CHAPTER 21

FEASIBILITY STUDY OF QUEZON BRIDGE REHABILITATION PLAN

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21.1 DETAILED BRIDGE SURVEY AND ASSESSMENT

This section is discussed inline with the flow chart shown in **Figure 13.1-1** of **Chapter 13** of this report. 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".

21.1.1 Review of Design and Repair Works

(1) Review of Design

(a) Outline of the Quezon Bridge

Photo 21.1.1-1 Panoramic View of Quezon Bridge

- Structure Type : Single Span Steel Arch Through Type Truss Bridge (2 Hinge Arch)
  Abutment foundation type is timber piles.
- Bridge Length : 102.40m (Abutment to Abutment)
- Date of Construction : 1946

(b) General Notes for Repair/Rehabilitation of Quezon Bridge in 1996

- 1989 A.A.S.H.T.O. Standard Specification for Highway and Bridges.
- Dead Load : 1.05 KPa for future wearing surface
- Live Load : M-18 (H-20-44)

Concrete	:	Class "A" concrete with a compressive strength of fc'=20.7MPa.
Structural Steel	:	AASHTO M-183 (ASTM A-36) with minimum yield strength of 248 MPa.
Reinforcing Steel	:	ASTM A-416 with a minimum yield strength fy = 275.80 MPa.

Welding : AWS Standard E-70 Electrodes.

Painting : All structural steel shall be cleaned thoroughly by sand blasting or appropriate means. All other field paints shall conform to Standard Specifications for Highway and Bridges revised 1988.

Scheme for the Repair of Bottom Chord

- Clean surface of existing bottom chord. Lateral truss members and gusset plates connections by sand blasting or other appropriate.
- Twisted and corroded parts of the bottom chord shall be replaced with plates of the same or bigger size.
- During repair, truss joints along the bottom chord shall be supported by false work.
- The bottom chord of lateral truss shall be repaired/replaced one side at a time and fully supported throughout its whole length in the duration of the repair.
- No heating of structural member shall be done during strengthening and re-alignment.
- In welding of plate together especially in horizontal position, it is required that all welding be closed to prevent rainwater or dirt to enter the space between and corrode the steel.
- Tightening of high tensile bolts should be done by the use of pneumatic equipments that can measure torque needed.

(1) Previous Repair Works

Outline of repair works of Quezon Bridge in 1996 is as follows:

(a) Superstructure

Deck Slab

- Rehabilitation for deck slab includes replacement of deck with new concrete and reinforcing bars and installing additional shear connectors. (shown in **Figure 21.1.1-1**)

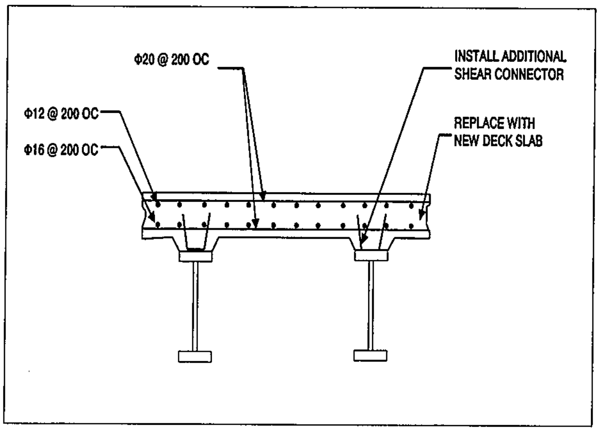


Figure 21.1.1-1 Section detail of Repair Works for Deckslab

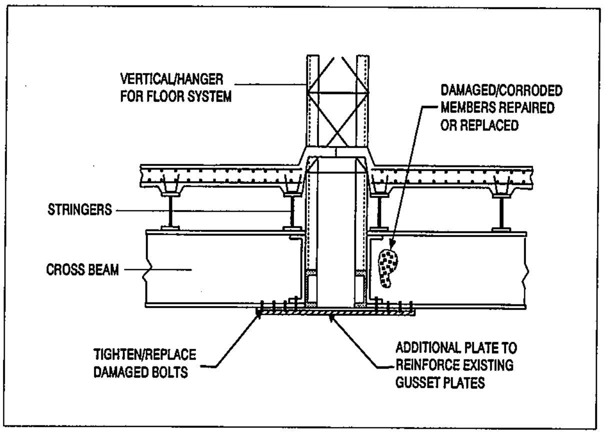


Figure 21.1.1-2 Section detail of Repair Works for Joints and Connection

Joints and Connections

- Damaged or corroded gusset plates are either repaired, reinforced or replaced by additional plates as shown in **Figure 21.1.1-2.**

- Rivets either remain in place or are replaced by bolts. When rivets are not removed, holes are provided to fit the new gusset plates as shown in **Figure 21.1.1-2.**

Truss Members

- Truss members especially lower part of vertical hangers and cross beams that are either damaged or corroded are repaired by steel patching or replacement.

- Corroded stringer sections are repaired with steel plate patching.

Additional Stringers

- Additional set of stringers are placed at the downstream side of the span next to the abutments.

Painting Works

- Painting works are carried out to all structural steel members.

Pylons

- Concrete spalling and cracks on the pylons are repaired by patching to cover exposed reinforcing bars on all pylons.

(c) Substructure

- Concrete cracks on abutments are sealed and repaired.

- Cracks on the South side abutment extend to almost the entire width while cracks on the North side abutment are observed to be minimal.

(3) Historical Background

The National Historical Institute interposes no objection to rehabilitate Quezon Bridge as long as the basic configuration is retained to preserve its historical authenticity.

21.1.2 Natural Condition Survey

(1) Topographic Survey

(a) Control Monuments

Two (2) GPS Stations were established as control points for Quezon Bridge as shown in **Table 21.1.2-1.**

Table 21.1.2-1 GPS Stationing and Coordinates

Station	Northing	Easting	Elevation
GPS-Q1	1614113.962	497780.651	12.348
GPS-Q2	1614194.083	497727.41	12.411

(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.

Table 21.1.2-2 shows the scope of works of topographic survey. Topographic plan is shown in **Appendix 21.1.2-1.**

Table 21.1.2-2 Scope of Work of Topographic Survey

Description	Original Scope	Actual Work
Control Point Survey (GPS)	1	2
Profile Survey	103m Bridge Section + 200m Each of Both approach Roads (200x2) Total = 503m.	106.45m Bridge Section + 187.34m + 233.85m at each approach roads. Total = 527.64m.
Road Cross-Section Survey	Bridge Section (103m) : 10m interval Approach Road (400m) : 20m Interval Width : Bridge '(21m) + 50m each on both sides = 121m. Total : 33 Sections	Bridge Sections (103m) : 11 sections Approach roads (421.19m) : 20 sections Width : 121m Total : 31 Sections
Topographic Survey	503m (Length) x 121m (Width) = 60863 sq.m.	527.64m (Length) x 121m (width) =63844 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 borehole was drilled at the bridge site. (See **Appendix 21.1.2-2**). It was drilled down to a final depth of 38.0 meters from the existing ground line. The borehole is characterized by an upper formation of granular soils consisting of poorly graded gravel, silty gravelly sand, silty sand, silty to clayey sand and poorly graded sand in the upper 11.0 meters, with N-values varying from 4 to 27. Underlying the sand layers is a thick cohesive formation extending down to 31.0 meter depth. The upper portion extending to 26.0m depth consists of clayey silt, elastic silt and sandy clay with corresponding N-values from 2 to 16. The lower portion is a highly consolidated and older deposit of elastic silt from 26.0 meters down to 31.0 meter depth. It is very stiff to hard with N-values varying from 28 to 46. Following the upper cohesive are very dense silty sand layer and very hard sandy clayey silt and elastic silt down to about 34.0 meters, with N-values in the refusal range (N>50). The bottom layer is made up of moderately to highly weathered siltstone formation extending to the bottom of the borehole at 38.0 meters.

21.1.3 Bridge Condition Survey and Identification of Damages

(1) Measurement of Shapes and Dimension

(a) Objective

The main purpose of this activity is 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 elements or members of the bridge.

(c) Coverage Areas

For inspection purposes, the bridge was divided into five (5) general inspection areas: Road/Deck Level (see **Photo 21.1.3-1**), Abutments (see **Photo 21.1.3-2**), Pylons (see **Photo 12.1.3-3**), Trusses (see **Photo 21.1.3-4**) and Deck Frame (see **Photo 21.1.3-5**).

Photo 21.1.3-1 Road Deck

Photo 21.1.3-2 Abutment

Photo 21.1.3-3 Pylons

Photo 21.1.3-4 Truss Members

Photo 21.1.3-5 Deck Frame

(d) Equipment and Procedure

To facilitate verification of truss members above the road deck level, scaffolds were installed at strategic locations on the sidewalks (see **Photo 21.1.3-4**). A tugboat with mounted scaffolds and working platforms on top was used in the verification of deck frame members. (See **Photo 21.1.3-6**).

Each inspection team was equipped with safety gear (including hard hats, safety belts, safety shoes, and rain coat), measurement tools (8.0m tape measure, caliper) for verification, hammer for damage inspection, verification forms and pencils for documentation. (See **Photo 21.1.3-7** and **Photo 21.1.3-8**).

Communication between the climber and the inspector was facilitated by two-way radios. In taking digital photos, a white board with scale and codes for location and direction was used. (See Photo **21.1.3-9**). The whiteboard was provided with magnets so it can be easily attached to steel members.

Photo 21.3-6 Tugboat with mounted Scaffolds and Working Platforms

Photo 21.3-7 Measurement of Top Chord Members

Photo 21.3-8 Measurement of Deck Gusset

Photo 21.3-9 Use of Whiteboard

(e) Results

Table 21.1.3-1 lists of drawings that summarize the data presented in the verification forms. Dimensions that were shown in the drawings were utilized in structural model for improvement plan and presumption of original design.

Table 21.1.3-1 Lists of Drawings

Sheet No.	Title	Appendix
1	General Elevation & Plan	21.1.3-1
2	Sections at Grid Line 9 and Grid Line 13	21.1.3-2
3	Partial Elevation from Grid Line 1 to Grid Line 9	21.1.3-3
4	Photos of Span, Approaches, and Deck Frame	21.1.3-4
1	Section near Grid Line 1, Section of South Pylons	21.1.3-5
2	Section near Grid Line 17, Section of North Pylons	21.1.3-6
3	Sections at South Abutment and Pylons	21.1.3-7
4	Sections at North Abutment and Pylons	21.1.3-8
1	Details of Truss A and Truss C – Top Chord and Bottom Chord members	21.1.3-9
2	Details of Truss A and Truss C – Diagonal Members and Vertical Members	21.1.3-10
3	Details of Truss A and Truss C – Hangers and Ties	21.1.3-11
4	Details of Truss B – Top Chord and Bottom Chord Members	21.1.3-12
5	Details of Truss B – Diagonal Members and Vertical Members	21.1.3-13
6	Details of Truss B – Hangers and Ties	21.1.13-14
7	Details of Sway Braces at Grid Line TC	21.1.13-15
8	Details of Sway Braces at Grid Line BC	21.1.3.-16
9	Details of Vertical Sway Braces	21.1.3-17
1	Details of Deck Frame Members	21.1.3-18
2	Details of Deck Frame Members – Additional Stringers	21.1.3-19

(2) Close-Up Visual Inspection

(a) Objective

To determine the damages on the bridge and to be able to make detailed documentation including digital still photos, close-up visual inspections was conducted.

(b) Inspection Teams

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

(c) Coverage Areas

The inspection covers the road/deck level, abutments, pylons, trusses, deck frame, accessories such as traffic barriers, sidewalk railings, and drainage facilities.

(d) Equipment and Procedure

The equipment and procedure used follows Item No. 3, **Section 13.3.2** of **Chapter 13.**

(e) Criteria for Damage Rating

The criteria used for damage ratings follows the criteria set in Item No. 5, **Section 13.3.2** of **Chapter 13.**

(f) Results

- Cracks were observed on the sidewalk and median concrete slabs.
- Cracks on the face of the North abutment.
- Corrosion damage was found on the members of truss A, B & C mostly on ties and hangers.
- Corrosion damage was found on deck frame system such as gusset plates, braces, ends of stringers, additional stringer at abutment and stringer supporting the sidewalk.

Photo 21.1.3-10 to **Photo 21.1.3-15** show typical damages by the structural components.

Table 21.1.3-2 lists drawings that summarize the data presented in the close-up visual inspection forms. The detail drawings are shown in **Appendix 21.1.3-20** to **Appendix 21.1.3-41.**

Photo 21.1.3-10 Vertical Hanger, Damage-Corrosion 10 ▯ Section Loss < 30%

Photo 21.1.3-11 Gusset Plates, Damage – Corrosion 10 ▯ Section Loss < 30%

Photo 21.1.3-12 Deck Frame End Braces, Damage – Corrosion Section Loss ▯ 30%

Photo 21.1.3-13 Longitudinal Tie Beam, Damage – Corrosion 10 ▯ Section Loss < 30%

Photo 21.1.3-14 Arch Bottom Chord, Damage – Corrosion 10 ▯ Section Loss < 30%

Photo 21.1.3-15 Arch Top Chord Damage – Corrosion Section Loss < 30%

Table 21.1.3-2 Damage Rating of Main Members by Close-Up Visual Inspection

Component	Item	Member / Location	Damage Rating *[1]	Description
	Material Damage	Truss A, B & C Bottom Chord	II	Corroded
	Material Damage	Truss A & C Top Chord	II	Corroded
	Material Damage	Truss A, B & C Vertical Members	II	Corroded
	Material Damage	Truss A, B & C Hangers	II	Corroded
	Material Damage	Truss A, B & C Tie Beam	II	Corroded
	Material Damage	Gusset Plates	I	Heavily Corroded
	Material Damage	Ends of Braces	I	Heavily Corroded
	Material Damage	End of Stringers (Abutments)	I	Heavily Corroded
	Material Damage	Additional stringers at abutments	I	Heavily Corroded
	Material Damage	Stringers supporting the sidewalks	I	Heavily Corroded
	Shapes and Dimension	North/South Abutment	II	Ok
	Material Damage	North Abutment	II	Cracks

NOTES: 1. Damage Rating Level is based on the XYZ Damage Rating Method.
2. I – Determined by engineering judgment by Team Leader through consultation with governing organization.

(3) Non-Destructive Test of Materials

(a) Objective

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

(b) Results

Table 21.1.3-3 shows the results of the test.

Table 21.1.3-3 Results of Non-Destructive Test

Test	Results	Reference Appendices
Brinell Hardness Test	Measured Brinell Hardness Numbers ranged from 97 to 160, corresponding to equivalent tensile strengths of 338 MPa to 550 MPa	21.1.3-42 21.1.3-43
Ultrasonic Flaw Detection Test (UFD)	The results of UFD showed no significant defect in any of the locations tested.	21.1.3-44 21.1.3-45
Dye Penetrant Test (DPT)	The results of the DPT showed no significant defect in any of the locations tested.	21.1.3-46 21.1.3-47
Compression Test	Results of the compression test ranged from 17 MPa to 29 MPa.	21.1.3-48
Phenolphthalein Test	Measured depth of carbonation on the core samples ranged from nil (0 mm) to 12mm.	21.1.3-48

(4) Special Test

(a) Microtremor

Objective

The purpose of this activity was to identify and confirm the modes relevant to the deformation due to dead load and governing live load cases (MS 18 lane loadings) considered in structural model.

Figure 21.1.3-1 shows the locations and orientations of the reference and roving sensors for each set up.

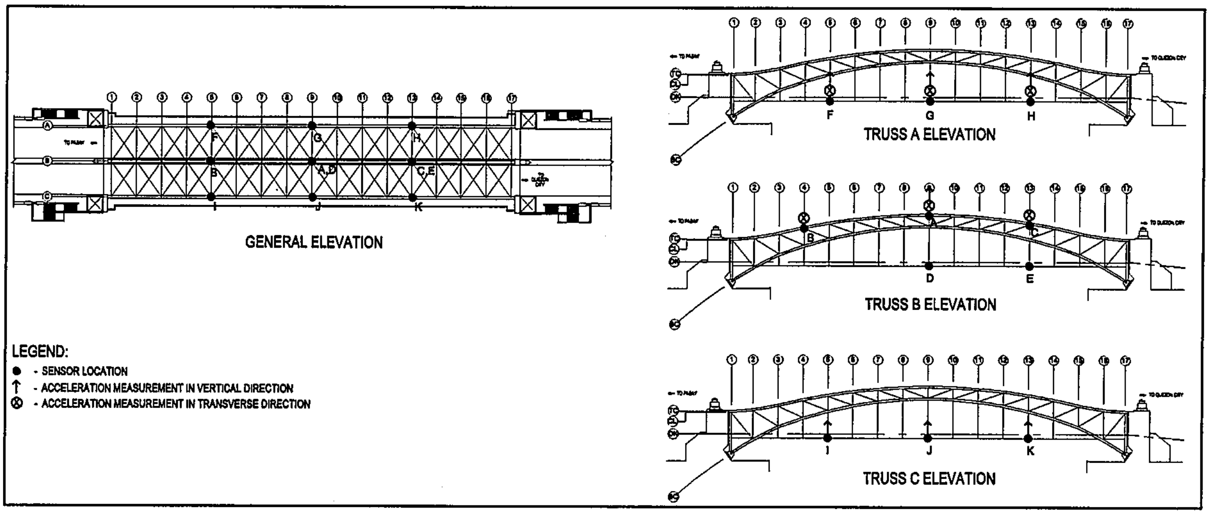


Figure 21.1.3-1 Location of Acceleration Sensors

Acceleration Sensors

Five (5) uniaxial force balance accelerometers (Kinemetrics Model ES-U) were used for the microtremor survey. All sensors were set to a full-scale range of 0.25g. Three (3) were set to capture vertical accelerations and two (2) were set to measure horizontal accelerations in the transverse direction. The sensors were mounted on designated locations using beeswax and connected to a data acquisition system with a 16-bit analog-to-digital converter using control cables.

Two (2) of the five sensors were allocated as reference sensors and the remaining three (3), as roving sensors. The reference sensors were fixed at their location (location A) throughout the survey.

One reference sensor was oriented in the vertical direction, and the other, in the transverse direction. Two roving sensors were oriented in the vertical direction, and one in the transverse direction.

Most Probable Natural Frequencies

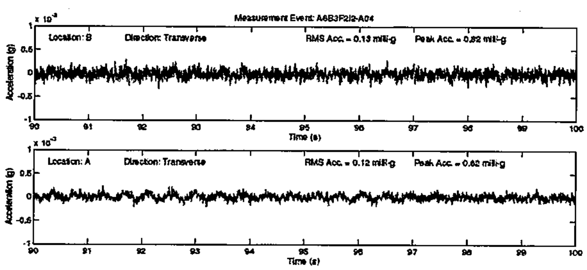
The most probable natural frequency of the 1st Vertical Mode is 2.20hz and that of the 1st Torsional Mode is 3.10hz as shown in **Table 21.1.3-5**.

Vibration measurement data were processed using routines and algorithms in the MATLAB environment. Processing consists of plotting the recorded vibration acceleration against time (time history), computing the root-mean-square (RMS) and peak values of the 4-minute history as indicators of the level of vibration, and plotting the amplitude spectrum (acceleration vs. frequency).

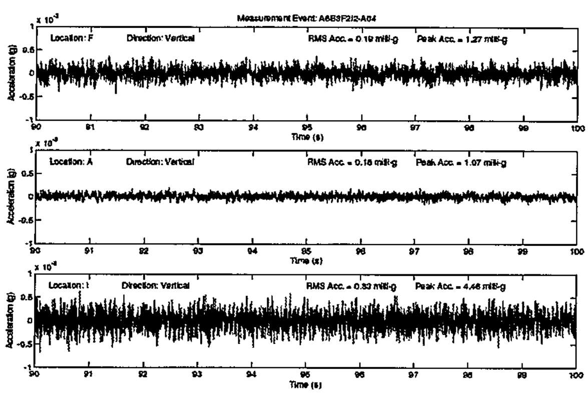
Sample plots of 10-second time histories of one of the measurement events are shown in **Figures 21.1.3-2.** Sample Amplitude Spectra are shown in **Figure 21.1.3-3**.

Table 21.1.3-5 Most Probable Natural Frequency

MODE SHAPE	Microtremor Test (Hz)	Structural Analysis (Hz)
1st Vertical Mode	2.20	2.21
1st Torsional Mode	3.10	3.12

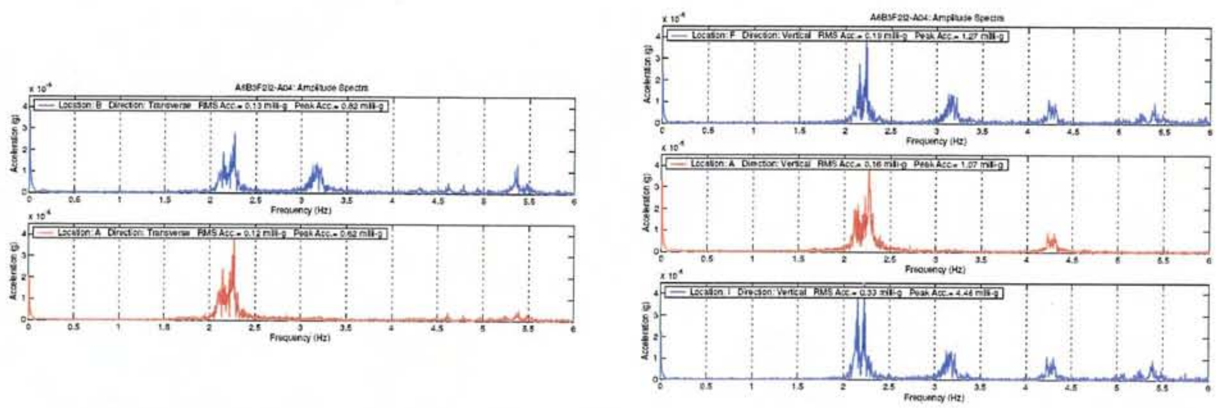


A. Horizontal Accelerations in Transverse Direction



B. Vertical Accelerations

Figure 21.1.3-2 Sample 10-Second Time History Plots



A. Horizontal Accelerations in Transverse Direction B. Vertical Accelerations

Figure 21.1.3-3 Sample Amplitude Spectra

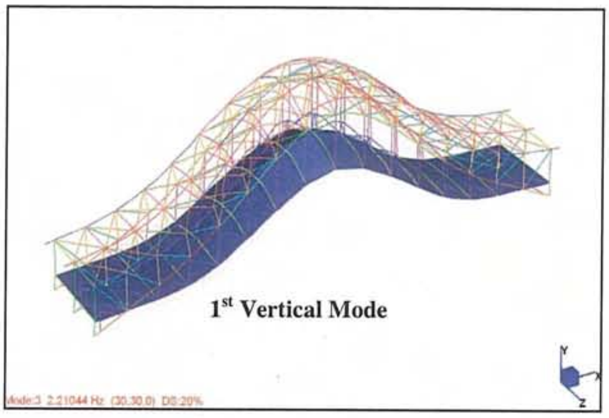
From the spectra of each measurement event, the resonant peaks (at least 5 micro-g of the spectral amplitude) were identified and their associated frequencies were noted. Where such peaks occurred clearly and consistently from one set-up, which consists of two measurement events, to the other set-up, a natural frequency was deemed indicated. Following this procedure, it was inferred that among the most probable natural frequencies of the bridge were 2.2 Hz, 2.3 Hz, 3.1 Hz, 4.3 Hz, and 5.4 Hz.

From the plots of amplitude spectra, natural frequencies around 2.2 Hz and 2.3 Hz can be associated with vertical modes (see **Figure 12.1.3-4**) with side sway of the trusses. The amplitudes of vertical vibration around these frequencies were slightly different in the three (3) trusses. Correspondingly, the transverse acceleration records on locations A, B, and C on top chord of Truss B at these frequencies have significant spectral amplitudes.

Meanwhile, the spectral peaks at around 4.3 Hz have almost the same amplitudes that can be associated with the vertical mode of the bridge.

The peaks at around 3.1 Hz and 5.4 Hz, on the other hand, can be associated with torsional modes (see **Figure 12.1.3-4**), taking into account that the amplitudes of vertical vibration are significant and almost the same at quarter-span locations on the outer trusses (F and H on Truss A, and I on Truss C), while the amplitude at locations in Truss B (D and E) is considerably less. At midspan locations on the outer trusses (G and J), spectral amplitudes were considerably less.

The transverse acceleration records on locations B and C (quarter-span locations on top chord of Truss B), which give significant spectral amplitudes as compared to location A (midspan), confirm that the modes associated around 3.1 Hz and 5.4 Hz are torsional.



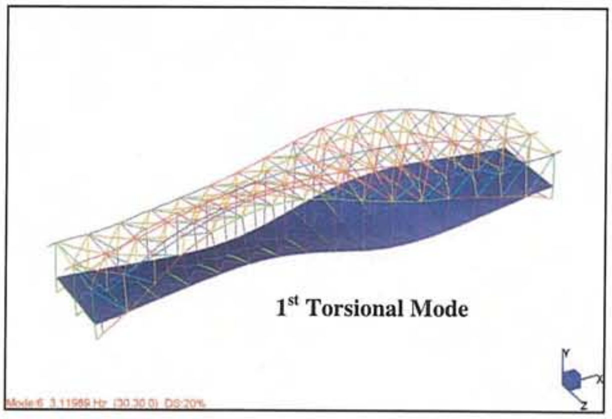


Figure 21.1.3-4 First Vertical and Torsional Modes

Transverse acceleration records at road deck level (locations F and H), which gave considerably less spectral amplitudes, provided adequate confirmation that the estimated mode shapes for these frequencies were torsional and not to be associated with lateral mode of the bridge.

(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 shown in **Section 13.3.7, Chapter 13**.

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

This section follows the procedures and criteria set forth in **Section 13.3.7, Chapter 13**.

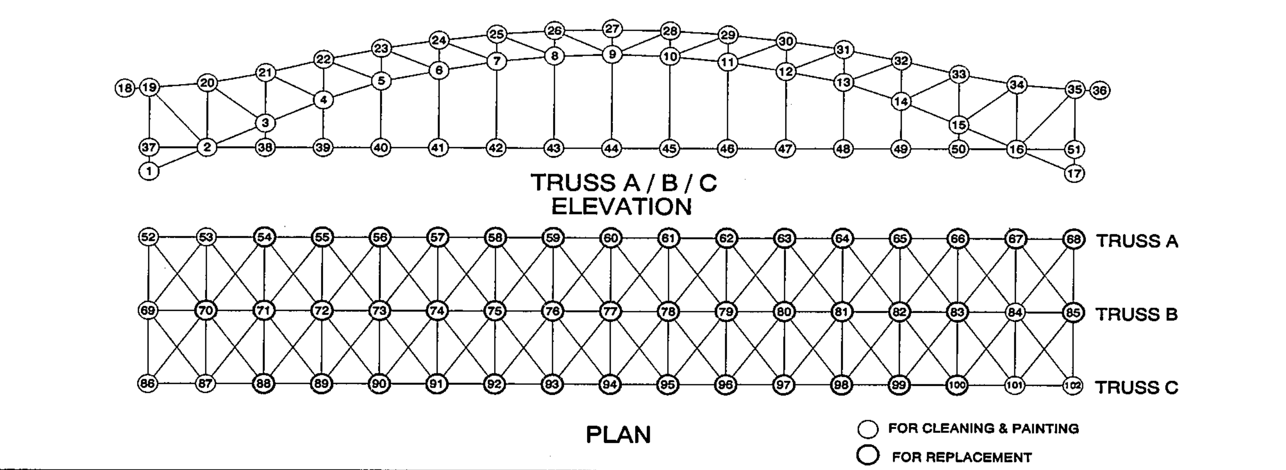
(c) Categorization of Damage Rating

The damage rating was categorized as discussed in **Chapter 6**.

(d) Evaluation of Damages

Evaluation results on damages of main members were summarized in **Table 21.1.3-6**.

Table 21.1.3-6 Evaluation of Major Damages on Quezon Bridge



	Location		Evaluation 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	Truss A - Bottom Chord	10-11	CO	II	B	20%	• Ultrasonic Flaw Detection Test – No Defects • Brinell Hardness Test – Tensile Strength 338 – 550 MPa • Dye Penetrant Test – No Defects	• Microtremor Test - OK
	Truss A - Top Chord	28-29	CO	II	B	20%		
	Truss A – Vertical Member	1-37	CO	II	B	20%		
	Truss A - Hangers	38-3, 39-4, 42-7, 46-11, 47-12, 43-8	CO	II	B	20%		
	Truss A - Hangers	48-13, 49-14, 50-15	CO	II	B	20%		
	Truss A - Tie Beam	55-67	CO	II	B	20%		
	Truss B - Bottom Chord	1-2, 16-17	CO	II	B	20%		
	Truss B - Bottom Chord	4-5, 12-14, 8-12	CO	II	B	20%		
	Truss B - Vertical Member	1-37	CO	II	B	20%		
	Truss B - Hangers	39-4, 40-5, 41-6, 42-7, 43-8, 44-9	CO	II	B	20%		
	Truss B - Hangers	45-10, 46-11, 47-12, 48-13, 49-14	CO	II	B	20%		
	Truss B - Tie Beams	70-85	CO	II	B	20%		
	Truss C - Bottom Chord	3-4, 8-9, 12-13	CO	II	B	20%		
	Truss C - Top Chord	33-34	CO	II	B	20%		
	Truss C - Vertical Member	1-37, 39-4, 43-8	CO	II	B	20%		
	Truss C - Vertical Member	46-11, 48-13, 49-14	CO	II	B	20%		
	Truss C - Tie Beams	87-100	CO	II	B	20%		
	Gusset Plates	54-68, 70-83, 85, 88-100	CO	I	A	40%		
	End of Braces	54-57, 59-61, 63-68, 86-89, 91-92, 96-102	CO	I	A	40%		
	End of Stringers	70-74, 76-77, 80-83	CO	I	A	40%		
Substructure	North Abutment	-	CR	II	B	-	• Compression Test – OK, fc' Range (17-29 MPa)	

21.1.4 Presumption of Original Design and Load Rating

(1) Objective

The purpose of the presumption of original design is to prepare the structural shapes, dimensions and properties of the analysis model for Load Rating. The following policies below were ensued:

- Inspection results were used for the determination of shapes and dimensions of visible portions such as superstructure, exposed parts of substructures.
- Material properties of members were basically determined from the non-destructive test. For the members that the non-destructive test was not carried out, the properties were estimated based on the relevant materials and as indicated in the as-built drawings.
- The foundation type and its embedment depth was determined from as-built drawings and geotechnical survey results.
- The number of piles was calculated based on the original design code.

(2) Structural Shapes and Dimensions

Superstructure

Most structural data of the superstructure does not need to be assumed because almost all the dimension and details were measured. **Appendices 21.1.3-1** to **21.1.3-19** shows the assumed shapes, dimensions, and details of the superstructure for the structural analysis.

Section properties of each member are presented in **Appendix 21.1.4-1.**

Substructure

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

The type of foundation was found from the as-built drawing. The availability of materials and construction was known, thus the pile capacity of each pile were estimated. **Appendices 21.1.3-5 to 21.1.3-8** shows the assumed shapes, dimensions and details of substructures.

(3) Structural Soundness (LOAD RATING)

Objective

The purpose of this analysis is to evaluate quantitatively the load carrying capacity of prime members of superstructure. With the bridge condition survey results and "Presumption of

Original Design", structural data were evaluated and integrated in order to build a complete and realistic structural frame model of the superstructure.

Procedure

See **Section 13.4.3** for the procedure and formula used in evaluating the capacity of members using the allowable stress method.

Structural member damages, corrosions and missing members were evaluated and taken into account in the model to truly reflect them on the actual status of the bridge. The resulting bridge structural model were then checked and verified with the Microtremor Test whether or not its model represented the actual motion.

Analysis Results

The calculated numeric data of Rating Factor for each main member was given in **Appendix 21.1.4-2.**

The results on minimum Rating Factor by the location were shown in **Table 21.1.4-1.**

Table 21.1.4-1 Minimum Rating Factor by the Location

LOCATION	MEMBER	INVENTORY LEVEL		OPERATING LEVEL	
		Rating Factor (RF)	Equivalent Truck (t)	Rating Factor	Equivalent Truck (t)
Truss A	Bottom Chord	3.3	106	3.3	106
	Top Chord	4.7	151	4.6	148
	Vertical Member	12.1	386	8.8	281
	Tie Beam	15.8	506	13.9	443
Truss B	Bottom Chord	1.1	36	2.4	76
	Top Chord	2.8	89	4.0	129
	Vertical Member	4.5	143	6.5	207
	Tie Beam	4.8	153	6.6	210
Truss C	Bottom Chord	3.3	105	3.3	106
	Top Chord	4.9	158	4.8	152
	Vertical Member	12.1	386	8.8	281
	Tie Beam	11.2	357	9.1	291
Deck Frame	Gusset Plate	**0.92**	**30**	1.59	51

The analysis yielded the following counter measures to be undertaken.

- Since RF is greater than 1, no need to provide load limits for passing vehicles. However, improvement works shall be required on corroded members.

- The Planning of the improvement works shall be implemented so as to meet the new design code requirements with consideration of the present traffic conditions.

(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

Quezon Bridge is located 13.5km from the Marikina Valley Fault System (MVFS). As a rule, bridge structures less than 5km distance are considered highly vulnerable. The 13km distance of Quezon Bridge makes it moderately 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 a single-column pier. This type of support could easily attain collapse mechanism due to lack of support redundancy.

Structural Configuration

The regular configuration of Quezon Bridge is structurally favorable.

Date of Construction

Quezon Bridge was restored / built in 1946. Before and during those times AASHTO have no recommendation with regards to seismic design. Therefore, analysis should be made using the latest code requirements.

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 was presented by capacity-demand ratio under earthquake.

The existing dimensions and structural data of the substructure was determined from actual field surveys, as-built plans and "Presumption of Original Design" using old code. **Appendix 21.1.4-3** shows the probable number of timber piles for substructures.

Table 21.1.4-2 shows the C/D ratio of substructures under the old code and the latest code.

Table 21.1.4-2 Summary of Capacity/Demand Ratio of Quezon Bridge

Substructure	Pile Foundation	
	Old Code	Latest Code
North Abutment	1.0 Safe	0.70 Fail
South Abutment	1.00 Safe	0.70 Fail

(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 Quezon Bridge is located was recorded in 1995 with a gust velocity reaching to 255 kph. This indicates that Quezon Bridge has been exposed to more than 200 kph basic design wind speed specified in the Philippine Code. Therefore Quezon Bridge is not vulnerable to wind action.

(c) Flood

Quezon bridge has no pier located in the river and the abutments built behind the river banks. Moreover, the deck elevation has more than sufficient freeboard to clear the maximum flood level. Under this condition, Quezon bridge has very low vulnerability to flood action.

(d) Special Issues

Vessel Collision

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

Vessel Collision with Girder

Although the regulated vertical clearance by PCG is 3.75m, the actual vertical clearance of Quezon Bridge is 6.75 m. more than that required by PCG. The present vertical clearance is adequate for vessels navigating under the Quezon Bridge.

Utilities

There are no existing Utility Lines for Quezon Bridge.

Informal Settler

There are informal settlers living around the south abutment.

21.1.5 Overall Assessment of Existing Bridge Condition

The present state of Quezon Bridge condition is based on the following informations:

(1) Superstructure

Major Damage Description and Causes:

- Floor system mostly on gusset plates, bracing and ties are heavily corroded due to water seepage through opening between vertical hangers and deck slab.
- A total of 44 joints at floor system are required for improvement works. There are 113 pieces of lateral braces to be cut and replaced, and at least 45 pieces of gusset plates are needed to be replaced due to heavy corrosion.
- A total of 8 joints at floor system are needed to be cleaned and painted due to corrosion.
- Removal and reconstruction of deck slab near abutment to replace corroded stringer near abutment and to provide new expansion joints.

(2) Substructure

- Minor cracks on the face of the abutment, however the stability of the piles are not enough to carry the load of the latest design code with 0.7 of capacity / demand ratios (C/D ratio)
- The lack of stability is caused due to the change of design code, especially the design the requirement.

(3) Social Environment

- As mentioned **Section 13.1.3**, the National Historical Institute declared that Quezon Bridge is one of the historical bridge, and strongly endorse the policy that the configuration of Historical Structures should be preserved for prosterity.
- Present navigational clearance will be maintained.

(4) Conclusion

Improvement work of the floor system especially on gusset plates, end braces and additional stringers in the abutment should be done. This will maintain and improve the durability of the existing bridge.

As for the substructure, a retrofit measure is recommended to conform with the requirement of the latest code.

Table 21.1.5-1 summarized the overall assessment of existing bridge.

Table 21.1.5-1 Overall Assessment for Quezon Bridge Condition

/Item			Member/ Location	Rating	Damage Condition	Diagnosis (IW or FI)
Structural Soundness	Substructure	Material/Damage	Truss A,B,C – Bottom Chord Members	II	Corroded Section Loss =20%	FI
			Truss A C – Top Chord Members	II	Corroded Section Loss =20%	FI
			Truss A,B,C – Vertical Members	II	Corroded Section Loss =20%	FI
			Truss A,B - Hangers	II	Corroded Section Loss =20%	FI
			Truss A,B,C – Longitudinal Tie Beam	II	Corroded Section Loss =40%	IW
			Gusset Plates	I	Corroded Section Loss = 40%	IW
			Ends of Braces	I	Corroded Section Loss = 40%	IW
			Ends of Stringers (abutments)	I	Corroded / ruptured Section Loss =40%	IW
			Additional stringers at abutment	II	Corroded Section Loss=30%	IW
			Stringers supporting the sidewalks	I	Corroded Section Loss =40%	IW
			20 test location for UFD	Ok	No Defects	-
			20 test location for Brinell Hardness	Ok	338MPa-550MPa	-
			20 test location for DPT	Ok	No Defects	-
		Operating Level	Truss A & C	Ok	Equiv. Truck=32.7 t	FI
			Truss B	OK	Equiv. Truck=32.7 t	FI
		Assessment of Superstructure	**Rating Factor (RF) = 0.92 for floor system at gusset plate, limit weight = 30 tons.**			
	Substructure	Shape/Dimension	North/South Abutment	Ok	-	FI
		Material/Damage	Abutments & Pylons	III	Crack	FI
			7 test locations for Coring & Comp.	Ok	17MPa to 29MPa	-
			8 phenolphthalein tests	Ok	0-12mm carbonation depth	-
		Stability	North Abutment	Ok	-	FI
			South Abutment	Ok	-	FI
		Assessment of Substructure	**Minor damage on existing substructure but design can not comply with the latest code.**			
	Foundation	Structure/Shape	North Abutment/Ø250 –Timber Piles	Ok	Underwater survey was not undertaken	FI
			South Abutment/Ø250 -Timber Piles	Ok	Underwater survey was not undertaken	FI
		Bearing Capacity/Stability	North/South Abutment/compression piles	Not Ok	C/D ratio = 0.70	FI
		Assessment of Foundation	**Existing Pile Foundation does not comply with the latest AASHTO code requirements.**			
	Assessment of Structural Soundness		**Existing Bridge structure needs strengthening to meet the present required strength for a bridge**			
Traffic Function	Vehicle Weight Limitation Use		30 tons			
	LOS		D (v/c = 1.22)			
	Geometrical Features		Fair			
	Safety of Vessel Transport		Existing vertical clearance is sufficient for a regulatory clearance.			
	Assessment of Traffic Function		**Traffic functionality reduced by the decrease in live load capacity.**			
Social Environment	Utilities Attached to the Bridge		3-ϕ340mm G.I. pipe water lines			
	Squatters		Squatters living around the south abutment			
	Historical Aspects		National Historical Institute lists Quezon Bridge as a historical structure.			
	Assessment of Social Aspects		**Historical aspects shall be considered**			
Overall Assessment			**Improvement works is necessary for superstructure. Substructure foundation strengthening is recommended to meet latest code requirements.**			

21.2 COMPARATIVE STUDY ON REHABILITATION METHOD

21.2.1 Proposition of Rehabilitation Method

With the aid of criteria set forth in **Chapter 13**, 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:

- Cleaning and painting corroded sections of floor system only
- Repair of expansion joint to seal water leakage
- Cleaning and painting of corroded gusset plates and replacement of heavily corroded gusset plates

This scheme provides only basic structural repair and maintenance to restore its intended function. This is the least durable among schemes and needs periodical painting works.

(2) Medium-Scale Rehabilitation

In addition to cleaning and painting of corroded section of floor system major works for this scheme are as follows:

- Cleaning and painting floor system including corroded vertical hanger and truss members.
- Replace expansion joint to seal water leakage
- Replacement of corroded section of floor beam, longitudinal tie beam and vertical members by cutting affected areas and placing new sections
- Sealing gap between vertical hanger and sidewalk concrete and provide plates to prevent further corrosion of vertical hanger
- Removal and reconstruction of deck slab near abutment
- Replacement of corroded stringers

This scheme will improve present condition and durability than the previous scheme. Construction cost is much higher in this scheme.

(3) Large-Scale Rehabilitation

The following additional/improvement works are considered for this scheme in addition to the medium-scale rehabilitation works:

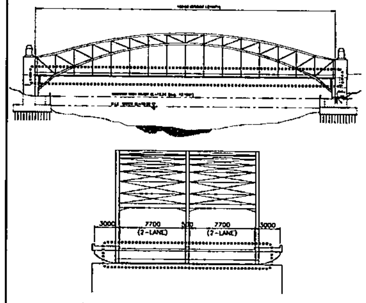
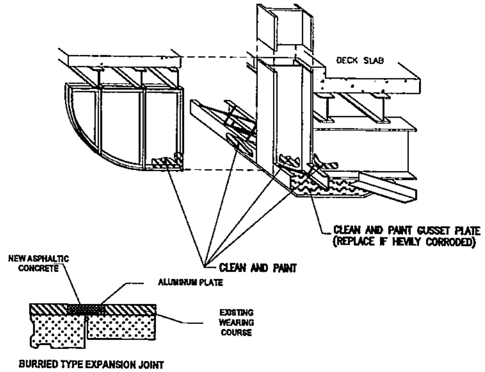
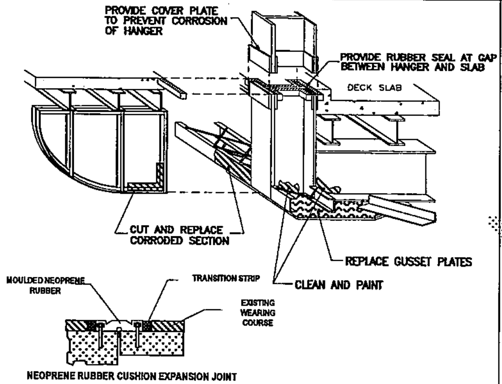
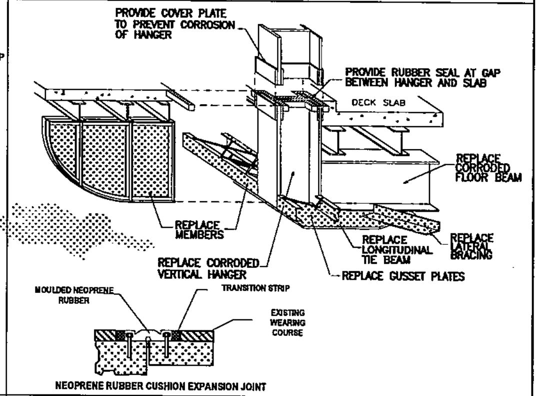
- Cleaning and painting the steel structures for the whole bridge
- Replacement of gusset plates, lateral bracing and longitudinal tie beam
- Replacement of corroded section of floor beam and vertical member by cutting affected areas and replacing with new sections.

Capacity and durability will be improved to a greater degree by replacement of gusset plates, bracing, tie beams and other corroded sections. The cost in this scheme is the most expensive among the alternatives.

21.2.2 Evaluation of Rehabilitation Method

From the three schemes formulated above, the recommended as best scheme in terms of engineering aspects is Scheme 2, Medium-Scale Rehabilitation. Structural capacity and durability of the bridge will be improved considerably by placing heavily corroded members and providing corrosion protection. Construction duration for this scheme is 18 months and cost is economical for the present state of the bridge. For the detailed description of possible rehabilitation schemes, see **Table 21.2.2-1.**

Table 21.2.2-1 Comparative Study of Possible Scheme for Quezon Bridge

IMPROVEMENT TYPE		PROPOSED SCHEMES: SCHEME 1 : SMALL SCALE REHABILITATION — REPAIR OF CORRODED SECTIONS	RATING	SCHEME 2 : MEDIUM SCALE REHABILITATION — REHABILITATION OF CORRODED SECTIONS	RATING	SCHEME 3 : LARGE SCALE REHABILITATION — REHABILITATION OF FLOOR SYSTEM	RATING
FIGURES  LOCATION OF SIGNIFICANT STRUCTURE DAMAGE							
SCHEME DESCRIPTION		• Cleaning and Painting Corroded Sections of Floor System Only • Repair of Expansion Joint to Seal Water Leakage • Cleaning and Painting of Corroded Gusset Plates and Replacement of Heavily Corroded Gusset Plates		• Cleaning and Painting Floor System including Corroded Vertical Hanger and Truss • Replace Expansion Joint to Seal Water Leakage • Replacement of Gusset Plates • Replacement of Corroded Section of Floor Beam, Longitudinal Tie Beam & Vertical Members by Cutting Affected Areas and Placing New Sections • Sealing Gap Between Vertical Hanger and Sidewalk Concrete and Provide Plates to Prevent Further Corrosion of Vertical Hanger • Removal and reconstruction of deck slab near abutment • Replacement of corroded stringers		• Cleaning and Painting the Steel Structures for the Whole Bridge • Replace Expansion Joint to Seal Water Leakage • Replacement of Gusset Plates, Lateral Bracing and Longitudinal Tie Beam • Replacement of Corroded Section of Floor Beam and Vertical Member by Cutting Affected Areas and Placing New Sections • Sealing Gap Between Vertical Hanger and Sidewalk Concrete and Provide Plates to Prevent Further Corrosion of Vertical Hanger • Removal and reconstruct ion of deck slab near abutment • Replacement of corroded stringers	
(1) STRUCTURAL ASPECT (30%)	CAPACITY / STABILITY	• Basic structural repair and maintenance to restore intended function • No capacity strengthening of reduced sections due to corrosion	D	• Present capacity condition is improved considerably by replacing gusset plates and corroded sections • New sections will have more capacity than corroded sections	B	• Present capacity is increased to a greater degree by replacement of gusset plates, bracing, tie beams and other corroded sections • Capacity higher than Schemes 1 & 2	A
	DURABILITY	• Least durable among schemes • Needs periodical painting works • No corrosion protection on vertical members		• More durable than Scheme 1 • Additional corrosion protection increases member durability (vertical hanger and floor system) and prevents member deterioration		• Highest degree of durability among the three schemes • New members require lesser painting maintenance	
(2) CONSTRUCTION ASPECT (30%)	COST	• PhP 38 Million • Least Expensive	A	• PhP 119.60 Million	B	• PhP 257 Million • Cost is the most expensive among the alternatives	C
	DURATION	• Shortest at ten (10) months		• Eighteen (18) months		• Longest at thirty (30) months	
	METHOD / DIFFICULTY	• Easiest to execute, minimal temporary works		• Temporary supports necessary during replacement of steel members • Construction methodology planning necessary for temporary supports		• Temporary supports necessary during replacement of steel members • Most difficult to execute among the three schemes specially when replacing members of floor system	
(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 of two lanes (1-lane each way) necessary during repair	B	• Closure of two lanes (1-lane each way) necessary during repair of gusset plates and secondary members	C	• Closure of two lanes (1-lane each way) necessary during replacement of members • Repair of vertical member and floor beams will require at least one lane for staging works, thus limiting traffic.	D
	LOAD CAPACITY REDUCTION	• Need to limit traffic load during replacement of gusset plates		• Need to limit traffic load during cutting and replacing members		• Need to limit traffic load during member replacement	
	DETOUR	• Rerouting of vehicles necessary during temporary closure • Only two lanes can be used for 5 months		• Rerouting of vehicles necessary during rehabilitation • Only two lanes can be used for 9 months		• Rerouting of vehicles necessary during rehabilitation • Only two lanes can be used for 15 months	
(5) SOCIAL / ENVIRONMENTAL IMPACT (5%)		• Painting works will have considerable environmental impacts	C	• Painting works will have considerable impact • Cutting and welding works will have considerable impact	C	• Painting works will have significant impact to river • Cutting and welding works will have significant impact	D
OVERALL EVALUATION AND RATING		2	3.85	1 (RECOMMENDED)	4.00	3	3.80

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

21.2.3 Lifecycle Cost Analysis of Bridges

(1) Procedure

Based on the bridge condition survey mentioned in **Section 21.1** and engineering study made in **Section 21.2**, the life cycle cost (LCC) analysis of the Quezon 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) Quezon Bridge Deterioration Situation

The standard deterioration curve of deck, superstructure and substructure between condition rating and age was adopted using the same equation mentioned in **Section 14.3**.

According to the bridge condition survey, the historical records of Quezon Bridge are as follows:

<u>**Vessel Collision**</u>

No vessel collision was reported for Quezon Bridge. The bridge maintains sufficient horizontal and vertical clearances.

<u>**Rehabilitation Record**</u>

In 1992, rehabilitation of deck was made

The deterioration curve of the Quezon Bridge is estimated and shown in **Figure 21.2.3-1**.

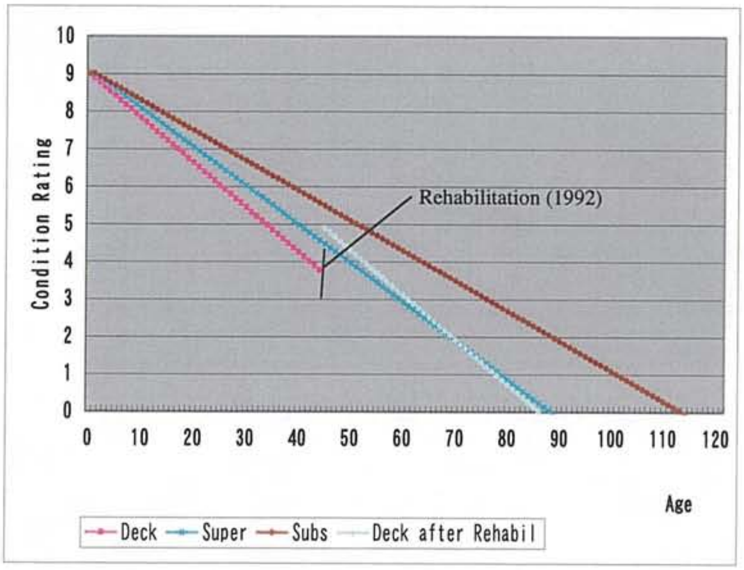


Figure 21.2.3-1 Deterioration Curve of Quezon Bridge

(3) Improvement Schemes

The engineering study proposed the improvement schemes which was shown in **Table 21.2.1-1**.

(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 was already explained in **Section 14.3**. The same model is used for this analysis.

(5) Extended Service Life by Improvement Schemes

Using the deterioration curve in **Figure 21.2.3-2**, 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 Quezon Bridge is calculated and shown in **Figure 21.2.3-3**. The service life of the Bridge is varied to extend by type of rehabilitation. If medium scale rehabilitation is implemented, the service life of the bridge is expected to extend 22 years so total service life will be 27 years from 2007.

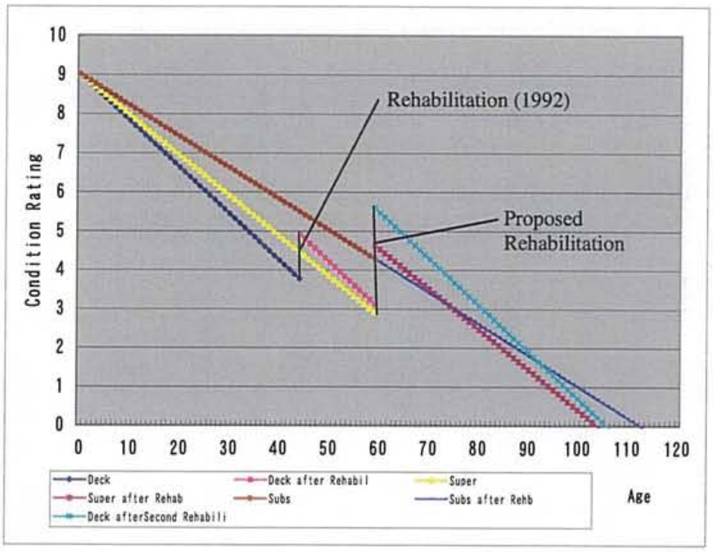


Figure 21.2.3-2 Estimated Deterioration Curve of Quezon Bridge in Case of Medium-Scale Rehabilitation Case

(6) Calculation of the Lifecycle Cost of the Bridges

The bridge lifecycle cost is calculated and shown in **Table 21.2.3-1.**

Table 21.2.3-1 Lifecycle Cost Estimates of Quezon Bridge by Rehabilitation Types

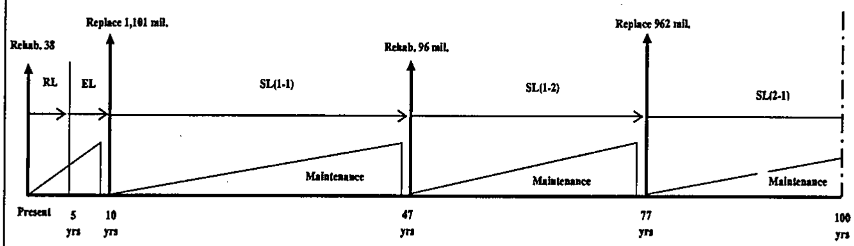
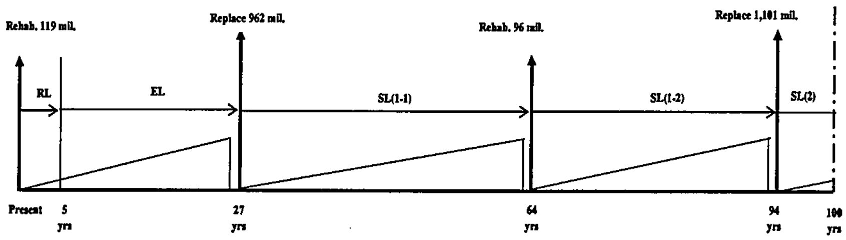
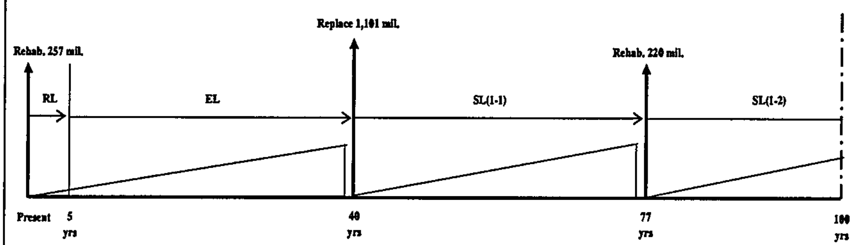
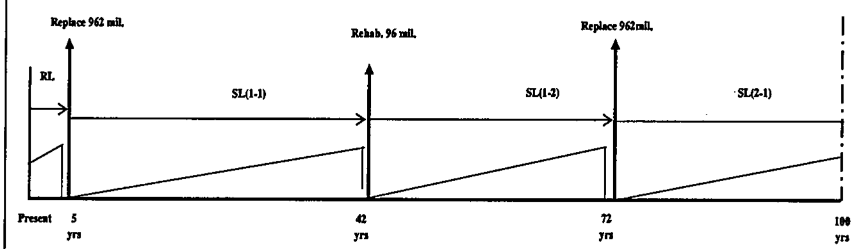
	LCC at Discount Rate15%	Recommended Improvement Ranking
Small Scale Rehabilitation	318 million (0.61)	3
Medium Scale Rehabilitation	197 million (0.39)	1
Large Scale Rehabilitation	292 million (0.56)	2
Replacement	518 million (1.000)	4

Note: ( ) Ratio of life cycle cost to replacement

(7) Recommendation

The result of the life cycle cost (LCC) analysis shows that the medium scale rehabilitation scheme has the least cost among the alternatives.

Judging from LCC view point of Quezon Bridge, the medium scale rehabilitation scheme is recommended.

Scheme	Life Cycle Cost Model	Service Life (Yrs)	LCC (Discounted rate 15 %)
Small Scale Rehabilitation		Type Year RL 5 EL 5 SL(1) 67 SL(2) 23 Total 100	318 million (0.61)
Medium Scale Rehabilitation		Year RL 5 EL 22 SL(1) 67 SL(2) 6 Total 100	197 million (0.39)
Large Scale Rehabilitation		Year RL 5 EL 35 SL(1) 60 Total 100	292 million (0.56)
Replacement		Year RL 5 EL 22 SL(1) 67 SL(2) 6 Total 100	586 million (1.00)

Notes: RL: Remaining life
EL: Extended life due to rehabilitation
SL: Bridge cycle life
SL(1): First bridge cycle life
SL(2): Second bridge cycle life
SL(1-1): First bridge cycle life before rehabilitation
SL(1-2): First bridge cycle life after rehabilitation

Figure 21.2.3-3 Life Cycle Analysis of Quezon Bridge

21.2.4 Recommendations

Based on the comparative study aiming at engineering aspects and life-cycle cost analysis, the most recommendable scheme is **the medium-scale rehabilitation** scheme.

21.3 PRELIMINARY DESIGN AND COST ESTIMATE

21.3.1 Rehabilitation Design

(1) Bridge Design

(a) Scope of Works for Rehabilitation

This preliminary design recommend the scheme, "**Rehabilitation of Corroded Section** (Medium Scale Rehabilitation)" as mentioned in **Section 21.2** of the Comparative Study of Rehabilitation Method. The major improvement measures in this scheme are as follows:

- Replacement of corroded section of floor beam, longitudinal tie beam & vertical members by cutting affected areas and placing new sections.
- Replacement of gusset plates, corroded stringers, expansion joint, and deck slab near the abutment.
- Sealing gap between vertical hanger and sidewalk with plates to prevent further corrosion of vertical hanger.
- Provision of drain pipe at the abutment.
- Cleaning and painting floor system Including corroded vertical hanger and truss.

Figure 21.3.1-1 shows the general view of the works.

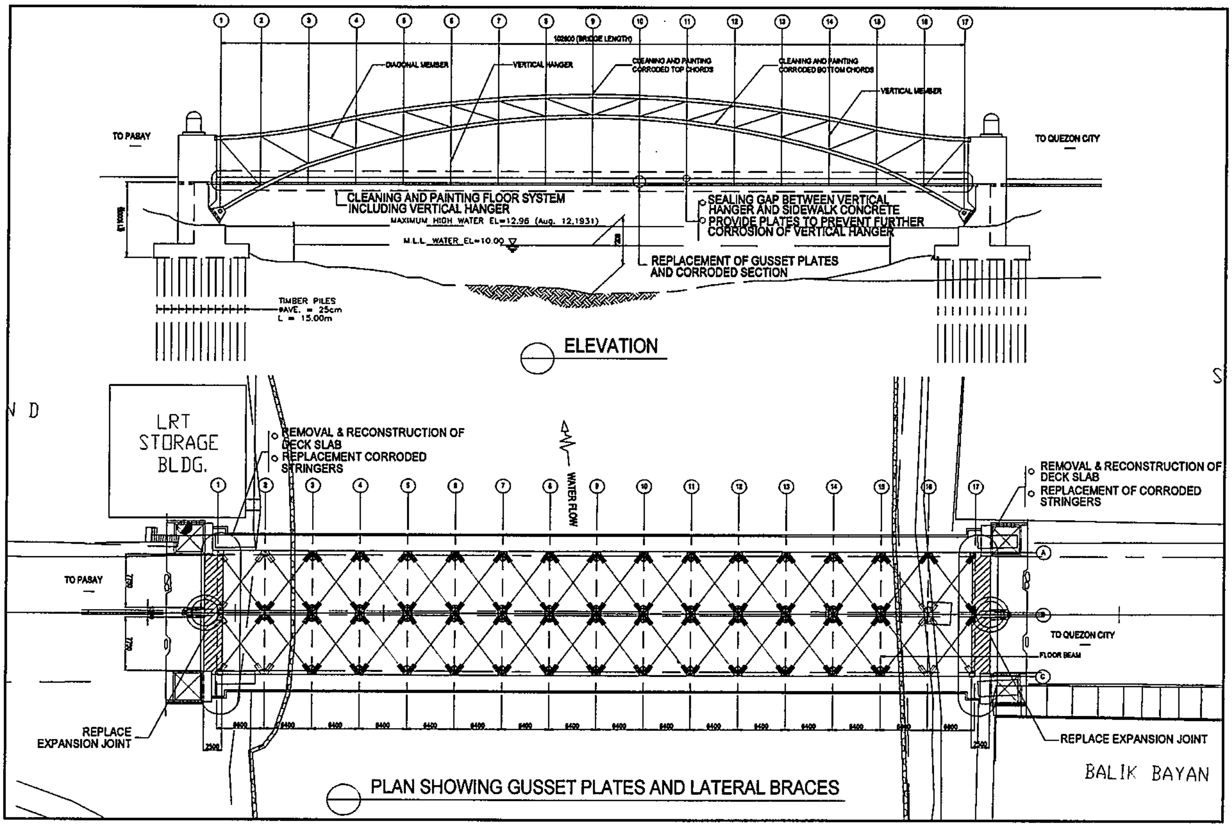


Figure 21.3.1-1 General View of Rehabilitation Works

(a) Design Criteria for Structure

Criteria for materials and Loads are shown in **Table 21.3.1-1.**

Table 21.3.1-1 Design Criteria

I. SPECIFICATION	- AASHTO Standard Specification for Highway Bridges, 16 Edition 2000 including Division IA, Seismic Design - Specification for Highway Bridges, Japan Road Association, 1994
II. MATERIALS	- Concrete; Fc' = 21 MPa - Reinforcing Steel; Fy = 275 MPa - Structural Steel, 248MPa A36 (36,000psi) Steel Plates & Rolled Shape, - Bolt; AASHTO M164 (ASTM A325) - Welds; AD1.1 – 183, E70xx Series
III. LOADS	- Deadloads Reinforced Concrete = 24.5 kN/cu.m Steel = 77 kN/cu.m Earth Compacted = 19 kN/cu.m - Highway Loads AASHTO MS – 18 Loading - Impact Loads I = 15.24/L + 38, Where L = Length in meters - Sidewalk Loads For Span more than 20m Sidewalk Loading shall be 2.50 KPa - Earthpressure Mononobe – Okabe Method

(c) Design of Superstructure

The design is to rehabilitate the sections of corroded members and seal the gap between slab and vertical hanger to prevent seepage of water that causes further corrosion of the members in accordance with the inspections carried out. The detail of the rehabilitation is shown in **Appendix 21.3.1-1.** The scope of major rehabilitations are as follows:

Replacement of Members

The corroded sections of floor beams, longitudinal tie beams, and vertical hangers are to be replaced with the new members by cutting as shown in **Figure 21.3.1-2.**

Waterproof Vertical Hanger at Deck slab

Waterproof plate shall be welded around the vertical hanger while the gap between the plate and deck slab are to be sealed with elastomeric rubber as shown in **Figure 21.3.1-3.**

Addition of new drain pipe at abutment

New drain pipe shall be connected with the existing drain pipe on deck slab as shown in **Figure 21.3.1-4** to prevent corrosion of bearing supports.

Replacement of Expansion Joint

Existing expansion joint are to be replaced with new water resistant expansion joint to prevent corrosion of floor system as shown in **Figure 21.3.1-5.**

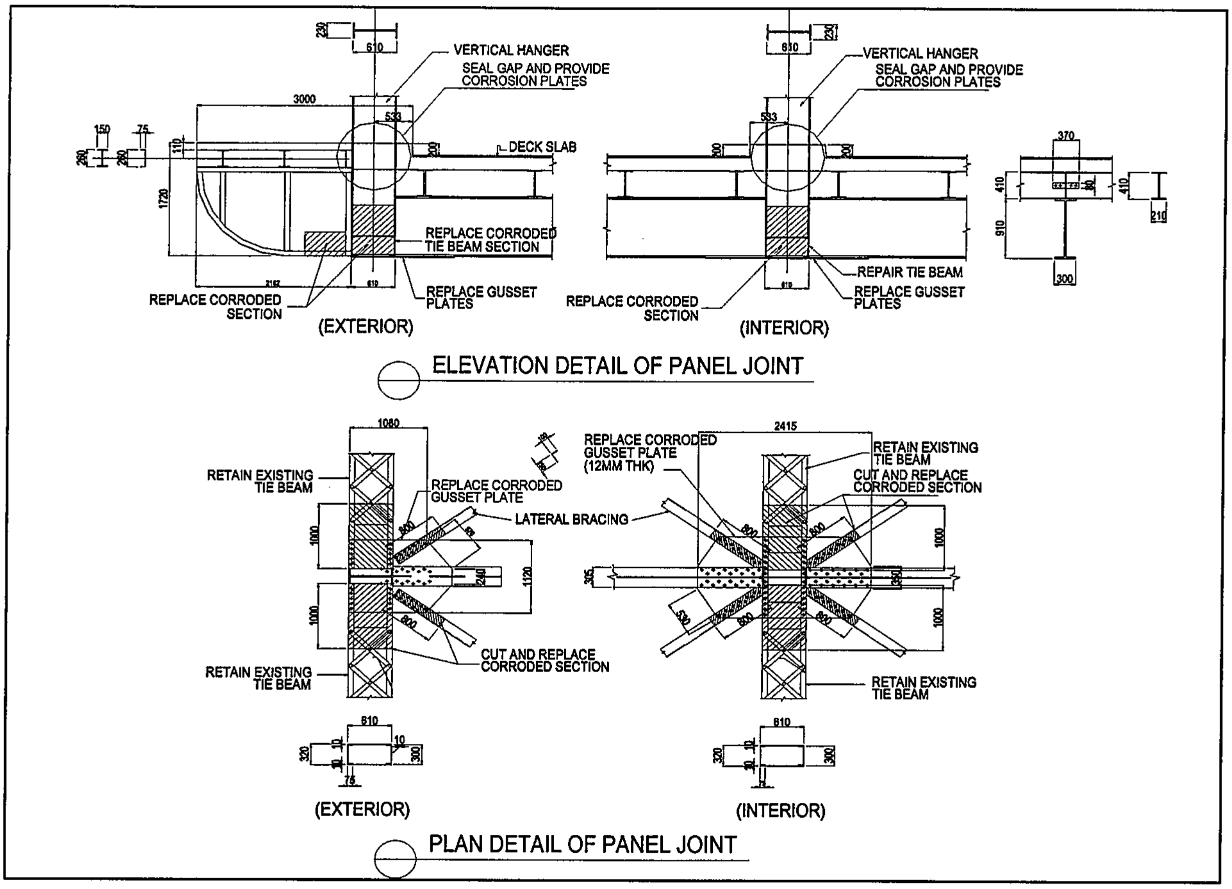


Figure 21.3.1-2 Replacement of Corroded Section

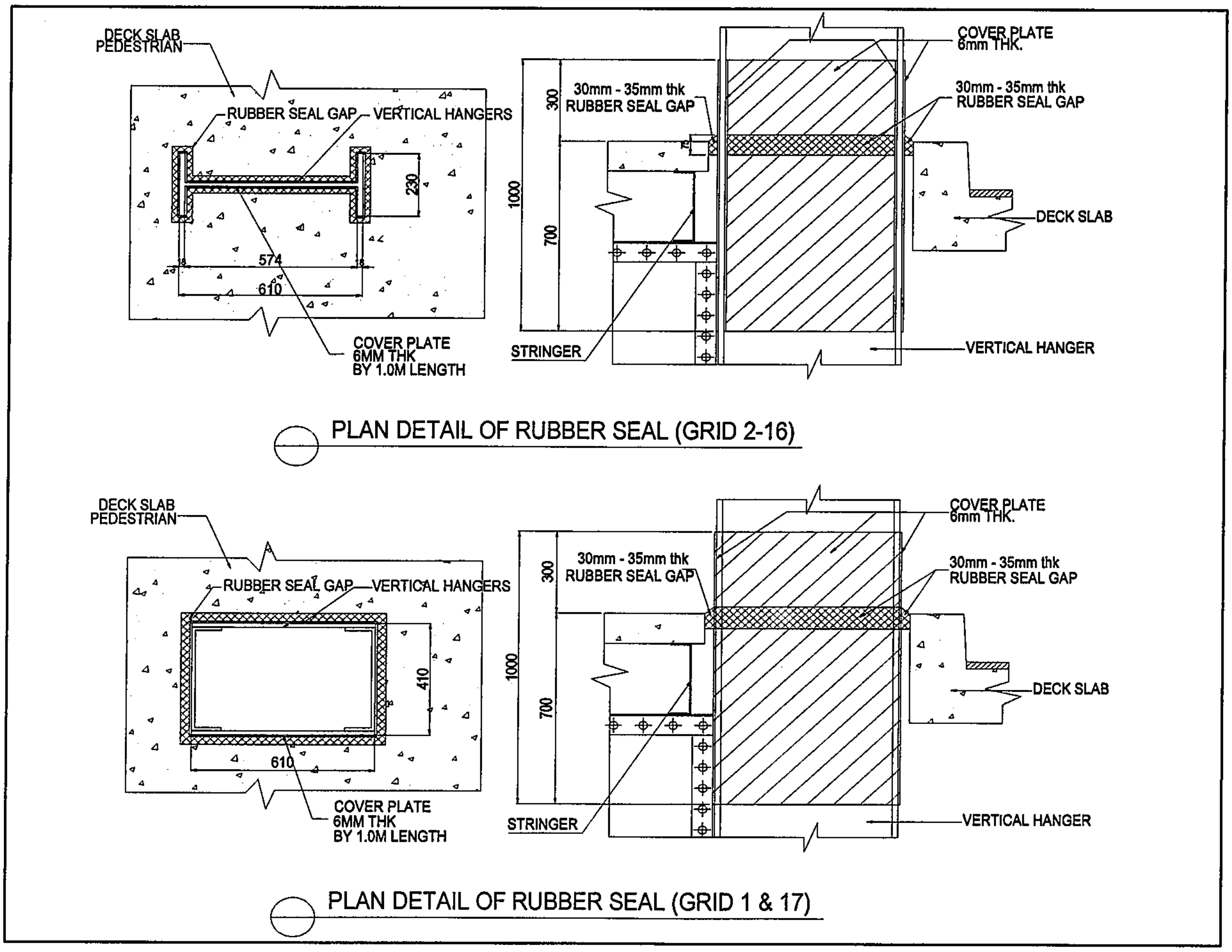


Figure 21.3.1-3 Sealing gap between Hangers and Deck Slab

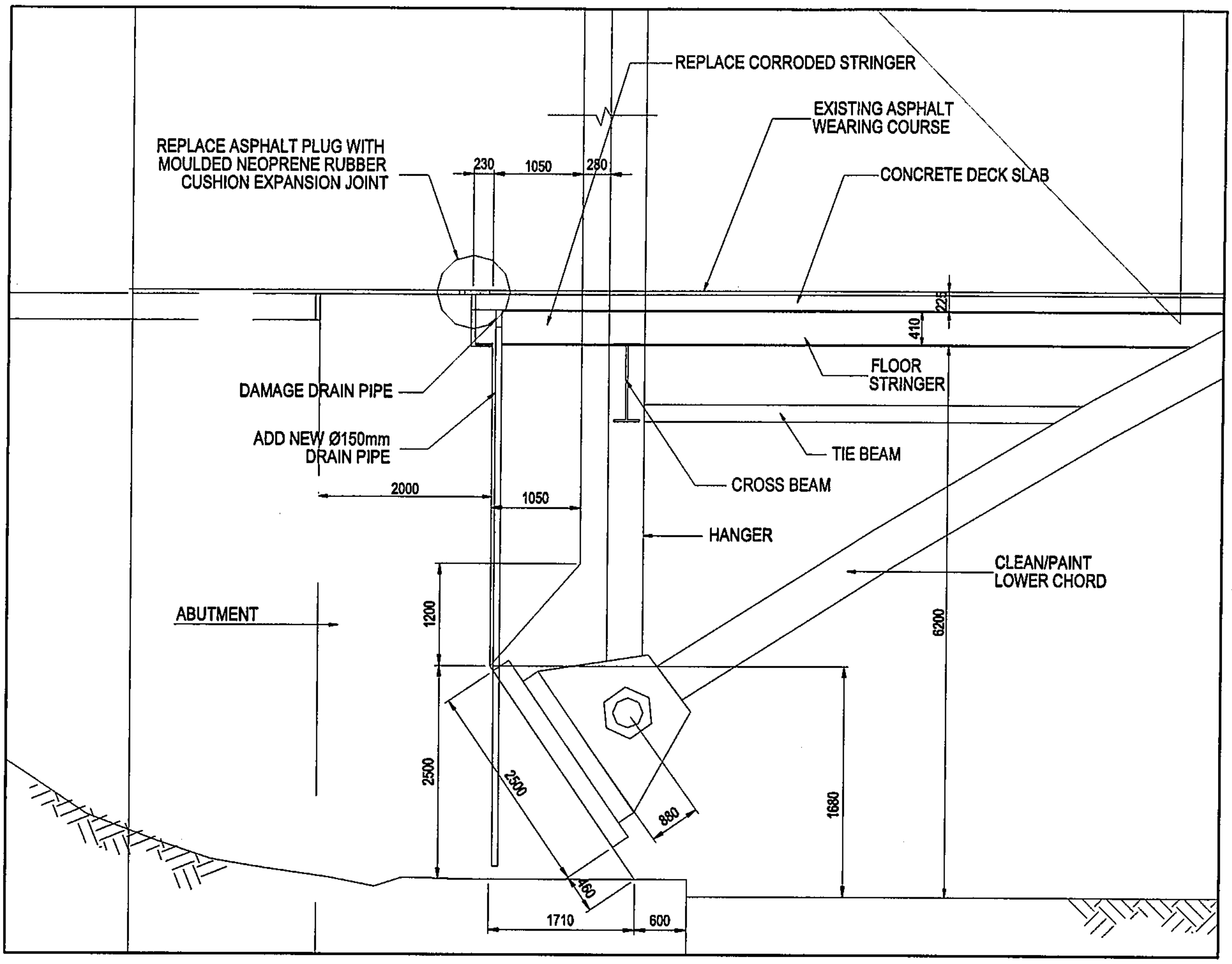


Figure 21.3.1-4 New Drain Pipe at Abutment

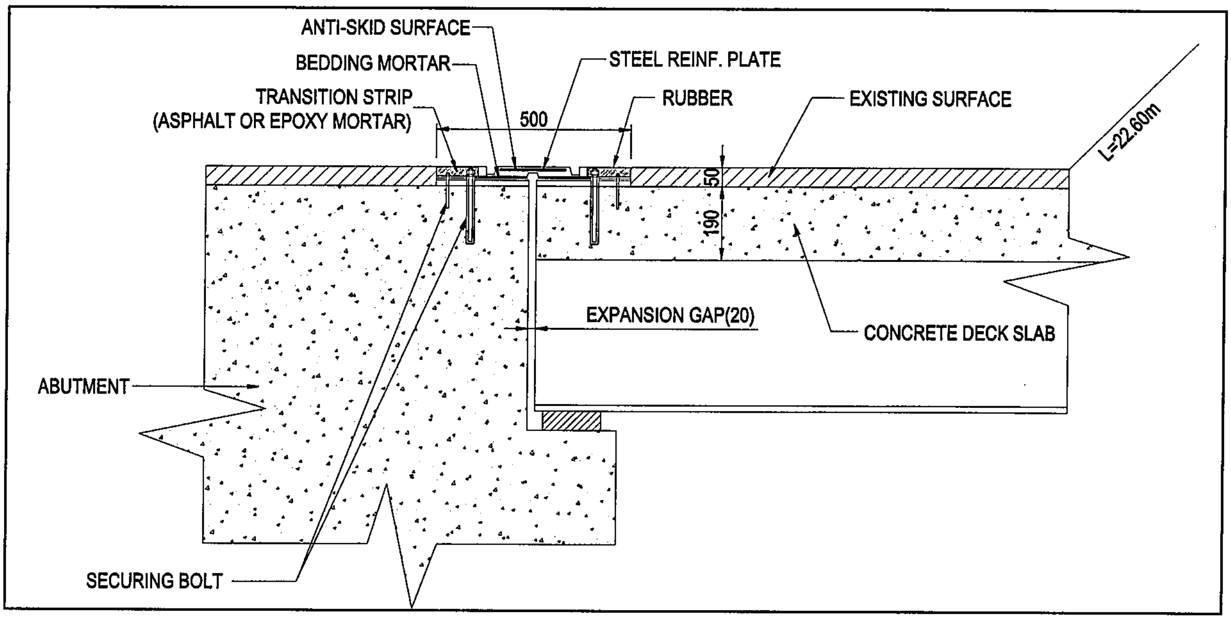


Figure 21.3.1-5 Waterproof Expansion Joint

Major Quantity

Major quantity for this rehabilitation work is shown in **Table 21.3.1-2**

Table 21.3.1-2 Major Quantity for Rehabilitation

Description	Unit	Quantity
A. Steel		
Replacement of Gusset Plate	kgs	11,268.75
Replacement of Tie Beam	kgs	7,868.75
Replacement of Lateral Bracing	kgs	3,341.25
Replacement of Stringers	kgs	1,737.50
B. Concrete Deck		
Replacement of Concrete Deck	cu.m.	25.00
Reinforcing Steel Bars	kgs	3,750.00
C. Removal		
Removal of Deck Slab	sq.m	125.00
Removal of Corroded Sections	l.s.	1.00
D. Miscellaneous		
Vertical Hanger Corrosion Protection	kgs.	7,231.25
Vertical Hanger Waterproofing Seal	l.m.	133.75
Expansion Joint	l.m.	58.75
Expansion Joint Drain Pipe	l.m.	65.00
Painting Floor System	sq.m.	14,377.99
Epoxy Injection	l.s.	1.00

(2) Highway Design

(a) Scope of Works

The highway works include following items:

- Approach roads of the bridge
- Improvement of two (2) intersections

(b) Design Criteria for Highway

The highway design was carried out based on the following criteria and standards:

- Design Guidelines and Standards for Public Works and Highways, Volume – 11
- A policy on Geometric Design of Highways and Streets, 1996 (AASHT0)
- Highway Capacity Manual, Special Report, Transportation Research Board, 1999
- Road Structure Ordinance, Japan Road Association, 1983 (JRA)

(c) Intersections

The problem with the existing intersection is the excessively wide area in the approach at Arroceros St. This has the tendency to pose traffic and pedestrian safety and create traffic confusion problems. The other approach intersection is already grade separated as it is part of the bridge approach ramp towards Quiapo area. However, the frequent bus and jeepney stops just before and after the bridge poses disruption of the traffic flow. The existing service roads at Approach 2 are wide but they are almost occupied by market vendors and are also being used as jeepney's end terminal routes.

Table 21.3.1-3 shows the existing condition of approaches/ intersections.

Table 21.3.1-3 Existing Conditions of Approaches /Intersections of Quezon Bridge

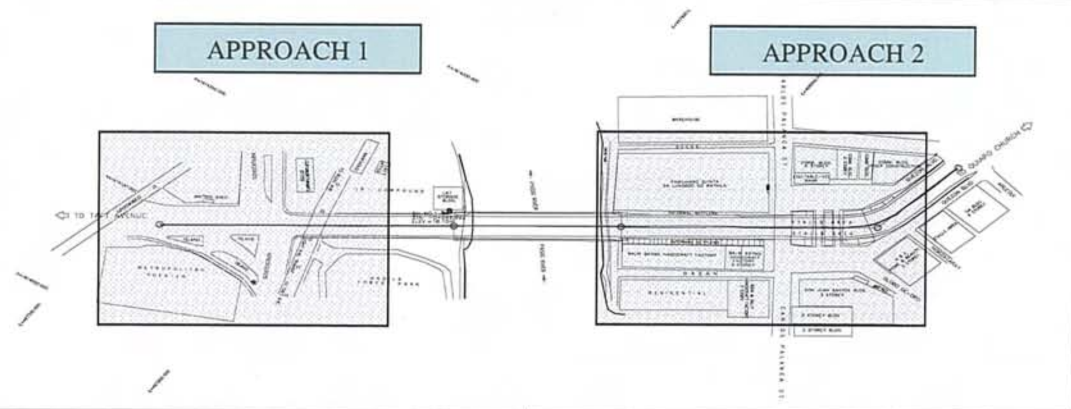
PLAN	 	
	APPROACH 1	**APPROACH 2**
Traffic	• **Several vehicles use Arroceros St. as alternate route towards the Quezon Bridge going to Quiapo, creating disruption of the through traffic flow.** • Arroceros St. on the other side is wide road used as bus terminal and parking.	• **Service road traffic are very heavy due to jeepney route and commercial district.** • Parking is allowed at both sides of the service road. • Heavy traffic at Carlos Palanca near the service road underneath the bridge due to parking and commercial district.
Pedestrian	• **Uncontrolled pedestrian crossing.** • Pedestrian traffic are heavy.	• **Uncontrolled pedestrian crossing at service roads.** • Underpass pedestrian crossing is provided.
Geometric and Pavement	• **Excessively wide road at Arroceros St.** • Pavement surface are worn out	• **Wide service road considering a very busy commercial district and roadside parking.** • Pavement is in good condition.
Traffic Signal, Markings and Signs	• Unsignalized intersection. • Pavement markings are worn out. • **Insufficient traffic signs.**	• Unsignalized intersection underneath the bridge at Carlos Palanca Street. • **Insufficient traffic signs.**
Recommendation	• Minor improvement is necessary to reduce unnecessary wide road at Arroceros St. • A pedestrian overpass is needed along through road of the Quezon Blvd.	• No improvement is necessary.

Figure 21.3.1-6 to **Figure 21.3.1-7** present before and after improvement of the Quezon Bridge approach intersections.

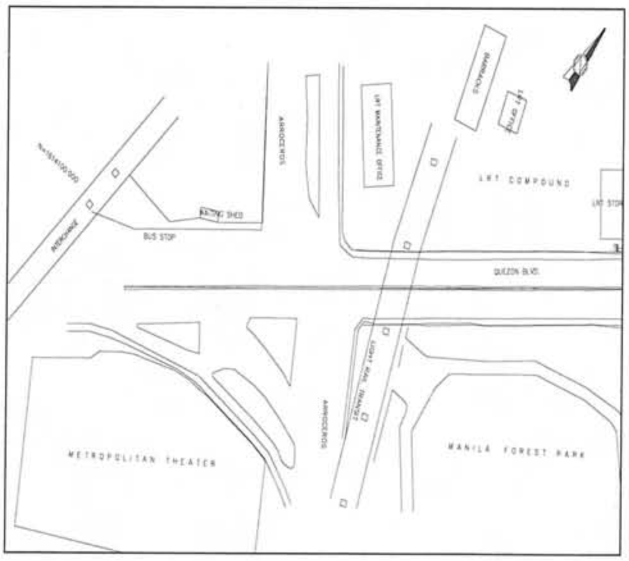


Figure 21.3.1-6 Existing Approach 1 Intersection before Improvement

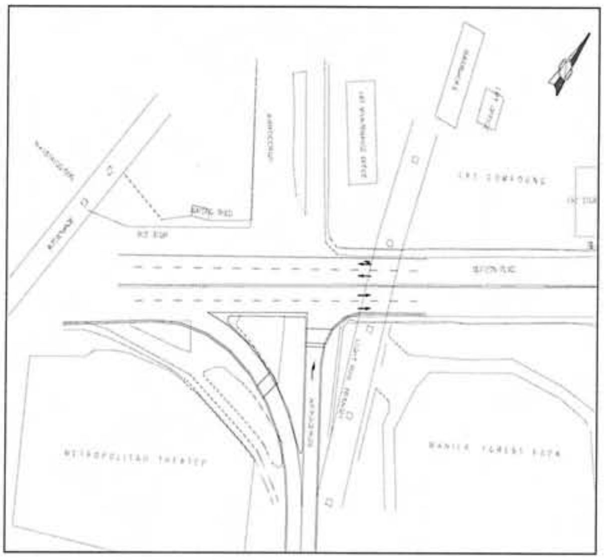


Figure 21.3.1-7 Proposed Approach 1 Intersection after Improvement

The existing intersection is permanently provided with raised median barrier. Bus stop is also provided at one side of the road coming from the Quiapo area. Traffic congestion is observed during peak periods approaching the bridge towards Quiapo area. Uncontrolled loading and unloading is observed near the bridge approaches.

The existing wide road at Arroceros St. is being used as bus and jeepney terminal. Right turn towards the Metropolitan Theater is temporarily closed with barrier fence due to jeepneys swerving to overtake towards the bridge.

Quezon Blvd. and Arroceros St. is a T-type intersection. This shall be improved to reduce wide area of the road that is unused to avoid confusion of traffic movements and to prohibit jeepney and bus terminal along busy road unless provided with specified bus stop such as the existing Bus Stop.

Barrier fence should be provided to control pedestrians crossing other than the specified zebra crossings. Pedestrian overpass is much recommended due to heavy pedestrian traffic.

Modification of the corner island shall be done to improve the wide road at Arroceros St. to accommodate only one lane going into the bridge. The right turning movement to Arroceros St. will be provided as this will serve as entry/exit road for the soon to be rehabilitated Metropolitan Theater.

Pavement markings, traffic signs and guide signs shall be installed.

(d) Approach Road and Access Road

The existing alignment dictates the alignment of improvement design; the horizontal and vertical alignment of the center line of the bridge is maintained.

Access Roads or driveways of nearby buildings should be discouraged to place entrances along the major road since this will obstruct efficient traffic flow especially along approach ramps of the bridge.

The existing site ocular inspections of both approaches are shown in **Appendix 21.3.1-2.**

(3) Design of Protection to Vessel Collision

According to recommendation in **Section 11.4**, the protection to vessel collision is not required.

21.3.2 Construction Plan and Traffic Management

(1) Construction Method

The rehabilitation works are to be carried out with the use of scaffoldings as shown in **Figure 21.3.2-1.**

When cutting and removing the existing deteriorated section, temporary supports will be used. The cut portions will be lifted by lifting boom on truck and will be transported for disposal. New parts will be brought to the scaffolding by the lifting boom.

The member/parts for center chord construction will be brought using rail on the scaffoldings under the bridge. The scaffoldings are designed to have space and capacity to stock construction members and materials. All the new members/parts will be connected by high-tension bolts

The rehabilitation works will be carried out by 2 working groups in order to shorten the construction period. The construction sequence of the proposed rehabilitation works is shown in **Appendix 21.3.2-1.**

(2) Traffic Management

In this construction method, the traffic will be controlled as shown in **Figure 21.3.2-2** to **21.3.2-3** in accordance with the construction work as shown in **Figure 21.3.2-1**. The Traffic constraints are as follows:

- During construction of vertical hanger-1, use 1 lane for construction and open 2 lanes for traffic.
- During construction of vertical hanger-2, use 3 lanes for construction and open 1 lane for traffic.
- During construction of vertical hanger-1, use 1 lane for construction and open 2 lanes for traffic.

Traffic shall be controlled to avoid additional effects from live load during removal and installation of replacing members.

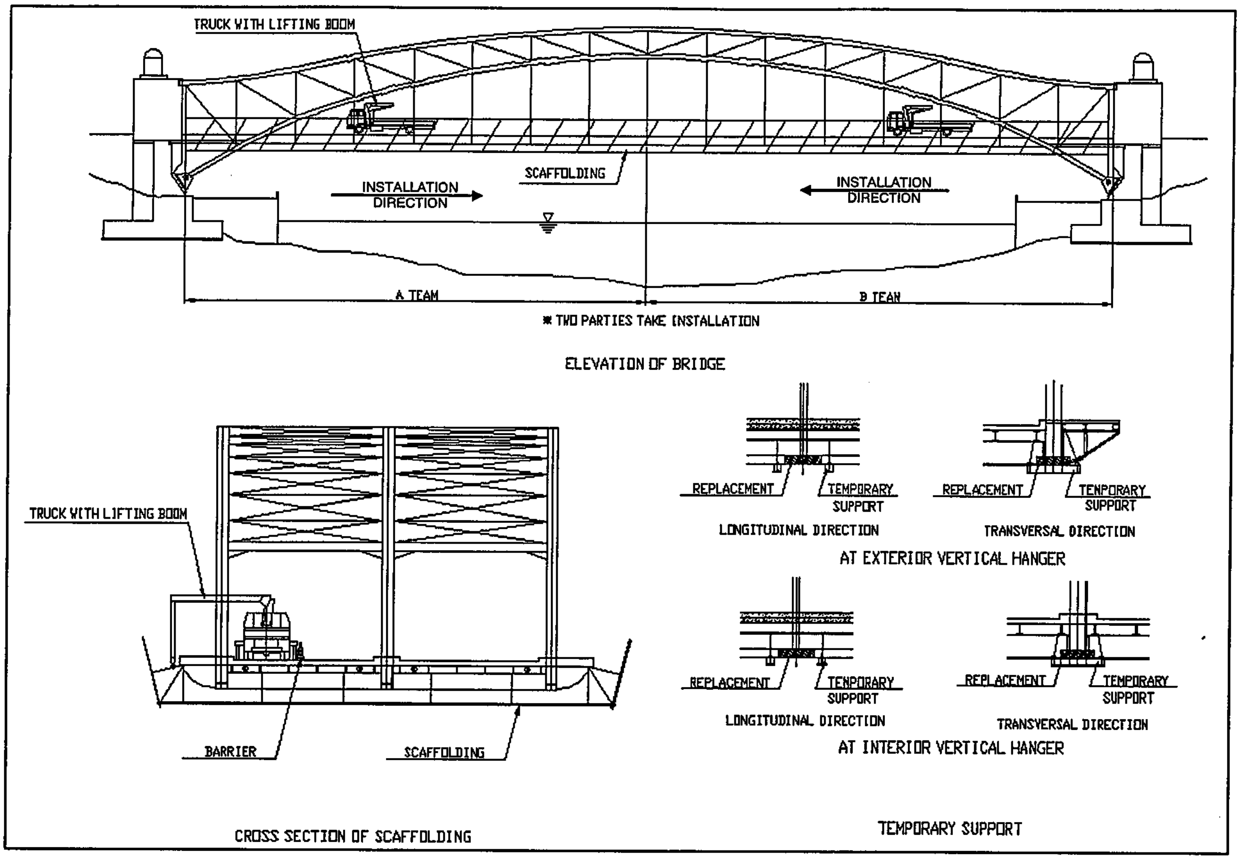


Figure 21.3.2-1 Construction Method

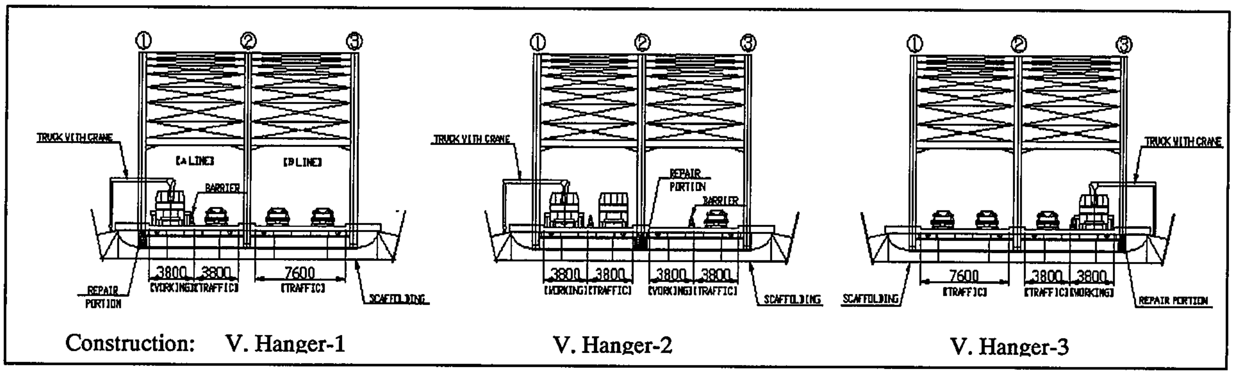


Figure 21.3.2-2 Traffic Control

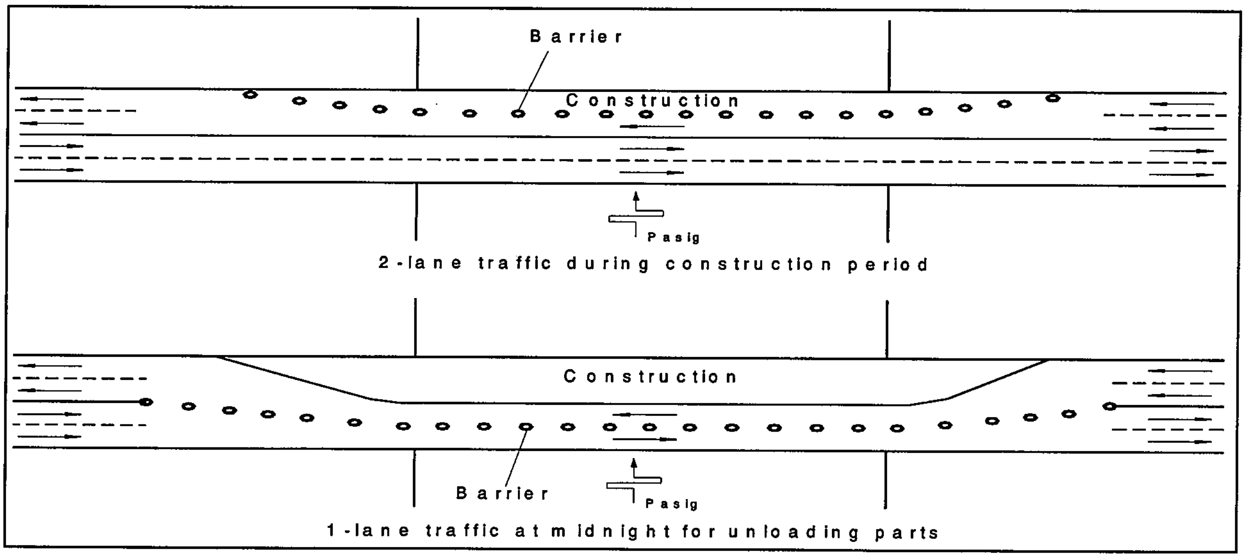


Figure 21.3.2-3 Traffic Lane Control

21.3.3 Preliminary Cost Estimate

The total project costs consist of total construction cost and engineering service cost. The construction cost was estimated by accumulation of each work item which is the combination of labor costs, material costs and equipment costs considering the construction method and procedure.

(1) Construction Cost

Construction cost was estimated by accumulating the cost of each work item which is the product of unit cost and quantity of each work. The unit costs of each item are estimated from the combination of the basic unit prices of the labor wages, material prices and equipment operation cost considering the construction method and procedure.

Unit costs were estimated from the previous similar practices in Manila. In order to cover the unforeseen works and conditions at this moment, and considering the allowance for some changes in the detailed design stage, a contingency of 5% was taken in account to the total construction cost.

The unit cost by construction item is shown in **Table 21.3.3-1**

Table 21.3.3-1 Unit Cost by Construction Items

June 2003 Prices

Item No.	Description	Unit	Unit Cost(PP)	Components (%)		
				Foreign	Local	Taxes
Annex I – CONSTRUCTION COST REHABILITATION						
A. Steel Structures (Fabricate & Transport)						
408(1)	Steel I Girder	kgs.	150.00	74%	11%	15%
B. Scaffolding (including scaffolding for painting)						
SPL	Scaffoldings/Temporary Works	sq.m.	1,200.00	68%	18%	14%
C. Siteworks						
SPL	Bridge Survey at Site	days	83,000.00	74%	11%	15%
SPL	Removal of Existing member	place	200,000.00	74%	11%	15%
SPL	Installation	place	134,000.00	74%	11%	15%
SPL	Section Repair	place	50,000.00	74%	11%	15%
411	Painting Works	sq.m.	4,020.00	74%	11%	15%
SPL	Expansion Joint	l.m.	130,000.00	74%	11%	15%
SPL	Expansion Joint Drain Pipe	l.m.	950.00	74%	11%	15%
SPL	Epoxy Injection	l.s.	1,250,000.00	74%	11%	15%
			Subtotal	74%	11%	15%
D. Total Direct Cost				74%	11%	15%
E. Indirect Cost						
	Traffic Management					
	Temporary Facilities					
	Mobilization/demobilization					
	40% of Total Direct Cost			71%	15%	14%
Annex II - ROADWAY IMPROVEMENT						
Earthworks						
101(3)a	Removal of Island	cu.m.	114.15	65%	21%	14%
Miscellaneous						
311	Concrete Median	sq.m.	272.93	65%	21%	14%
600(1)	Concrete Curb	l.m.	562.46	65%	21%	14%
612(1)	Pavement Markings	sq.m.	862.13	65%	21%	14%
xxx	Contingencies	l.s.	18,223.90	75%	15%	10%

Detailed computation is presented in **Appendix 21.3.3-1.**

Total Construction Cost

The total construction cost of the bridge rehabilitation estimated on the basis described above is shown in **Table 21.3.3-2.**

Table 21.3.3-2 Estimated Construction Cost

June, 2003 Prices

Items		Cost(x MP)
Superstructure	Foreign	87.00
	Local	14.80
	Tax	17.50
	Subtotal	119.20
Highway	Foreign	0.30
	Local	0.10
	Tax	0.10
	Subtotal	0.40
Total Construction Cost		**119.60 MP**

(2) Road Right-Of-Way Acquisition Cost

No acquisition of Right-Of-Way for this bridge.

(3) Engineering Cost

Engineering service cost consists of the engineering design services at the detailed design stage and the construction supervision at the construction stage. The engineering service cost varies depending on the scale of the project, tender processing and contract method.

Based on previous experiences, the engineering service costs for the project are estimated as 5% and 8% of the total construction cost for the detailed design and construction supervision respectively.

The estimated engineering cost is shown in **Table 21.3.3-3**

Table 21.3.3-3 Estimated Engineering Cost

June, 2003 Prices

Items		Cost (x MP)
Detailed Design	Foreign	3.30
	Local	2.10
	Tax	0.60
	Subtotal	6.00
Construction Supervision	Foreign	5.30
	Local	3.30
	Tax	1.00
	Subtotal	9.60
Total Engineering Cost	**Total**	**15.60**

(4) Project Cost

The total project cost consist of construction cost, land acquisition cost and engineering service cost. The summary of the estimated project cost is given in **Table 21.3.3-4**

Table 21.3.3-4 Summary of Estimated Project Cost

June, 2003 Prices

Items		Cost (x MP)
Construction Cost	Foreign Local Tax	87.30 14.80 17.50
	Subtotal	119.60
Engineering Cost	Foreign Local Tax	8.60 5.40 1.60
	Subtotal	15.60
Grand Total	Foreign Local Tax	95.80 20.30 19.10
Grand Total		**135.20**