CHAPTER 24

FEASIBILITY STUDY OF VARGAS BRIDGE REHABILITATION PLAN

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24.1 DETAILED BRIDGE SURVEY AND ASSESSMENT

Discussion in this Chapter is with reference to the flow chart shown in **Figure 13.1-1** of **Chapter 13**. The survey level is the detailed survey which follows the procedure as defined in the Manual prepared by the Study Team which corresponds to "Feasibility Study Level".

24.1.1 Review of Design and Repair Works

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

(1) Review of Design

(a) Outline of Vargas Bridge (Upstream)

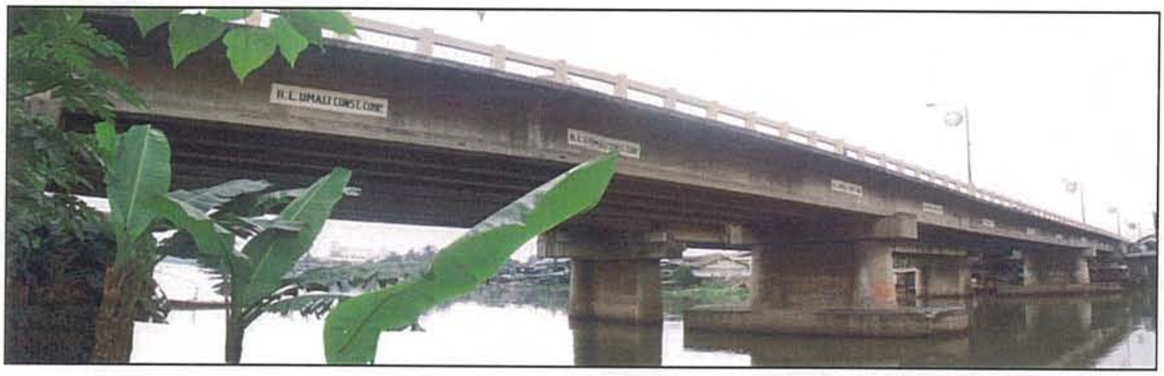


Photo 24.1.1-1 Panoramic View of Vargas Bridge (Upstream)

- Structure Type : Four-Span Cantilever Type Prestressed Concrete Deck Girder Bridge
  Four (4) PC AASHTO Type V girders support the 2-lane concrete deck. The PC girders rest on Two-column bent piers supported by retrofitted foundation on steel piles.
- Bridge Length : 122.44m (19.30m + 30.50m + 50.60m + 22.04m)
- Date of Construction : 1992

(b) General Notes on Vargas Bridge Construction Drawings

<u>**Design**</u>

- Design of Vargas Bridge (Upstream) was prepared by Department of Public Works and Highways, Bureau of Design, August 1988

- Design Code Reference: 1977 AASHTO Standard Specification for Highway Bridges
- Design Loads :

Live load	:	MS18
Dead load (surfacing)	:	1.05 kpa of roadway for future wearing surface

- Allowable Stresses :

Cast in place concrete	:	f'c = 20.68 MPa fc = 8.27 MPa
Precast prestressed concrete	:	f'c = 39 MPa @ design load f'c = 36 MPa @ transfer
Reinforcing steel	:	fs = 137.9 MPa (Grade 40)
Prestressing steel	:	fs = 1862 MPa (Grade 270)

Construction and Materials

- Design criteria were based on Government Standard Specification for Highway Bridges and Airports revision 1988.
- All concrete types indicated were class "A", except railings which were indicated as Class "C".
- The bridge was constructed in 1992, by contractor R.L. Umali Construction Corporation.

Reinforcing Steel

- Reinforcement steel specifications was not indicated.

Tubular Steel Piling

- All piles indicated were YN18" (450 mmϕ) tubular steel piles. Exact length of piles was determined from the results of driving test piles, and was driven to a minimum bearing.

Note on Approaches

- Embankment Section construction method used was based on the Standard Specifications for Highways and Bridges, revised 1988.

(2) Review of Repair Works

- The widening of Vargas Bridge (Upstream) was completed in 1992.
- Retrofit work was done in 1997.

The following items were recommended in the report of Briefing Information concerning the BRP and the Retrofit of selected Metro Manila Bridges, February 1997:

- To make deck continuous with reconstruction of end diaphragms.
- To install vertical cable restrainers on abutments & Piers (see **Photo 24.1.1-2**).
- To add shear keys at top of piers to adequately transmit seismic forces from the superstructure to the substructure (see **Photo 24.1.1-2**).
- To drive four steel piles at each pier footing to reduce seismic forces being absorbed by the existing piles.
- To provide footing overlay to accommodate new piles and to increase flexural and shear capacity of the footing (see **Photo 24.1.1-3**).

Photo 24.1.1-2 Shear keys on all piers and vertical cable restrainers on Pier 1

Photo 24.1.1-3 Enlarge pile cap and additional piles (not seen on picture)

Problems / Issues of Previous Repair Works

Previous rehabilitation works made no improvement measure in the deflection of girders (see **Photo 24.1.1-4**) and propagation of cracks in PC girders. Cracks on girders are evident at top of piers and gerber hinge parts.

Photo 24.1.1-4 Deflection of Girders

(3) Historical Background

Vargas Bridge (Upstream) has no historical significance as per National Historical Institute report and interposes no objection in the rehabilitation of the bridge.

24.1.2 Natural Condition Survey

(1) Topographic Survey

(a) Control Monument

Two (2) GPS Stations were established as control points for Vargas Bridge as shown in **Table 24.1.2-1.**

Table 24.1.2-1 GPS Stationing and Coordinates

STATION	GPS COORDINATES		
	NORTHING	EASTING	ELEVATION
BM-1	1610980.803	507712.018	21.325
BM-2	1610967.675	507862.940	21.329

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 24.1.2-2 shows the scope of works of topographic survey. Topographic plan is shown in **Appendix 24.1.2-1**.

Table 24.1.2-2 Scope of Work of Topographic Survey

Description	Original Scope	Actual Work
CONTROL POINT SURVEY (GPS)	1	2
PROFILE SURVEY	123 m Bridge Section + 200 m Each of Both Approach Roads (200 x 2) Total = 523 m	123 m Bridge Section + 200 m Each of Both Approach Roads (200 x 2) Total = 523 m
ROAD CROSS-SECTION SURVEY	Bridge Section (123m) : @ 10m Interval Approach Roads (400m) : @ 20 m Interval Width: Bridge 24m + 50m each at both sides = 124m Total = 34 Sections	Bridge Section (123m) : @ 10m Interval Approach Roads (400m) : @ 20 m Interval Width: Bridge 24m + 50m each at both sides = 124m Total = 34 Sections
TOPOGRAPHIC SURVEY	523 m (Length) x 108 m (Width) = 56,000 sq. m	523 m (Length) x 108 m (Width) = 56,000 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

The uppermost 7.0 meter portion of the borehole undertaken in one of the abutment locations of the existing bridge is made up of a granular soil formation. Dense with N-value = 40 only in the topmost 1.0 meter portion, the rest of the layers are loose to medium dense with N-values varying from 7 to 22.

Cohesive layers of sandy elastic silt with gravel and sandy clay underlies the upper granular soils from 7.0 to 10.0 meter depth. They are generally stiff in consistency with N-values varying from 9 to 13.

Another granular layer is present from 10.0 to 13.0 meters of the borehole. It consists of poorly graded sand with gravel and silt and silty gravelly sand generally non-plastic and dense with N-values varying from 30 to 31.

A predominantly cohesive formation is present down to 19.0 meter depth of the borehole. The silty clay and clayey silt layers are soft to medium stiff with N-values varying from 3 to 8. A loose (N=9) silty gravelly sand layer is present at 19.0 to 20.0 meter with N-values of 32 to 34.

The rock formation under the bridge site occurs at 23.0 meter depth. It is made up of generally moderately fractured tuff formation.

Geotechnical survey results were used in the estimation of seismic forces and resistance of foundation. Borehole log and location are shown in **Appendix 24.1.2-2**.

(3) Scour Survey

An Echo Sounder (Hondex PS-7 LCD Digital Sounder) combined with Total Station was used for the determination of the riverbed configuration, with observations taken at every 1-meter intervals at the abutments and piers. These are shown in **Appendix 24.1.2-3.**

As shown in **Figure 24.1.2-1**, scouring around substructures is not evident.

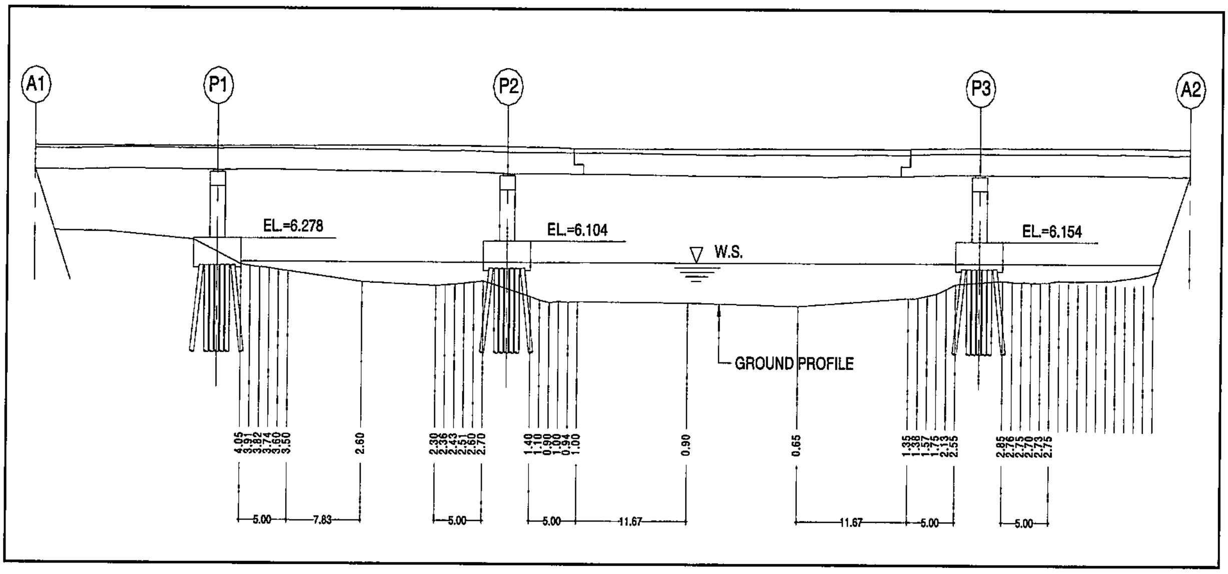


Figure 24.1.2-1 Result of Scour Survey conducted on Vargas Bridge

24.1.3 Bridge Condition Survey and Identification of Damages

(1) Shape and Dimension Measurement

(a) Objective

The main purpose of this activity is to perform measurements on the main and secondary members of Bridge.

(b) Inspection Teams

The inspectors conducted measurement on deck and below superstructure, and substructure.

(c) Coverage Area

The bridge was divided into two (2) general inspection areas namely: (1) road deck level, and (2) below deck level (including substructure).

Verification on road deck measurements includes deck slab, length of bridge, sidewalk, post, railings and expansion joint gap (see **Photo 24.1.3-1**).

Photo 24.1.3-1 Road / Deck Level Inspection

Photo 24.1.3-2 Below Deck Level Inspection

Verification of measurements below deck level were taken on all 4 girders in every span (see **Photo 24.1.3-2**). The spacing of girders was also measured as well as intermediate and end diaphragm. Gerber hinges were also measured. Substructures were also measured including the exposed areas of pile cap.

The deflection of girders was also surveyed as shown in **Figure 24.1.3-1**.

(d) Reference Information

The Study Team was furnished with copies of as-built plans of the bridge including retrofitting drawings. They were used as references in planning various activities, and in filling-up the preliminary verification forms.

(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.

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

Photo 24.1.3-3 Use of Gondola on below deck inspection

Photo 24.1.3-4 Use of motorized boat to transport inspection teams from pier to pier

Verification of measurements below deck level required the use of suspended foot bridge or Gondola as shown in **Photo 24.1.3-3**. This system was used on high superstructure locations. Erection of Gondola was done manually. The motorized boat was used to transport inspection teams from pier to pier as shown in **Photo 24.1.3-4**.

Each activity and inspected damages were supported with photos and dimensions were recorded.

(f) Miscellaneous Structure

Miscellaneous structure, including non-structural elements, were noted and photographs were taken. These includes 8-100 mm ϕ PVC Telecommunication Lines as shown in **Photo 24.1.3-5**.

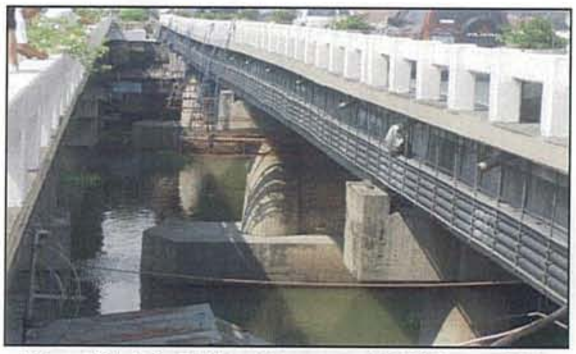


Photo 24.1.3-5 Miscellaneous / Utilities - PVC pipe lines attached at side of exterior girders

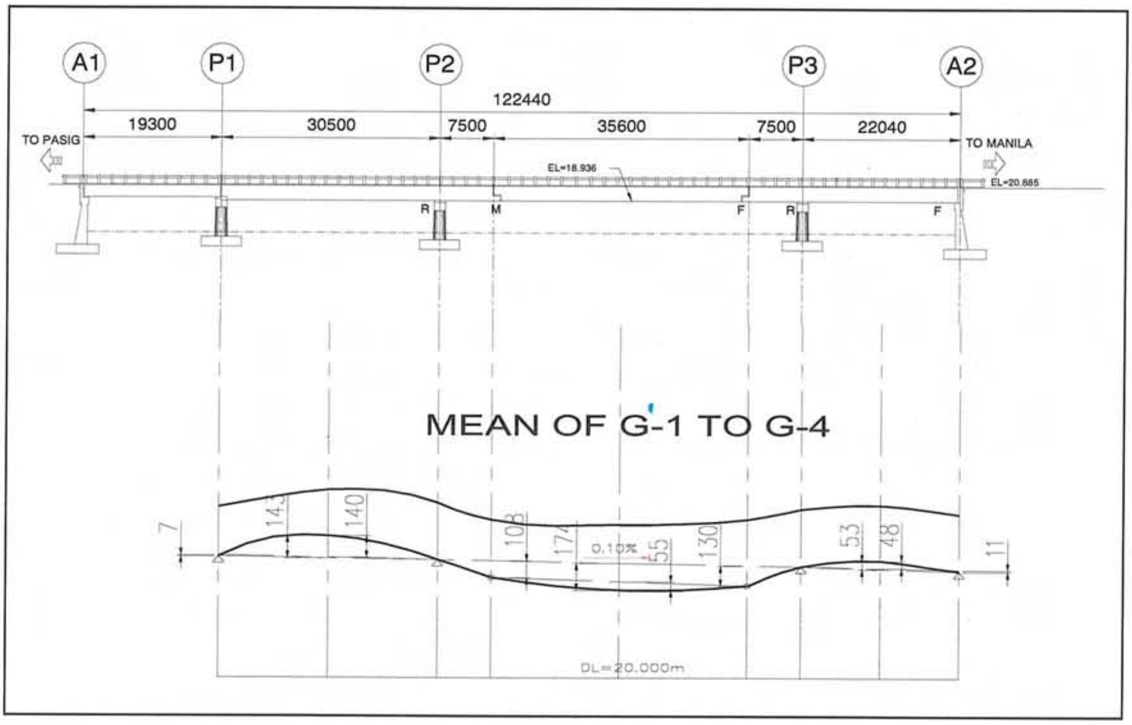
(g) Results

Table 24.1.3-1 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.

Profile of the superstructure verification yielded girder deflections at cantilever portions that ranged from 110mm to 120mm. The profile-elevation of girder soffit deflection is presented in **Figure 24.1.3-1.**

Table 24.1.3-1 Lists of Drawings

Sheet No.	Title	Appendix
1	General Elevation and Plan	Appendix 24.1.3-1 (1/6)
2	Main Girder and Section	Appendix 24.1.3-1 (2/6)
3	Gerber Details	Appendix 25.2.3-1 (3/3)
4	Abutment A1 and A2	Appendix 24.1.3-1 (4/6)
5	Pier P1, P2 and P3	Appendix 24.1.3-1 (5/6)
6	Bearing Details	Appendix 24.1.3-1 (6/6)



PROFILE OF GIRDER DEFLECTION

Figure 24.1.3-1 Result of Measurement on Deflection of PC Girders on the above figure can be seen evidently along the side of the superstructure (see photo)

(2) Close-up Visual Inspection

(a) Objective

This activity aimed to summarize the observed damages, condition of structural members, conditions of accessories and documentation by taking digital photos of damages.

(b) Inspection Team

Teams of inspectors were deployed to conduct a Close-up Visual Inspection of bridge condition.

(c) Coverage Area

The activity was divided into three (3) general inspection areas namely: (a) road deck level, (b) below deck level, and (c) substructure.

Foundation inspection however, were only limited to visible area.

(d) Reference Information

The study team was furnished with copies of as-built drawings and retrofitting drawings of the Bridge (Upstream). They were used as reference in planning this activity, and in filling the verification forms.

(e) Equipment and Procedure

This activity was also taken up during the verification of shapes and dimension.

(f) Criteria for Damage Rating

The damages follow the X-Y-Z Method as stated in the criteria and procedures in rating the damages in **Section 6.4, Chapter 6.**

(g) Results

- Damages found were mostly cracks on gerber hinge parts. The cracks were drawn in a matrix as part of documentation.
- The Bridge have noticeably large deflection problem. The largest of which is located at the cantilever portion of the girder.
- The vertical restraining cables at abutments were provided for falling prevention against longitudinal displacement of girders and not for uplift/hold down purposes.
- The damages of the Bridge are shown in **Figure 24.1.3-2.** The damage rating of main members based on Close-up Visual Inspection is shown on **Table 24.1.3-2**.

The damage sheets were documented in **Appendix 24.1.3-2 (1/4) to (4/4).**

(3) Non-Destructive Test of Superstructure

(a) Objective

This activity aimed to assess the strength and condition of materials and structural components of bridges by conducting test procedures.

Result of close-up visual inspection and the importance of the member/joint were considered in deciding the specific location where the non-destructive tests was conducted.

CRACK AT GIRDER 1 ON TOP OF PIER 2
WIDTH OF CRACK = 2 mm
DEPTH OF CRACK = 17 mm

CRACK AT GIRDER 1 AT GERBER HINGE 1
WIDTH OF CRACK = 1 mm
DEPTH OF CRACK = 44 mm

CRACK AT GIRDER 1 AT GERBER HINGE 2
WIDTH OF CRACK = 21 mm
DEPTH OF CRACK = 43 mm

CRACK AT GIRDER 1 ON TOP OF PIER 3
WIDTH OF CRACK = 2 mm
DEPTH OF CRACK = 217 mm

DEFLECTION OF GIRDERS

SPALLING WITH EXPOSED REBARS SPAN 3
AREA = 0.45 SQ. M.

CRACK AT BACKWALL OF ABUTMENT A
WIDTH OF CRACK = 1 mm

CRACK AT BOTTOM OF COPING AT PIER 3
WIDTH OF CRACK = 5 mm

FRACTURE OF RAILING BECAUSE OF
DISMOUNTED CONCRETE POST

Figure 24.1.3-2 Close-Up Visual Inspection of Damages in Vargas Bridge (Upstream)

(b) Results

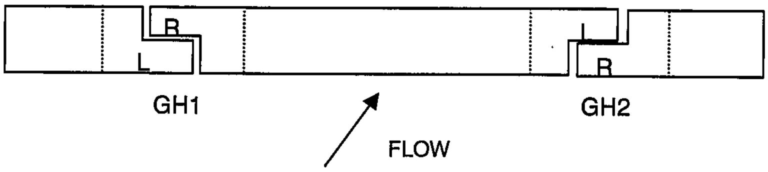
Table 24.1.3-3 shows the results of the tests conducted in specific locations of the bridge.

Table 24.1.3-3 Results of Non-Destructive Test

Test	Results	Reference Appendices
Ultrasonic Pulse Velocity Test (To determine depth of cracks on concrete members	A total of four (4) test locations were chosen for UPV. Results of crack depths vary from 17mm to 217mm.	Appendix 24.1.3-3 (1/4) to (4/4)
Phenolphthalein Test (To determine the depth of carbonation)	Two (2) Cores were examined, the depth of carbonization ranges from 0mm to 6mm. Portions tested for Carbonation Test were as follows: (1) Pier 2, Footing (2) Pier 3, Footing	Appendix 24.1.3-4 (1/1)

Table 24.1.3-2 Damage Rating of Main Members by Close-up Visual Inspection

GH: Gerber Hinge



Reference Location

ITEM		Member / Location	Damage Rating [1]	Description
Superstructure	Shape / Dimension	G1/ Span 4/ Mid	III	Spalling
		Suspended Girder	II	Deflection
	Material/ Damage	Exterior Main Girder, Girder 1 at Pier 2	I	Concrete Cracks
		Exterior Main Girder, GH1, L, Girder 1 of Span 3	I	Concrete Cracks
		Exterior Main Girder, GH2,R, Girder 1 of Span 3	I	Concrete Cracks
		Exterior Main Girder, G1/ GH1 at Pier 3	I	Concrete Cracks
		Exterior Main Girder, G1/ GH2, L at Pier 3	I	Concrete Cracks
		Main Exterior Girder, G1/ GH2,R at Pier 3	I	Concrete Cracks
		G-1/ Pier 3	I	Concrete Cracks
Substructure/ Bearing	Shape/ Dimension	Pier 1, Pier Coping	III	Spalling/ Exposed Rebars
		Pier 3, Top of Pier Coping	III	Spalling/ Exposed Rebars
	Material / Damage	Abutment A1, Backwall	III	Concrete Cracks
		Abutment A1, Bearing Anchorage	III	Corrosion
		Pier 3, Pier Coping	III	Concrete Cracks

Note: 1. Damage Rating Level is based on the XYZ Damage Rating Method.

(4) Special Test

(a) Microtremor Measurement Survey

Objective

This survey was conducted to determine the fundamental frequencies and natural dynamic characteristics of the bridge. The results were compared with the structural analysis model and a conclusion was made regarding the structural soundness of the superstructure.

Acceleration Sensors

Five (5) force balance accelerometers (Kinemetrics ES-U) were used in the survey in each measurement event: Four (4) were set to measure vertical accelerations and one (1) was set to measure horizontal acceleration. All the accelerometers were set to a full scale range of ± 0.25g.

Figure 24.1.3-3 shows the locations of sensors during the survey.

Most Probable Natural Frequencies

The survey results yielded the most probable natural frequencies of the superstructure as 2.30hz, 3.7hz, 3.9hz and 4.3hz. The results were compared with the analysis and is tabulated in **Table 24.1.3-4**.

The result of analytical model using STRAND7 software and compared with the microtremor survey results is shown in **Table 24.1.3-4** below.

Table 24.1.3-4 Comparison of Natural Frequency from Test and Analysis

From Microtremor Test		From STRAND7 Analysis	
Most Probable Mode Shapes	Most Probable Natural Frequency (Hz)	Mode Number	Natural Frequency (Hz)
Vertical	2.30	1	2.15

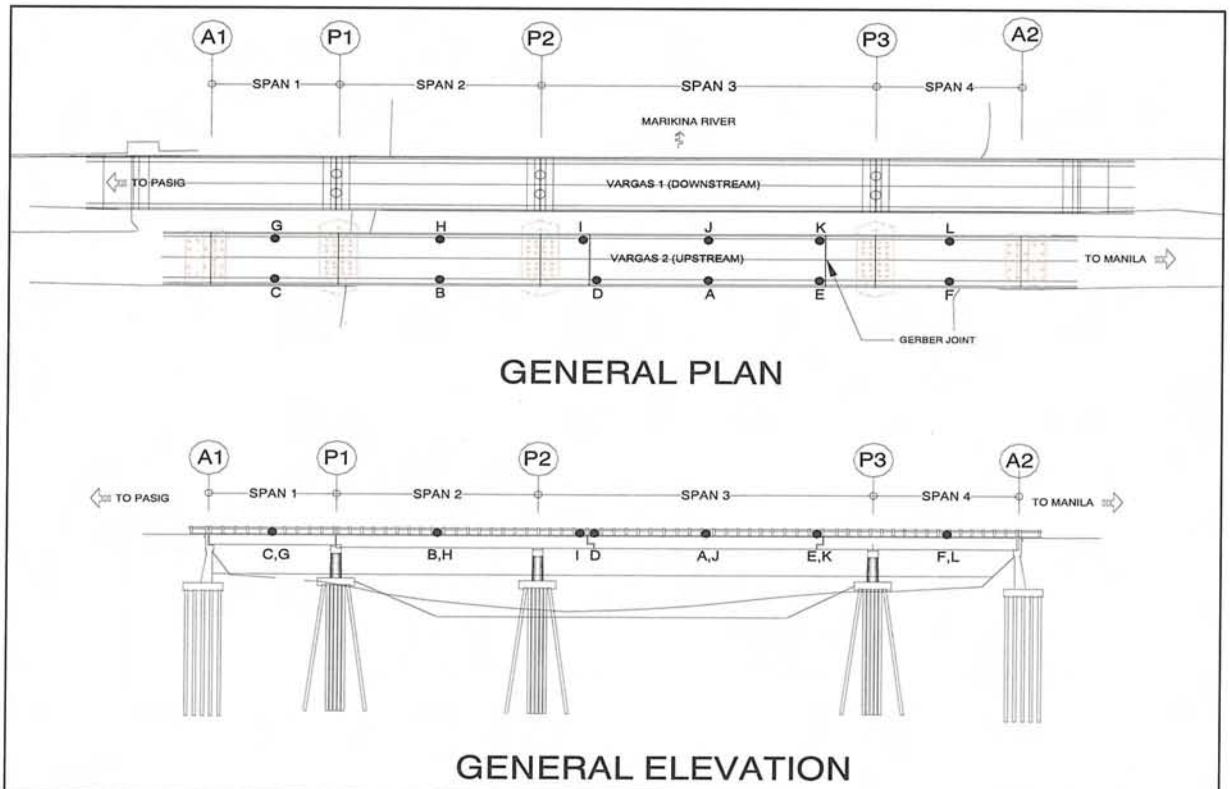


Figure 24.1.3-3 Location of Acceleration Sensors on Vargas Bridge Microtremor Survey

(b) Impact Vibration Test of Substructure

Objective

This test was conducted to evaluate the substructure soundness by focusing on the natural frequency of the pier.

Procedure

An impact pendulum was specially fabricated for this activity. The pendulum was positioned to impact the centerline of the coping beam at Pier 2 (Transverse Direction). (See **Photo 24.1.3-6**). The impacting head is 300 mm diameter steel ball with a mass of about 60 kg.

Photo 24.1.3-6 Impact Pendulum Setup

Sensors and Locations

Two (2) sensors were mounted on top of Pier P2, at location P. One was directed along the transverse direction, while the other was oriented vertically. Two (2) more sensors, set to capture transverse accelerations, were installed along the height of the pier. One was placed approximately 1.5m from the top of pile cap at location Q, and the other approximately 0.3m above the pile cap at location R. Lastly, a sensor oriented vertically was mounted also on top of pier, at the far side from the point of impact at location S. **Figure 24.1.3-4** illustrates the locations of sensors. Resulting time history of 14 stacked impact record and Amplitude Spectra are shown in **Figure 24.1.3-5**.

The results of impact test is compared with the result analysis conducted in **Table 24.1.3-5**.

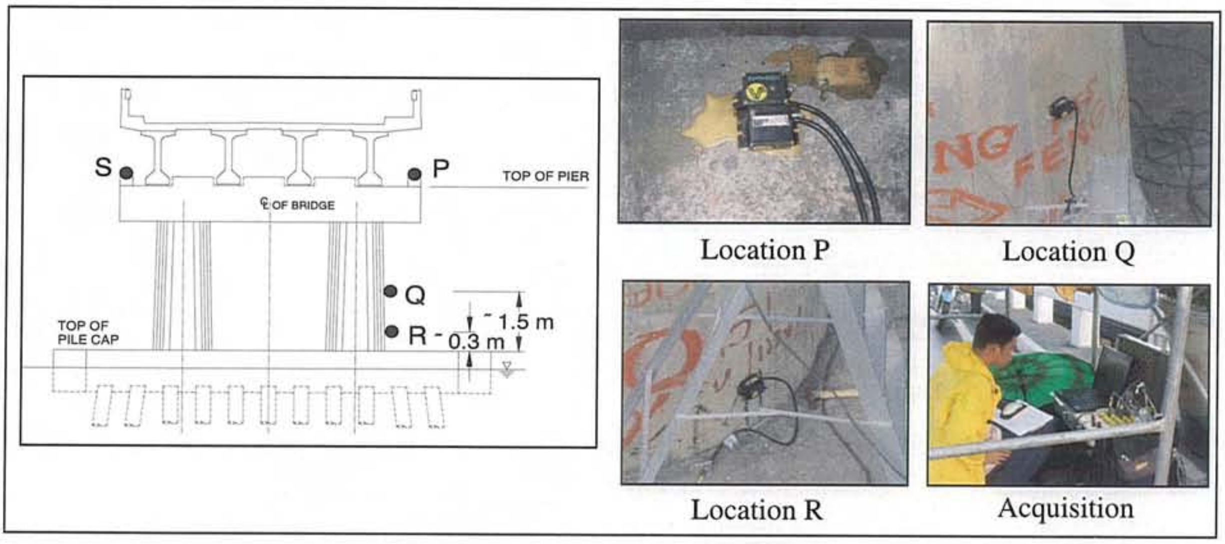


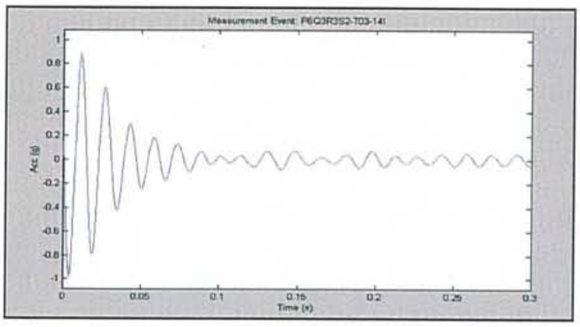
Figure 24.1.3-4 Sensor Locations at Pier

Results of impact vibration test and compared with STRAND7 analysis are shown below:

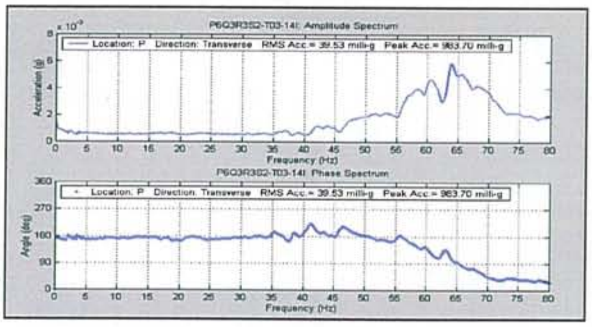
Table 24.1.3-5 Structural Soundness of Pier P2

Mode Shape	Natural Frequency		Rating Index*	Remarks
	From Impact Test (Hz)	From STRAND7 Analysis (Hz)		
Transverse	66.00	65.70	1.005	(>0.85) ok!

* Rating Index calculation and evaluation, see Table 13.3.6-3, Item 8, Section 13.3.6, Chapter 13.



Time History of 14 Stacked Impact Record



Amplitude Spectra

Figure 24.1.3-5 Time History and Amplitude Spectra from Impact Vibration Test Results

(5) Assessment of Critical Damages

(a) Evaluation Criteria

Damages of bridge members 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

Damage rating follows the procedures and criteria set forth in **Section 6.4** of **Chapter 6**.

(c) Categorization of Damage Rating

The damage rating was categorized based on Section 4.10 of the Manual.

Evaluation of Damages

Results of evaluation on damages for main members were tabulated in **Table 24.1.3-6**.

24.1.4 Presumption of Original Design

(1) Objective

The purpose of the presumption of original design was to reconstruct the unavailable information or data that will be the basis of bridge inventory and inspection.

(2) Structural Shape and Dimensions

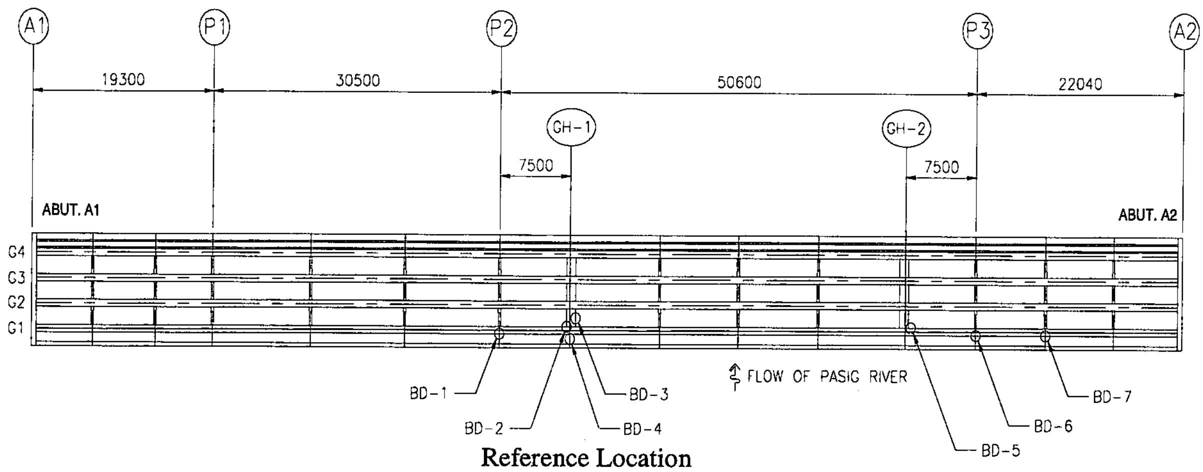
Superstructure

All necessary data and informations were available in as-built drawings and construction drawings. In addition, retrofit drawings were available. Structural shapes and dimensions were verified and documented in **Appendix 24.1.4-1 (1/18 to 10/18)**. Section properties of Superstructure used in the analysis is shown in **Appendix 24.1.4-1 (11/18 to 18/18)**.

Substructure

The structural details and dimensions were made available by construction drawings and retrofit drawings for this bridge. Verified informations and data for substructure are documented in **Appendix 24.1.4-2 (1/2 to 2/2)**.

Table 24.1.3-6 Evaluation of Major Damages on Vargas Bridge



Reference Location

	Location		Evaluation Based on Field Survey				Evaluation Based on Non-Destructive Test	Evaluation based on Special Test
	Member	Node	Damage Type	Damage Rating[1]	Diagnostic Category[2]	Estimated Section Loss		
SUPERSTRUCTURE	Exterior Girder 1	Pier 2	CR	I	A	-	• Ultrasonic Pulse Velocity Test– **Crack Depths vary from 17mm to 217mm**	• Micro tremor Test **-OK**
	Exterior Girder 1	GH1, L, Span3	CR	I	A	-		
	Exterior Girder 1	GH2, R, Span 3	CR	I	A	2 %		
	Exterior Girder 1	GH1, L, Span3	CR	I	A	2 %		
	Exterior Girder 1	Span 4	S	III	B	2 %		
	Girder		DE	II	B	-		
SUBSTRUCTURE	Abutment A1	Back wall	CR	III	B	-	• Phenolphthale in Test – **Depth of carbonization for Pier 1 and Pier 2 ranges from 0mm to 6mm** • Compression test of concrete samples and gave results of compressive strength ranging from **21.1 MPa to 30.7 MPa**	-
	Abutment A1	Bearing Anchor-age	CO	III	B	3 %		Impact Vibration Test, Pier P2: **Rating Index = 1.005>0.85 Ok**
	Pier 1	Coping	SER	III	B	0.12 sq. m.		
	Bearing	GH2, L	CR	III	B	-		
	Bearing	GH1, R	CR	III	B	-		
	Bearing	Pier 3 Coping	SER	III	B	0.10 sq. m.		
	Bearing	Pier 3 Coping	CR	III	B	-		

Note 1. Based on X-Y-Z rating method.
2. Based on Diagnosis of Damages as discussed in Section 4.10 of the Manual.

(3) Structural Soundness (LOAD RATING)

The demand forces under different loading conditions was determined from a structural analytical model using the as-built conditions and data gathered from verified shapes and dimensions.

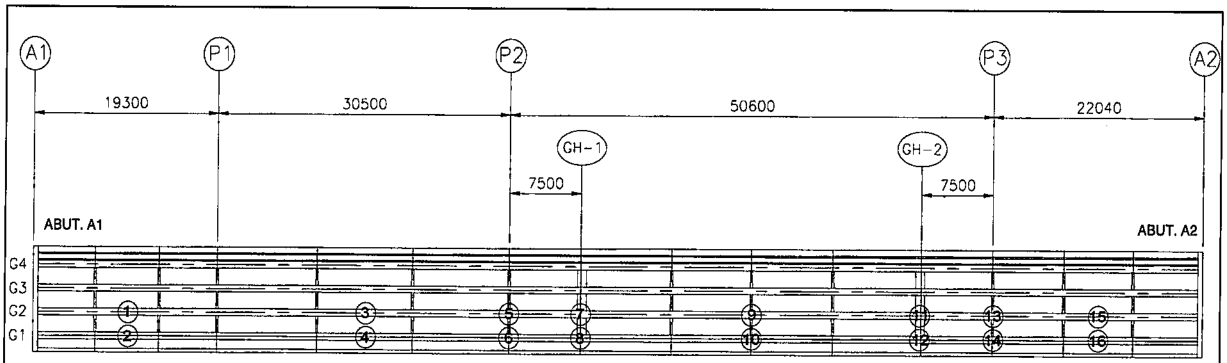
The flow of load rating and evaluation of bridge soundness was based in **Section 7.4, Chapter 7.** Prestressed concrete members were rated both in inventory and operating level by the established strength requirements of AASHTO Design Specifications.

At inventory level, the rating considered the allowable stress at service load. At operating level, the stresses was limited to 0.90 of yield point stress in the prestressing steel nearest the extreme tension fiber of the member.

The bridge superstructure was modeled using 3D beam elements and an analysis of load effects was performed.

Results of load rating analysis for superstructure are tabulated in **Table 24.1.4-1**.

Table 24.1.4-1 Minimum Rating Factor by Location



LOCATION		SERVICEABILITY LIMIT STATE		STRENGTH LIMIT STATE			
		ALLOWABLE STRESS		INVENTORY LEVEL		OPERATING LEVEL	
Member	Number	RF	EQ. TRUCK (tons)	RF	EQ. TRUCK (tons)	RF	EQ. TRUCK (tons)
G1	②	2.41	78.81	2.76	90.25	4.61	150.75
G1	④	3.10	101.37	3.81	124.59	6.37	208.30
G1	⑥	**0.00**	**0.00**	1.42	46.43	2.38	77.83
G1	⑧	-	-	**0.83**	**27.14**	1.39	45.45
G1	⑩	4.80	159.96	2.04	156.96	3.40	111.18
G1	⑫	-	-	**0.83**	**27.14**	1.39	45.45
G1	⑭	**0.00**	**0.00**	1.83	59.84	3.05	99.74
G1	⑯	5.21	170.37	5.71	186.72	9.54	311.96
G2	①	2.89	94.50	3.31	108.24	5.52	180.50
G2	③	3.98	130.15	4.97	162.52	8.30	271.41
G2	⑤	**0.38**	**12.43**	2.49	81.42	4.16	136.03
G2	⑦	-	-	1.01	33.02	1.68	54.94
G2	⑨	2.00	65.40	2.82	92.21	4.71	154.02
G2	⑪	-	-	1.01	33.02	1.68	54.94
G2	⑬	**0.38**	**12.43**	2.63	86.00	4.38	143.23
G2	⑮	5.98	195.55	6.72	219.74	11.21	366.57

Analysis Results

From the load rating analysis conducted, the Rating Factor at top of Pier No. 2 (Node 5 & 6) and Pier No. 3 (Node 13 & 14) yielded a value less than 1.0. This indicates that the section has no longer any capacity under live load at service level. The section, during dead load and live load condition is under tension and is already in its strength limit state level.

The gerber hinge supports which yields a Rating Factor of 0.83 at Strength Limit Level , also signifies that the section is at its critical stage. Load rating calculations are presented in **Appendix 24.1.4-3 (1/30 to 30/30)**.

(4) Vulnerability to Disaster

(a) Earthquake

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

Bridge Site

Vargas Bridge (Upstream) is located 1.0km from the Marikina Valley Fault System (MVFS). As a rule, bridge structures less than 5.0km distance are considered highly vulnerable. Therefore, Vargas Bridge is 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 2-Column Bent Pier with 300 mm diaphragm in between columns. Girder supports were pin connected over the piers.
- Shear keys were constructed during retrofitting of the bridge in 1997.
- Vertical cable restrainers at abutments and Pier were installed in 1997 as hold-down device.

Structural Configuration

The regular configuration of Vargas Bridge (Upstream) is structurally favorable.

Date of Construction

The Vargas Bridge (Upstream) was constructed in 1992. The seismic code used in the design was AASHTO's seismic provisions using ATC recommendations. Thus, retrofitting works was recommended later in 1997 using the widely accepted code of AASHTO Seismic Design Division 1-A.

Analysis Results

The results of analysis conducted in calculating the structural soundness of substructures based on the latest code yielded satisfactory results.

The capacity demand - ratio of substructure is tabulated in **Table 24.1.4-2**. Calculation of the capacity-demand ratio is presented in **Appendix 24.1.4-4**.

Table 24.1.4-2 Capacity-Demand Ratio of Substructures of Vargas Bridge

Member	Abutment		Piers	
	A	B	P1	P2
Foundation	1.07	1.03	6.18	6.18
Column/Pier Wall	N/A	N/A	1.68	1.68
Abut. Main wall	2.48	2.12	N/A	N/A

(b) Wind

The National Structural Code of the Philippines (NSCP 2001) recommends a design basic wind speed of 200 kph. AASHTO recommends only 160 kph, the maximum cyclone center wind velocity of 225 kph passing Metro Manila where the bridge (upstream) is located was recorded in 1995 with a gust velocity reaching to 255 kph. This indicates that the bridge (upstream) has been exposed to more than 200 kph basic design wind speed specified in the Philippine Code. Therefore, the Vargas Bridge (upstream) 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. Furthermore, Vargas bridge has more than sufficient freeboard under maximum flood level.

(d) Special Issues

Vessel Collision

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

The actual vertical navigation clearance of the bridge is 5.7m, while the regulated vertical clearance set by the Philippine Cost Guard in 3.75m.

The horizontal navigational space between piers of the bridge is 40.4 m, while the regulated space is 43.0 m.

Therefore, pier protection is needed for the bridge.

Utilities

The existing utility lines of the bridge attached at the side is 8-100mm ϕ Telecommunication Lines.

Informal Settlers

There are informal settlers at both abutments that will be affected when improvement works will be implemented.

24.1.5 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 24.2.4.**

(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 (35.60m) of the Vargas Bridge,
- To simulate the existing condition of the gerber hinge part and the girder part at top of pier, specifically the occurrence and pattern of cracks under service load, and
- To estimate the live load capacity of the gerber hinge support with the proposed rehabilitation measure.

(2) Methodology

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

Four (4) finite element models of a typical gerber hinge supporting the middle post-tensioned girder spans (35.60m) 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 24.1.5-1**.

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 zones 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 to 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 22.1.5-2, Item 22.1.5, Chapter 22.**

Table 24.1.5-1 Description of Finite Element Models

Model		Description	Remarks
1	E	• To simulate existing condition : Effective post-tensioning force, Peff, is 5000kN for the sum of all tendons specified in the construction drawings. : The amount of reinforcements shown in the construction drawings is used (See **Figure 24.1.5-1**).	• For checking the design, fpu = 1860 Mpa
2	Et	• To simulate existing condition : Same as Model E except that the effective post-tensioning force is reduced, Peff = 4500kN for all tendons. : The amount of reinforcements shown in the construction drawings is used (See **Figure 24.1.5-1**).	• 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 : Same as Model E except that spacing of longitudinal reinforcement at the pier support is wider in the upper half of the girder.	• 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 24.2.4.**

(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 24.1.5-2** shows the loading stages for this simulation.

Table 24.1.5-2 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 23.2kN loading increment

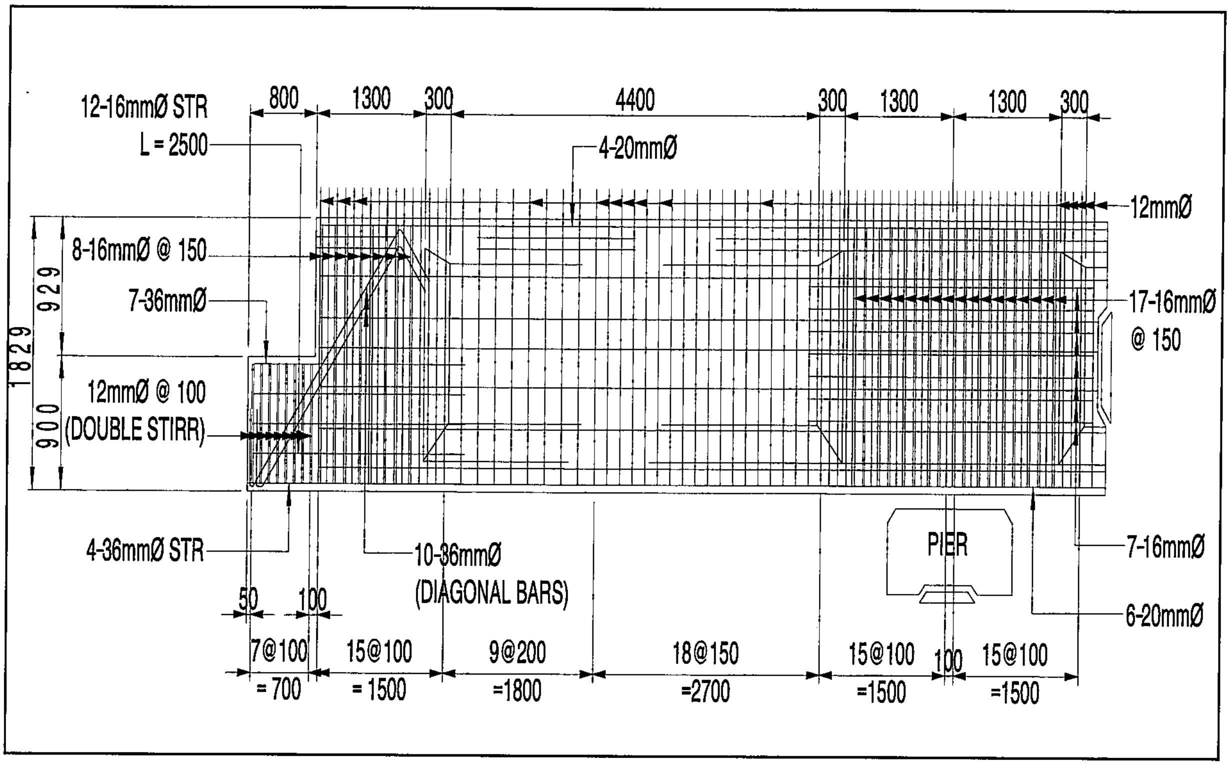


Figure 24.1.5-1 FEM Model and Construction Drawing of Gerber Hinge Part (Cantilever Portion)

(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 24.1.5-3** shows the loading stages for this analysis.

Table 24.1.5-3 Loading Stages for Simulation of Rehabilitated Condition

Action	No. of Loading Stages
Self-weight	1
Effective Post-tensioning Force (Model Et)	3
Dead Load	5
Effective Post-tensioning Force of Slanted Cables	5
Effective Post-tensioning force of additional longitudinal external tendons	5
Live Load	To be loaded until failure at 23.2kN loading increment

(3) Analysis Results

The observed cracks in gerber hinge parts and the girder at top of Pier are shown in **Figure 24.1.5-2; Figure 24.1.5-3** shows patterns and locations of cracks in each model for comparison. From these figures, the following can be deduced:

- 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 reduced amount of reinforcement as compared to the results of Model Et and Es.
- The rehabilitation measure recommended in **Section 24.2.4** is shown to be effective.

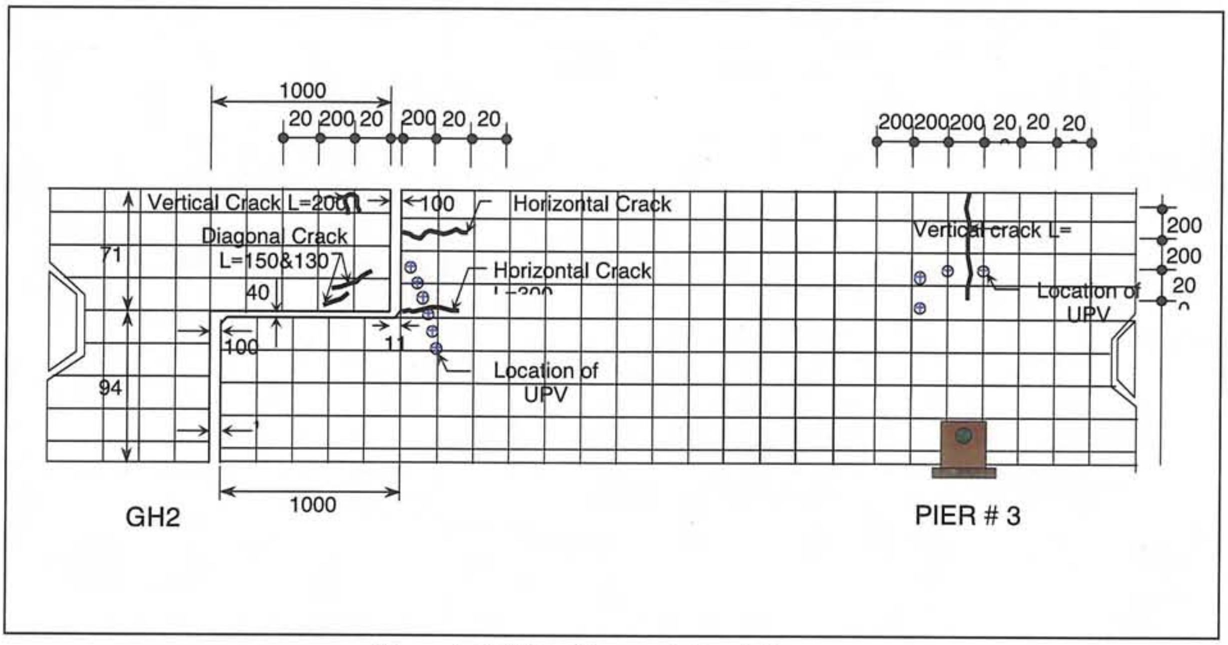


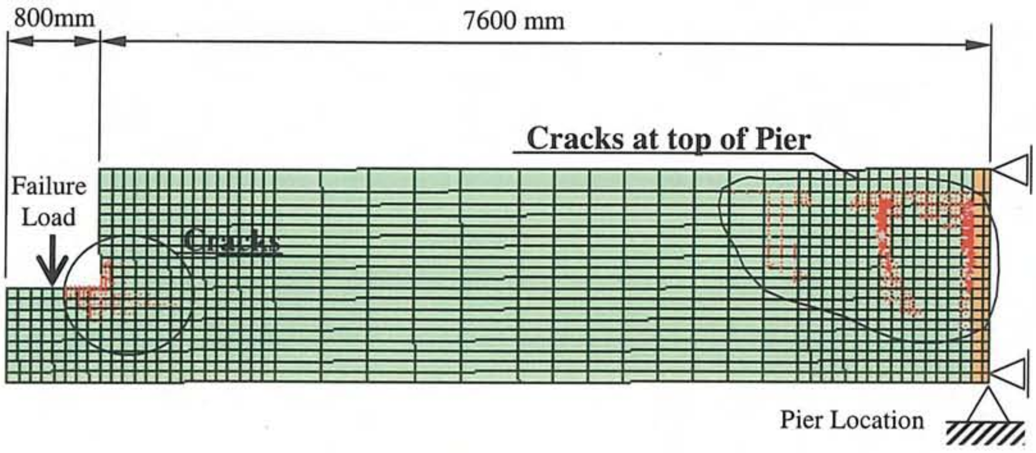
Figure 24.1.5-2 Observed Crack Pattern

Figure 24.1.5-4 indicates the crack pattern prior to failure of Model Et and load-deflection relationship at the loaded point.

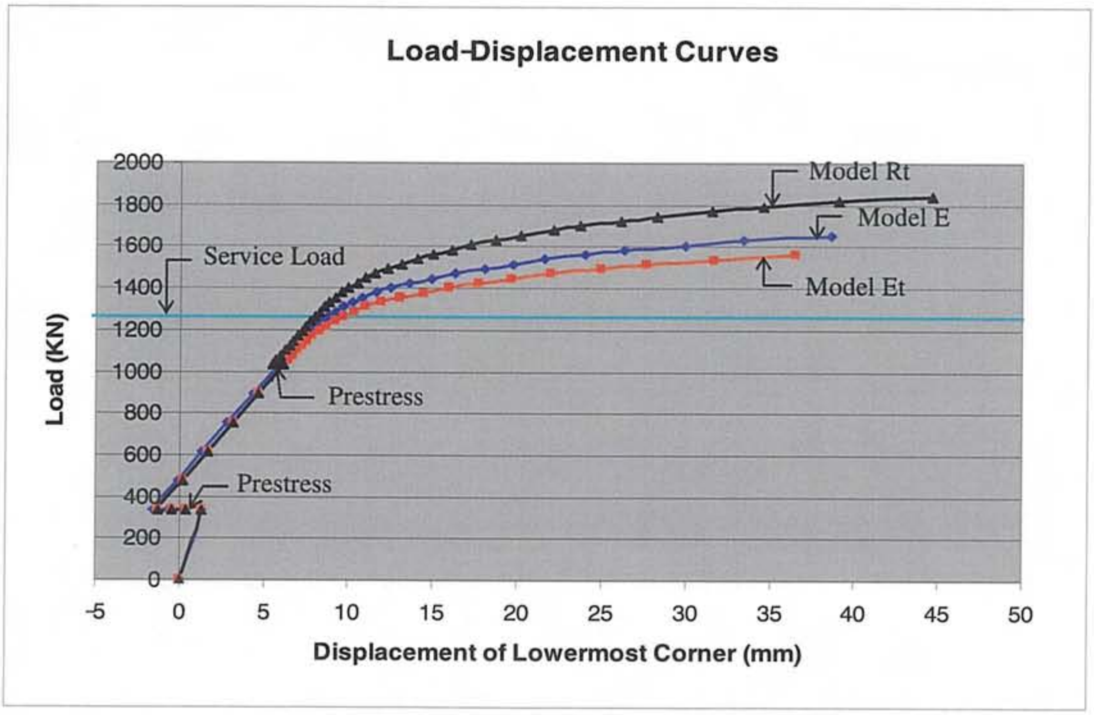
From the results, the following were observed:

- In case of Model Et which simulates well the existing crack condition among the analysis models, integrated behavior is under the yield zone condition 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 under the elastic zone condition.
- Even at load level prior to failure, any crack occurrences at girder on pier top were not identified.

Figure 24.1.5-3 Crack Patterns and Locations at Service Load in Each Model



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



(b) Load – Deflection Relationship

Figure 24.1.5-4 Crack Pattern Prior to Failure and Load-Deflection Relationship at Reaction Force Loaded Point.

24.1.6 Overall Assessment of Bridge Condition

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

(1) Superstructure

Major Damage Description and Causes

- Cracks in gerber hinge parts observed were caused by insufficient hanger reinforcement provided on the girder and unforeseen losses of prestressing forces.

- Flexural cracks of girder on top of pier 2 and pier 3 were caused by tension stresses on top fiber at service loads.

- Large deflections at the cantilever portion were due to insufficient number of longitudinal tendons.

(2) Substructure

- There are no damage observed in substructures

The following results of analysis conducted on substructure are:

- As shown in **Table 24.1.4-2** the existing pier is sound to carry the design load based on the latest code C/D_{PL} = 1.68 (Pier 1 & 2).

- The stability of foundation is enough to carry the load required by the latest code with 6.18 Capacity/Demand Ratio (C/D Ratio)

(3) Social Environment

- The Bridge has no historical importance as per NHI report.
- Dislocation of people will be unavoidable during improvement works.

(4) Conclusion

From the series of surveys and analysis made, the following are the major findings in the existing condition of Vargas Bridge:

- Cracks were evident on gerber hinge parts and an immediate counter-measure is necessary to stop its propagation

- Major cracks on girder at top of piers 1 and 2 were also evident. Immediate counter-measure is necessary.

- Large rotation/deformation of the suspended spans at the cantilever portion were observed to affect the rideability.

- Cracks, honeycomb and spalling of concrete on deck slab were also observed.

The Overall Assessment of existing Bridge is tabulated in **Table 24.1.6-1.**

Table 24.1.6-1 Overall Assessment for Vargas Bridge (Upstream) Condition

Items			Member / Location	Damage Rating	Damage Condition	Diagnosis (IW of FI)
Structural Soundness	Superstructure	Shape / Dimension	Span 3 Exterior Main Girder, GH2,R, Girder 1 of Span 3	II	Spalling with exposed rebar A = $0.45m^2$	IW
			Exterior Main Girder, Girder 1, GH1 at Span 3	II	Spalling with exposed rebar A = $0.24m^2$	IW
			Span 4 Exterior Main Girder, Girder 1 of Span 4	III	Spalling at bottom of flange equal to 0.15sq.m. Rebars are not exposed.	FI
		Material / Damage	Span 2 Exterior Main Girder, Girder 1 at Pier 2	II	One crack only with thickness equal to 2mm. depth of crack, d = 17mm. Condition of rebars are severe. Concrete coverage of 40~70mm	IW
			Span 3 Exterior Main Girder, GH1, L, Girder 1 of Span 3	II	Crack width equal to 1~3mm with spacing of less than 50cm. d=44mm. Condition of rebars are severe. Concrete coverage of 40~70mm	IW
			4 Test Locations for UPV	II	Crack depth of 17mm to 217	IW
			G1 / GH2, L	I	Crack width 2mm, and spacing less than 50cm. d=43mm. Concrete cover of 40~70mm	IW
			G1 / GH2, R	I	Crack width 1mm, and spacing less than 50cm. Concrete cover of 40-70mm	IW
			G1, P3	I	Crack width 2mm, spacing less than 50cm. d=217mm. Concrete cover of 40-70mm	IW
		Inventory Level	Gerber hinge exterior girder (0.83)		Equivalent Truck 27.0 tons	IW
		Assessment of Superstructure	**Measure for cracks at Gerber Hinge parts is necessary.**			
	Substructure	Shape / Dimension	Pier1, Pier Coping	II	SER with an area of 0.12sq.m.Condition of rebars are severe. Concrete coverage of 40~70mm	FI
			Pier 3, Top of Pier Coping	II	Spalling with exposed rebars in top portion of pier coping. Area of SER equal to 0.10sq.m.	FI
			Suspended Girder	II	Deformation at the center of suspended girder and cantilever span	FI
		Material / Damage	Abutment A1, Backwall	II	Crack of Backwall of Abutment A1, Crack width = 1mm and spacing of less than 50cm. Condition of rebars are severe. Concrete coverage of 40~70mm	IW
			Abutment A1, Bearing Anchorage	III	Light rust on steel. No section loss due to corrosion	FI
			Pier 3, Pier Coping	II	Wide cracks near bottom of coping, crack width = 5mm and spacing less than 50cm.	IW
			Phenolphthalein Test	Ok	Depth of carbonation 0~6mm	-
			Chloride Content	Ok	Non-Detected	-
			Petrographic Analysis	Not Ok	Alakali-silica reaction were observed	FI
		Strength of Pier Body	Pier P1	Ok	Strength of pier body sufficient	-
			Pier P2	Ok	Strength of pier body sufficient	-
		Assessment of Substructure	**Piers are sufficient as per latest code requirements**			
	Foundation	Structure / Shape	Pier P1 Tubular Steel Pile	-	Underwater survey was not undertaken	FI
			Pier P2 Tubular Steel Pile	-	Underwater survey was not undertaking	FI
		Scouring	Under Water	Ok	Scouring not occurred	-
		Bearing Capacity / Stability	Pier P1 Pier	Ok	Bearing capacity of pile foundation sufficient	-
			Pier P2 Pier	Ok	Bearing capacity of pile foundation sufficient	-
		Assessment of Foundation	Sufficient as per latest code requirements			
	Assessment of Structural Soundness		**Measure for cracks at gerber hinge and at top of pier are necessary**			
Traffic Function	Vehicle Weight Limitation Use		27 tons (0.83 load rating) at inventory level			
	LOS		E (0.85)			
	Geometrical Features		Fair including approach road			
	Safety of Vessel Transport		Collision Protection for Pier Column is needed			
	Assessment of Traffic Function		Traffic functionality reduced by decrease in live load capacity			
Social Environment	Utilities Hanged at the Bridge		8 – ø 100mm Telecommunication line			
	Squatters		Heavy. More than 26 families live under the Vargas Bridge			
	Historical Aspects		Not included in NHI preservation list			
	Assessment of Social Aspects		Moderate social and environmental impact			
Overall Assessment			**Measure against cracks of PC Girder is necessary. Pier protection are needed.**			

24.2 COMPARATIVE STUDY ON REHABILITATION METHOD

24.2.1 Proposition of Rehabilitation Method

Three (3) schemes were prepared 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

This scheme involves repair and sealing of concrete cracks, honeycomb and spalling as major works to improve local condition of the bridge.

(2) Medium-Scale Rehabilitation

Major works included in this scheme are the following:

- Repair and sealing of concrete cracks, honeycomb and spalling.
- Installation of CFRP (Carbon Fiber Reinforced Polymer) longitudinally at top of girder and horizontally at web over pier support; and horizontally at Gerber hinge.
- Partial replacement of deck slab over pier support.

(3) Large-Scale Rehabilitation

This scheme covers the following major works in improving the condition of the bridge:

- Repair and sealing of concrete cracks, honeycomb and spalling.
- Partial replacement of deck slab over pier support.
- Rehabilitation of gerber hinge portion with slanted P/S cables.
- Reconstruction of diaphragm and slab at gerber hinge.
- Installation of external cables on each side of the girder to counter the deflection.

24.2.2 Evaluation of Rehabilitation Method

Each scheme were evaluated by corresponding rating given on structural aspect, constructability, traffic and navigational functionality, its impact to traffic during construction and social and environmental impact. Points accumulated were tallied for each scheme and evaluated based on the highest total points.

From the results of each three schemes prepared and compared in **Table 24.2.2-1** the **large-scale rehabilitation** was recommended as the best scheme in terms of engineering aspects.

24.2.3 Lifecycle Cost Analysis of the Bridge

(1) Procedure

Based on the bridge condition survey mentioned in **Chapter 24.1** and engineering study made in **Section 24.2.1**, the life cycle cost (LCC) analysis of the Vargas 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) Vargas Bridge Deterioration Situation

The standard deterioration curve of deck, superstructure and substructure between condition rating and age can be considered the same equation mentioned in **Section 14.3**. According to the bridge condition survey, however, deck and superstructure of Vargas Bridge have been deteriorated more than three (3) times than the standard deterioration due to construction of superstructure being not appropriate.

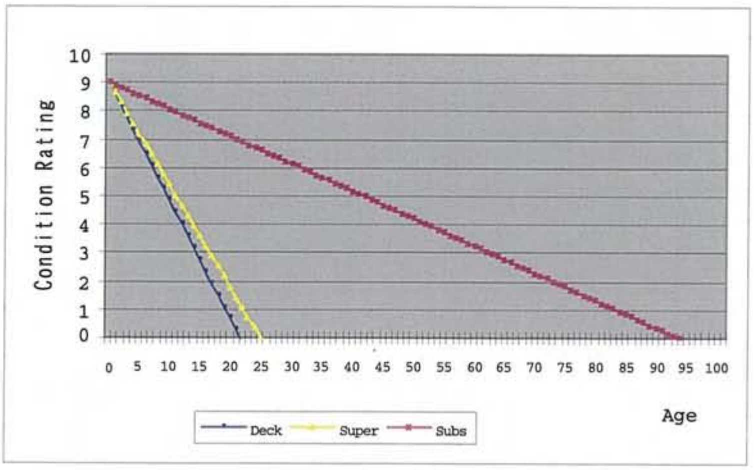


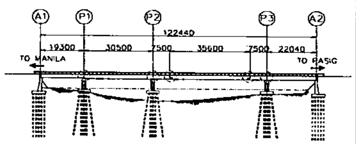
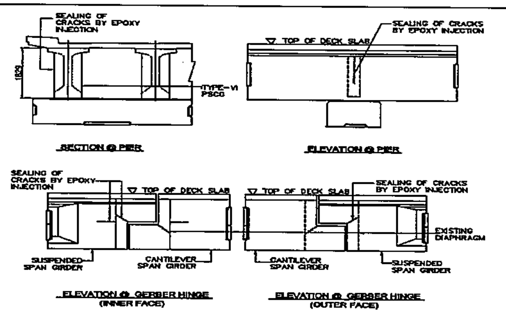
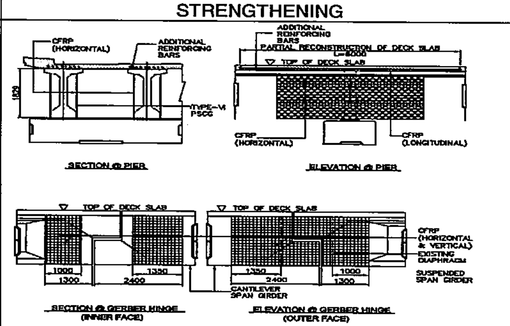
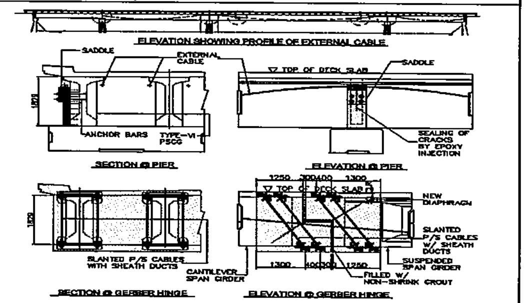
Figure 24.2.3-1 Deterioration Curve of Vargas Bridge

The deterioration curve of the Vargas Bridge is estimated and shown in **Figure 24.2.3-1**.

(3) Rehabilitation Schemes and Cost Estimates

The engineering study proposed the rehabilitation schemes and cost estimates which are shown in **Table 24.2.2-1**.

Table 24.2.2-1 Comparative Study of Possible Schemes for Vargas Bridge

IMPROVEMENT TYPE		PROPOSED SCHEMES: SCHEME 1 : SMALL SCALE REHABILITATION CRACK SEALING	RATING	SCHEME 2 : MEDIUM SCALE REHABILITATION CARBON FIBER & REINFORCING BAR STRENGTHENING	RATING	SCHEME 3 : LARGE SCALE REHABILITATION SLANTED P/S CABLE & LONGITUDINAL EXTERNAL P/S CABLE STRENGTHENING	RATING
FIGURES  LOCATION OF SIGNIFICANT STRUCTURE DAMAGE							
MAJOR WORKS		• **Repair and sealing** of concrete cracks, honeycomb and spalling.		• Repair and sealing of concrete cracks, honeycomb and spalling. • **Installation of CFRP** (Carbon Fiber Reinforced Polymer) longitudinally at **top of girder** and horizontally at **web over pier support**; and vertically & horizontally at **Gerber hinge.** • **Partial replacement of deck slab** over pier support		• Repair and sealing of concrete cracks, honeycomb and spalling. • - • Partial replacement of deck slab over pier support • **Rehabilitation of Gerber hinge portion with slanted P/S cables.** • **Reconstruction of diaphragm and slab at Gerber hinge.** • **Installations of external cables** on each side of the girder for the cantilever span and drop span.	
(1) STRUCTURAL ASPECT (30%)	STABILITY	• Countermeasures taken is not permanent because shear capacity at Gerber hinge portion is insufficient. • Rectification of deflection is not addressed on this scheme. • No strengthening done for the girders.	E	• Longitudinally & horizontally installed CFRP; and additional reinforcing bar will improve flexural capacity of girder over pier support. • Vertically and horizontally installed CFRP will increase the shear capacity of the Gerber hinge portion.	B	• Slanted P/S cables at the Gerber hinge portion will deter shear failure and diagonal tension. • External P/S cables increases girder capacity and controls deflection.	A
	DURABILITY	• Least durable among schemes especially at the Gerber hinge portion. • Requires periodic inspection of pier supports and Gerber hinge connections.		• Long term effect of the bond between CFRP and concrete needs periodic inspection to check for debonding. • Durability on the whole structure is improved by strengthening girders over pier support and Gerber hinge portion using CFRP.		• The slanted P/S cables will effectively improve the performance of the Gerber hinge portion. • External P/S cable at the face of the girders will strengthen the girders and corrects deflection.	
(2) CONSTRUCTION ASPECT (30%)	COST	• PhP 9.7 Million • Least Expensive	A	• PhP 18.50 Million	B	• PhP 24.30 Million • Cost is the most expensive.	B
	DURATION	• Shortest at two (2) months.		• Six (6) months.		• Longest at ten (10) months.	
	METHOD / DIFFICULTY	• Easiest to execute among the schemes.		• Installation of CFRP at pier support and Gerber hinge portion will be quick. • Additional reinforcing bar on top of girder over pier support will entail. • Requires partial removal and reconstruction of deck slab & diaphragm.		• 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 and more complicated than scheme 2. • Longer construction period is anticipated due to setting out and stressing of external P/S cables for all cantilever and drop span girders.	
(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	• Temporary closure not necessary during repair works.	A	• Partial closure of affected traffic lane is necessary during repair work. • Temporary full closure is necessary during concreting works.	C	• 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 works.	C
	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 and reconstruction of existing diaphragm and deck slab.	
	DETOUR	• Rerouting of traffic is not necessary.		• Rerouting of traffic is necessary during removal and reconstruction of existing deck slab.		• Rerouting of traffic is necessary during removal and reconstruction of existing diaphragm and deck slab; and stressing operation.	
(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	4.05	1 (RECOMMENDED)	4.35

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

(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 discussed in **Chapter 14.3.4**.

(5) Extended Service Life by Improvement Proposals

Using the deterioration curve in **Figure 24.2.3-1** and the relationship between investment cost and improvement condition rating shown in **Figure 14.3.5-1** of **Section 14.3.5,** the expected extended service life of Vargas Bridge is calculated and shown in **Figure 24.2.3-2** and **Figure 24.2.3-3**. The service life of the bridge is varied to extend by type of rehabilitation. If large scale rehabilitation is implemented, the service life of the bridge is expected to extend 46 years so total service life will be 50 years from 2007.

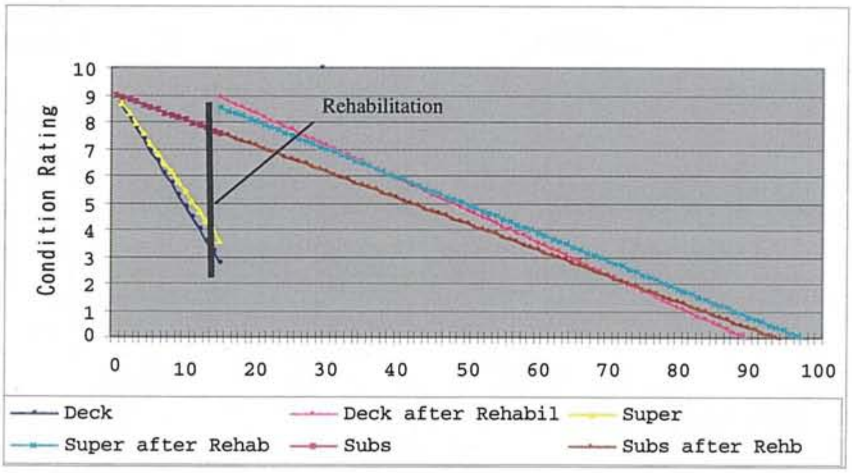


Figure 24.2.3-2 Estimated Service Life by Rehabilitation Types of Vargas Bridge under Large Scale Rehabilitation

(6) Calculation of the Lifecycle Cost of the Vargas Bridge and Recommendation

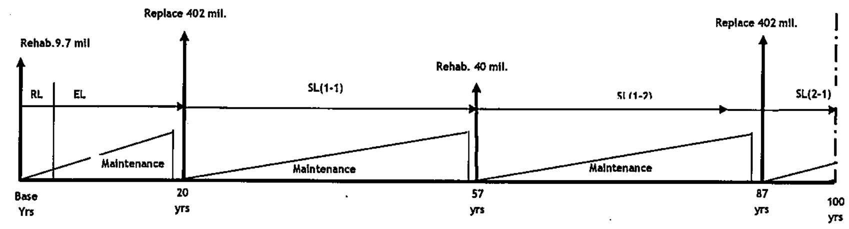
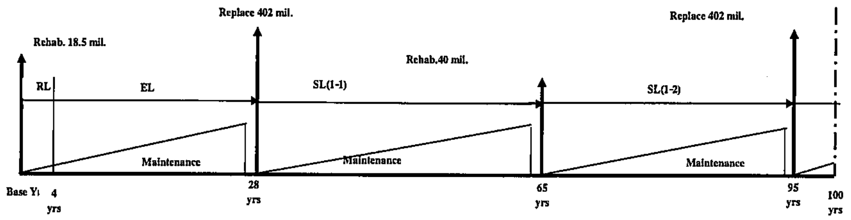
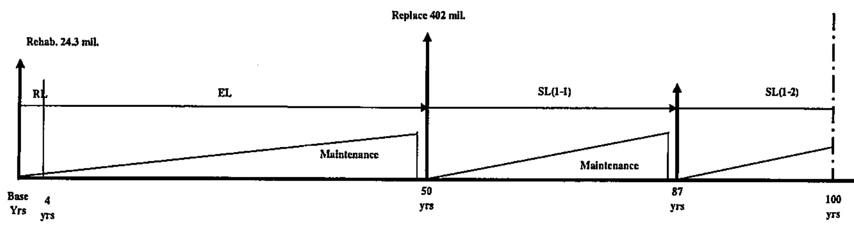
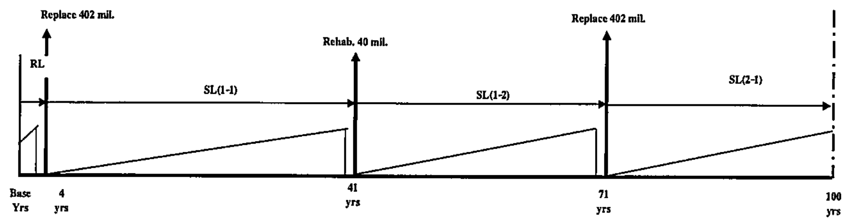
The lifecycle cost of Vargas Bridge is calculated and shown in **Table 24.2.3-1** and judging from the LCC of alternative rehabilitation works, the large scale rehabilitation scheme is recommended to be employed as the type of rehabilitation work.

Table 24.2.3-1 Lifecycle Cost Estimates of Vargas Bridge by Rehabilitation Types

Unit : Million Pesos

Scheme	LCC at Discount Rate of 15%	Recommended Improvement from LCC Analysis
Small Scale Rehabilitation	49.0 (0.21)	3
Medium Scale Rehabilitation	84.4 (0.16)	2
Large Scale Rehabilitation	31.9 (0.14)	1
Replacement	245.6 (1.00)	

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 4 EL 16 SL(1) 67 SL(2) 13 Total 100	49.0 million (0.21)
Medium Scale Rehabilitation		Year RL 4 EL 24 SL(1) 67 SL(2) 5 Total 100	84.4 million (0.34)
Large Scall Rehabilitation		Year RL 4 EL 46 SL(1) 50 Total 100	31.9 million (0.13)
Replacement		Year RL 4 SL(1) 67 SL(2) 29 Total 100	245.6 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

Figure 24.2.3-3 Life Cycle Analysis of Vargas Bridge

24.2.4 Recommendation

Based on the life-cycle cost analysis conducted on the possible rehabilitation schemes, the most recommended scheme is the large scale rehabilitation scheme.