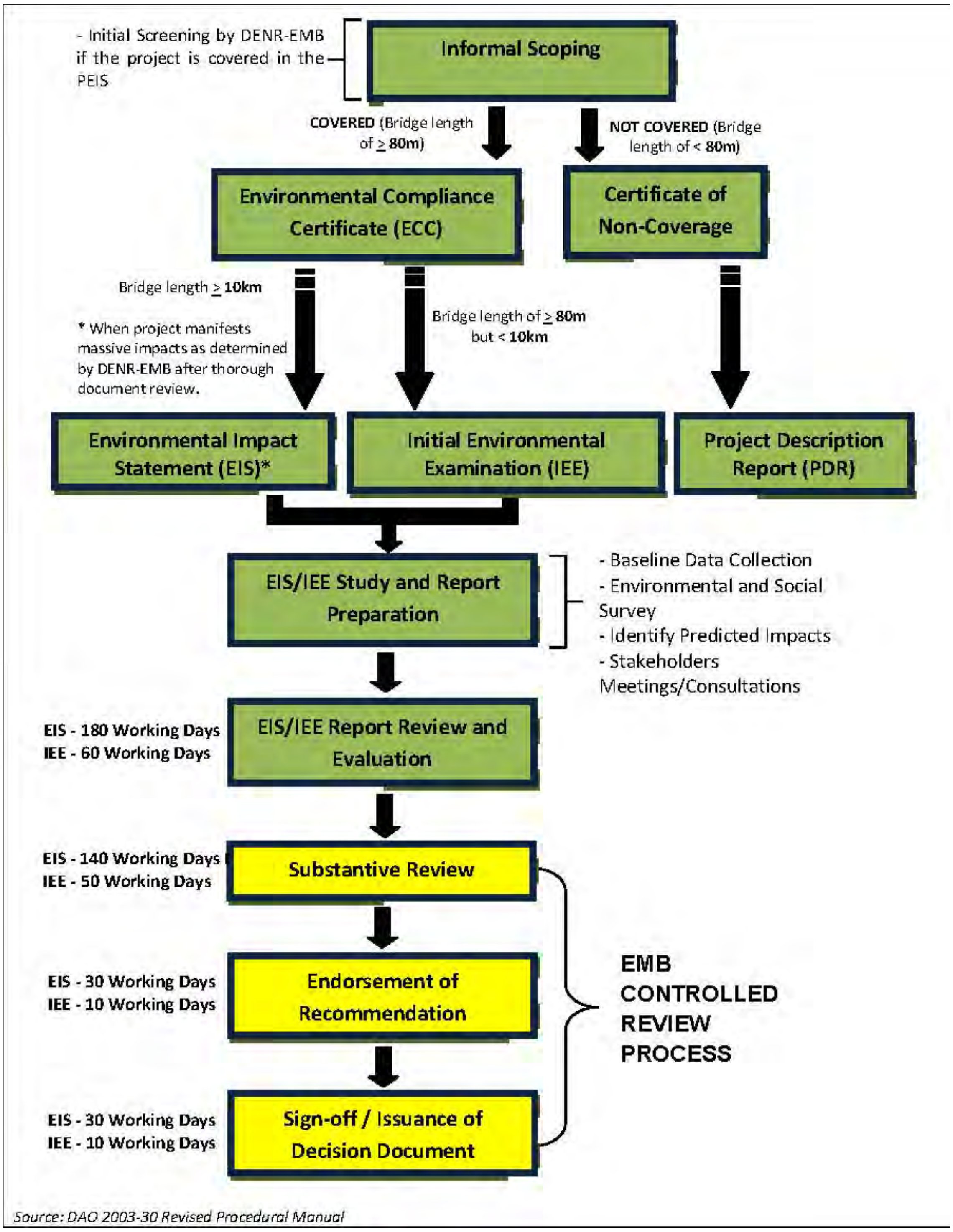


Figure 20.1.1-1 Flowchart for ECC applications and review processes

(4) Required EIA Process for Candidate Bridges

Infrastructure Projects including construction of Major Road and Bridge (80m <length< 10km) is still not considered as Environmental Critical Project under Philippine EIS system. (Shown in)
The entire Projects sites are not located in historical, cultural and national reserve but with water bodies are technically considered to locate Environmental Critical Area.
Thus all Replacement/Retrofitting Bridge Projects are required IEE Report as shown in Table 20.1.1-3.
Max time to grant or deny ECC (Environmental Compliance Commitment) Application is 60 working days for Replacement/Retrofitting Bridge Projects as shown in Table 20.1.1-3.
In case that PAPs is over 200 full RAP Report and procedures are required and also it is necessary to consult JICA committee (Advisory committee for environmental and social considerations).
Supposed schedule of EIA process after second screening is shown in.

After decided the Bridges to replace or retrofit, DPWH of proponent will initiate EIA processes obtained result of outline design such as construction method and construction yards.
Then submitted IEE are reviewed at EMB for at least 45 days and ECC (Environmental Compliance Commitment) issued.
After the L/A between Japan and the Philippines, proponent of Project (DPWH) will initiate LARAP (Land Acquisition and Resettlement Action Plan).
It is though that necessary period for LARAP will takes more than one year.

Table 20.1.1-3 Summary Table of Project Groups, EIA Report Types, Decision Documents, Processing/Deciding Authorities and Processing Duration

Bridge Replacement/ Retrofitting Project

PROJECT GROUPS/		APPLIED TO	DOCUMENTS REQUIRED FOR ECC/CNC APPLICATION	DECISION DOCUMENT	PROCESSING RESPONSIBILITY (Endorsing Official)	DECIDING AUTHORITY	MAX TIME TO GRANT OR DENY ECC APPLICATION (Working Days)
I: Environmentally Critical Projects (ECPs) in either Environmentally Critical Area (ECA) or Non- Envir Critic	I - A: New	Single Projects	Environmental Impact Statement (EIS)	ECC	CO: EIAMD Chief / EMB Director	EMB Director / DENR Secretary	120 days
	I - B: Existing Projects for Modification or Re-start up (subject to conditions in Annex 2-1c)	Single Projects	Environmental Performance Report and Management Plan (EPRMP) *	ECC	CO: EIAMD Chief/ EMB Director	EMB Director / DENR Secretary	90 days
	t ECC	Single Projects	Environmental Performance Report and Management Plan (EPRMP) *	ECC	CO: EIAMD Chief/ EMB Director	EMB Director / DENR Secretary	90 days
II: Non-Environmentally Critical Projects (NECPs) in Environmentally Critical Area (ECA)	II - A: New	Single Projects	Environmental Impact Statement (EIS)	ECC	RO: EIAMD Chief	EMB RO Director	60 days
			Initial Environmental Examination Report (IEER)	ECC	RO: EIAMD Chief	EMB RO Director	60 days
			Initial Environmental Examination Checklist (IEEC)	ECC	RO: EIAMD Chief	EMB RO Director	30 days
			Project Description Report (PDR)	ECC	RO: EIAMD Chief	EMB RO Director	15 days
	II - B: Existing Projects for Modification or Re-start up (subject to conditions in Annex 2-1c)	Single Projects	Environmental Performance Report and Management Plan (EPRMP) *	ECC	RO: EIAMD Chief	EMB RO Director	30 days
	II - C: Operating without ECC	Single Projects	Environmental Performance Report and Management Plan (EPRMP) *	ECC	RO: EIAMD Chief	EMB Director	30 days
III: Non-Environmentally Critical Projects (NECPs) in Non-Environmentally Critical Area (NECA)	III – A1: New (Enhancement and Mitigation Projects)	Single Projects	Project Description Report (PDR) (REQUIRED)	CNC	CO: EIAMD Chief	EMB Director	15 days
					RO: EIAMD Chief	EMB RO Director	
	III – A2: New (All Other Grp II Project Types/Sub-types in NECA)	Single Projects	Project Description Report (PDR) (AT OPTION OF PROPONENT)	CNC	CO: EIAMD Chief	EMB Director	15 days
					RO: EIAMD Chief	EMB RO Director	
IV: Co-located Projects	IV - A: New	Co-located Projects majority of which are Group I Projects	Programmatic Environmental Impact Statement (PEIS)	ECC	CO: EMB Director	DENR Secretary	180 days
IV: Co-located Projects	IV - A: New	Co-located Projects majority of which are Group I Projects	Programmatic Environmental Impact Statement (PEIS)	ECC	CO: EMB Director	DENR Secretary	180 days

Source: REVISE PROCEDURAL MANUAL FOR DAO 2003-30

(5) Environmental Standard

Environmental standard and regulations for Air, water and noise are shown in following tables.

Table 20.1.1-4 National Ambient Air Quality Guideline Values

Pollutants	Short Term[a]			Long Term[b]		
	µg/Ncm	ppm	Averaging Time	µg/Ncm	ppm	Averaging Time
Suspended Particulate Matter[c] – TSP PM-10	230[d] 150[f]		24 hours 24 hours	90 60		1 year[e] 1 year[e]
Sulfur Dioxide[c]	180	0.07	24 hours	80	0.03	1 year
Nitrogen Dioxide	150	0.08	24 hours			
Photochemical Oxidants as Ozone	140 60	0.07 0.03	1 hour 8 hours			
Carbon Monoxide	35 mg/Ncm 10 mg/Ncm	30 9	1 hour 8 hours			
Lead[g]	1.5		3 months[g]	1.0		1 year

[a] Maximum limits represented by ninety-eight percentile (98%) values not to exceed more than once a year.
[b] Arithmetic mean.
[c] SO_2 and Suspended Particulate matter are sampled once every six days when using the manual methods. A minimum of twelve sampling days per quarter or forty-eight sampling days each year is required for these methods. Daily sampling may be done in the future once continuous analyzers are procured and become available.
[d] Limits for Total Suspended Particulate Matter with mass median diameter less than 25-50 µm.
[e] Annual Geometric Mean.
[f] Provisional limits for Suspended Particulate Matter with mass median diameter less than 10 µm and below until sufficient monitoring data are gathered to base a proper guideline.
[g] Evaluation of this guideline is carried out for 24-hour averaging time and averaged over three moving calendar months. The monitored average value for any three months shall not exceed the guideline value.

Table 20.1.1-5 Effluent Standard: Conventional and Other Pollutants in Land Waters Class C and Coastal Waters Class

Parameter	Unit	Inland waters (Class C) for NPI	Coastal Waters (Class SC) for NPI
Color	PCU	150[(C)]	-
Temperature (max rise in deg.)	˚C rise	3	3
PH (range)		6.5 – 9.0	6.0 – 9.0
COD	mg/L	100	200
Settleable Solid (1hr)	mg/L	0.5	-
BOD (5days, 20˚C)	mg/L	50	100
Total suspended Solids	mg/L	70	150
Surfactant (MBAS)	mg/L	5.0	10
Oil/Grease (Petroleum Ether Extract)	mg/L	5.0	10
Phenolic Substances as Phenols	mg/L	0.1	0.5
Total Coliforms	MPN/100mL	10,000	-

NPI: New/Proposed Industry or wastewater treatment plant to be constructed (applied for during construction)

Table 20.1.1-6 Ambient Noise Level (unit:db(A))

Category of Area	Description of Area	Daytime	Morning & Evening	nighttime
AA	Within 100 m from school sites, nursery schools, hospitals and special homes for the aged.	50	45	40
A	Primarily used for Residential purpose	55	50	45
B	Zone or used as heavy industrial area	65	60	55
C	zone or used as light industrial area	70	65	60
D	Reserved or used as a heavy industrial area	75	70	65

Areas directly fronting or facing a four-lane road +5db (A)
Areas directly fronting or facing a four-lane or wide road +10db (A)

Table 20.1.1-7 Noise standards for construction activities

Classification	Construction activities	At a distance of 30m from the noise source (unit db(A))
Class 1	Pile drivers, pile extractors, reveting, hammers or combination thereof	90
Class 2	Rock drills, jack hammers, pavement breakers	85
Class 3	Air compressors	75
Class 4	Batching plant	75

20.1.2 Project Rationale

The proposed Project is one of the bridges selected under the screening and investigation for possible retrofitting or replacement. The bridge was constructed in 1962-1979 and the structural integrity may have been weakened over time. This Project aims to restore the stability and structural guarantee of the bridge especially during strong earthquakes.

20.1.3 Brief Discussion and Assessment of Predicted Impact

The proposed Project will inevitably create various impacts on the surrounding land, air, water, biological environment and local population throughout its construction, operations and abandonment phases.

Table 20.1.3-1 summarizes the identified environmental impact that may be created based on the proposed Project's different activities. The most affected sector and the significance of each impact are also marked to determine the following:

Will the identified/perceived impact generate positive/negative impacts;
Will the identified/perceived impact cause direct/indirect effects;
Will the identified/perceived impact cause long/short term effects; and
Will the identified/perceived impact be reversible/irreversible effects on the surrounding environment

Table 20.1.3-1 Matrix of Proposed Project's Environmental Impacts

Activities	Aspects	Environmental Impacts	Parameter Most Affected	Significance of Impact			
				+/-	D/In	L/S	R/I
A. Construction							
Implementation of major civil and construction activities along the proposed Project and Road Right of Way (ROW)	Earth-movement and other civil works	Generation of solid wastes	Land	-	D	S	R
		Dust propagation and migration	Air	-	D	S	
		Restriction or alteration of stream flows	Water	-	D	S	R
		Stormwater run-off	Water	-	In	S	R
		Siltation and increased water turbidity	Water	-	D	S	R
		Disturbance/ displacement of flora and fauna	Flora Fauna	-	D	S	R
		Traffic congestion	People	-	D	S	R
		Displacement of human settlements	People	-	D	L	I
	Use of heavy equipment	Ground vibration	Land	-	D	S	R
		Generation of hazardous wastes (i.e. used oil)	Land	-	D	S	R
		Increase in air emission levels	Air People	-	D	S	R
		Increase in noise levels	Air People	-	D	S	R
		Increased risks to occupational safety	People	-	D	S	R
	Influx of heavy equipment and construction personnel	Generation of solid wastes	Land	-	D	S	R
		Generation of wastewater	Water	-	D	S	R
		Traffic congestion	People	-	D	S	R
		Generation of employment	People	+	D	S	R
B. Operations							
Bridge operation	Bridge maintenance	Stormwater run-off	Water	-	In	L	R
		Faster traffic flow	People	+	D	L	R
C. Abandonment							
Closure	Bridge demolition	Generation of solid wastes	Land	-	D	S	R
		Generation of wastewater	Water	-	D	S	R
		Traffic congestion	People	-	D	L	R

LEGEND:
(+) positive, (-) negative
(D) direct, (In) indirect
(L) long-term, (S) short-term
(R) reversible, (I) irreversible

20.1.4 Brief Discussion on the Proposed Mitigation Measures

Table 20.1.4-1 details the summary of the proposed Project's environmental aspects and impacts, with corresponding mitigating and enhancement measures, including responsible parties and guarantees involved.

Table 20.1.4-1 Matrix of the Proposed Project's Environmental Mitigation and Enhancement Measures

Activity	Environmental Aspects	Environmental Impacts	Mitigation and Enhancement Measures	Responsibility	Cost	Guarantees
A. Construction						
Implementation of major civil and construction activities along the proposed Project and Road Right of Way (ROW)	Earth-movement and other civil works	Generation of solid wastes	Application of Solid Waste Management Plan (SWMP) Segregation of solid waste according to recyclables and non-recyclables Repair or re-use of available construction materials and equipment Hauling of discarded/recyclable items by licensed haulers	DPWH contractor	Part of construction costs	MOA
		Dust propagation and migration	Minimize/prevent unnecessary earth movement Regular watering of construction sites that have high dust concentration Avoid long exposure of excavated soil and sand piles to strong winds by applying canvass covers Establishment of construction buffer zones and containment barriers Regular clean-up and housekeeping of construction areas Equip trucks with canvass that haul dusty items (i.e., dry soil, sand) Provide construction personnel with PPEs (i.e., goggles, masks)	DPWH contractor	Part of construction costs	MOA
		Restriction or alteration of stream flows	Fast-track construction activities (i.e., foundation laying) Provide alternative drainages or channeling for affected water bodies	DPWH contractor	Part of construction costs	MOA

Activity	Environmental Aspects	Environmental Impacts	Mitigation and Enhancement Measures	Responsibility	Cost	Guarantees
			Establishment of construction buffer zones and containment barriers			
		Stormwater run-off	Avoid long exposure of excavated soil to rain Prevent/minimize chemical spills and unauthorized discharges Establishment of construction buffer zones and containment barriers	DPWH contractor	Part of construction costs	
		Siltation and increased water turbidity	Avoid long exposure of excavated soil to rain Establishment of construction buffer zones and containment barriers	DPWH contractor	Part of construction costs	MOA
		Disturbance/ displacement of flora and fauna	Perform earth balling for applicable trees Avoidance of unnecessary tree cutting Implement tree re-planting activities after full completion of the project Record/inventory of affected trees	DPWH contractor	Part of construction costs	MOA
		Displacement of human settlements	Perform additional consultations and IEC activities, with the coordination of the LGUs, with the affected residents about the relocation/resettlement Give sufficient time for the affected residents to perform relocation Provide rightful and immediate compensation to affected residents	DPWH in coordination with LGUs	To be determine	RA 8974, DPWH Ministry Order 65
		Possible traffic congestion	Provide alternate routes through a Traffic Management Plan in coordination with LGUs Provide directional signage and traffic control officers	DPWH contractor	Part of construction costs	MOA
	Use of heavy equipment	Ground vibration	Apply non-vibrating methods (i.e., bored piles) in construction sites that are nearby to residential areas If piling is necessary, perform monitoring for nearby concrete structures that may be	DPWH contractor	Part of construction costs	MOA

Activity	Environmental Aspects	Environmental Impacts	Mitigation and Enhancement Measures	Responsibility	Cost	Guarantees
			affected Notify nearby residents about the activities of using heavy equipment For hauling trucks, comply with road weight limit standards to avoid ground vibration			
		Generation of hazardous wastes (i.e. used oil)	Segregation of hazardous wastes from regular wastes Storage of hazardous items on sealed, sturdy, and properly marked containers Hauling of hazardous items by accredited haulers/treaters	DPWH contractor	Part of construction costs	MOA
		Increase in air emission levels	Regular maintenance of equipment Installation of air emission control devices for air emitting equipment	DPWH contractor	Part of construction costs	MOA
		Increase in noise levels	Installation of mufflers Perform noisy activities during daytime Use of low-noise machines/equipment Strict maintenance of equipment and vehicles	DPWH contractor	Part of construction costs	MOA
		Increased risks to occupational safety	All personnel are required to wear proper PPEs All works must be supervised by trained and competent engineers and workers First aid stations, safety equipment and signage shall be made available on working areas	DPWH contractor	Part of construction costs	MOA, Labor Code
	Influx of heavy equipment and construction personnel	Generation of solid wastes	Application of Solid Waste Management Plan (SWMP) Segregation of solid waste according to recyclables and non-recyclables Repair or re-use of available construction materials and equipment Hauling of discarded/recyclable items by licensed haulers	DPWH contractor	Part of construction costs	MOA

Activity	Environmental Aspects	Environmental Impacts	Mitigation and Enhancement Measures	Responsibility	Cost	Guarantees
		Generation of wastewater	Follow basic housekeeping policies Provision of sanitation facilities (i.e., portable comfort rooms)	DPWH contractor	Part of construction costs	MOA
		Traffic congestion	Provide alternate routes through a Traffic Management Plan in coordination with LGUs Provide directional signage and traffic control officers	DPWH contractor	Part of construction costs	MOA
		Generation of employment	Prioritize hiring of qualified residents in the host communities	DPWH contractor	Part of construction costs	MOA
B. Operations						
Bridge operation	Bridge maintenance	Stormwater run-off	Provide adequate drainage systems and direct flows into the nearest outfall	DPWH	Part of maintenance cost	
		Faster traffic flow	Regular maintenance and monitoring of the bridge Remove stalled vehicles immediately	DPWH MMDA	Part of maintenance cost	
C. Abandonment						
Closure	Bridge demolition	Generation of solid wastes	Segregation of solid waste according to recyclables and non-recyclables Hauling of discarded/recyclable items by licensed haulers	DPWH Contractor	To be determined	EMP, Abandonment Plan
		Generation of wastewater	Follow basic housekeeping policies Provision of sanitation facilities (i.e., portable comfort rooms)	DPWH Contractor	To be determined	EMP, Abandonment Plan
		Traffic congestion	Provide alternate routes through a Traffic Management Plan in coordination with LGUs Provide directional signage and traffic control officers	DPWH Contractor	To be determined	EMP, Abandonment Plan

20.1.5 Environmental Monitoring Plan

The Environmental Monitoring Plan (EMoP) presents the proposed protocols that the DPWH and its designated contractor undertake to continuously check and supervise the environmental performance of the proposed Project. This EMoP will allow DPWH to monitor, verify, and make the necessary corrective actions on the Project's various environmental impacts. Table 20.1.5-1details the matrix of environmental monitoring plan to be conducted by DPWH during the different phases of the Project.

Table 20.1.5-1 Matrix of the Proposed Project's Environmental Monitoring Plan

Concern	Parameter to be Monitored	Sampling Measurement Plan			Responsibility	Estimated Cost
		Method	Frequency	Location		
A. Construction						
Affected houses and trees	No. of houses and other establishments to be directly affected	Survey	Twice (Initial and Confirmatory)	Along the bridge ROW	DPWH DPWH Contractor	Part of Construction Cost
	No. of trees	Terrestrial Survey/ Inventory				
Air Quality	Dust	Visual observation	Daily	Immediate vicinity of construction sites	DPWH Contractor	Minimal
	NOx, SOx	Air sampler	Quarterly	Identified sampling stations	DPWH Contractor	PhP 10,000 per sampling station
	TSP	High volume sampler	Quarterly		DPWH Contractor	
	Noise	Digital sound level meter	Quarterly		DPWH Contractor	
Water Quality	TSS, Oil& Grease, color	Grab sampling	Monthly	Upstream and downstream portions of identified/affected water bodies	DPWH Contractor	PhP 5,000 per sampling activity
Solid Wastes	Tons/day, no. of items/day	Visual observation,	Daily	Construction field office/warehouse	DPWH Contractor	Part of Construction Costs
Hazardous Wastes	Liters/No. of drums (liquids) Kilograms (solids)	Visual inspection/ weighing	Monthly	Construction field office/warehouse	DPWH Contractor	Minimal
Occupational Safety	No. of work-related	Log-book	Daily	Immediate vicinity	DPWH Contractor	Minimal

Concern	Parameter to be Monitored	Sampling Measurement Plan			Responsibility	Estimated Cost
		Method	Frequency	Location		
	injuries No. of safety man-hours	registration		of the construction sites, command center		
Public Perception/ Acceptability	No. of valid complaints	Consultations with local officials and residents	Variable	Affected barangay/s	DPWH Contractor	To be determined
B. Operations						
Stormwater Run-off	BOD, COD, pH, heavy metals, TPH	Grab sampling	Quarterly	Drainage outlets	DPWH Maintenance Dept	PhP 20,000 per sampling activity
Occupational Safety	No. of work-related injuries No. of safety man-hours	Log-book/database registration	Daily	Field Operations Center	DPWH Maintenance Dept	Part of Operations Costs
Bridge Safety	No. of vehicular accidents	Log-book/database registration	Daily	Field Operations Center	DPWH Maintenance Dept	Part of Operations Costs
C. Abandonment						
Water Quality	BOD, TSS, Total coliforms,	Grab Sampling	To be determined	To be determined	To be determined	To be determined
Solid/Hazardous Wastes	Liters/No. of drums (liquids) Kilograms (solids)	Visual inspection/ weighing	To be determined	To be determined	To be determined	To be determined

20.1.6 Stakeholder Meeting

First time courtesy Meetings were held to get permission to conduct survey and to introduce proposed project as shown in Table.

Table 20.1.6-1 First time courtesy Meeting

	Name of Bridge	Date	Place
C07	1stMandaue-Mactan Bridge	September 17 & 19, 2012	Mandaue and Lapu-lapu Barangay Hall
C14	Liloan Bridge	September 12, 2012	San Roque Barangay Hall
B08	Lambingan Bridge	September 4 & 5, 2012	Brgy894, 892, 888, 891 Hall
B10	Guadalupe Bridge	September 11 & 12, 2012	Viejo, Nuevo, Llaya Barangay Hall
C09	Palanit Bridge	September 3 & 4, 2012	Palanit Barangay Hall
C11	Mawo Bridge	September 3 & 4, 2012	Poblacion Victoria
C15	Wawa Bridge	September 26 & 27, 2012	San Vicente Covered Court

Summary of Consultation such as concern/issues are in Appendix.
Second time Stakeholder Meetings were held as shown in table.
Attendance list and minutes were shown in Appendix.

Table 20.1.6-2 Second time Stakeholder Meeting

	Name of Bridge	Date	Place	No. of Attendance
C07	1stMandaue-Mactan Bridge	July 1, 2013	Looc Barangay Hall	35
C14	Liloan Bridge	July 3, 2013	San Roque Covered Court	77
B08	Lambingan Bridge	June 20, 2013	Brgy894 Multipurpose Hall	23
B10	Guadalupe Bridge	June 21, 2013	Llaya Multipurpose Hall	24
C09	Palanit Bridge	June 25, 2013	Palanit Barangay Hall	34
C11	Mawo Bridge	June 26, 2013	Poblacion Victoria	34
C15	Wawa Bridge	June 20, 2013	San Vicente Covered Court	55

20.2 Land Acquisition and Resettlement Action Framework

20.2.1 Justification of the Land Acquisition with Respect to the Bridge Repair and Rehabilitation

Table 20.2.1-1 shows two possible project implementation options for the bridges as well as their corresponding activities.

Table 20.2.1-1 Possible Implementation Options for the Project

Options	Possible Activities
Option 1 – Retrofitting Estimated Implementation Period: 6 to 12 months C07 1stMandaue-Mactan Bridge C14 Liloan Bridge	Retrofit of substructure Strengthening for collapse protection Improvement or strengthening of ground approach and bridge Navigation protection works against collision (from ships, ferries) Rehabilitation of bridge drainage
Option 2 – Replacement Estimated Implementation Period: 12 to 18 months B08: Lambingan Bridge B10: Guadalupe Bridge C09 Palanit Bridge C11 Mawo Bridge C15 Wawa Bridge	Replacement of foundation, substructure and superstructure Navigation protection works against collision (i.e., from ships, ferries) Replacement of approach road Bridge pavement and installation of bridge railings Installation of drainage systems Installation of street lights, if necessary

20.2.2 Land Acquisition and Resettlement Action Framework

(1) National and Local Environmental Law

The detailed RAP should be governed by existing Philippine laws and regulations for the protection of the rights of families and establishments who will be displaced by the proposed Project. These regulations shall be consistent with the JICA and World Bank policies on Involuntary Resettlement. It should also be guided by DPWH land acquisitions and resettlement policies specifically for this type of project. Table 20.2.2-1 shows the national and local laws, regulations and standards on involuntary resettlement applicable for the proposed Project. Table 20.2.2-1 also shows the various land acquisition and resettlement manuals and references currently used by DPWH.

Table 20.2.2-1 National and Local Laws, Regulations and Standards for Involuntary Resettlement

Reference	Title/Description
1987 Philippine Constitution	Indicating that "no person shall be deprived of life, liberty or property without due process of law, nor shall any person be denied the equal protection of the laws" and that "private property shall not be taken for public use without just compensation"
PD 1472	Amending Republic Acts Nos. 4852 And 6026 By Providing Additional Guidelines In The Utilization, Disposition And Administration Of All Government Housing And Resettlement Projects.
RA 8974	An Act To Facilitate The Acquisition Of Right-Of-Way, Site Or Location For National Government Infrastructure Projects And For Other Purposes
RA 7835	Comprehensive and Integrated Shelter Financing Act of 1994
RA 7279	An Act To Provide For A Comprehensive And Continuing Urban Development And Housing Program, Establish The Mechanism For Its Implementation, And For Other Purposes.
RA 6657	Comprehensive Agrarian Reform Law of 1988
RA 6389	Code of Agrarian Reforms of the Philippines of 1971
EO 1035	Providing The Procedures And Guidelines For The Expeditious Acquisition By The Government Of Private Real Properties Or Rights Thereon For Infrastructure And Other Government Development Projects
CA 141	An Act To Amend And Compile The Laws Relative To Lands Of The Public Domain
DPWH DO 2007-34	Simplified Guidelines for the Validation and Evaluation of Infrastructure

Reference	Title/Description
	Right-Of-Way Claims
DPWH DO 2003-05	Creation Of The Infrastructure Right Of Way And Resettlement Project Management Office (PMO) And The Implementation Of The Improved IROW Process
DPWH DO 2002-187	Strict Compliance to Inclusion of Preparation of Parcellary Plans and Cost Estimates for ROW Acquisition in Detailed Engineering of Infrastructure Projects
DAO 1996-34	Guidelines on the Management of Certified Ancestral Domain Claims
DAO 1993-02	Rules And Regulations For The Identification, Delineation And Recognition Of Ancestral And Domain Claims
DPWH LARR	Policy Framework for Land Acquisition, Resettlement, and Rehabilitation (LARR)
LARRIPP	Land Acquisition, Resettlement Rehabilitation and Indigenous People Policy
IROW	Infrastructure Right-of-Way Procedural Manual 2003

Legend:
PD – Presidential Decree
RA – Republic Act
EO – Executive Order
CA – Commonwealth Act
DAO – Department Administrative Order (DENR)
DO – Department Order (DPWH)

(2) JICA Land Acquisition Requirement

As stipulated in the JICA guidelines on Compliance with Laws, Standards and Plans, "Projects must comply with the laws, ordinances and standards related to environmental and social considerations established by the governments that have jurisdiction over project sites (including national and local governments) They must also conform to the environmental and social consideration policies and plans of the governments that have such jurisdiction."
Given that land acquisition and resettlement are necessary in the implementation of this project, relevant laws and guidelines will serve as the basis for lawful and proper procurement acts.

(3) Philippine LAPRAP for Road and Bridges Project

Based on DPWH's Department Order No 1993-05, a Land Acquisition Plan and Resettlement Action Plan (LAPRAP) report shall be prepared for all locally funded or foreign assisted infrastructure projects.
Prior to any land acquisition, construction or resettlement activities of this project a RAP, containing detailed description and procedures on how social and economic resettlement concerns will be addressed, will be drafted. Should there be less than 200 people affected or if land acquisition is minor and no physical relocation is required, then an ARAP will be acceptable.
The following are the necessary information that must be included in the full report:

- Number and identity of Project Affected Persons (PAPs)
- Degree (marginal or severe) and scale of adverse impacts that will be brought about as a consequence of project implementation, particularly in terms of loss of land and fixed assets as well as income
- Mitigation measures to minimize foreseeable said adverse socio-economic impacts;
- Appropriate compensation package for PAPs
- LAPRAP implementation schedule
- Overall estimated resettlement cost

(4) Justification of the Land Acquisition with Respect to the Bridge Repair and Rehabilitation Works

Justification of land acquisition will be based on the following issues:

- Final location of proposed project
- Number of households or structures that will be affected
- Type of structures (i.e., residential, commercial)
- Type of construction work

(5) Gaps in JICA and Philippine Involuntary Resettlement Frameworks

To ensure that all issues on land acquisition and resettlement will be addressed and are consistent with JICA and Philippine policies, the JICA Guidelines on Involuntary Resettlement were reviewed and compared to existing Philippine IR guidelines. The results of the comparison are summarized in Table 20.2.2-2.

Table 20.2.2-2 Gaps in JICA and Philippine Involuntary Resettlement Frameworks

No.	(A) JICA Guidelines	(B) Philippine IR Guidelines	Gaps Between (A) and (B)	Considerations in this PRAP
1	Involuntary resettlement and loss of means of livelihood are to be avoided when feasible by exploring all viable alternatives. (JICA GL)	Same (LARRIPP*)	None	Houses and other structures within the DIA where already identified and tagged in this PRAP
2	When population displacement is unavoidable, effective measures to minimize impact and to compensate for losses should be taken. (JICA GL)	Same (LARRIPP)	None	Depending on the type of rehabilitation works, possible households that will be displaced can be assessed in this PRAP.
3	People who must be resettled involuntarily and people whose means of livelihood will be hindered or lost must be sufficiently compensated and supported, so that they can improve or at least restore their standard of living, income opportunities and production levels to pre-project levels. (JICA GL)	Same (LARRIPP)	None	Guided by the LARRIPP, DPWH will initiate the inventory of the affected households and conduct appropriate valuation of properties and livelihood that will be affected.
4	Compensation must be based on the full replacement cost. (JICA GL)	Same (LARRIPP)	None	Whenever applicable, compensation will be based on full replacement cost.

No.	(A) JICA Guidelines	(B) Philippine IR Guidelines	Gaps Between (A) and (B)	Considerations in this PRAP
5	Compensation and other kinds of assistance must be provided prior to displacement. (JICA GL)	LARRIPP does not clearly state the timing of provision. In socially accepted procedure, compensation and other kinds of assistance for resettling informal setters is provided on site, prior to displacement, right after the ISFs and staff of governmental institutions together inspect the completion of the demolition of existing structures.	None	Guided by the LARRIPP, DPWH will initiate the inventory of the affected households and conduct appropriate valuation of properties and livelihood that will be affected.
6	For projects that entail large-scale IR, resettlement action plans must be prepared and made available to the public. (JICA GL)	Same (LARRIPP)	None	This PRAP will be submitted to DPWH. The proponent can do public disclosure especially to the LGU affected.
7	In preparing a resettlement action plan, consultations must be held with the affected people and their communities based on sufficient information made available to them in advance. (JICA GL)	Same (LARRIPP)	None	This PRAP will be the basis of a detailed RAP. Initial consultations with the barangay officials and several stakeholders were already conducted. A more intensive consultation will be done after the detailed engineering has been completed.
8	When consultations are held, explanations must be given in a form, manner, and language that are understandable to the affected people. (JICA GL)	Same (LARRIPP)	None	Consultations shall be conducted in local dialect supported by illustrations on the scope of the project
9	Appropriate participation of affected people must be promoted in planning, implementation, and monitoring of resettlement action plans. (JICA GL)	Same (LARRIPP)	None	An IEC (Information, Education and Communication) plan will be an integral component of the full RAP.
10	Appropriate and accessible grievance mechanisms must be established for the affected people and their communities. (JICA GL)	Same (LARRIPP)	None	The GRS (Grievance and Redress System) will also be imbedded in the RAP. The framework is initially discussed in this PRAP.

No.	(A) JICA Guidelines	(B) Philippine IR Guidelines	Gaps Between (A) and (B)	Considerations in this PRAP
11	Affected people are to be identified and recorded as early as possible in order to establish their eligibility through an initial baseline survey (including population census that serves as an eligibility cut-off date, asset inventory, and socioeconomic survey), preferably at the project identification stage, to prevent a subsequent influx of encroachers of others who wish to take advance of such benefits. (WB OP 4.12 Para.6)	LARRIPP states the cut-off date as the date of commencement of the census. Resettlement project conducted by LGUs nationwide notifies to public the last day of the census work, and use the date as the cut-off date, so that no eligible PAFs are left out in the inventory.	General public, including PAFs, may have pre-conception that cut-off date is either the starting date or the ending date of the census work	A more detailed census will be conducted taking from the initial inventory presented in this PRAP. At this point of the project, the cut-off date is not yet identified since the exact number of PAFs is not yet determined. The setting of the cut-off date shall be guided by the LARRIPP.
12	Eligibility of benefits includes: the PAPs who have formal legal rights to land (including customary and traditional land rights recognized under law), the PAPs who don't have formal legal rights to land at the time of census but have a claim to such land or assets, and the PAPs who have no recognizable legal right to the land they are occupying. (WB OP 4.12 Para. 15)	Professional Squatters (as defined by Republic Act 7279) applies to persons who have previously been awarded home lots or housing units by the government but who sold, leased or transferred the same to settle illegally in the same place or in another urban area, and non bona fide occupants and intruders of lands reserved for socialized housing. Squatting Syndicates (as defined by Republic Act 7279) refers to groups of persons who are engaged in the business of squatter housing for profit or gain. Those persons are ineligible for structure compensation, relocation, and rehabilitation/ inconvenience/ income-loss assistance in case their structures are to be demolished in resettlement project	Professional "squatters" and "squatting syndicates" are not eligible for compensation. They may salvage the structure materials by themselves.	All affected people will be eligible for compensation and rehabilitation assistance, regardless of tenure of status, social or economic standing and any such factors that may discriminate against achievement of the objectives of JICA Guidelines. However, those who have previously been awarded home lots or housing units by the government but who sold, leased or transferred the same to settle illegally in the same place or in another urban area, and non bona fide occupants and intruders of lands reserved for socialized housing will not be eligible for compensation.

No.	(A) JICA Guidelines	(B) Philippine IR Guidelines	Gaps Between (A) and (B)	Considerations in this PRAP
		according to Republic Act 7279. This definition excludes individuals or groups who simply rent land and housing from professional squatters or squatting syndicates.		
13	Preference should be given to land based resettlement strategies for displaced persons whose livelihoods are land-based. (WB OP 4.12 Para. 11)	If feasible, land for land will be provided in terms of a new parcel of land of equivalent productivity, at a location acceptable to PAFs. (LARRIPP)	None	This shall be one of the main considerations during resettlement
14	Provide support for the transition period (between displacement and livelihood restoration). (WB OP 4. 12 Para. 6)	Same (LARRIPP)	Specific details are provided in the LARRIPP.	All PAFs shall be considered for Livelihood Rehabilitation Assistance whose details will be provided in the full RAP after intensive and participatory consultations.
15	Particular attention must be paid to the needs of the vulnerable groups among those displaced, especially those below the poverty line, landless, elderly, women and children, ethnic minorities etc. (WB OP 4.12 Para. 8)	Same (LARRIPP)	None	The LARRIPP requires that all vulnerable groups are included in the resettlement process. This will be considered in this project.
16	For projects that entail land acquisition or involuntary resettlement of fewer than 200 people, abbreviated resettlement plan is to be prepared. (WB OP 4.12 Para. 25)	Minimum number of PAPs for regular RAP is not mentioned in related laws.	Minimum number of PAPs for regular RAP is not mentioned in Laws of the Republic of Philippines.	This PRAP shall be reviewed and updated when the study on the bridge rehabilitation works is completed. This will serve as a guide in drafting the full RAP or ARAP, depending on the number of PAPs. At that point, the ROW and the exact number of PAFs would have been determined.

*LARRIPP: Land Acquisition, Resettlement, Rehabilitation and Indigenous Peoples' Policy (LARRIPP), Department of Public Works and Highways, Republic of the Philippines, April.2007.

Source: JICA

20.2.3 Status of settlement around the Bridge

Based on environmental survey status of settlers around the Projects area is summarize in Table 20.2.3-1 and Table 20.2.3-2.
Table 20.2.3-1 and Table20.2.3-2 show Environmental Category in Philippine and Category in JICA Guideline.
As shown in Table 20.1.1-3 Ⅱ-A category project required IEE report and RAP report. In case the number of PAPs exceeds 200 in each Bridge, full RAP is required, but not required for EIA. Full RAP is necessary to provide resettlement place for PAPs.

Table 20.2.3-1 Status of settlers around candidate Bridges (Package-B)

Name	Along Approach and Crossing Road	Under Bridge	Environmental Category in Philippine	Environmental Category in JICA Guideline
B08 Lambingan	There are many legal and illegal houses, factory and venders. Also there are many informal houses are confirmed immediately beneath the Bridge. There is water pipe bridge adjacent.	(Right side) Out of new dyke wall there is one house with 5 PAPs.	Ⅱ-A	B
B10 Guadalupe	(North side) Alongside walk and immediately beneath the Bridge there are many houses and business buildings. (South side) Both sides of the road are used for parks.	(North side) There are 12 units' informal houses and some stores with 27 PAPs were confirmed.	Ⅱ-A	B

Table 20.2.3-2 Status of settlers around candidate Bridges (Package-C)

Name	Along Approach and Crossing Road	Under Bridge	Environmental Category in Philippine	Environmental Category in JICA Guideline
1 st Mandaue	(North side) Around the bridge there are many houses and stores.	There are 189 houses and Number of PAPs are733.	Ⅱ-A	A
Mawo	There are many houses immediately beside the Bridge (within the ROW that is 10 meter from the centre of the road each side).	(North side) Under the bridge is used for shed of boat. Within the ROW (=20m), there are two informal settlers families. Number of people is 12. (South side) Under the bridge is used for breeding place for domestic animal such as fighting cock, pig and for hanging out the washing to dry. Within the ROW there is no housing.	Ⅱ-A	B

Name	Along Approach and Crossing Road	Under Bridge	Environmental Category in Philippine	Environmental Category in JICA Guideline
Palanit	There are many houses immediately beside the Bridge. (within the ROW that is 10 meter from the centre of the road each side). Water pipe is held by the bridge.	(North side) Under the bridge is used for shed of fishing tool (bawn). Within the ROW (=20m), there are 7 PAPs.	II -A	B
Liloan	There is no house along the road near the bridge.	(South side) Under the Bridge near strait is used for basket court. There are two venders under the Bridge. Some parts of under the Bridge are used for orchard, block storage site, chicken house, waste collection point and dock for boat.	II -A	B
Wawa	(North side) Along the road there are some thatch houses. In case of replacement of the approach road between existing bridge and dam structure, there may be some PAPs.	(South side) There is no object under the Bridge.	II -A	B

Table 20.2.3-3 Estimated Number of Household members to be resettle

Name of Bridge	House/ Structure	Household members
Lambingan	10	52
Guadalupe	17	67
1st Mandaue-Mactan(cebu)	107	444
1st Mandaue-Mactan(mactan)	63	213
Palanit	9	42
Mawo	13	70
Liloan	18	85
Wawa	25	90

Table 20.2.3-4 Number of Households/Structures within the DIA

Bridge Name	Formal Settlers			Informal Settlers			Total		
	Structures	HH	Members	Structures	HH	Members	Structures	HH	Members
Lambingan								10	52
Guadalupe		15			2			17	67
1st Mandaue-Mactan								170	657
Palanit								9	42
Mawo								13	70
Liloan								18	85
Wawa	56	54	235	2	2	4	58	56	244

20.2.4 Compensation and Entitlements

(1) Compensation

When directly affected residents are clearly identified and validated, Compensation packages and entitlements must then be established as guided by the matrix in
Figure 20.2.4-1, prescribed in the Land Acquisition, Resettlement, Rehabilitation and Indigenous People's Policy (LARRIPP) 3rd Edition (2007). Eligible residents, compensation packages, channels and procedures for grievances should be clearly communicated to the PAPs.

Establishing the Cut-off Date for Compensations

The cut-off date will be on the last day of the detailed census of PAPS. This is established to ensure reliable documentation of eligible PAPS and to control speculators and illegal settlers after the census and survey of the project area. People who are not covered by the census will not be entitled to compensations.

For Residential and Commercial Land Owners

The title holder will be entitled to cash payment or land-for-land compensation. RA 8974 shall govern the computation for the replacement cost of the land. Existing zonal laws and practices issued by the Bureau of Internal Revenue (BIR) will be the basis for the initial offer to the PAF. Should the PAF decline the initial offer, the second offer will then be the actual market value of the land at the time of taking.

As for land-for-land compensation the new replacement land must be of equivalent size or at least a size acceptable to the owner, with adequate physical and social infrastructure as based on existing zoning laws. If the lost land is larger than the lot sizes for relocation, cash compensation will be paid to cover the difference.

For Residential Land Tenants/Renters

Concerned tenants/renters shall only be entitled to compensation if they physically reside in the directly affected areas at the cut-off date. Residential tenants or renters are entitled to rental subsidy equivalent to the current average monthly rental for structures similar to the house lost.

For Crops and Trees Lost

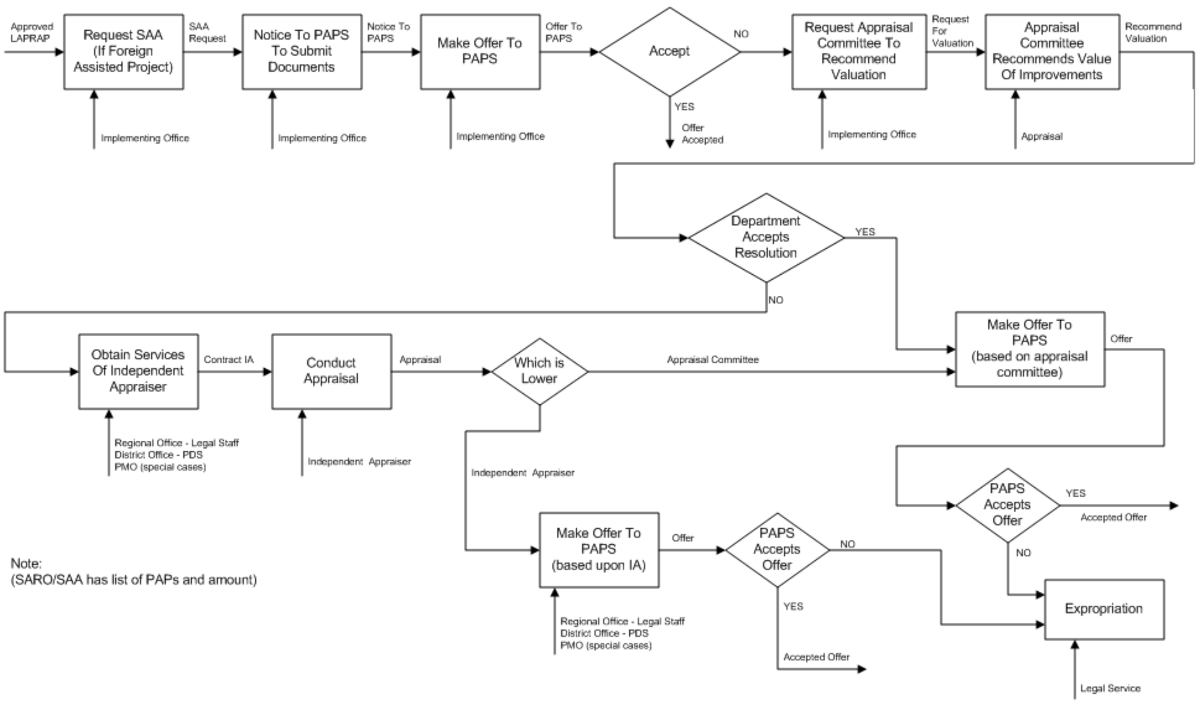
Owners of the trees lost shall be entitled to cash compensation calculated on the basis of type, age, and productive value of affected trees. For fruit-bearing trees, payment shall be based on tax declaration or schedule of values by the Provincial Assessor. For perennials of commercial value, valuation can be based on DENR schedule of valuation or concerned Appraisal Committee.

For Informal Settlers (Squatters)

Informal settlers or squatters who built their own house shall be entitled to compensation in full for their affected house or structure, without deduction for salvaged building materials. Professional squatters can collect salvaged materials but will not be entitled to receive compensation. As described in RA 7279, professional squatters are the intruders of the land or people who have previously received housing from the government but sold, leased or transferred it and settled in the same place or another urban area. This term also applies for people who live in areas without the consent of the landowner and who have sufficient funds to live in legitimate housings.

For Temporary Relocates

As stipulated in RA 7279, adequate relocation, whether temporary or permanent, will be provided for all relocates. Temporary relocates will also be considered as PAPs. As such, if relocation is not immediately available or if means of livelihood is directly affected then just compensation, other entitlements and assistance, such as income loss, inconvenience allowance, rehabilitation assistance and other compensations agreed upon by the PAPs and the proponent, should be provided.



Source: Infrastructure Right-of-Way (IROW) Procedural Manual.2003

Figure 20.2.4-1 Flow Chart for Payment of Compensation to PAPs

(2) Restoration Guidelines

After resettlement, income restoration and livelihood rehabilitation of PAPs should be undertaken. DPWH will be actively involved in the implementation of the rehabilitation and livelihood restoration programs in partnership with NGOs and national government agencies.

Income restoration and livelihood rehabilitation planning will begin during the final engineering design phase. The plan should be responsive to the needs of the PAPs and in consonance with the development thrust of the affected LGU. Sample restoration needs and possible solutions to these concerns are summarized in Table 20.2.4-1.

Table 20.2.4-1 Sample Restoration and Possible Solutions

Sample Restoration Needs	Possible Solutions
Agricultural	Sustainable agriculture, agro-forestry and food security programs Related processing activities to products/harvest for value added
Cash Income / Job Opportunities/ Regular Employment	Income restoration strategy to provide for the immediate need for employment and economic opportunities at the relocation site Match manpower needs of project during construction and operations phase. Development of comprehensive and sustainable program that will address continuous supply of raw materials and food resources and to provide for sustainable income for the relocates even after construction
Social	Establishment of the Homeowners Association to address concerns on-site maintenance, peace and order, sanitation and cleanliness, building social relationships and network, among others Establishment of cooperatives to promote self-reliance among PAPs through capital build-up and savings formation and to serve as conduits of capital/loan assistance, micro-enterprise and livelihood programs. Capacity enhancement of cooperatives to ensure sustainability in their efforts at providing services to their members
Educational	Skills training/vocational technical education to provide opportunities for technical jobs Educational Scholarship Program to upgrade their educational achievement up to college

(3) Entitlement Matrix

Table 20.2.4-2 Sample Entitlements Matrix

Type of Loss	Application	Entitled Person	Compensation/Entitlements
LAND (Classified as Agricultural/ Residential/ Commercial/ Industrial/Institutional)	More than 20% of the total landholding is lost or where less than 20% lost but the remaining land holding becomes economically unviable.	PAPs with Transfer Certificate of Title (TCT) or tax declaration (tax declaration can be legalized to full title)	PAPs will be entitled to: Cash compensation for loss of land at 100% replacement cost at the informed request of PAPs If feasible, land for land will be provided in terms of a new parcel of land of equivalent productivity at a location acceptable to PAPs, or Holders of free or homestead patents and CLOAs under CA 141. Public Lands Act will be compensated on land Improvements only Holders of Certificates of Land Ownership Award (CLOA) granted under will be compensated for the land at zonal value Cash compensation for damaged crops at market value at the time of taking Rehabilitation assistance in the form of skills training equivalent to the amount of Php 15,000.00 per family, if the present

Type of Loss	Application	Entitled Person	Compensation/Entitlements
			means of livelihood is no longer viable and the PAPs will have to engage in a new income activity
		PAPs without TCT	Cash compensation, for damaged crops at market value at the time of taking Agricultural lessors are entitled to disturbance compensation equivalent to 5 times the average of the gross harvest, for the past 3 years but not less than Php 15,000.00
	Less than 20% of the total landholding loss or less than 20% loss or where the remaining structures still viable for use	PAPs with TCT or tax declaration (tax declaration can be legalized to full title)	PAPs will be entitled to: Cash compensation for loss of land at 100% replacement cost at the informed request of PAPs Holders of free or homestead patents and CLOAs under CA 141. Public Lands Act shall be compensated on Land Improvements only Holders of Certificates of Land Ownership Award (CLOA) granted under the Comprehensive Agrarian Reform Act shall be compensated for the land at zonal value Cash compensation for damaged crops at market value at the time of taking
		PAPs without TCT	Cash compensation for damaged crops at market value at the time of taking Agricultural lessors are entitled to disturbance compensation equivalent to 5 times the average of the gross harvest, for the past 3 years but not less than Php 15,000.00
STRUCTURES (Classified as Residential/ Commercial/ Industrial)	More than 20% of the total landholding is loss or where less than 20% loss but the remaining structures no longer function as intended or no longer viable for continued use	PAPs with TCT or tax declaration (tax declaration can be legalized to full title)	PAPs will be entitled to: Cash compensation for entire structure at 100% of replacement cost Rental subsidy for the time between the submission of complete documents and the release of payment on land
		PAPs without TCT	Cash compensation for entire structure at 100% of replacement cost Rental subsidy for the time between the submission of complete documents and the release of payment on land
	Less than 20% of the total landholding lost or where the remaining structure can still function and is viable for continued use	PAPs with TCT or tax declaration (tax declaration can be legalized to full title)	Compensation for affected portion of the structure
		PAPs without TCT	Compensation for affected portion of the structure
IMPROVEMENTS	Severely or marginally affected	PAPs with or without TCT, tax declaration, etc.	PAPs will be entitled to: Cash compensation for the affected improvements at replacement cost
CROPS, TREES, PERRENIALS			PAPs will be entitled to: Cash compensation for crops, tress and perennials at current market value as prescribed by the concerned LGUs and DENR

20.2.5 Grievance Redress System

A Grievance Redress System (GRS) should be established to ensure transparency in the use of funds and that grievances regarding the project are effectively and expeditiously resolved. This will provide the affected communities the opportunity to voice out any complaints and grievances regarding the overall implementation and process of the proposed Project.

The Resettlement Implementation Committee (RIC) will be responsible for receiving these and in the preparation and implementation of appropriate measures. Project-affected persons (PAPs) may also forward their concerns to the Regional Director or the concerned division of the LGU.

During community meetings, hand-outs/leaflets indicating the channels and related procedures in the submission of grievances shall be distributed to the public during community meetings. The same hand-out shall be used to explain GRS procedures to PAPs that come to file their grievances. Received documentation of their concerns will then be discussed during meetings for immediate action.

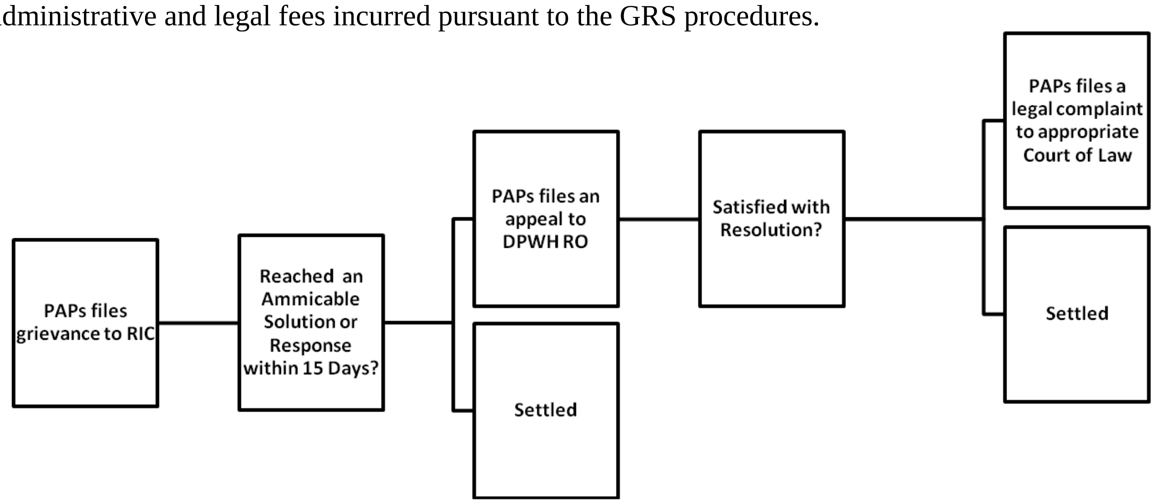
The grievances will be addressed through negotiations that aim to reach a consensus and will abide by the following procedure:

The PAPs will file their grievances by writing to the RIC for immediate resolution. When received verbally, the grievances may be translated in writing by the staff of the regional director, LGU, or PMO, or staff assigned by PMO, for submission.

If the complaint is not properly addressed, no understanding or amicable solution is attained or if PAPs does not receive a response from the RIC in 15 days, PAPs can file an appeal to the DPWH NCR Regional Office (RO).

As a last resort, if the PAP is still not satisfied with the resolution from the DPWH RO, the PAPs can file a legal complaint in any appropriate Court of Law

Grievances of PAPs shall be handled free of monetary charge and PAPs shall be exempted from all administrative and legal fees incurred pursuant to the GRS procedures.



Source: Land Acquisition, Resettlement Rehabilitation and Indigenous People Policy.2007

Figure 20.2.5-1 Redress Grievance Flow Chart

20.2.6 Implementation Framework

Proper implementation and monitoring of the resettlement action plan should be done by the following institutions, which will be responsible for specific roles:

Table 20.2.6-1 Implementation Framework

Institution	Roles
The Project Implementation Office/ Project Management Office (PMO) of the DPWH	In-charge of overall implementation of the project Manage and supervise all activities of the project, including resettlement and land acquisition Safeguard funds for the RAP, with regards to its timely implementation and accounting of expenses
Environmental and Social Services Office (ESSO)	Provide technical guidance and support to PMO in the implementation of the RAP Assist in the preparation and planning of the RAP, including the RAP budget plan Guide District Engineering Offices and Regional Offices in their tasks (verification of PAPs, information dissemination and others) With PMO, amend/revise the RAP to incorporate resettlement concerns identified during monitoring Monitor actual payment of compensation to PAPs Prepare periodic supervision and monitoring reports on RAP implementation prior to submission to DPWH and JICA
District Engineering Offices of the DPWH	Shall serve as the technical coordinator for the project Member of the Resettlement Implementation Committee Oversee the staking-out, verification and validation of PAPs assets Conduct inventories of properties that will be affected Approve disbursement vouchers/payments Submit reports on compensation to PAPs and monthly progress reports to the regional office and PMO
Regional Office (RO) of the DPWH	Shall serve as liaison between the ESSO and the District office Monitor RAP implementation and fund disbursement, including payment to PAPs and submit monthly reports to ESSO Address grievances and concerns of PAPs with regards to the project
Resettlement Implementation Committee (RIC)	Shall be composed of representatives from the Regional Office and District Engineering Office, LGU, PAPs Assist in the RAP activities, including validation of PAPs and their affected assets, payment of compensation to PAPs and in monitoring and implementation of RAP Take part during public information campaign, public participation and consultation activities Receive and address grievances and concerns of PAPs Records all public meetings, grievances and solutions to these Assist in enforcing the laws regarding Right-of-Way (ROW)
Department of Social Welfare and Development (DSWD), Technical Education and Skills Development Authority (TESDA), and Cooperative Development Authority (CDA)	Assist in the monitoring of the PAPs and resettlement activities, including consultations and ensuring just compensation Provide livelihood rehabilitation trainings to relocated PAPs

Source: LARRIPP, 3rd Edition (2007)

20.2.7 Schedule

Supposed schedule of IEE and LARAP procedures are shown in table.

Table 20.2.7-1 Schedule of IEE & LARAP

Implementation of LARAP	year	2014					2015												
	month	Aug	Sep	Oct	Nov	Dec	Jan	Feb	Mar	Apr	May	Jun	Jul	Aug	Sep	Oct	Nov	Dec	
	responsibility																		at least one year is necessary for clearance of land after detailed survey
L/A between Japan and Philippine																			after the issue of ECC actual
Approval of Project in Philippine																			LARAP process will start
																			45days to 6 months in case IEE
Stakeholders Meeting/Consultation	DPWH/LGU																		
submission of IEE	DPWH/ESSO																		
Report Review and Evaluation	EMB																		
Substantive Review	EMB																		
Endorsement of Recommendation	EMB																		
Sign-off/Issurance of Decision Document	EMB																		
Detailed Measurement Survey	DPWH, LGU(MDAA)																		
Installation of cut-off date	DPWH, LGU(MDAA)																		
Consultation meeting for Resettlement	DPWH, LGU(MDAA)																		
Cost estimation, budjeting	DPWH, LGU(MDAA)																		
Clearance of site																			
Monitoring of resettlement	DPWH, LGU(MDAA)																		will continue for two years

20.2.8 Cost Estimation

Outline of cost estimation of Land acquisition and compensation based on survey are shown in table.

Cost Estimation for 1st Mandaue-Mactan Bridge

Type	Unit	Number	Cost/Unit	Total Cost (Php)	Basis of Unit Numbers	Source of Estimation	Remarks
Compensation	**Land (m^2)**	0	1,900	0.00		Inquired from Municipal Hall or BIR Zonal Value	
	Structures						
	Severe	170	20,000	3,400,000.00	Structures Under the Bridge	Re-validated Resettlement Action Plan for CP-RI 2.1: Magapit-Sta. ana Road (Magapit-Mission Section) under the National Roads Improvement and Management Program Phase II (NRIMP-2). June 2012	Value from Sample RAP was estimated at a lower cost
	Marginal	0	10,000	0.00	Structures Near the Bridge	Re-validated Resettlement Action Plan for CP-RI 2.1: Magapit-Sta. ana Road (Magapit-Mission Section) under the National Roads Improvement and Management Program Phase II (NRIMP-2). June 2012	Value from Sample RAP was estimated at a lower cost
	Tree						
	Bearing Trees	0		0.00			
	Non-Bearing Trees	0		0.00			
Assistance and Allowance	**Transportation Allowance**	170	1,050	178,500.00	For relocating severely affected HH	RESETTLEMENT ACTION PLAN- PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III). September 2011	Value from Sample RAP was increased by 5%
	Rental Subsidy	0	3,000	0.00	No. of Severely Affected Renting Residents	RESETTLEMENT ACTION PLAN- PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III). September 2011	Value from Sample RAP was increased by 5%
	Inconvenience Allowance	170	10,000	1,700,000.00	No. of Severely Affected Structures (under the bridge)	Incovenience Allowance Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Livelihood Rehabilitation Assistance	0	15,000	0.00	No. of All Residential Structures	Livelihood Rehabilitation Assistance Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Disturbance Compensation	0	15,000	0.00	Severely Affected Agricultural Land	Disturbance Compensation Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Income Loss	0	15,000	0.00	No. of Commercial Structures Under the Bridge	Income Loss Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
			Subtotal	**5,278,500.00**			
			5 % Management Cost	**263,925.00**			
			10 % Contingencies	**527,850.00**			
			Estimated	**6,070,275.00**			

Cost Estimation for Liloan Bridge

Type	Unit	Number	Cost/Unit	Total Cost (Php)	Basis of Unit Numbers	Source of Estimation	Remarks
Compensation	**Land (m^2)**	0		0.00		Inquired from Municipal Hall or BIR Zonal Value	
	Structures						
	Severe	3	70,000	210,000.00	Structures Under the Bridge	Re-validated Resettlement Action Plan for CP-RI 2.1: Magapit-Sta. ana Road (Magapit-Mission Section) under the National Roads Improvement and Management Program Phase II (NRIMP-2). June 2012	Value from Sample RAP was increased by 10%
	Marginal	15	20,000	300,000.00	Structures Near the Bridge	Re-validated Resettlement Action Plan for CP-RI 2.1: Magapit-Sta. ana Road (Magapit-Mission Section) under the National Roads Improvement and Management Program Phase II (NRIMP-2). June 2012	Value from Sample RAP was increased by 10%
	Tree						
	Bearing Trees			0.00			
	Non-Bearing Trees			0.00			
Assistance and Allowance	**Transportation Allowance**	15	1,050	15,750.00	For relocating severely affected HH	RESETTLEMENT ACTION PLAN- PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III). September 2011	Value from Sample RAP was increased by 5%
	Rental Subsidy	0	3,000	0.00	No. of Severely Affected Renting Residents	RESETTLEMENT ACTION PLAN- PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III). September 2011	Value from Sample RAP was increased by 5%
	Inconvenience Allowance	15	10,000	150,000.00	No. of Severely Affected Structures (under the bridge)	Incovenience Allowance Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Livelihood Rehabilitation Assistance	0	15,000	0.00	No. of All Residential Structures	Livelihood Rehabilitation Assistance Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Disturbance Compensation	0	15,000	0.00	Severely Affected Agricultural Land	Disturbance Compensation Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Income Loss	1	15,000	15,000.00	No. of Commercial Structures Under the Bridge	Income Loss Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
			Subtotal	**690,750.00**			
			5 % Management Cost	**34,537.50**			
			10 % Contingencies	**69,075.00**			
			Estimated	**794,362.50**			

Cost Estimation for Lambingan Bridge

Type	Unit	Number	Cost/Unit	Total Cost (Php)	Basis of Unit Numbers	Source of Estimation	Remarks
Compensation	**Land (m^2)**	0	5,240	0.00		Inquired from Municipal Hall or BIR Zonal Value	
	Structures						
	Severe	1	70,000	70,000.00	Structures Under the Bridge	Re-validated Resettlement Action Plan for CP-RI 2.1: Magapit-Sta. ana Road (Magapit-Mission Section) under the National Roads Improvement and Management Program Phase II (NRIMP-2). June 2012	Value from Sample RAP was increased by 10%
	Marginal	41	20,000	820,000.00	Structures Near the Bridge	Re-validated Resettlement Action Plan for CP-RI 2.1: Magapit-Sta. ana Road (Magapit-Mission Section) under the National Roads Improvement and Management Program Phase II (NRIMP-2). June 2012	Value from Sample RAP was increased by 10%
	Tree						
	Bearing Trees	0		0.00			
	Non-Bearing Trees	0		0.00			
Assistance and Allowance	**Transportation Allowance**	1	1,050	1,050.00	For relocating severely affected HH	RESETTLEMENT ACTION PLAN- PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III). September 2011	Value from Sample RAP was increased by 5%
	Rental Subsidy	0	3,000	0.00	No. of Severely Affected Renting Residents	RESETTLEMENT ACTION PLAN- PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III). September 2011	Value from Sample RAP was increased by 5%
	Inconvenience Allowance	1	10,000	10,000.00	No. of Severely Affected Structures (under the bridge)	Incovenience Allowance Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Livelihood Rehabilitation Assistance	0	15,000	0.00	No. of All Residential Structures	Livelihood Rehabilitation Assistance Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Disturbance Compensation	0	15,000	0.00	Severely Affected Agricultural Land	Disturbance Compensation Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Income Loss	0	15,000	0.00	No. of Commercial Structures Under the Bridge	Income Loss Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
			Subtotal	**901,050.00**			
			5 % Management Cost	**45,052.50**			
			10 % Contingencies	**90,105.00**			
			Estimated total compensation	**1,036,207.50**			

Cost Estimation for Guadalupe Bridge

Type	Unit	Number	Cost/Unit	Total Cost (Php)	Basis of Unit Numbers	Source of Estimation	Remarks
Compensation	**Land (m^2)**	0	17,500	0.00		Inquired from Municipal Hall or BIR Zonal Value	
	Structures						
	Severe	6	70,000	420,000.00	Structures Under the Bridge	Re-validated Resettlement Action Plan for CP-RI 2.1: Magapit-Sta. ana Road (Magapit-Mission Section) under the National Roads Improvement and Management Program Phase II (NRIMP-2). June 2012	Value from Sample RAP was increased by 10%
	Marginal	11	20,000	220,000.00	Structures Near the Bridge	Re-validated Resettlement Action Plan for CP-RI 2.1: Magapit-Sta. ana Road (Magapit-Mission Section) under the National Roads Improvement and Management Program Phase II (NRIMP-2). June 2012	Value from Sample RAP was increased by 10%
	Tree						
	Bearing Trees	0		0.00			
	Non-Bearing Trees	0		0.00			
Assistance and Allowance	**Transportation Allowance**	6	1,050	6,300.00	For relocating severely affected HH	RESETTLEMENT ACTION PLAN- PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III). September 2011	Value from Sample RAP was increased by 5%
	Rental Subsidy	0	3,000	0.00	No. of Severely Affected Renting Residents	RESETTLEMENT ACTION PLAN- PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III). September 2011	Value from Sample RAP was increased by 5%
	Inconvenience Allowance	6	10,000	60,000.00	No. of Severely Affected Structures (under the bridge)	Incovenience Allowance Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Livelihood Rehabilitation Assistance	0	15,000	0.00	No. of All Residential Structures	Livelihood Rehabilitation Assistance Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Disturbance Compensation	0	15,000	0.00	Severely Affected Agricultural Land	Disturbance Compensation Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Income Loss	0	15,000	0.00	No. of Commercial Structures Under the Bridge	Income Loss Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
			Subtotal	**706,300.00**			
			5 % Management Cost	**35,315.00**			
			10 % Contingencies	**70,630.00**			
			Estimated	**812,245.00**			

Cost Estimation for Palanit Bridge

Type	Unit	Number	Cost/Unit	Total Cost (Php)	Basis of Unit Numbers	Source of Estimation	Remarks
Compensation	**Land (m^2)**	0	150	0.00		Inquired from Municipal Hall or BIR Zonal Value	
	Structures						
	Severe	2	70,000	140,000.00	Structures Under the Bridge	Re-validated Resettlement Action Plan for CP-RI 2.1: Magapit-Sta. ana Road (Magapit-Mission Section) under the National Roads Improvement and Management Program Phase II (NRIMP-2). June 2012	Value from Sample RAP was increased by 10%
	Marginal	0	20,000	0.00	Structures Near the Bridge	Re-validated Resettlement Action Plan for CP-RI 2.1: Magapit-Sta. ana Road (Magapit-Mission Section) under the National Roads Improvement and Management Program Phase II (NRIMP-2). June 2012	Value from Sample RAP was increased by 10%
	Tree						
	Bearing Trees	0		0.00			
	Non-Bearing Trees	0		0.00			
Assistance and Allowance	**Transportation Allowance**	2	1,050	2,100.00	For relocating severely affected HH	RESETTLEMENT ACTION PLAN- PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III). September 2011	Value from Sample RAP was increased by 5%
	Rental Subsidy	0	3,000	0.00	No. of Severely Affected Renting Residents	RESETTLEMENT ACTION PLAN- PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III). September 2011	Value from Sample RAP was increased by 5%
	Inconvenience Allowance	2	10,000	20,000.00	No. of Severely Affected Structures (under the bridge)	Incovenience Allowance Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Livelihood Rehabilitation Assistance	0	15,000	0.00	No. of All Residential Structures	Livelihood Rehabilitation Assistance Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Disturbance Compensation	0	15,000	0.00	Severely Affected Agricultural Land	Disturbance Compensation Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Income Loss	0	15,000	0.00	No. of Commercial Structures Under the Bridge	Income Loss Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
			Subtotal	**162,100.00**			
			5 % Management Cost	**8,105.00**			
			10 % Contingencies	**16,210.00**			
			total compensation	**186,415.00**			

Cost Estimation for Mawo Bridge

Type	Unit	Number	Cost/Unit	Total Cost (Php)	Basis of Unit Numbers	Source of Estimation	Remarks
Compensation	**Land (m^2)**	0	150	0.00		Inquired from Municipal Hall or BIR Zonal Value	
	Structures						
	Severe	9	70,000	630,000.00	Structures Under the Bridge	Re-validated Resettlement Action Plan for CP-RI 2.1: Magapit-Sta. ana Road (Magapit-Mission Section) under the National Roads Improvement and Management Program Phase II (NRIMP-2). June 2012	Value from Sample RAP was increased by 10%
	Marginal	4	20,000	80,000.00	Structures Near the Bridge	Re-validated Resettlement Action Plan for CP-RI 2.1: Magapit-Sta. ana Road (Magapit-Mission Section) under the National Roads Improvement and Management Program Phase II (NRIMP-2). June 2012	Value from Sample RAP was increased by 10%
	Tree						
	Bearing Trees	0		0.00			
	Non-Bearing Trees	0		0.00			
Assistance and Allowance	**Transportation Allowance**	9	1,050	9,450.00	For relocating severely affected HH	RESETTLEMENT ACTION PLAN- PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III). September 2011	Value from Sample RAP was increased by 5%
	Rental Subsidy	0	3,000	0.00	No. of Severely Affected Renting Residents	RESETTLEMENT ACTION PLAN- PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III). September 2011	Value from Sample RAP was increased by 5%
	Inconvenience Allowance	9	10,000	90,000.00	No. of Severely Affected Structures (under the bridge)	Incovenience Allowance Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Livelihood Rehabilitation Assistance	0	15,000	0.00	No. of All Residential Structures	Livelihood Rehabilitation Assistance Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Disturbance Compensation	0	15,000	0.00	Severely Affected Agricultural Land	Disturbance Compensation Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Income Loss	0	15,000	0.00	No. of Commercial Structures Under the Bridge	Income Loss Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
			Subtotal	**809,450.00**			
			5 % Management Cost	**40,472.50**			
			10 % Contingencies	**80,945.00**			
			total compensation	**930,867.50**			

Cost Estimation for Wawa Bridge

Type	Unit	Number	Cost/Unit	Total Cost (Php)	Basis of Unit Numbers	Source of Estimation	Remarks
Compensation	**Land (m^2)**	2,620	80	209,612.89		Inquired from Municipal Hall or BIR Zonal Value	
	Structures						
	Severe	25	70,000	1,750,000.00	Structures Under the Bridge	Re-validated Resettlement Action Plan for CP-RI 2.1: Magapit-Sta. ana Road (Magapit-Mission Section) under the National Roads Improvement and Management Program Phase II (NRIMP-2). June 2012	Value from Sample RAP was increased by 10%
	Marginal	0	20,000	0.00	Structures Near the Bridge	Re-validated Resettlement Action Plan for CP-RI 2.1: Magapit-Sta. ana Road (Magapit-Mission Section) under the National Roads Improvement and Management Program Phase II (NRIMP-2). June 2012	Value from Sample RAP was increased by 10%
	Tree						
	Bearing Trees			0.00			
	Non-Bearing Trees			0.00			
Assistance and Allowance	**Transportation Allowance**	25	1,050	26,250.00	For relocating severely affected HH	RESETTLEMENT ACTION PLAN-PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III). September 2011	Value from Sample RAP was increased by 5%
	Rental Subsidy	0	3,000	0.00	No. of Severely Affected Renting Residents	RESETTLEMENT ACTION PLAN-PASIG-MARIKINA RIVER CHANNEL IMPROVEMENT PROJECT (PHASE III). September 2011	Value from Sample RAP was increased by 5%
	Inconvenience Allowance	25	10,000	250,000.00	No. of Severely Affected Structures (under the bridge)	Incovenience Allowance Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Livelihood Rehabilitation Assistance	0	15,000	0.00	No. of All Residential Structures	Livelihood Rehabilitation Assistance Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Disturbance Compensation	0	15,000	0.00	Severely Affected Agricultural Land	Disturbance Compensation Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
	Income Loss	0	15,000	0.00	No. of Commercial Structures Under the Bridge	Income Loss Rate from LARRIPP: DPWH Policy and Guideline on Resettlement (2007)	
			Subtotal	**2,235,862.89**			
			5 % Management Cost	**111,793.14**			
			10 % Contingencies	**223,586.29**			
			Estimated	**2,571,242.32**			

20.2.9 Internal and External Monitoring and Evaluation

(1) Internal Monitoring Evaluation

Internal monitoring will be conducted by the Multi-partite committee, composed of representatives of the following:

- Local Government Units (LGUs)
- AP representatives (cooperatives/HOAs)
- NGOs
- Resettlement Unit
- DPWH-PMO
- National Agencies (i.e., DENR, DAR, DA, DTI)

The committee will assess the status and progress of the delivery of entitlements and assistance, income restoration and rehabilitation efforts as defined in the resettlement plan. It will also determine the PAPs socio-economic conditions at the relocation site and what type of assistance is further needed to improve their living conditions. This internal monitoring will ensure the immediate response to the problems and issues that may arise immediately after relocation.

Monitoring will be undertaken at the household level on a monthly basis for the first (1^{st}) year after relocation and quarterly on the second (2^{nd}) year onwards.

Monthly monitoring will also be undertaken by the Community Relations and Resettlement Unit to document the entitlement received and the involvement of each family in the programs and services being provided. It will also look at the extent of community participation to determine the level of social interaction and degree of relationships established within the relocation site.

(2) External Monitoring and Evaluation

An external monitoring and evaluation will be undertaken by a qualified independent agency which can be from an NGO, academe or a consulting group.

External monitoring and evaluation will be conducted to assess the implementation of the resettlement process, the operation and management of resettlement sites and the delivery and responsiveness of the entitlement and benefit package. It will also verify the results of the internal monitoring and evaluate whether objectives of the resettlement program are met.

The independent group will conduct semi-annual reviews for the entire duration of the resettlement process and an End-of-Project Evaluation. Both qualitative and quantitative methods will be used in the process. Table 20.2.9-1 shows a sample of monitoring/evaluation indicators.

Table 20.2.9-1 Sample of Monitoring/Evaluation Indicators

Scope	Indicators
a. Monitoring Indicators	
Project Implementation	Number of home lots and farm lots developed and used by relocates
	Resettlement procedure carried out as planned and scheduled
	Social preparation carried out and results achieved
Delivery of Compensation/ Delivery of Entitlement	Number of AFs provided with transport services during relocation
	Number of AFs provided with residential/farm lots
	Number of AFs provided with livelihood skills training
	Number of AFs employed with the project
	Number of AFs trained and have access to loans/micro-credit
	Number of women/men engaged in productive activities
	Number of AFs provided with employment and job referrals
Consultation and Grievance	Frequency of community meetings and consultation
	Number of relocates in the resettlement site who are assisted in their grievances
b. Evaluation Indicators	
Benefit Impact /Evaluation	Changes in housing conditions of AFs
	Changes in income and expenditures
	Changes and improvement in the general community situation
	Changes in health condition of women and children
	Changes in relationship of family and community
	Changes in quality life among relocates
Sustainability Mechanisms Established	Number of organizations established and number of members at the relocation sites (HOA, livelihood associations, cooperatives, etc.)
	Level of savings/capital saved by livelihood associations and cooperatives
	Linkages and Network established by associations and cooperatives
	Financial net worth of cooperatives and associations

20.3 Others

20.3.1 Categorization on JICA Guidelines for Environmental and Social Considerations

JICA classifies projects into four categories according to the extent of environmental and social impacts, taking into account an outline of projects, scale, site condition, etc.

According to this study result proposed project of 1st Mandaue rehabilitation is classified as Category A because of significant adverse impact on society, large-scale involuntary resettlement. (See Table 20.2.3-3 Estimated Number of Household members to be resettle)

(Abstract from JICA Guidelines refer to Category A procedures as follows)

On necessary projects among Categories A and Category B, the Advisory Committee for Environmental and Social Considerations gives advice on environmental and social considerations in preparatory surveys. JICA reports to the Committee, and the Committee gives advice as needed at the environmental review and monitoring stages. On the projects of technical cooperation for development planning, the Advisory Committee gives advice at full-scale study stage.

Project proponents etc. must submit EIA reports for Category A projects. For projects that will result in large-scale involuntary resettlement, a Resettlement Action Plan (RAP) also must be submitted.

JICA publishes the status of host countries' submission of major documents on environmental and social considerations on its website. Prior to its environmental review, JICA also discloses the following: (1) EIA reports and environmental permit certifications, (2) RAPs for projects that will result in large-scale involuntary resettlement. Specifically, JICA discloses EIA reports 120 days prior to concluding agreement documents. In addition, JICA discloses a translated version of these major documents, subject to approval by project proponents etc.

PART 5

PROJECT IMPLEMENTATION AND RECOMMENDATIONS

CHAPTER 21 PROJECT IMPLEMENTATION

21.1 Project Outline

As a result of the evaluation of conditions of existing bridges thru the first and second screening, the priority ranks and improvement measures of the target bridges are studied. Based on the studies, the outline design of two (2) bridges from Package B and five (5) bridges from Package C are conducted with recommendable improvement measures as shown in Table 21.1-1.

The project in Metro Manila shall be implemented under severe urban environment such as construction work at traffic congested areas and narrow working space and satisfactory work to minimize the traffic disturbance, safety construction. Most of Japanese contractors have enough experiences and technologies to cope with such severe conditions in densely urbanized areas. Moreover, the Project needs special technology for the bridge seismic improvements. Thus, this project can be one of the model projects for which Japanese contractors can exercise their technologies in the following fields,

(1) Seismic Retrofitting of Bridge Pier
(2) Installation of Unseating/Fall-down Prevention System
(3) Seismic Retrofitting of Foundation
(4) Ground Improvement against Liquefaction
(5) Base Isolation/Menshin Technology
(6) Neighboring/Proximity Construction Technology
(7) Rapid Construction Technology

Table 21.1-1 Project Outline

Package B		
Bridge Name	Proposed Improvement Measures	Description
1 Lambingan Br.	Replacement	**Length** Bridge: 90 m Approach Rd.: 240 m (119 m+121 m) **Type** Superstructure: Simple Steel Deck Lohse Arch Stiffening Box Girder Substructure: RC Reversed T Type Abutment Foundation: Cast-in-place Concrete Pile
2. Guadalupe Br.	Replacement/ Partial Seismic Retrofit	**Length** Bridge: 125 m (41.1 m + 42.8 m + 41.1 m) Approach Rd.: N/A **Type** Superstructure: 3-span Continuous Steel Deck Box Girder Substructure: RC Wall Type Pier/RC Reversed T Type Abutment Foundation: Steel Pipe Sheet Pile Foundation **Seismic Retrofit** Soil Improvement

Table 21.1-1 Project Outline

Package C		
Bridge Name	Proposed Improvement Measures	Description
1. 1st Mandaue-Mactan Br.	Seismic Retrofit	**Length** Bridge: 860 m (Existing) **Seismic Retrofit** Seismic Damper, Concreting Jacket, Cast-in-place Concrete Pile, Steel Pipe Sheet Pile Foundation and Unseating Prevention System
2. Palanit Br.	Replacement	**Length** Bridge: 82 m (27 m + 28 m + 27 m) Approach Rd.: 135 m (98 m + 37 m) **Type** Superstructure: 3-span PC-I Girder Substructure: RC Single Column Pier (Circular Type)/ RC Reversed T Type Abutment Foundation: Spread Footing Foundation
3. Mawo Br.	Replacement	**Length** Bridge: 205 m (62.5 m + 80.0 m + 62.5 m) Approach Rd.: 267 m (151 m + 112 m) **Type** Superstructure: 3-Span Continuous PC Fin-back Box Girder Substructure: RC Wall Type Pier/RC Reversed T Type Abutment Foundation: Cast-in-place Concrete Pile
4. Lilo-an Br.	Seismic Retrofit	**Length** Bridge: 298 m (Existing) **Seismic Retrofit** Seismic Damper, Concreting Jacket, Cast-in-place Concrete Pile and Unseating Prevention System
5. Wawa Br.	Replacement	**Length** Bridge: 230 m (75.0 m + 80.0 m + 75.0 m) Approach Rd.: 296 m (197 m + 99m) **Type** Superstructure: 3-Span Continuous Composite Steel Truss Substructure: RC Wall Type Pier/RC Reversed T Type Abutment Foundation: Cast-in-place Concrete Pile

Note: All replacement bridges including instauration of unseating prevention system.

21.2 Implementation Schedule

The proposed implementation schedule is shown in Table 21.2-1.

Table 21.2-1 Proposed Implementation Schedule

	2014	2015	2016	2017	2018	2019	2020	2021
ECC (Environmental Compliance Certificate)	X							
NEDA-ICC (NEDA Board, Investment Coordination Committees)	X							
Appraisal Mission	X							
Detailed Design and Tender Assistance		Selection 12 month						
Tendering				15 month				
Construction					32 month			
Operation & Maintenance								Jan. 2021

21.3 Project Organization

Project implementing agency is the Department of Public Works and Highways (DPWH) and project implementing office is Project Management Office (PMO).

The proposed project organization is shown in Figure 21.3-1.

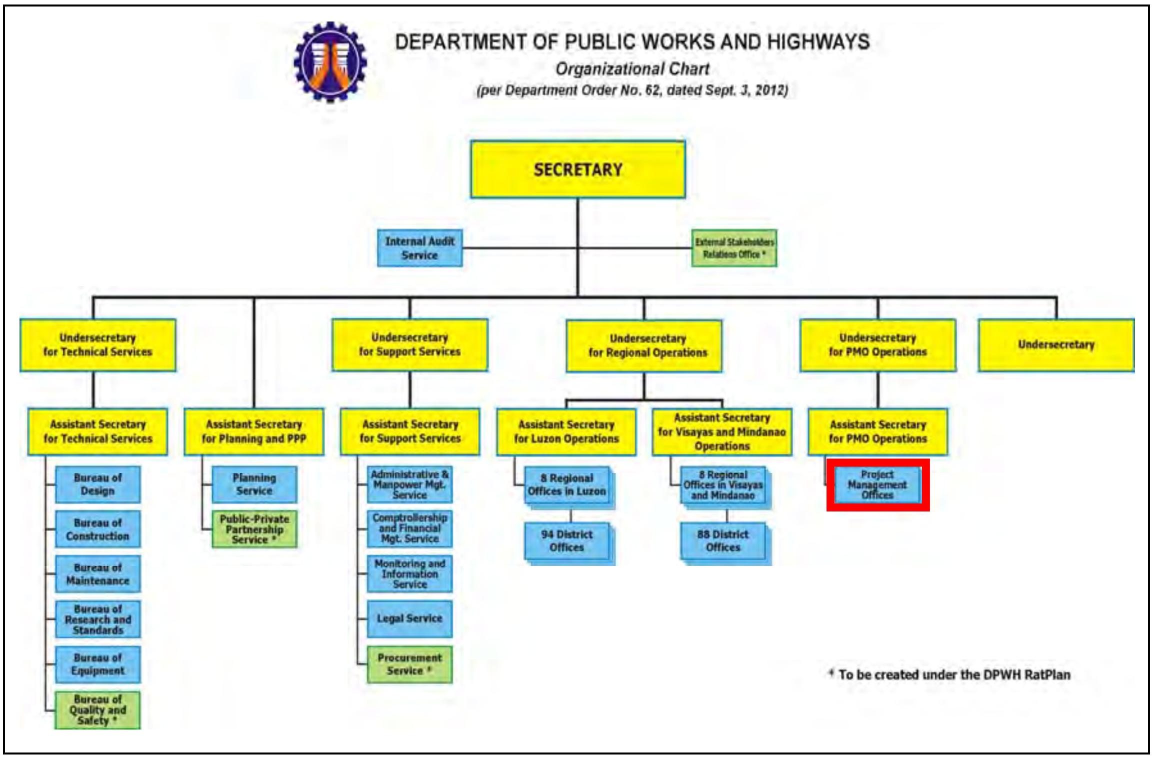


Figure 21.3-1 Proposed Project Organization

21.4 Financial Analysis and Funding

The economic evaluation of the bridge improvement project is carried out by comparing the economic cost of the project with the economic benefit that will be brought about by the bridge replacement/retrofit.

The following three indexes are used to assess the project viability:

- Economic Internal Rate of Return (EIRR)
- Benefit Cost Ratio (B/C Ratio)
- Net Present Value (NPV)

The result of economic evaluation by bridges is shown in Table 21.4-1. All bridges were evaluated as economically feasible.

Table 21.4-1 Results of Economic Evaluation by Bridges

Bridge	EIRR	B/C Ratio	NPV (Million Peso @ i=15%)
Lambingan	27.1%	1.90	
Guadalupe	26.8%	2.08	
1st Mandaue- Mactan	20.3%	1.42	
Palanit	19.1%	1.27	
Mawo	16.1%	1.06	
Liloan	19.6%	1.25	
Wawa	15.4%	1.02	
Projects (all seven bridges)	**22.9%**	**1.60**	

Source: JICA Study Team

The Project Sensitivity to the identified risks is shown in Table 21.4-2.

Table 21.4-2 Project Sensitivity

	Base	Cost plus 10%	Cost plus 20%
Base	22.9%	21.1%	19.6%
Benefit less 10%	20.9%	19.3%	17.8%
Benefit less 20%	18.9%	17.3%	16.0%

Source: JICA Study Team

Results show that the project is able to hurdle the minimum acceptable criteria of EIRR that is 15%. Even if cost goes up and/or benefit goes down as shown in the following condition, the minimum criteria of 15% EIRR would still meet.

- Cost plus 60%
- Benefit less 47%
- Cost plus 23% and Benefit less 23%

CHAPTER 22 RECOMMENDATIONS

22.1 Proposed Bridge Seismic Design Specifications (BSDS)

The major points of the proposed BSDS that is different from the current bridge seismic design specifications are as follows.

(A) Establishment of Seismic Performance Requirements

- Seismic performance requirements and bridge operational classification were established, which is to be for the first time in the Philippines.

(B) Localized Seismic Hazard Maps

- Distribution of active faults and ocean trenches in the Philippines were reflected in the seismic hazard maps which are shown as design seismic ground acceleration and response spectral acceleration coefficient contour maps by region at the surface of soil type B specified in AASHTO, which is likewise for the first time in the Philippines.
- The design seismic ground accelerations specified in the BSDS will be the basis for sustainable development of the bridge seismic design in the Philippines because the future data gained from new earthquake events in the Philippines can be reflected into the specifications following the process done in this study.

(C) Adoption of Latest Design Method

- The Load and Resistance Factor Design (LRFD) method was employed following AASHTO 2012 version as the base specifications, including change in design earthquake return period from 500 years to 1,000 years.

(D) Introduction of JRA Falling Down Prevention System

- The JRA falling down prevention system was introduced, considering similarity of ground conditions between the Philippines and Japan.
- Components of the system are: (a) design method on effects of seismically unstable ground, (b) unseating prevention system, and (c) requirements for seismically isolated bridges.

(E) Other major points

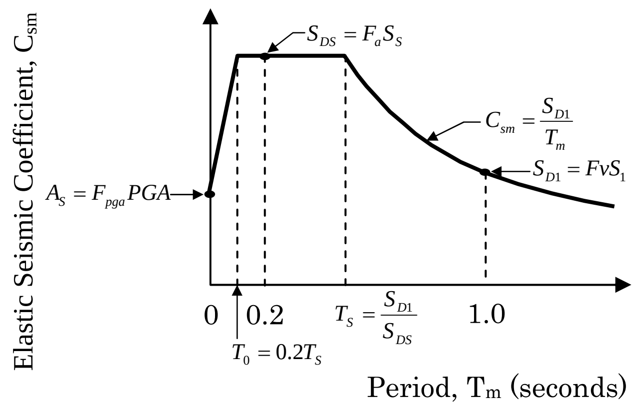
- Ground types for seismic design were classified into three types based on the JRA methods, which can be identified with the Characteristic Value of Ground (TG(s)) which are to be calculated with N-values.
- Effects and extent of liquefaction were reflected in the foundation design.

In this study, a seismic design manual and two seismic design examples were prepared to deepen the understanding and prevent misunderstanding of the proposed BSDS. The following six (6) actions are recommended for DPWH in order to make the proposed BSDS effective and useful, leading to mitigation of disasters caused by large scale earthquakes.

(1) Since the major points of Items (A), (B) and (C) above largely affect the scale of bridge substructures including foundations, the DPWH should make careful trial design and accumulate design experiences from the various angles so as to avoid sudden large change in the scale of bridge substructures including foundations compared to the one designed by the current seismic design procedures. When determining the acceleration response spectra acting on the structure as seismic forces, administrative judgment sometimes is required considering uncertainties of the analysis results without referring to actual recorded ground motion data and the country's budgetary capacity.

Figure 22.1-1 shows recommendation on the acceleration response spectra at present for Level-2 earthquake, which recommends setting the upper and lower limits for PGA considering the present situations of experience and the progress of technology and research in this field.

		Future Issue		Recommended Range of PGA for Level-2 Earthquake										Future Issue	
		As and S_{DS} corresponding to PGA													
PGA (Soil Type B)		PGA= 0.10	S_{DS}/As	PGA= 0.20	S_{DS}/As	PGA= 0.30	S_{DS}/As	PGA= 0.40	S_{DS}/As	PGA= 0.50	S_{DS}/As	PGA= 0.60	S_{DS}/As	PGA= 0.80	S_{DS}/As
Reference (Soil Type B)	**Fpga(=Fa)**	1.00		1.00		1.00		1.00		1.00		1.00		1.00	
	As(Fpga*PGA)	0.10	2.50	0.20	2.50	0.30	2.50	0.40	2.50	0.50	2.50	0.60	2.50	0.80	2.50
	S_{DS}(Fa*Ss)	0.25		0.50		0.75		1.00		1.25		1.50		2.00	
Soil Type I (C)	**Fpga(=Fa)**	1.20		1.20		1.10		1.00		1.00		1.00		1.00	
	As(Fpga*PGA)	0.12	2.50	0.24	2.50	0.33	2.50	0.40	2.50	0.50	2.50	0.60	2.50	0.80	2.50
	S_{DS}(Fa*Ss)	0.30		0.60		0.83		1.00		1.25		1.50		2.00	
Soil Type II (D)	**Fpga(=Fa)**	1.60		1.40		1.20		1.00		0.90		0.89		0.88	
	As(Fpga*PGA)	0.16	2.50	0.28	2.50	0.36	2.50	0.40	2.50	0.45	2.51	0.54	2.50	0.70	2.50
	S_{DS}(Fa*Ss)	0.40		0.70		0.90		1.00		1.13		1.35		1.75	
Type III (E, F)	**Fpga(=Fa)**	2.50		1.70		1.20		0.90		0.80		0.77		0.75	
	As(Fpga*PGA)	0.25	2.50	0.34	2.50	0.36	2.50	0.36	2.50	0.40	2.50	0.47	2.51	0.60	2.50
	S_{DS}(Fa*Ss)	0.63		0.85		0.90		0.90		1.00		1.18		1.50	



Future Issues*):
Upper and lower limits will be expanded based on the future experience, development of technology and research in this field including recommended As and S_{DS} corresponding to each PGA.
Regarding Fv corresponding to the above, please refer to this report.

Minimum PGA=0.2 is recommended only for Palawan and Sulu islands, for the other areas 0.3 is recommended as a minimum PGA taking account of the 2013 Bohol Earthquake.

Figure 22.1-1 Recommendation on Acceleration Response Spectra

(2) Major points of Items (B), (D) and (E) above should be authorized immediately after submission of this final report because they are directly linked with the safety of bridges during earthquakes. DPWH does not need to fix, at present, the return periods in the major point Item (B) for the seismic design. It is better to improve the proposed BSDS through the above trial design, which means that transition period is to be required.

(3) Through the above process, the proposed BSDS should be totally authorized as soon as possible, and the DPWH should take actions to disseminate the authorized BSDS nationwide in order to firmly make it rooted in bridge seismic design practice.

(4) The Standard design procedure and the standard design drawings should be revised based on the new BSDS.

In addition to the above, action (5) and (6) below is recommended to be taken.

(5) With data on the new fault of the 2013 Bohol Earthquake, seismic hazard maps are recommended to be verified and updated.

(6) The BSDS categorizes bridges according to its operational class, which is a function of the bridge importance. In this regards, it is recommended that DPWH-BOD coordinates with the Planning Service division in order to designate the bridge operational classification according to the road function especially roads belonging to the regional disaster prevention routes.

(7) Since the current design practice in AASHTO has been shifting from the force-based R factor design approach to the displacement-based design approach, it is recommended for DPWH to consider the displacement-based design approach in the future so that design engineers could easily imagine and judge the behavior of the structures' displacement according to the scales of the seismic design lateral forces. It should be noted that the BSDS is based on the current design procedure being employed by the DPWH.

With respect to the activities or items shown in Table 22.1-1, further supports seem to be needed as a transition period so as to make the outcome of this study meaningful and sustainable.

Table 22.1-1 Transition Period Recommended for Sustainable Development

<table>
<tr><th></th><th>1st Year</th><th>2nd Year</th></tr>
<tr><td>(1) Trial Design/Accumulation of Design Experience</td><td>Design Trial and Accumulation Stage</td><td>Revision Stage</td></tr>
<tr><td>(2) Capacity Development</td><td colspan="2">Repeated Training and Holding Seminar</td></tr>
<tr><td>(3) Implementation of a Pilot Project</td><td>Target Bridge Selection and Detailed Design</td><td>Implementation</td></tr>
<tr><td>(4) Preparation of New Standard Design Procedure and Drawings</td><td></td><td>Preparation</td></tr>
<tr><td>(5) Preparation of Bridge Retrofit Manual</td><td>Preparation</td><td></td></tr>
<tr><td>(6) Inter Agency Committee Meeting*</td><td></td><td></td></tr>
<tr><td>Remarks</td><td colspan="2">* Inter Agency Committee Meeting (IACM) consists of DPWH, PHILVOCS, ASEP, UP, Geological Society, under which working group will be needed to maintain close coordination.</td></tr>
</table>

22.2 Implementation of the project for seismic strengthening of bridges recommended in the Study

(1) Urgency of Project Implementation

Seismic resistance capacities of seven (7) bridges out of 33 subject bridges are recommended to be strengthened urgently after conducting the various careful investigation and study in this project. Among them, Lambingan Bridge and the outer section of Guadalupe Bridges are strongly recommended to be replaced immediately in terms of not only seismic safety but also the superstructures' safety against traffic loads considering their importance. Though both bridges are located on the soft ground having high potential of liquefaction, nobody knows the foundation types and conditions of both bridges including whether the foundations are being placed in the stable bearing layers. If Guadalupe Bridge collapses similar to the bridges which collapsed mainly due to liquefaction by the 2013 Bohol Earthquake, the 2012 Negros Earthquake and the 1990 North Luzon Earthquake, its impact on the Philippine economy and the human lives cannot be imagined which may lead to devastation.

Properly designed and constructed new Lambingan Bridge and Guadalupe Bridges will have reliable resistance capacity against expected large earthquakes, which will perform as if they were the "Savior Bridges" because the real seismic resistance capacities of the other old bridges crossing over the Pasig and Marikina Rivers against expected large earthquakes are unknown.

The other five (5) bridges of Package C, of which three (3) bridges are to be replaced and two (2) bridges are to be retrofitted, are all vulnerable to large scale earthquakes and recommended to be implemented according to the implementation schedule of this report at appropriate timing, considering their importance.

(2) Utilization of Japanese Technology for Project Implementation

Seismic resistance improvements of bridges require experience and special technology for design and construction. Therefore, it is recommended that this project be a model project for Philippine seismic performance improvement of bridges utilizing Japan's rich technology in the area of:

(a) Seismic Retrofitting of Bridge Piers.
(b) Installation of Unseating/Fall-down Prevention System.
(c) Seismic Retrofitting of Foundation
(d) Improvement of soil layers with liquefaction potential.
(e) Base Isolation/ Menshin Technology.
(f) Construction Technology under limited space or constrained working conditions / very near by existing Structures).
(g) Rapid Construction Technology.

(a) Seismic retrofitting of bridge pier

(b) Installation of falling down prevention system and dampers

(c) Seismic retrofitting of foundation

(f) Foundation construction technology under limited space and constrained working conditions with press-in method for piles

(3) Importance of Construction Quality and Proper Maintenance Activities

Seismic resistance capacity of structures will not be governed only by appropriate seismic design but also by the construction quality. Proper maintenance activities, on the other hand, are also essential to maintain the quality of the constructed structures having appropriate seismic resistance capacity. It is recommended that the DPWH take proper care to construct structures with high quality and maintain their quality through proper maintenance activities.

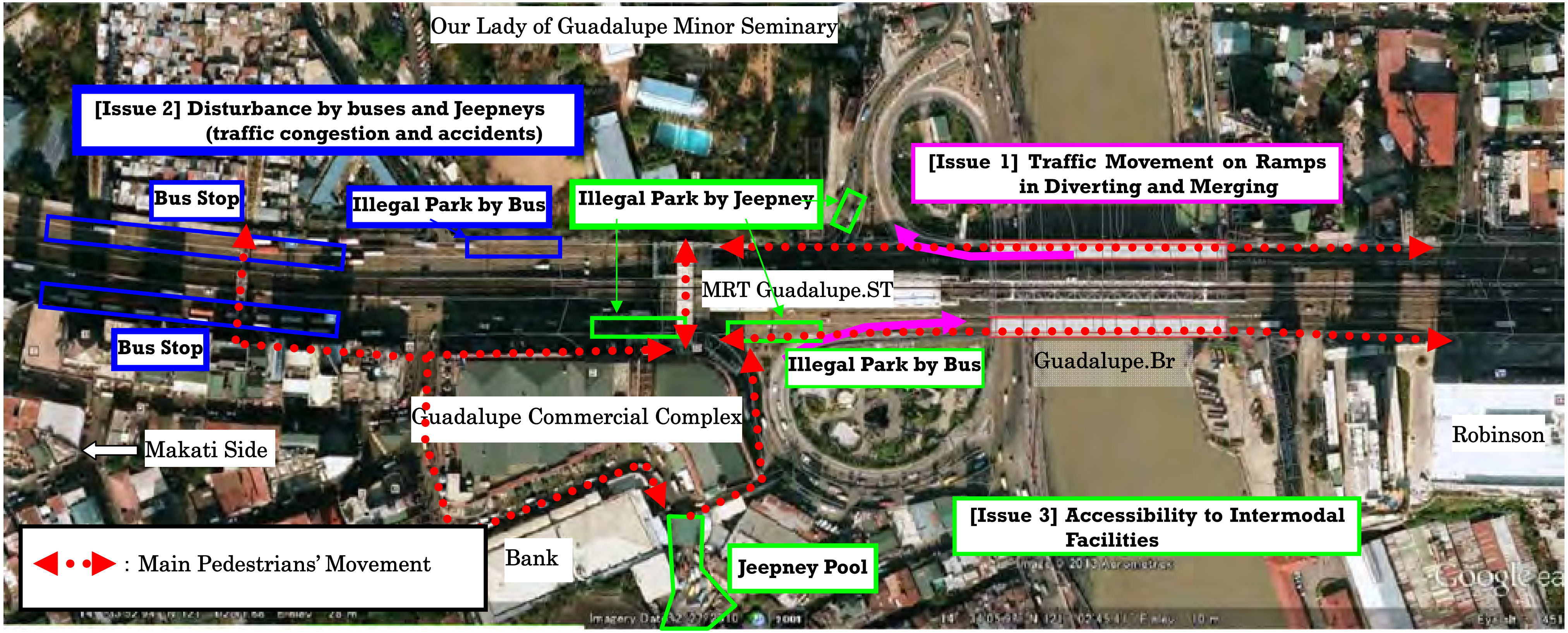
22.3 Recommendation of Improvement Project for Traffic Conditions in Traffic Intermodal Area through Guadalupe Bridge Seismic Strengthening Project

Makati side of Guadalupe Bridge is the intermodal area connecting such public transport as MRT, buses, taxies and Jeepneys, the situation of which has been giving rise to traffic confusion involving their passengers' and customers' movement using the public market located near by the area. By making the most of the opportunity of the Guadalupe Bridge seismic strengthening works, solving the traffic situation above is strongly recommended, because there is no room but the bridge section for widening and improving the area.

22.3.1 Present Issues on the Traffic Intermodal Area

The following three (3) issues on traffic conditions in the intermodal area are summarized, which is shown visually in Figure 22.3.1-1

- [**Issue 1**] Traffic movement on ramps in diverting and merging.
- **[Issue 2**] Disturbance by buses and Jeepneys, which may cause traffic congestion and accidents.
- [**Issue 3**] Accessibility to traffic intermodal facilities.



(1) Present Issues on Traffic Conditions in the Intermodal Area

Converging by Low Speed Vehicles

Disturbance by Pedestrians Crossing

Traffic Congestion near Bus Stop

Occurrence of Accidents when Traffic Congested

Buses' Illegal Parking near Station

Passengers Waiting for Jeepneys in a Queue

[**Issue 2**] Disturbance by Buses and Jeepneys (Traffic congestion and accidents)

[**Issue 3**] Accessibility to Intermodal Facilities

(2) Description of the Issues

Figure 22.3.1-1 Present Issues on Traffic Conditions in the Intermodal Area

22.3.2 Improvement Measures

(1) Improvement Level

The improvement measures for the three (3) issues above are to be expressed as improvement levels closely related to the project costs as shown in Figure 22.3.2-1.

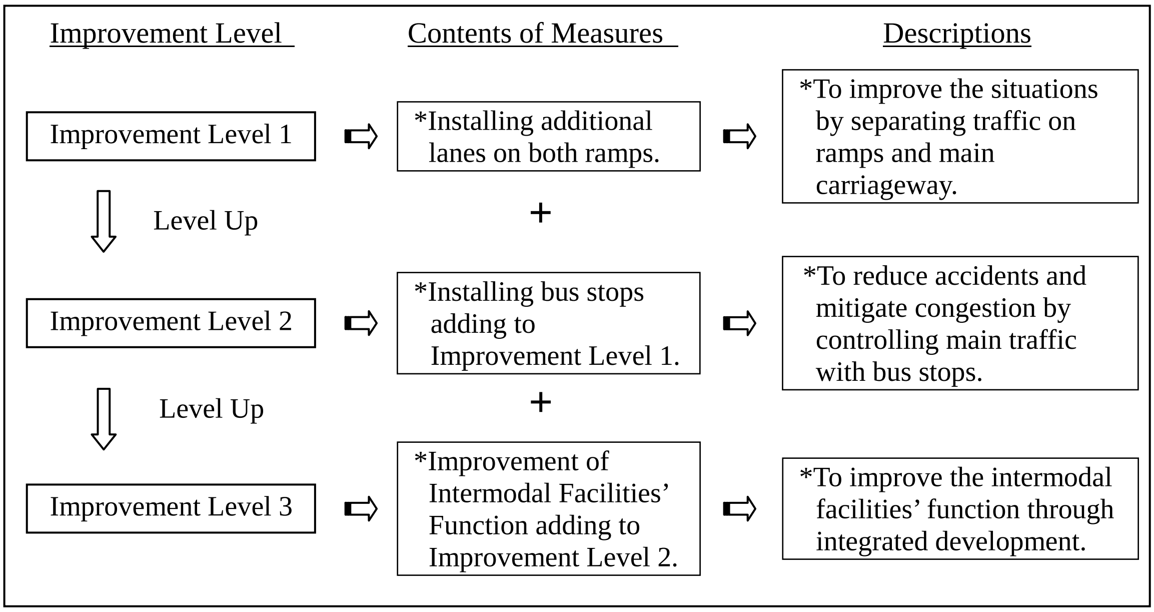


Figure 22.3.2-1 Improvement Measures

(2) Comparison on Improvement Measures

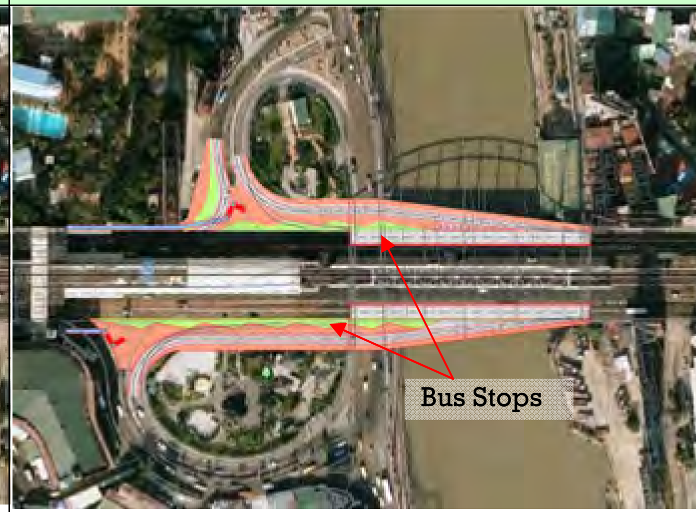
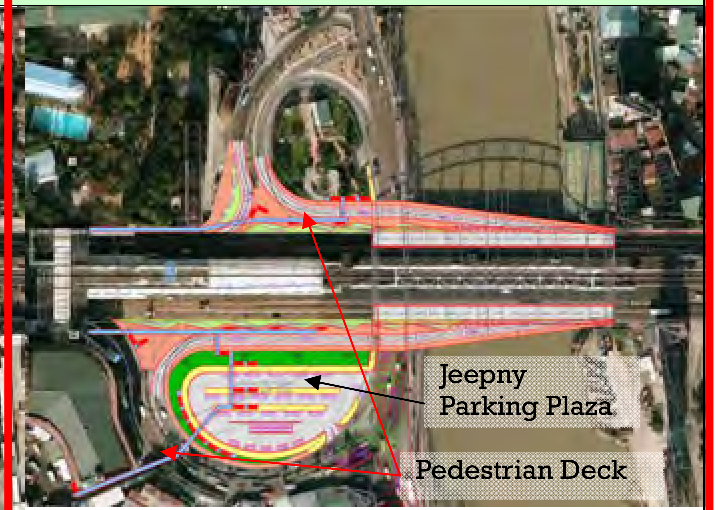
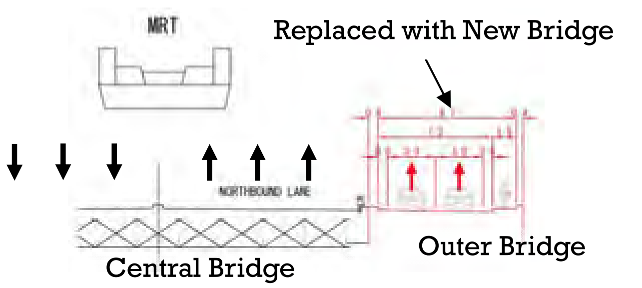
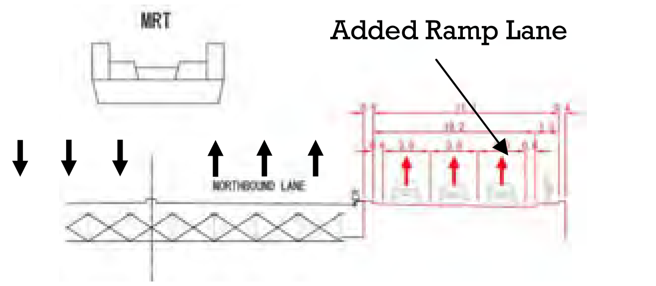
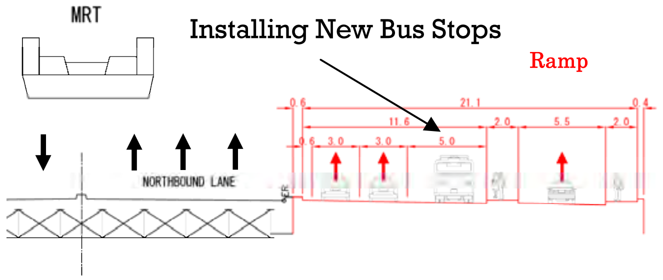
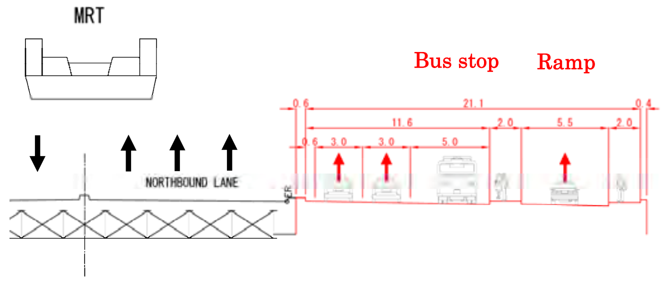
Table 22.3.2-2 shows comparison on improvement measures by improvement levels. Seismic strengthening work itself does not contribute to traffic conditions' improvement around the bridge, which is shown for reference.

From the following reasons described in Table 22.3.2-1, Improvement Level 3 is recommended not only for solving traffic confusion around the traffic intermodal facilities but also for improving environmental circumstances in the area, which leads to the mitigation measures for the climate change.

Table 22.3.2-1 Features of Improvement Levels

Improvement Level 1	Improvement Level 2	Improvement Level 3
<u>Merits</u> ▪ Merging at an appropriate speed is possible with the additional lane ▪ Waiting space for pedestrian crossing is to be secured. **<u>Demerits</u>** ▪ Traffic congestion due to buses and Jeepneys is not to be solved. ▪ Traffic safety of pedestrians is to be low because they need to cross roads.	**<u>Merits</u>** ▪ Traffic jam will be mitigated with dispersed bus stops preventing passengers converging. ▪ Accidents due to traffic jam will be reduced, resulting in reducing traffic confusion by accidents. **<u>Demerits</u>** ▪ Disturbance by Jeepneys and taxies is not to be prevented ▪ Since crossing the road for pedestrians is needed, traffic safety will not be secured.	**<u>Merits</u>** ▪ Accessibility will be improved by integrating public transport. ▪ Traffic jam will be improved by controlling illegal parking. ▪ Traffic safety of pedestrians will be improved utilizing pedestrian decks. ▪ Environmental circumstances will be drastically improved, which has a potential to become a major attraction point. **<u>Issues</u>** ▪ Securing existing park beside the bridge for Jeepney pool is needed. ▪ (Existing Jeepney pool is to be transformed to park in exchanging new Jeepney pool) ▪ Agreement between stakeholders including land owners is needed.

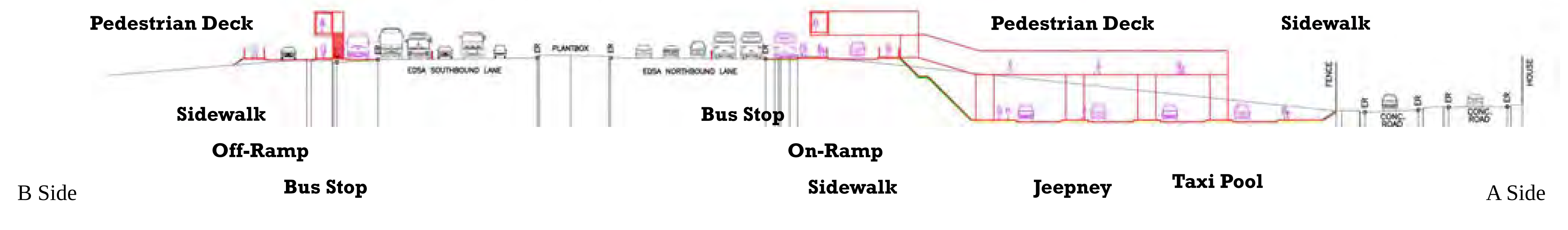
Table 22.3.2-2 Proposal for the Improvement of Traffic Situations around MRT Guadalupe Station

	Present Condition (P-0) (Bridge Seismic Strengthening Only)	**Improvement Level 1 (L-1)** (Improvement of Traffic Conditions on Ramps)	**Improvement Level 2 (L-2)** (L-1 + Providing New Bus Stops)	**Improvement Level 3 (L-3)** (L-2 + Development of Traffic Intermodal Facilities)
Plan View				
Cross Section of Bridges				
Cross Section Plan	• To maintain present configuration including the number of lanes and lane width.	• To separate main traffic and traffic on ramps, with one lane added for each side.	• To install bus stop adding to **L-1**.	• **L-2** + to develop such traffic square as pedestrian decks connecting intermodal facilities, Jeepney and taxi pools.
Mitigation of Traffic Congestion	• No improvement in terms of traffic congestion around Guadalupe Station and Bridge. 0	• To improve traffic conditions entering/going out main carriageway from/to ramps. 1	• To mitigate traffic congestion due to buses' illegal parking on main carriage way. • To intend to prevent buses' double parking. 2	• **L-2** + to intend to prevent buses, Jeepneys and taxies illegal parking. **3**
Traffic Safety	• No change in traffic safety after the bridge seismic strengthening project 0	• To reduce traffic accident potential in diverting and merging. 1	• **L-1** + to reduce traffic accident potential due to changing travel lane in loading and unloading. 2	• **L-2** + to largely improve traffic safety for pedestrians **3**
Accessibility	• No Change in accessibility after the bridge seismic strengthening project. 0	• No Change in accessibility after the project.. 0	• To improve accessibility for passengers between bus stops and MRT station. 1	• **P-2** + to largely improve accessibility for people between MRT station, public transport and commerce facilities. **3**
Additional Cost	• No increase in bridge surface area ratio after the bridge seismic strengthening project. **3**	• The number of lanes in one direction from 5 lanes to 6 lanes (area ratio increase of 1.2 times) 2	• **L-1** + bus stop area (area ratio increase of 1.7 times) 1	• **P-2** + traffic square (area ratio increase of 1.7 times) 0
Evaluation	• No change in the situations of traffic congestion, traffic safety and accessibility. 3	• To improve the extent of disturbance by traffic on ramps to main traffic 4	• **L-1** + to improve traffic congestion through preventing buses' illegal parking 6	• **L-1** + to improve traffic congestion through preventing buses, Jeepneys and taxies illegal and random parking. • To improve traffic safety and accessibility for pedestrians by developing traffic plaza including additional pedestrian decks (**Recommended**). **9**

22.3.3 Recommendations

Improvement Level 3 is recommended for solving traffic confusion and improving environmental circumstances in and around traffic intermodal area by utilizing the opportunity of seismic strengthening project. Figure 22.3.3-1 shows the recommended scheme.

(1) Plan View of Recommended Improvement Scheme



(2) Section View (A-B)

Figure 22.3.3-1 Recommended Improvement Scheme in and around Traffic Intermodal Area near Guadalupe Bridge