CHAPTER 23

FEASIBILITY STUDY OF GUADALUPE BRIDGE REHABILITATION PLAN

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FEASIBILITY STUDY OF GUADALUPE BRIDGE (BOTH SIDES) REHABILITATION PLAN

23.1 DETAILED BRIDGE SURVEY AND ASSESSMENT

This section is discussed in line with the flow chart shown in **Figure 13.1-1, Chapter 13** of this report. The survey level follows the procedure as defined in the Manual prepared by the Study Team which corresponds to "Feasibility Study Level".

23.1.1 Review of Design and Repair Works

(1) Review of Design

The references of the review of design are the construction drawings of the Guadalupe Bridge and the Bridge Retrofit Program Report of BRP both furnished by DPWH.

(a) Outline of Guadalupe Bridge (Both Sides)

Photo 23.1.3-1 Panoramic View of Guadalupe Bridge

- Structural Type : Three (3) Span PC Gerber Bridge
  Foundation Type for Pier and Abutment are PSC Piles.
- Bridge Length : 114.44m (abutment to abutment)
- Date of Construction : 1979

(b) General Notes for Widening of Guadalupe Bridge (Both Sides)

- Drawing of the Guadalupe Bridge (Both Sides) was prepared by Ministry of Public Works.
- Standard Specification for Highway Bridge, Eleventh Edition, 1973 adopted by AASHTO.
- Government Standard Specifications for Highway and Bridges, revised 1972.

Construction and Materials

- Standard Specifications for Highway Bridges, Eleventh Editions 1973, adopted by AASHTO.
- Government Standard Specifications for Highway and Bridges, revised 1972.
- The Guadalupe Bridge (Both sides) was constructed by general contractor UMALI-PAJARA Construction Company in 1979.

Dimensions

- Dimension and elevation are written in meters, weight and stresses are written in pound, feet and inch.
- October 17, 1974 Ministry of Public Highway for Widening of The Guadalupe Bridge,

Concrete

- Reinforced concrete fc' = 3000 psi (21 MPa)
- Prestressed concrete fc' = 5000 psi (35 MPa)

Reinforcing Steel

- ASTM A615 or AASHTO M31 (fy = 40,000 psi)

Structural Steel

- ASTM A36 or AASHTO M 183

Prestressing Steel

- ½" ø seven-wire high tensile strands confirming to ASTM A416 or AAHSTO M203

Foundations

- 0.35m x 0.35m (14" x 14") precast prestressed concrete piles with safe bearing capacity of 90 Metric tons each.

(2) Review of Repair Works

The actual field verification for retrofitting were cross checked with as-built drawings of retrofitting of the Guadalupe Bridge under the Bridge Retrofit Program of 1997. The retrofitting works were verified on the bridge, as follows:

- Longitudinal and transverse shear keys at coping of piers, and abutments.
- Longitudinal cable strainers at all piers to prevent the spans from falling off since the deck remain discontinuous.

- Vertical cable restrainers at abutments for uplift prevention.

Problems/Issues of Previous Repair Works

- Cracks at gerber hinge parts of exterior girders.
- Exposed rebar of mostly PC members.

(3) Historical Background

The Guadalupe Bridge (Both Sides) has no historical importance according to the National Historical Institute and interposes no objection to its rehabilitation.

23.1.2 Natural Condition Survey

(1) Topographic Survey

(a) Control Monument

Two (2) GPS Stations were established as control points for the Guadalupe Bridge as shown in **Table 23.1.2-1**.

Table 23.1.2-1 GPS Stationing and Coordinates

STATION	GPS COORDINATES		
	NORTHING	EASTING	ELEVATION
BM-1	1611106.327	504752.409	23.352
BM-2	1611481.641	504856.150	29.047

All elevations were reckoned from existing PCGS, BM and were added a constant 10.475 meter to be consistent with the previous study's vertical control system.

(b) Topographic Survey

Topographic Survey was conducted using the established control points and through the use of Calibrated Total Station Survey Instrument with Electronic Data Recorder. Two (2) GPS Stations were established and were tied to existing NAMRIA GPS Stations MMA-1 and MMA-46 located at Fort Bonifacio and Cultural Center of the Philippines to conform with PRS-92 coordinates system.

Table 23.1.2-2 shows the scope of works of topographic survey. Topographic plan is shown in **Appendix 23.1.2-1**.

Table 23.1.2-2 Scope of Works of Topographic Survey

Description	Original Scope	Actual Work
CONTROL POINT SURVEY (GPS)	1	2
PROFILE SURVEY	115 m Bridge Section + 200 m Each of Both Approach Roads (200 x 2) Total = 515 m	125.83 m Bridge Section + 217.07m + 217.10 m at each approach roads Total = 560 m
ROAD CROSS-SECTION SURVEY	Bridge Section (115m) : @ 10m Interval Approach Roads (400m) : @ 20 m Interval Width: Bridge 19m + 50m each at both sides = 119m Total = 35 Sections	Bridge Section (115m): 14 sections Approach Roads (466.31m): 21 sections Width: 119 m Total = 35 Sections
TOPOGRAPHIC SURVEY	515 m (Length) x 119 m (Width) = 61,285 sq. m	560 m (Length) x 119 m (Width) = 66,640 sq. m
RIVER CROSS-SECTION SURVEY	Edges of Bridge: 2 Upstream Side: 2 Downstream Side: 2 Total = 6 Sections	Edges of Bridge: 2 Upstream Side: 2 Downstream Side:2 Center Profile of Bridge: 1 Total = 7 Sections

(2) Geotechnical Survey

One offshore borehole was drilled at the bridge site (see **Appendix 23.1.2-2**). It was drilled down to a final depth of 30.0 meters from the top of the existing riverbed.

The granular formation extends in thickness from the surface down to the 25.0 meter depth. It is made up of poorly graded sand, silty gravelly sand and silty sand with traces of gravel. Relative density varies from loose (N=9) near the surface to medium dense and dense (N=11 to 50) down to 22.0 meters. The rest of the formation is very dense (N>50) down to 25.0 meter depth. A thin silty clay layer with fine sand is sandwiched at 12.0 to 13.0 meter depth (N=40). Underlying the predominantly granular formation is a tuffaceous sandstone formation described as very poor.

(3) Scour Survey

There was no scouring observed around substructure of the bridge.

23.1.3 Bridge Condition Survey and Identification of Damages

(1) Shape and Dimension Measurement

(a) Objective

The main purpose of this activity was to perform measurements on the main and secondary members of the bridge.

(b) Inspection Teams

Teams were formed to conduct hands-on verification of shapes and dimensions of all elements or members of the bridge.

(c) Coverage Area

The bridge was divided into three (3) general inspection areas namely: road deck level, below deck level, and substructure including bearings (see **Photo 23.1.3-2** to **23.1.3-4**).

Photo 23.1.3-2 Road Deck Level

Photo 23.1.3-3 Below Deck Level

Photo 23.1.3-4 Substructure

Verification below deck level were taken on all 8 girders on every span. The spacing of girders were measured as well as intermediate and end diaphragms. Gerber hinges were also measured.

(d) Reference Information

The Study Team was furnished with copies of as-built drawings, and retrofitting drawings of Guadalupe Bridge (Both Sides), and used them as reference in planning different activities.

(e) Equipment and Procedure

Each team was equipped with safety gear (hard hats, safety belts, safety shoes, and goggles), measurement tools (steel tape and caliper) for verification, hammer, steel brush, digital still camera, forms and pencils for documentation (see **Photo 23.1.3-5**).

Photo 23.1.3-5 Truck Mounted Scaffoldings

In verification of measurements on road deck level and sidewalk, dimensions were made easily using tape measure.

Verification of measurements below deck level required the use of suspended foot bridge or Gondola (see **Photo 21.1.3-6**). This system was used on high superstructure locations.

Photo 23.1.3-6 Gondola Platform

Each activity and inspected damages were supported with photos and dimensions were recorded. Results of special tests were also supplemented with photos.

(f) Miscellaneous Structures

Miscellaneous structure, including non-structural elements, were noted and photographs of these were taken. These includes 20 - ϕ 100mm PVC Telecommunication Pipe attached on the downstream side of exterior girder of upstream bridge. (See **Photo 21.1.3-7**).

Photo 23.1.3-7 Utilities

(g) Results

Table 23.1.3-1 lists the drawings that summarize the data presented in the verification forms. Dimensions that were shown in the drawings were utilized in structural modeling and analysis.

Table 23.1.3-1 List of Drawings

Sheet No.	Title	Appendix
1	General Plan and Elevation	Appendix 23.1.3-1 (1/3)
2	Girder Section and Gerber Hinge Details, Girder Elevation and Framing Plan	Appendix 23.1.3- 1 (2/3)
3	Details of Abutment A1 & A2, Details of Pier 1 & 2.	Appendix 23.1.3-1 (3/3)

(2) Close-up Visual Inspection

(a) Objective

The close up visual inspection's purpose is to determine the damages on the bridge and to be able to make detailed documentation including digital still photos.

A few available drawings were taken and used in planning for this activity.

(b) Inspection Team

The inspection teams were tasked to conduct close-up visual inspection of damages on the bridge.

(c) Coverage Area

The inspection covered the entire superstructure, road/deck, below deck level and substructure including bearings.

(d) Reference Information

The Study Team was furnished with copies of revised design drawings of the bridge and used them as reference in various activities and analysis of the structure.

(e) Equipment and Procedure

This activity was conducted simultaneously with the verification of shapes and dimensions.

(f) Criteria for Damage Rating

The criteria used for damage rating follows the criteria set forth **Section 6.4** of **Chapter 6.**

(g) Results

- Large cracks were observed mostly on exterior girder. The suspended portion of the girder at gerber hinge connection exhibited deflection.
- The major damages of the bridge are shown in **Figure 23.1.3-1.** The damage rating of main members based on Close-up Visual Inspection is shown on **Table 23.1.3-2**.
- The damage sheets were completely documented in **Appendix 23.1.3-2 (1/12** to **12/12**).

Girder 1, Gerber Hinge Damage – Crack Width 0.5mm – 1mm

Photo 23.1.3 Sidewalk Fascia Damage – Exposed Rebar

Girder 1, Gerber Hinge Damage – Crack Width 0.5mm – 1mm

Girder 8, Gerber Hinge Damage – Crack Width 2.0mm

Girder 8, Gerber Hinge Damage – Crack Width 3.0mm

Girder 5, Gerber Hinge Damage – Crack Width 2.0mm – 5.0m

Figure 23.1.3-1 Close-Up Visual Inspection of Major Damages in the Guadalupe Bridge

Table 23.1.3-2 Damage Rating of Members by Close-up Visual Inspection

Component	Item	Member / Location	Damage Rating	Damage Description
SUPERSTRUCTURE	Material Damage	Span 1 : Deck Slab, Sidewalk	II	Corrosion of rebar
	Material Damage	Span 2 : Gerber Hinge 1, L, of Girder 5	II	Corrosion of rebar
	Material Damage	Span 2 : Gerber Hinge 1, R, of Girder 8	II	Corrosion of rebar
	Material Damage	Span 2 : Middle of Span of Girder 5	II	Corrosion of rebar
	Material Damage	Span 2 : Gerber Hinge 2, L, of Girder 1	II	Corrosion of rebar
	Material Damage	Span 1 : Bottom of Center Span of Girder 1,4 ,5 & 8	III	Cracks thickness of 0.10mm, Severe corrosion of rebar
	Material Damage	Span 2 : Gerber Hinge 1, L, of Girder 1	I	Wide cracks at gerber hinge with crack widths of 0.5mm to 2mm with some spalling and exposed rebars.
	Material Damage	Span 2 : Gerber Hinge 1, R, of Girder 1	I	2mm wide, cracks. Condition of corrosion of rebars is severe.
	Material Damage	Span 2 : Gerber Hinge 1, L, of Girder 5	I	3 locations of wide cracks from 1.5mm to 5mm. Condition of corrosion of rebars is severe.
	Material Damage	Span 2 : Gerber Hinge 1, R, of Girder 5	I	0.50mm wide cracks condition of corrosion of rebars is severe.
	Material Damage	Span 2 : Gerber Hinge 1, R of Girder 8	I	0.10mm cracks at center of span 2. Condition of corrosion of rebars is severe.
	Material Damage	Span 2 : Middle of Span of Girder 8	III	Many cracks with thickness ranging from 0.10mm to 0.20mm. Condition of corrosion of rebars is severe
	Material Damage	Span 2 : Gerber Hinge 2, R, of Girder 1	I	Cracks width varies from 1.0mm to 5.0mm. Condition of corrosion of rebars is severe.
	Material Damage	Span 2 : Gerber Hinge 2, R, of Girder 5	I	2mm to 5mm wide cracks. Condition of corrosion of rebars is severe.
	Material Damage	Span 2 : L, Girder 5	I	Crack width of 1.0mm spacing less than 50 cm. Condition of corrosion of rebars is severe.
	Material Damage	Span 2 : Gerber Hinge 2, L, of Girder 8	I	3 Crack locations with crack widths of 1.0mm to 3mm.Condition of corrosion of rebars is severe
	Material Damage	Span 2 : Gerber Hinge 2, R, of Girder 8	I	2mm to 3mm wide cracks. Condition of corrosion of rebars is severe.
	Material Damage	Span 2 : Gerber Hinge 2, L-R, Girder 6	II	Corrosion of rebar
	Material Damage	Span 2 : Gerber Hinge 1, L-R, Girder 7	II	Corrosion of rebar
	Material Damage	Span 1 : Bottom of Center Span of Girder 3,4,6 & 7	III	Alligator cracks having thickness of 0.1mm. Condition of corrosion of rebars is severe.
	Material Damage	Span 2 : Gerber Hinge 1, R, Girder 6,	I	0.20mm thick cracks. Condition of corrosion of rebars is severe.
	Material Damage	Span 2 : Gerber Hinge 1, L, Girder 7	I	One crack location with thickness of 0.20mm. Condition of corrosion of rebars is severe.
	Material Damage	Span 2 : Gerber Hinge 2, L, Girder 6	I	Crack thickness of 2 mm with spacing of 50 cm. Condition of corrosion of rebars is severe.
SUBSTRUCTURE	Material Damage	Bearing of G8, Pier 1	III	Rust scattered and generated extensively is observed, section loss is small, less than 10%.
	Material Damage	Bearing of G5, Pier 2	III	Rust scattered and generated extensively is observed, section loss is small, less than 10%.
	Material Damage	Pier 2 Wall D/S	III	Corrosion of rebar
	Material Damage	Pier 2 Wall U/S	III	Corrosion of rebar
	Material Damage	Abutment A2	II	Alligator cracks with thickness of 0.3mm located on the downstream face of abutment.

Note: 1 - Damage Rating Level is based on the X,Y,Z Damage Rating Method
2 - Rating I – Determined through engineering judgment by Team Leader through consultation with governing organization.

(3) Non-Destructive Test of Superstructure

(a) Objective

In conducting this activity, results of close-up visual inspection and importance of the member/joint were considered in deciding the location of the non-destructive test.

(b) Results

Table 23.1.3-3 shows the results of non-destructive tests conducted in the Guadalupe Bridge.

Table 23.1.3-3 Results of Non-Destructive Test

Test	Description & Results	Reference Appendices
Ultrasonic Pulse Velocity Test (To determine depth of cracks on concrete members)	Results of crack depths vary from no evident cracks to full depth cracks.	Appendix 23.1.3-3 (1/3 to 3/3)
Schmidt Rebound Hammer Test (To determine the in-situ uniformity, surface hardness, and approximate compressive strength of concrete)	Measured compressive strength ranged from 38 Mpa to 49 Mpa.	Appendix 23.1.3-4
Phenolphthalein Test (To determine the depth of carbonation)	The depth of carbonization ranges from 5mm to 12mm.	Appendix 23.1.3-4
Chloride Test (To assess the distribution of chlorides)	Test revealed chloride levels were not detected from the two (2) core samples.	Appendix 23.1.3-4
Compression Test (To obtain the compressive strength of concrete)	Results of the compression test ranged from 21.1MPa to 30.7MPa.	Appendix 23.1.3-4

(4) Special Test

No special test was performed for this bridge.

(5) Assessment of Critical Damages

(a) Evaluation Criteria

Damages of bridge members were inspected under the close-up visual inspection and non-destructive test of material were identified and evaluated in compliance with the procedure set forth in Section 4.10 of the Manual.

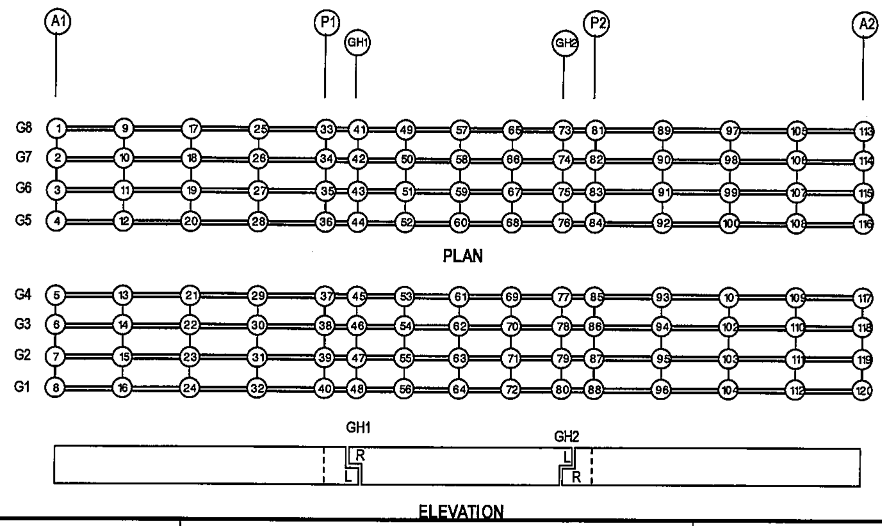
(b) Damage Rating with X, Y, Z Method

This section follows the procedures and criteria set forth in **Section 6.4, Chapter 6.**

Evaluation of Damages

Evaluation results on damages of main members were summarized in **Table 23.1.3-4.**

Table 23.1.3-4 Evaluation of Major Damages on Guadalupe Bridge



	Location		Evaluation Based on Field Survey				Evaluation Based on Non-Destructive Test	Evaluation based on Special Test
	Member	Node	Damage Type	Damage Rating	Diagnostic	Estimated Section Loss		
SUPERSTRUCTURE	Sidewalk Slab	33	SER	II	B	5%	•Ultrasonic Pulse Velocity Test – **Varies from no evidence to full depth cracks OK**	-
	Gerber GH1, L	44	SER	II	B	3%		
	Gerber GH1, L-R	43	SER	II	B	3%		
	Gerber GH 1, L-R	42	SER	II	B	3%		
	Gerber GH 1, R	41	SER	II	B	3%		
	Girder	60	SER	II	B	3%		
	Gerber GH2, L	80	SER	II	B	3%		
	Girder	17-21, 23, 24	CR	III	B	-		
	Gerber GH1, L	48	CR	I	A	10%		
	Gerber GH1, R	48	CR	I	A	10%		
	Gerber GH1, L	44	CR	I	A	10%		
	Gerber GH1, R	44	CR	I	A	10%		
	Gerber GH1, R	43	CR	I	A	10%		
	Gerber GH1, L	42	CR	I	A	10%		
	Gerber GH1, R	41	CR	I	A	10%		
	Girder	57	CR	III	B	-		
	Gerber GH2, R	80	CR	I	A	10%		
	Gerber GH2, L	76	CR	I	A	10%		
	Gerber GH2, R	76	CR	I	A	10%		
	Gerber GH2, L	75	CR	I	A	10%		
	Gerber GH2, L	73	CR	I	A	10%		
	Gerber GH2, R	73	CR	I	A	10%		
SUB-STRUCTURE	Bearing	33	CO	III	B	4%	•Schmidt Rebound Hammer Test – **OK** •Compression Test – **OK, fc' Range 21.1 – 30.7 MPa**	-
	Bearing	84	CO	III	B	4%		
	Pier 2 Wall, D/S	81-84	SER	III	B	3%		
	Pier 2 Wall, U/S	81-84	SER	III	B	3%		
	Abutment A2	113-116	CR	II	B	3%		

23.1.4 Presumption of Original Design

(1) Objective

The purpose of the presumption of original design is to prepare the structural shapes, dimensions and properties for the analysis model of the Load Rating.

(2) Structural Shapes and Dimensions

Superstructure

Most structural data of the superstructure does not need to be assumed because all the dimensions and details were measured. **Appendix 23.1.4-1 (1/10** to **10/10)** shows the shapes, dimensions, and details of the superstructure for the structural analysis.

Substructure

The dimension of exposed portion of the substructure were all measured. It is necessary to calculate the dead load and live load for the estimation of the scale of foundation.

The type of foundation were determined from the as-built drawings. The availability of materials and construction were known.

Appendix 23.1.4-2 shows the shapes and dimensions of substructures.

(3) Structural Soundness (LOAD RATING)

The bridge superstructure was modeled with 3D beam elements using a commercial FEM software. An analysis for Load Effects was performed on the model under different loading conditions. The AASHTO MS-18 live load was used in the analysis.

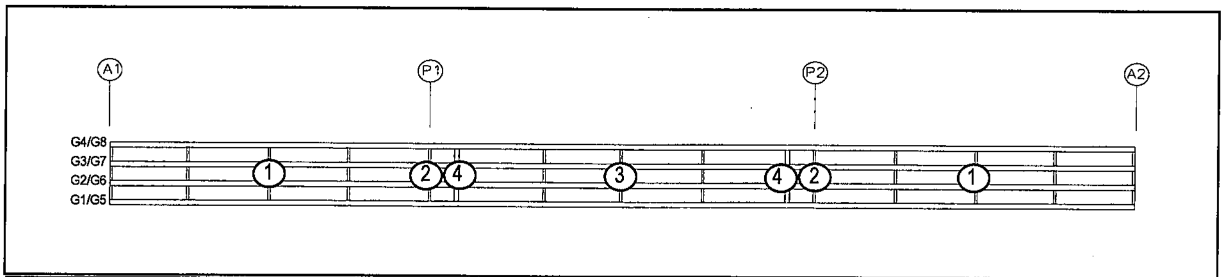
Live load rating factors were calculated at two levels: inventory and operating levels. The allowable stress that was adopted for inventory level evaluation was 21 MPa in compression and 2.96 MPa in tension for prestressed girders.

For the formula in calculating the Rating Factor, see **Section 7.4**, of **Chapter 7.**

Analysis Result

The calculated values of Rating Factor for each main member is presented in **Appendix 23.1.4-3**. The results on minimum RF by the location were shown in **Table 23.1.4-1.**

Table 23.1.4-1 Minimum RF by the Location



LOCATION		SERVICEABILITY LIMIT STATE		STRENGTH LIMIT STATE			
MEMBER	NODE	ALLOWABLE STRESS		INVENTORY LEVEL		OPERATING LEVEL	
		RF	EQ. TRUCK (tons)	RF	EQ. TRUCK (tons)	RF	EQ. TRUCK (tons)
Girder	1	2.03	65	1.78	57	2.97	95
Girder	2	4.92	157	3.61	116	6.02	193
Girder	3	1.64	52	2.55	82	4.25	136
Gerber	4	0	0	**0.44**	14	**0.74**	24

(4) Vulnerability to Disaster

(a) Earthquake

The earthquake vulnerability of a bridge can be assessed by considering the following factors discussed in details in **Chapter 10.**

Bridge Site

Guadalupe Bridge (Both Sides) is located 1.5km from the Marikina Fault Valley System (MVFS). As rule, bridge structures less than 5km distance are considered highly vulnerable. The 1.5km distance of Guadalupe Bridge (Both Sides) make it highly vulnerable to earthquake. However, the type of soil and its response characteristics will have to be properly evaluated and considered in the design of strengthening.

Construction Details

The existing superstructure is supported by wall-type pier. The piers were extended to accommodate the new Prestressed Girder Superstructure. Shear keys on piers and longitudinal restraining cables at gerber hinge were installed during retrofitting works in 1997.

Structural Configuration

The regular configuration of Guadalupe Bridge (Both Sides) is structurally favorable.

Date of Construction

Guadalupe Bridge (Both Sides) was constructed in 1979. Before and during those times AASHTO have minimal recommendations with regards to seismic designs. But, the existence of Caltrans 1973 provisions may have been covered in the detailed design, wherein seismic force is dependent on distance to fault, soil at site and dynamics of bridge. However, Guadalupe Bridge (Both Sides) could still be prone to present seismic forces.

Analysis Results

The safety of the bridge under earthquake was determined from the capacity and stability of the substructure. Vulnerability of the bridge to earthquake is presented by capacity-demand ratio under earthquake.

The existing dimensions and structural data of the substructure were determined from actual field surveys, as-built plans and "Presumption of Original Design" using the original code.

Table 23.1.4-2 shows the C/D ratio of substructures under the original code and the latest code.

Table 23.1.4-2 Summary of Capacity/Demand Ratio of Guadalupe Bridge (Both Sides)

Substructure	Original Code		Latest Code	
	Pier Wall	Foundation	Pier Wall	Foundation
Pier 1	1.24	1.19	**0.85**	**0.61**
Pier 2	1.24	1.19	**0.85**	**0.61**

The calculations of the above assessment is presented in **Appendix 23.1.4-4**.

(b) Wind

The National Structural Code of the Philippines (NSCP 2001) recommends a design basic wind speed of 200 kph but AASHTO recommends only 160 kph. The maximum cyclone center wind velocity of 225 kph passing Metro Manila where Guadalupe Bridge (Both Sides) is located was recorded in 1995 with a gust velocity reaching to 255 kph. This indicates that Guadalupe Bridge (Both Sides) has been exposed to more than 200 kph basic design wind speed specified in the Philippine Code. Therefore, Guadalupe Bridge (Both Sides) is not vulnerable to wind action.

(c) Flood

The pressure from flood water flow is usually quite small comparing to the lateral design force adopted under the earthquake in the Philippines. This means that the earthquake forces dictate the scale and the safety of the bridge. Moreover, the present profile of Guadalupe Bridge deck is more than sufficient to clear the maximum flood level.

(d) Special Issues

Vessel Collision

Pasig River is a major river for water navigation, and the vessels navigating the river are the motorized tug boats, barges motor tankers, bankers and fishing boats.

- Vessel Collision with Girder

  The regulated vertical clearance by PCG is 3.75m, the actual vertical clearance of Guadalupe Bridge is sufficient at 8.3m.

- Vessel Collision with Pier

  The ideal navigation span of piers should be more than the maximum vessel length of 60m. However, the preferable span of bridge piers for one vessel at Pasig River is 43 meters to allow passage at the deflection angle of 45 degrees. The navigational clearance between piers of Guadalupe Bridge is 34.2m which is less than the preferable space of 43m.

Utilities

The existing utility lines attached to the Bridge are listed below:

a) 20-100 mmø PVC Telecommunication lines

Informal Settlers

There are two (2) families living under the Bridge.

23.1.5 FEM Analysis of Gerber Hinge Support

The Finite Element Method (FEM) analysis was employed to estimate mathematically the causes of cracks on gerber hinge support parts and to verify the effectiveness of the rehabilitation measure which was recommended in **Section 23.2.4.**

(1) The approach for estimation of crack causes and verification of rehabilitation measure is as follows:

- To set-up nonlinear finite element models of a typcial gerber hinge supporting the suspended post-tensioned girder spans (36.70m) of the Guadalupe Bridge,
- To simulate the existing condition of the gerber hinge part, specifically the occurrence and pattern of cracks under servie load, and

- To estimate the live load capacity of the gerber hinge support with the proposed rehabilitation memasure.

(2) Methodology

(a) Nonlinear Finite Element Modeling of Typical Gerber Hinge part

Four (4) finite element models of a typical Gerber Hinge supporting the center span post-tensioned girder (36.70m) was set-up using the software Ucwin/WCOMD, which had been developed by Prof. Maekawa and his colleagues at the University of Tokyo. Descriptions of the models are listed in **Table 23.1.5-1**.

Two types of 8-noded quadrilateral plane stress (2D) elements were used in modeling: namely, RC Plates for zones of concrete in which reinforcements are embedded and Plain Concrete Plates for areas in which no reinforcement is embedded. Both element types take into account the highly nonlinear behavior of reinforced concrete.

Post-tensioning was applied as an initial force in the analysis.

The Model Failure Criteria used in the software were the following:

- Maximum tensile strain normal cracks (Et): 3.00%
- Maximum compressive strain parallel to cracks (Ec): -1.00%
- Maximum shear strain parallel to cracks (Esh): 2.00%

Outline of the crack model used in the software is illustrated in the **Figure 22.1.5-1, Item 22.1-5, Chapter 22.**

Table 23.1.5-1 Description of Finite Element Models

Model		Description	Remarks
1	E	• To simulate existing condition : Effective post-tensioning forces are 0.50fsu for all tendons specified in the design code. : The amount of reinforcements shown in the construction drawings is used (See **Figure 23.1.5-1**).	• For checking the design, fpu = 1860 Mpa
2	Et	• To simulate existing condition : Effective post-tensioning forces are 0.35fsu for Tendon 3, 0.20fpu for Tendon 2 and 0.50fsu for Tendons 1 and 4. : The amount of reinforcements shown in the construction drawings is used (See **Figure 23.1.5-1**).	• For checking the construction quality • Reduced values of effective post-tensioning forces are the results which would explain the occurrence of cracks observed.
3	Es	• To simulate existing condition : Effective post-tensioning forces are the same as Model E : The amount of vertical reinforcements is reduced from 75mm spacing to 150mm.	• For checking the difference in the scale of influence on the crack ocurrence between Models Et and Es
4	Rt	• To simulate rehabilitated condition : Assumptions of existing post-tensioning forces and/or the amount of reinforcements are the same as Model Et.	• For checking the effectiveness of rehabilitation measure recommended in **Section 23.2.4.**

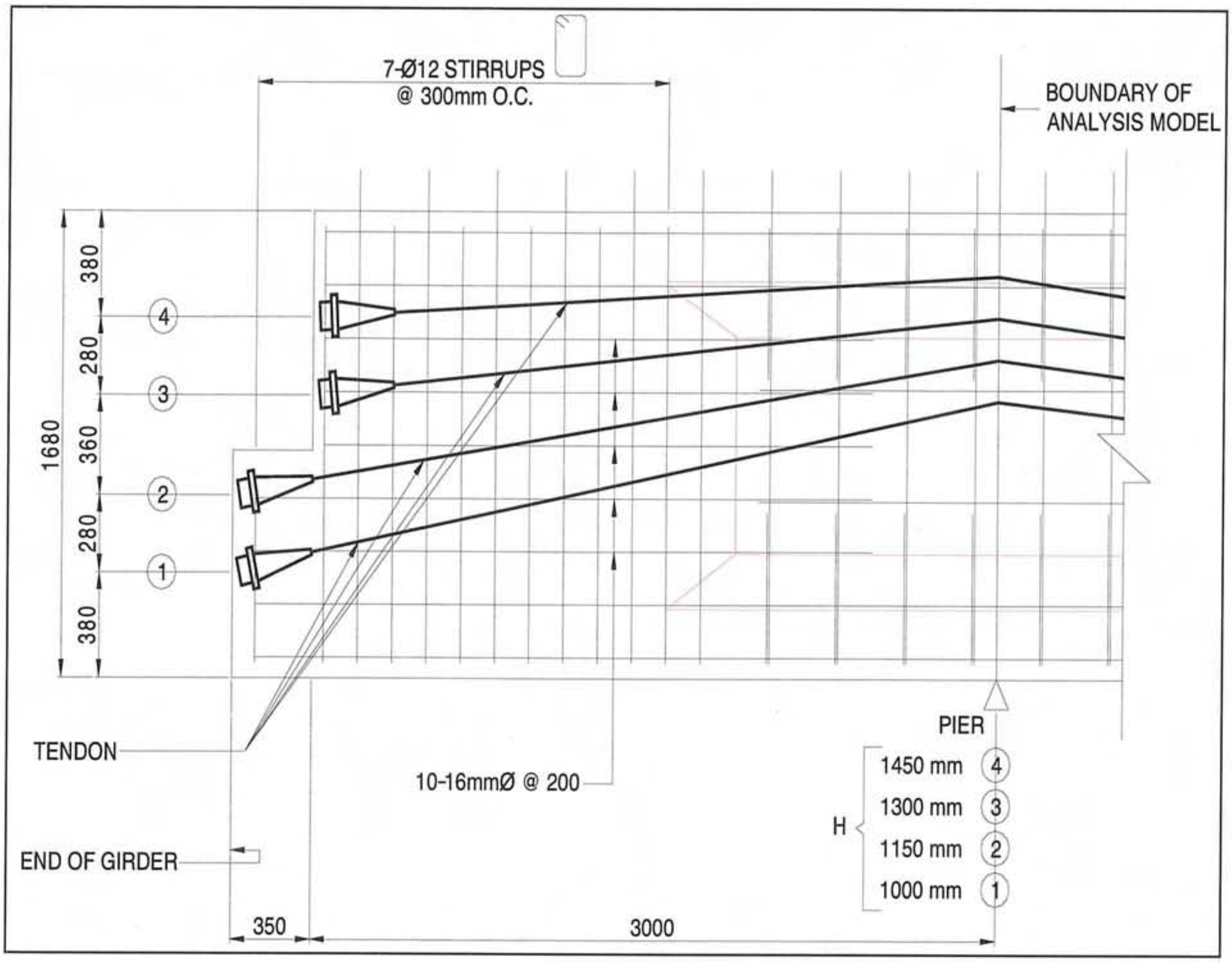


Figure 23.1.5-1 FEM Model and Construction Drawing of Gerber Hinge Part

(b) Simulation of Existing Condition and Ocurrence of Cracks

Models E, Et and Es were used to simulate the existing condition of the gerber hinge, and the ocurrence and pattern of cracks. Loads were applied monotonically in stages. **Table 23.1.5-2** shows the loading stages for this simulation.

Table 23.1.5-2 Loading Stages for Simulation of Existing Condition

Action	No. of Loading Stages
Self weight	1
Effective Post-tensioning Force (0.50fsu for Model E and Model Es)	3
Dead Load	5
Live Load	To be loaded until failure at 25kN loading increment

(c) Estimation of Live Load Capacity with the Rehabilitation Measure

Models R, Rt, or Rs were used to model the rehabilitated condition of the gerber hinge, considering the most probable mechanism of cracking as would be evident from the results of the simultations using the Models E, Et and Es.

Similar to (b) of this section of the proposed methodology, loading was applied monotonically in stages, with live load last to be applied. **Table 23.1.5-3** shows the loading stages for this analysis.

Table 23.1.5-3 Loading Stages for Simulation of Rehabilitated Condition

Action	No. of Loading Stages
Self weight	1
Effective Post-tensioning Force (0.50fsu for Model E and Model Es)	3
Dead Load	5
Post-tensioning Force of Slanted Cables	10
Live Load	To be loaded until failure at 25kN loading increment

(3) Analysis Results

Figure 23.1.5-2 show the patterns and locations of cracks for each Model; those observed cracks are shown in **Figure 23.1.5-4** for comparison. From these figures, the following were observed :

- The most probable model to explain the observed crack pattern is Model Et.
- This suggests that effective post-tensioning under construction were smaller than that specified in the design.
- The effective post-tensioned forces affect the occurrence of cracks more than the amount of reinforcement as compared to the results of Model Et and Es.
- The rehabilitation measure recommended in **Section 23.2.4** is revealed to be effective.

Figure 23.1.5-3 indicates the crack pattern prior to failure of Model Et and load-deflection relationship at the loaded point.

From the results, the following are observed:

- In case of Model Et which simulates well the existing crack condition among analysis models, integrated behavior is under the situation of yield zone or almost failure zone when the service load (design live load) is loaded.
- Model Rt presenting rehabilitation measure is very effective to increase the capacity of gerber hinge part.
- The integrated behavior of rehabilitated gerber hinge part is still within the elastic zone.
- Even at load level prior to failure, any crack occurrences at girder on top of the pier did not appear.

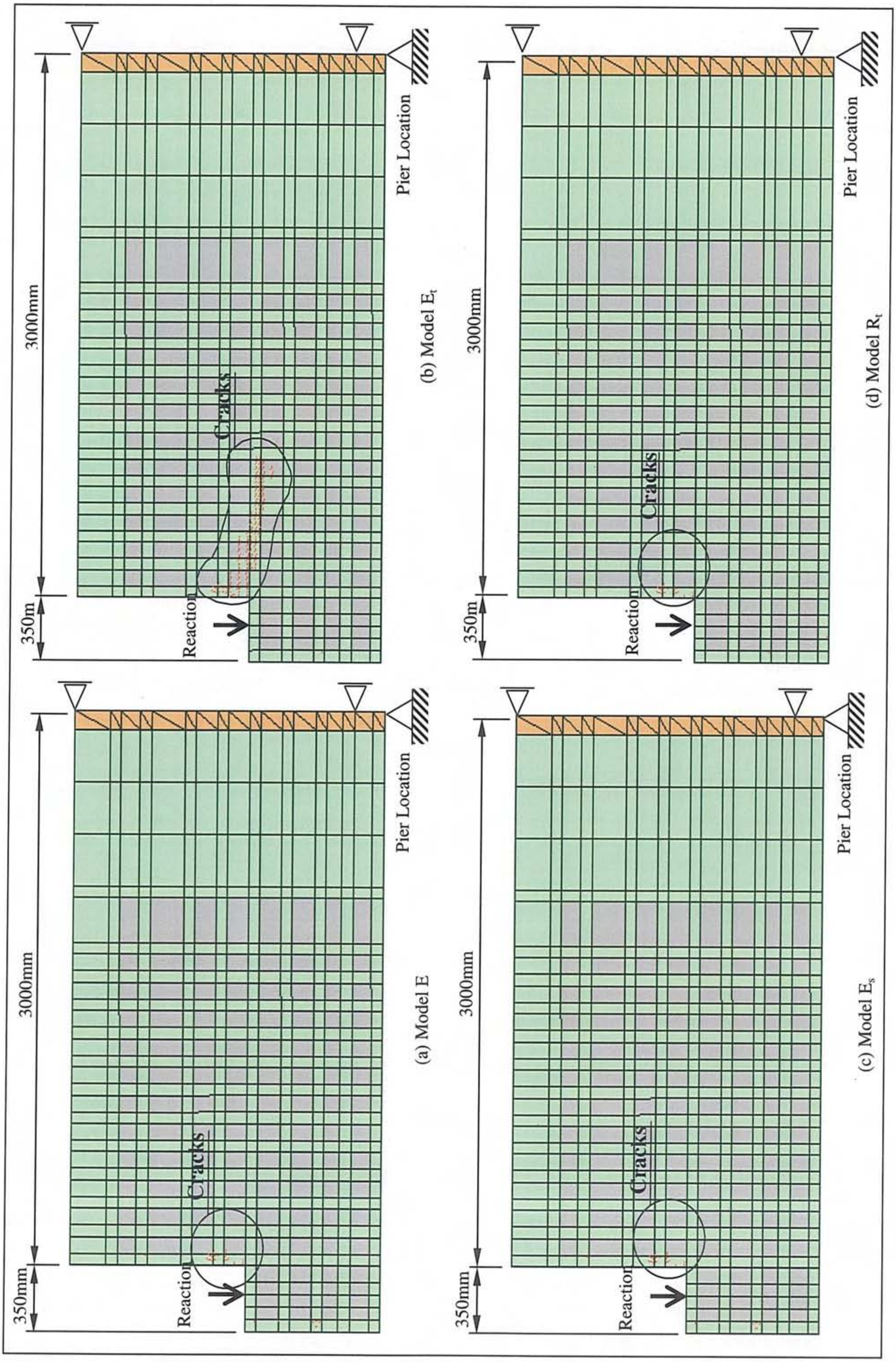
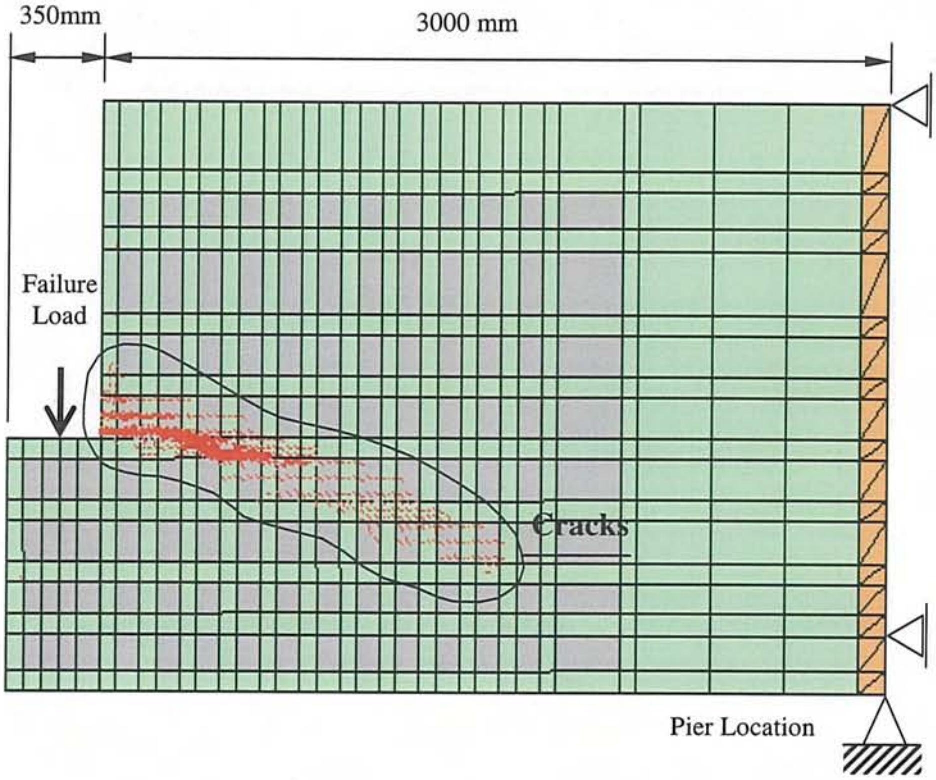
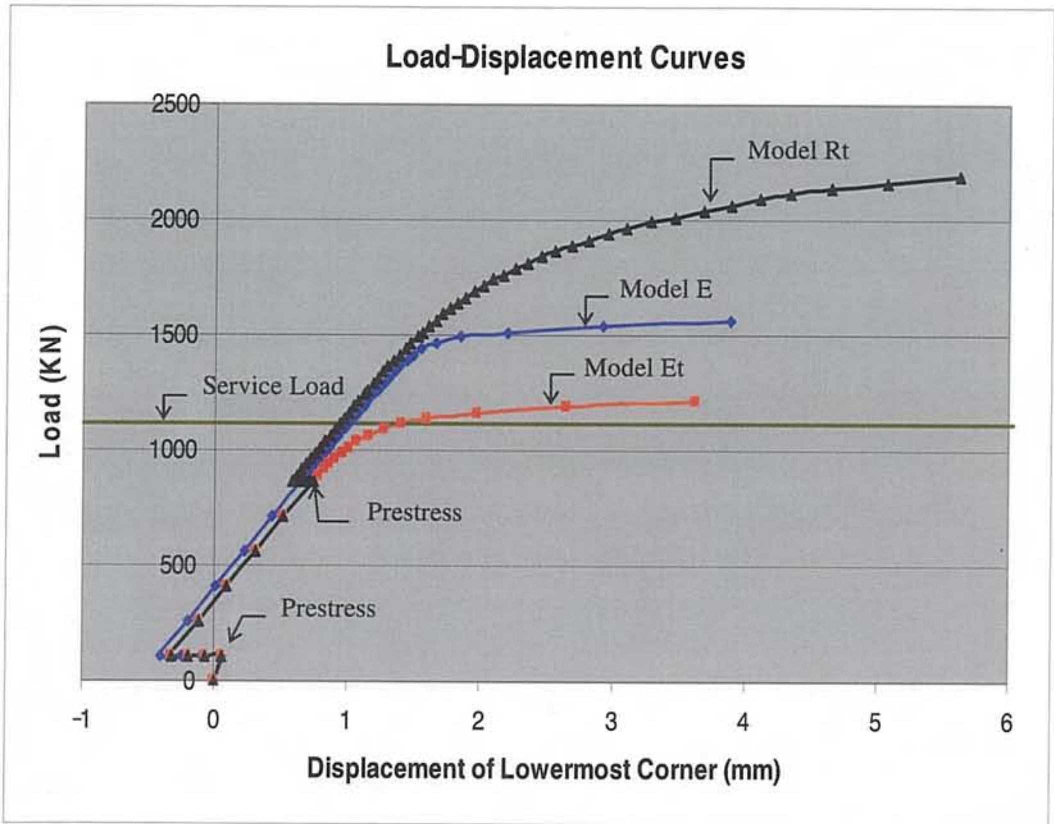


Figure 23.1.5-2 Crack Patterns and Locations at Service Load in Each Model



(a) Crack Pattern Prior to Failure (Model E_t)



(b) Load – Deflection Relationship

Figure 23.1.5-3 Crack Pattern Prior to Failure and Load-Deflection Relationship at Reaction Force Loaded Point.

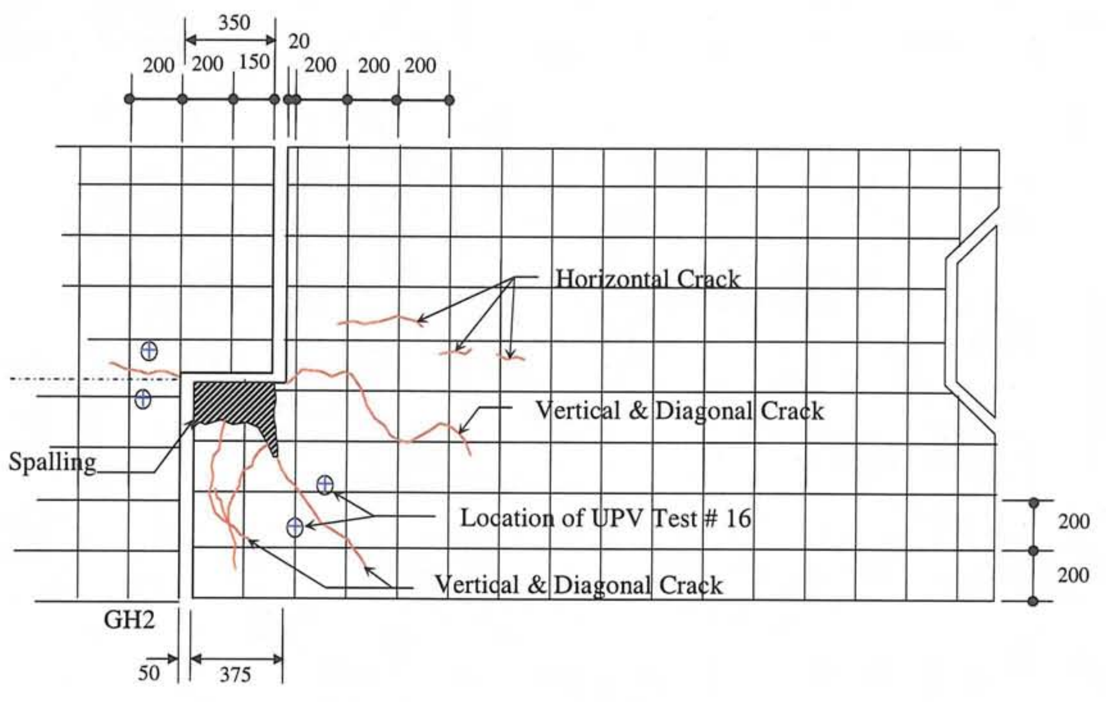


Figure 23.1.5-4 Observed Crack Pattern in Gerber Hinge Part of Guadalupe Bridge

23.1.6 Overall Assessment of Bridge Condition

The present state of the bridge was assessed based on the following informations:

(1) Superstructure

Major Damage Description and Causes

- The outer faces of exterior girders at gerber hinge parts have many cracks. Most of the cracks were full depth as tested non-destructively.These cracks are due to insufficient flexural and shear reinforcement and unforeseen losses of prestressing forces.

(2) Substructure

The following results of analysis conducted on substructure were:

- Existing Pier walls and foundations are sound to carry the original design load but not enough to carry the loads required under the latest code as shown in **Table 23.1.4-2**.
- The instability of pier walls and foundations are due to the change of design code especially the design requirement.

(3) Social Environment

- Dislocation of people living under the bridge is compulsory upon project implementation.

(4) Conclusion

- Improvement work in the gerber hinge part is necessary to improve the durability of the existing superstructure.
- As for the substructure, a retrofit measure is recommended to conform with the requirement of the latest code.

Table 23.1.6-1 summarizes the overall assessment of existing bridge.

Table 23.1.6-1 Overall Assessment for Guadalupe Bridge (Both Sides) Condition

Items			Member/Location		Damage Rating	Damage Condition	Diagnosis (IW or FI)
Structural Soundness	Superstructure	Shape/Dimension	Span 1	Deck Slab, Sidewalk, Span 1	II	Not so large section loss due to corrosion.	FI
			Span 2	GH1, L, Ext. Girder 5, Span 2	II	Not so large section loss due to corrosion.	FI
				GH 1, R, Ext. Girder 8, Span 2	II	Not so large section loss due to corrosion.	FI
				Ext. Girder 5, Middle of Span 2	II	Not so large section loss due to corrosion.	FI
				GH2, L, Ext. Girder 1, Span 2	II	Not so large section loss due to corrosion.	FI
				GH1, L-R, Int. Girder 6, Span 2	II	Not so large section loss due to corrosion.	FI
				GH 1, L-R, Int. Girder 7, Span 2	II	Not so large section loss due to corrosion.	FI
		Material/Damage	Span 1	Bottom of Center Span of Main Girder, G1,G4 , G5, and G8 of Span 1	III	Alligator cracks was observed at bottom of girders having crack thickness of 0.10mm, Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	FI
				Bottom of Center Span of Main Girder, G3,G4,G6 and G7 of Span 1	III	Alligator cracks having thickness of 0.1mm. Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	FI
			Span 2	GH1, L, Ext. Girder 1, Span 2	II	Wide cracks at gerber hinge with crack widths of 0.5mm to 2mm with some spalling and exposed rebars. Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	IW
				GH1, R, Ext. Girder 1, Span 2	II	2mm wide, cracks. Condition of corrosion of rebars is severe. Concrete coverage 40 – 70mm.	IW
				GH1, L, Ext. Girder 5, Span 2	II	3 locations of wide cracks from 1.5mm to 5mm. Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	IW
				GH1, R, Ext. Girder 5, Span 2	II	0.50mm wide cracks condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	IW
				GH1, R, Ext. Girder 8, Span 2	II	0.10mm cracks at center of span 2. Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	FI
				Ext. Girder 8, Middle of Span 2	III	Many cracks with thickness ranging from 0.10mm to 0.20mm. Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	FI
				GH2, R, Ext. Girder 1, Span 2	II	Cracks width varies from 1.0mm to 5.0mm. Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	IW
				GH2, R, Ext. Girder 5, Span 2	II	2mm to 5mm wide cracks. Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	IW
				GH2, L, Ext. Girder 5, Span 2	II	Crack width of 1.0mm spacing less than 50 cm. Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	IW
				GH2, L, Ext. Girder 8, Span 2	II	3 Crack locations with crack widths of 1.0mm to 3mm. Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	IW
				GH2, R, Ext. Girder 8, Span 2	II	2mm to 3mm wide cracks. Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	IW
				GH1, R, Int. Girder 6, Span 2	II	0.20mm thick cracks. Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm. with spacing less than 50cm.	FI
				GH1, L, Int. Girder 7 Span 2	II	One crack location with thickness of 0.20mm. Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	FI
				GH2, L, Int. Girder 6, Span 2	II	Crack thickness of 2 mm with spacing of 50 cm. Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	IW
			21 Test Locations for UPV		II	Result of no evident cracks to full depth cracks	IW
		Inventory Level	Gerber Hinge part		Not Ok	Equivalent Truck = 14 tons RF = 0.44	FI
		Assessment of Superstructure	**Gerber Hinge part should be improved**				
	Substructure	Shape/ Dimension	Pier 2 Wall D/S		III	Not so large section loss due to corrosion	FI
			Pier 2 Wall U/S		III	Not so large section loss due to corrosion	FI
			4 Test locations of Schmidt Hammer Test for Piers & Abutment		Ok	Concrete strength of 28~ 49Mpa	-
		Material/ Damage	Bearing of G8, Pier 1		III	Rust scattered and generated extensively is observed, section loss is small, less than 10%	FI
			Bearing of Pier 2		III	Rust scattered and generated extensively is observed, section loss is small, less than 10%.	FI
			Abutment 2		II	Alligator cracks with thickness of 0.3mm located on the downstream face of abutment. Condition of corrosion of rebars is severe. Concrete coverage of 40 – 70mm.	FI
			4 Test locations for Phenolphthalein Test		Ok	Depth of Carbonation = 5mm and 12mm	-
			4 Test locations for Petrographic Analysis		Ok	Presence of alkali silica reaction	-
			1 Core sample each of Piers 1 & 2 for Chloride Test		Ok	Chloride levels were not detected	-
		Strength of Pier Body	Pier P1& Pier P2		Ok	Strength of pier body sufficient (C/D ratio = 0.85)	-
		Assessment of Substructure	**Strength of Pier is insufficient, does not comply with the latest code requirement.**				
	Foundation	Structure/Shape	Pier P1 PSC Piles		Not Ok	Under water survey was not undertaken	FI
			Pier P2 PSC Piles		Not Ok	Under water survey was not undertaken	FI
		Scouring	Underwater		Not ok	Under water survey was not undertaken	FI
		Bearing Capacity/ Stability	Pier P1 (PSC Piles = 350mm x 350mm)		Not Ok	Capacity of pile insufficient (C/D ratio = 0.61)	FI
			Pier P2 (PSC Piles = 350mm x 350mm)		Not Ok	Capacity of pile insufficient (C/D ratio = 0.61)	FI
		Assessment of Foundation	Existing Pile Foundation does not comply with the latest code requirement.				
	Assessment of Structural Soundness		**Existing Bridge structure needs strengthening to meet the present required strength.**				
Traffic Function	Vehicle Weight Limitation Use		14 tons (RF = 0.44, gerber hinge part)				
	LOS		F (v/c = 1.512)				
	Geometrical Features		Fair including approach road				
	Safety of Vessel Transport		Space between piers 34.2 m <43.0m, collision protection for Girder and piers needed				
	Assessment of Traffic Function		Insufficient				
Social Environment	Utilities Suspended at the Bridge		20-ϕ 100 mm Telecommunication Line				
	Squatters		Light. More than 2 families live around the Guadalupe Bridge				
	Historical Aspects		No historical importance				
	Assessment of Social Aspects		Historical situation shall not be considered				
Overall Assessment			**Superstructure improvement work is necessary especially on gerber hinge part Retrofit of substructure is recommended**				

23.2 COMPARATIVE STUDY ON REHABILITATION METHOD

23.2.1 Proposition of Rehabilitation Method

With the aid of criteria set forth in **Chapter 14**, three (3) schemes were proposed and compared for the best possible rehabilitation scheme. These three schemes were prepared based on engineering aspects needed to improve the present condition of the bridge. These are itemized as small-scale rehabilitation, medium-scale rehabilitation and large-scale rehabilitation.

(1) Small-Scale Rehabilitation

Major works for this scheme are as follows:

- Repair and sealing of concrete cracks, honeycomb and spalling.

Countermeasure in this scheme is not permanent because shear capacity at gerber hinge portion is insufficient. The least durable among schemes and no improvement in the overall bridge strength and integrity.

(2) Medium-Scale Rehabilitation

In addition to the repair and sealing of concrete cracks, honeycomb and spalling, major works for this scheme include:

- Rehabilitation of gerber hinge part by installation of slanted P/S cables.
- Reconstruction of diaphragm and partial reconstruction of deck slab at gerber hinge portion.

This scheme will improve the durability of the whole structure because the shear capacity is effectively increased on the gerber hinge portion.

(3) Large-Scale Rehabilitation

The following major works were considered for these scheme in addition to the medium-scale rehabilitation works.

- Additional elastometric bearing pads at diaphragm.
- Installation of transverse P/S cables at diaphragm of Gerber hinge portion.

More stability will be attained in this scheme by the addition of the transverse cables and is more durable and effective than the previous schemes. Addition of support at the center between each girder will lessen the loads of individual bearing support.

23.2.2 Evaluation of Rehabilitation Method

From the three schemes formulated above, the recommended as best scheme in terms of engineering aspects is Scheme 3 Large-scale Rehabilitation Scheme. Structural capacity and durability of the bridge will be improved considerably by installation of P/S cables at gerber hinge and at the diaphragm. Construction duration for this scheme is 12 months and cost is justifiable for the overall improvement of the bridge. For the detailed description of possible rehabilitation schemes, see **Table 23.2.2-1.**

23.2.3 Lifecycle Cost Analysis of the Bridge

(1) Procedure

Based on the bridge condition survey mentioned in **Section 23.1** and engineering study made in **Section 23.2.1**, the life cycle cost (LCC) analysis of the Guadalupe Bridge is carried out in this section. The procedure of the LCC analysis of the Bridge employed is the same as that of Ayala Bridge as shown in **Figure 14.3.1-1**.

(2) Guadalupe Bridge Deterioration Situation

The standard deterioration curve of deck, superstructure and substructure between condition rating and age using the same equation mentioned in **Section 14.3.** According to the bridge condition survey, however, the deck and superstructure of Guadalupe Bridge have deteriorated more than two (2) times that of the standard deterioration due to the construction of superstructure being not appropriate.

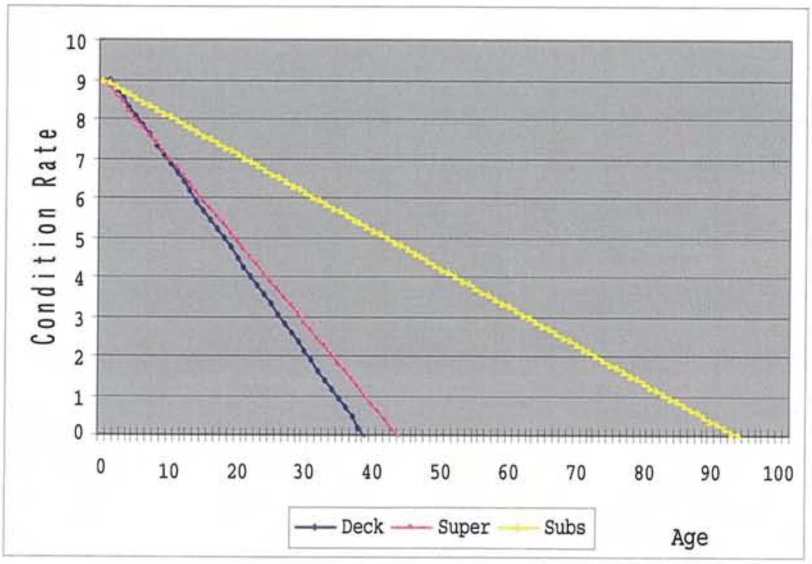
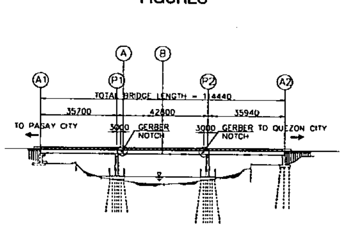
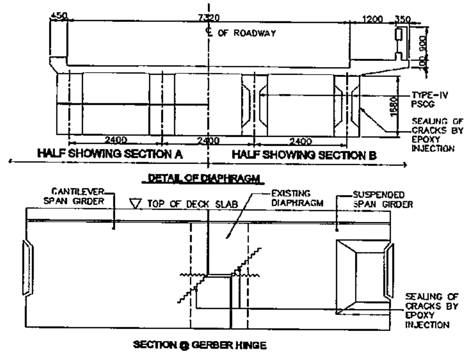
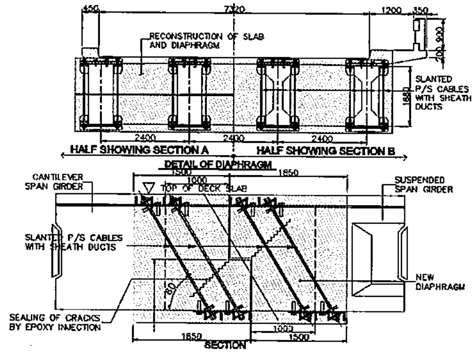
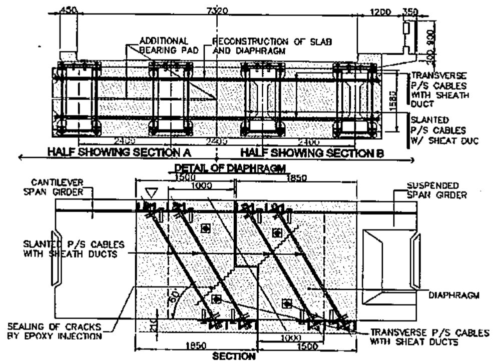


Figure 23.2.3-1 Deterioration Curve of Guadalupe Bridge

The deterioration curve of the Guadalupe Bridge is estimated and shown in **Figure 23.2.3-1**.

Table 23.2.2-1 Comparative Study of Possible Schemes for Guadalupe Bridge

IMPROVEMENT TYPE		PROPOSED SCHEMES					
		SCHEME 1 : SMALL SCALE REHABILITATION CRACK SEALING		SCHEME 2 : MEDIUM SCALE REHABILITATION SLANTED P/S CABLE STRENGTHENING		SCHEME 3 : LARGE SCALE REHABILITATION TRANSVERSE AND SLANTED P/S CABLE STRENGTHENING	
FIGURES  LOCATION OF SIGNIFICANT STRUCTIRE DAMAGE							
MAJOR WORKS		• Repair and sealing of concrete cracks, honeycomb and spalling.	RATING	• Repair and sealing of concrete cracks, honeycomb and spalling. • Rehabilitation of Gerber hinge portion with slanted P/S cables. • Reconstruction of diaphragm and partial reconstruction of deck slab at Gerber hinge portion.	RATING	• Repair and sealing of concrete cracks, honeycomb and spalling. • Rehabilitation of Gerber hinge portion with slanted P/S cables. • Reconstruction of diaphragm and partial reconstruction of deck slab at Gerber hinge portion. • Additional elastomeric bearing pads at diaphragm. • Installation of transverse P/S cables at diaphragm of Gerber hinge portion.	RATING
(1) STRUCTURAL ASPECT (30%)	STABILITY	• Countermeasures taken is not permanent because shear capacity at Gerber hinge portion is insufficient.	E	• Slanted P/S cables at Gerber hinge will prevent shear failure due to diagonal tension and make cracks close. • Cracks at the Gerber hinge will be addressed efficiently. This is more stable than scheme 1.	C	• Slanted P/S cables at Gerber hinge will deter shear failure and diagonal tension. • Additional new support in between the girders will distribute stresses. • Transverse P/S cables at the diaphragm will effectively integrates the girder together. • More stability is attained by this scheme than scheme 2.	A
	DURABILITY	• Least durable among schemes. • No improvement is concentrated to overall bridge integrity because rehabilitation is only on repairing of cracks but not addressing or preventing girder from developing further cracks.		• Durability of the whole structure is improved because the shear capacity is effectively increased on the Gerber hinge portion. • The slanted P/S cables will effectively improve the durability of the Gerber hinge.		• Installation of transverse tendons is more durable and effective than scheme 2. • Addition of support at the center between each girder will lessen the loads of individual bearing support.	
(2) CONSTRUCTION ASPECT (30%)	COST	• PhP 11.34 Million • Least Expensive	A	• PhP 18.20 Million	B	• PhP 20.50 Million • Cost is the most expensive.	C
	DURATION	• Shortest at three (3) months.		• Ten (10) months.		• Longest at twelve (12) months.	
	METHOD / DIFFICULTY	• Easiest and fastest to execute.		• Requires partial removal of deck slab and full replacement of diaphragm. • Setting out and stressing of slanted P/S cable at Gerber hinge will be laborious.		• Requires partial removal of deck slab and full replacement of diaphragm. • Setting out and stressing of transverse P/S cable at Gerber hinge is more complicated than scheme 2.	
(3) TRAFFIC / NAVIGATION FUNCTIONALITY (20%)		• Existing condition for traffic functionality and navigation clearance will be maintained.	A	• Existing condition for traffic functionality and navigation clearance will be maintained.	A	• Existing condition for traffic functionality and navigation clearance will be maintained.	A
(4) IMPACT TO TRAFFIC DURING CONSTRUCTION (15%)	AFFECTED LANES	• Traffic disturbance is minimal. • No temporary closure is necessary during repair of concrete cracks, honeycomb and spalled concrete.	A	• Partial closure of affected traffic lane is necessary during repair. • Temporary full closure is unavoidable during concreting of diaphragm and stressing of tendons.	B	• Partial closure to traffic is necessary during removal of existing diaphragm and partial removal of deck slab. • Temporary full closure of bridge is necessary during stressing operation and concreting work.	B
	LOAD CAPACITY REDUCTION	• Full lane vehicle load can be maintained during construction.		• Lane vehicle load will be limited during removal and reconstruction of existing diaphragm and deck slab.		• Lane vehicle load will be limited during removal of existing diaphragm and partial removal of deck slab up to the installation of rebars, tendons and formworks.	
	DETOUR	• Not necessary.		• Rerouting is necessary during removal and reconstruction of existing diaphragm and deck slab.		• Rerouting is necessary during concreting of diaphragm and stressing of tendons.	
(5) SOCIAL / ENVIRONMENTAL IMPACT (5%)		• Application of structural epoxy will have minimal impact to river.	A	• Application of structural epoxy and concreting works will have slight impact to river.	B	• Application of structural epoxy and concreting works will have slight impact to river.	B
OVERALL EVALUATION AND RATING		3	3.80	2	3.90	1 (RECOMMENDED)	4.20

NOTES : A = EXCELLENT (5 pts) B = GOOD (4 pts) C = SATISFACTORY (3 pts) D = BELOW SATISFACTORY (2 pts) E = NOT RECOMMENDED (1 pt)

(3) Rehabilitation Schemes and Cost Estimates

The engineering study proposed the rehabilitation schemes and cost estimates as shown in **Table 23.2.2-1**

(4) Lifecycle Cost Analysis Model

In the life cycle analysis model, there are principally two (2) cases;

a. Replacement case
b. Rehabilitation case

The explanation of the lifecycle cost analysis model for these two (2) cases are as shown in **Section 14.3.4**;

(5) Extended Service Life by Improvement Proposals

From the deterioration curve in **Figure 23.2.3-2**, and using the relationship between investment cost and improvement condition rating shown in **Figure 14.3.5-1** in **Section 14.3.5,** the expected extended service life of Guadalupe Bridge is calculated and shown in **Figures 23.2.3-2** and **23.2.3-3**. The service life of the Bridge is varied and extended depending on the type of rehabilitation. If large scale rehabilitation is implemented, the service life of the bridge is expected to extend 32 years so the total service life will be 38 years from 2007.

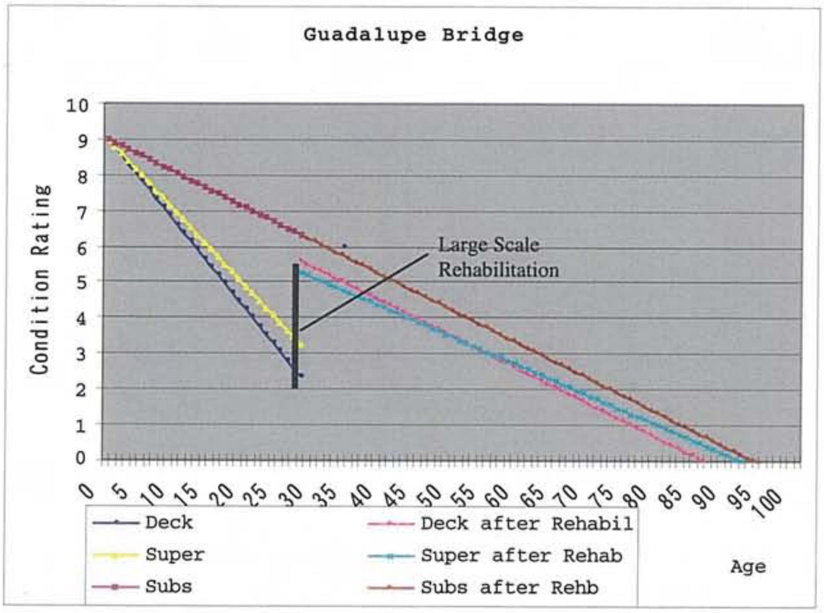


Figure 23.2.3-2 Estimated Service Life by Rehabilitation Types of Guadalupe Bridge under Large Scale Rehabilitation

(6) Calculation of the Lifecycle Cost of the Guadalupe Bridge

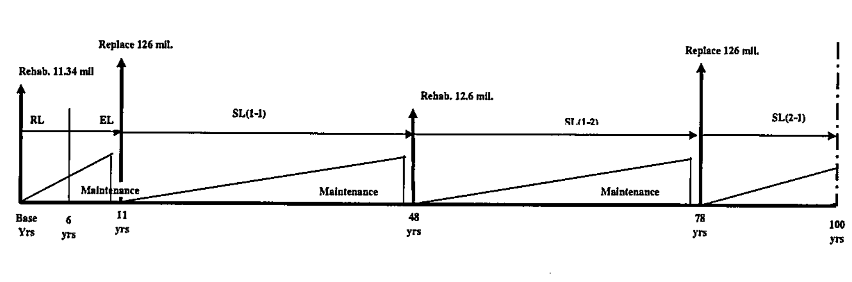
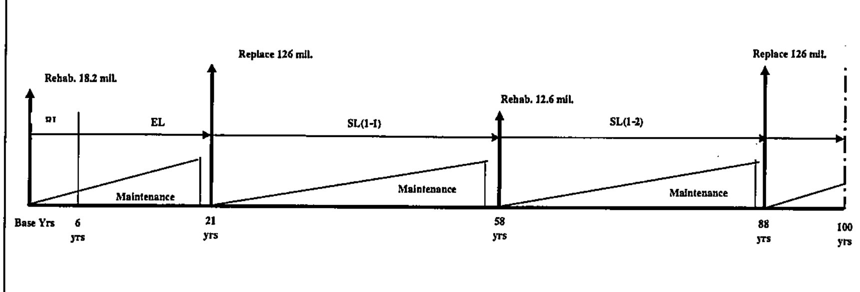
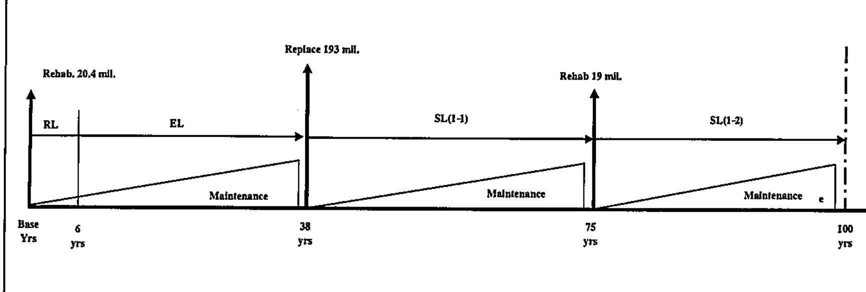
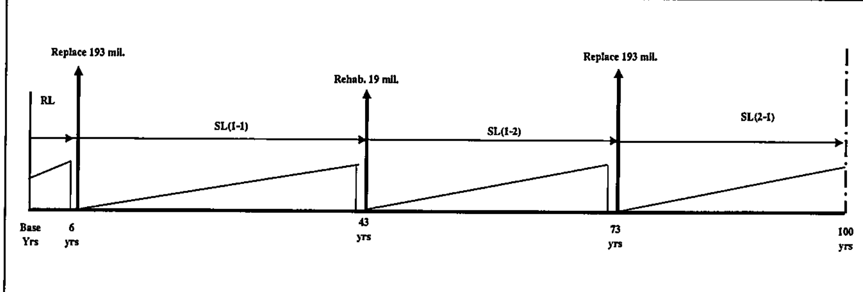
The lifecycle cost of the Guadalupe Bridge is calculated and shown in **Table 23.2.3-1** and judging from the LCC of alternative rehabilitation works, the large scale rehabilitation scheme is recommended for the rehabilitation of Guadalupe Bridge.

Table 23.2.3-1 Lifecycle Cost Estimates of Guadalupe Bridge by Rehabilitation Types

Unit: Million Pesos

	LCC at Discount Rate of 15%	Recommended Rehabilitation Ranking from LCC Analysis
Small Scale Rehabilitation	43.9 (0.79)	3
Medium Scale Rehabilitation	43.9 (0.79)	2
Large Scale Rehabilitation	42.8 (0.76)	1
Replacement	55.8 (1.00)	4

Notes:1) Discount rate is assumed to be 15%.
2) Recommended improvement ranking is based on the LCC
3) ( ) Ratio of life cycle cost to replacement

Scheme	Life Cycle Cost Model	Service Life (Yrs)	LCC (Mil Peso)(Discounted rate 15 %)
Small Scale Rehabilitation		Year RL 6 EL 4 SL(1) 67 SL(2) 23 Total 100	43.9 million (0.79)
Medium Scale Rehabilitation		Year RL 6 EL 15 SL(1) 67 SL(2) 12 Total 100	43.9 million (0.79)
Large Scall Rehabilitation		Year RL 6 EL 32 SL(1) 62 Total 100	42.8 million (0.76)
Replacement		Year RL 6 SL(1) 67 SL(2) 27 Total 100	55.8 million (1.00)

Notes: RL: Remaining life
EL: Extended life due to rehabilitation
SL: Bridge cycle life
SL(1): Service life of first bridge cycle
SL(2): Service life of second bridge cycle life
SL(1-1): Service life of first bridge cycle life before rehabilitation
SL(1-2): Service life of first bridge cycle life after rehabilitation

Figure 23.2.3-3 Life Cycle Analysis of Guadalupe Bridge

23.2.4 Recommendation

Based on the life-cycle cost analysis conducted on the possible rehabilitation schemes, the most recommendable scheme is the **medium-scale rehabilitation scheme** because it is the least expensive at a discount rate at 15%.