CHAPTER 20

FEASIBILITY STUDY OF JONES BRIDGE REHABILITATION PLAN

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20.1 DETAILED BRIDGE SURVEY AND ASSESSMENT

The discussion in this section 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".

20.1.1 Review of Design and Repair Works

(1) Review of Design

The references of the review of design are the construction drawings of Jones Bridge provided by the DPWH and the Bridge Retrofit Program Report of ERP also furnished by the DPWH.

(a) Outline of Jones Bridge

Photo 20.1.1-1 Panoramic View of Jones Bridge

- Structure Type : Three-Span Continuous Steel Girder Bridge
  Eight (8) steel girders support the four-lane separated concrete deck. The girders rest on a 22m wall type piers.
- Bridge Length : 114.43m (35.46m + 43.31m + 35.46m)
- Date of Construction : 1948

(b) General Notes on Jones Bridge Construction Drawings

- 1944 AASHO Standard Specifications for Highway Bridges.
- Drawings of Jones Bridge were prepared by Public Road Administration, Washington D.C. in January 1948, and revised in June 1948.

- Jones Bridge was designed by Design Division, Bureau of Public Works, Manila, Philippines.

- Pier body was designed and completed in 1948 on existing caisson foundation which was built in 1916.

Construction and materials

- Design Criteria and materials were not available in the collected plans.

- Outline of existing caissons and piers as originally constructed, Department of Engineering and Public Works, City of Manila, dated September 1916.

Concrete

- All concrete types indicated are class "A".

Reinforcing Steel

- Field strength of reinforcing steel not specified due to lack of adjoining sheets in the collected plans.

Structural Steel

- Yield strength of structural steel is not specified.

Pier Notes

- Parts of damaged existing pier shall be removed.

- The voids of pier body shall be cleaned out and filled with concrete, diameter 25 mm. anchor bolt shall be placed where possible existing reinforcing steel shall be cleaned and incorporated in the new concrete. No horizontal construction joints shall be permitted.

- Parts of damaged existing pier shall be removed. The voids shall be filled with concrete and diameter 25 mm anchors.

- Possible existing reinforcing steel shall be cleaned and incorporated in the new concrete.

(2) Review of Repair Works

- The date of restoration was 1948 by US Bureau of Public Roads and Philippine Bureau of Public Works under the US-Philippine Rehabilitation Act of 1946

- Foundation was built on original caisson which was constructed in 1916

- Seismic Rehabilitation was done in 1997 under the DPWH Bridge Retrofit Program (BRP) of the Earthquake Reconstruction Project (ERP)

Photo 20.1.1-2 Shear keys as shown on Top of Piers and Abutment are part of ERP Retrofit Program in 1997.

In 1997, the works undertaken for the seismic rehabilitation were demolition and reconstruction of deck slab and sidewalk, replacement of defective structural members, replacing of steel railing members, painting of metal surfaces, replacement of expansion joints, and provision of asphalt overlay at approaches. Shear keys on top of the piers were also constructed to properly transmit the forces from superstructure to the substructure. The support condition of the girders at Pier P2 (near Binondo side) was changed from roller condition to hinge condition.

Problems /Issues of Previous Repairs Works

- Deck slab were fully replaced, and no major damage or defect was detected in this study.
- The exterior girders at upstream & downstream sides were temporarily repaired, and they are still prone to vessel collision because of the lower vertical clearance at near piers.
- The existing vertical clearance near the piers is 3.6m, which is lower than the regulatory vertical clearance of 3.75m as shown in **Figure 20.1.1-1** and was not improved during the rehabilitation.

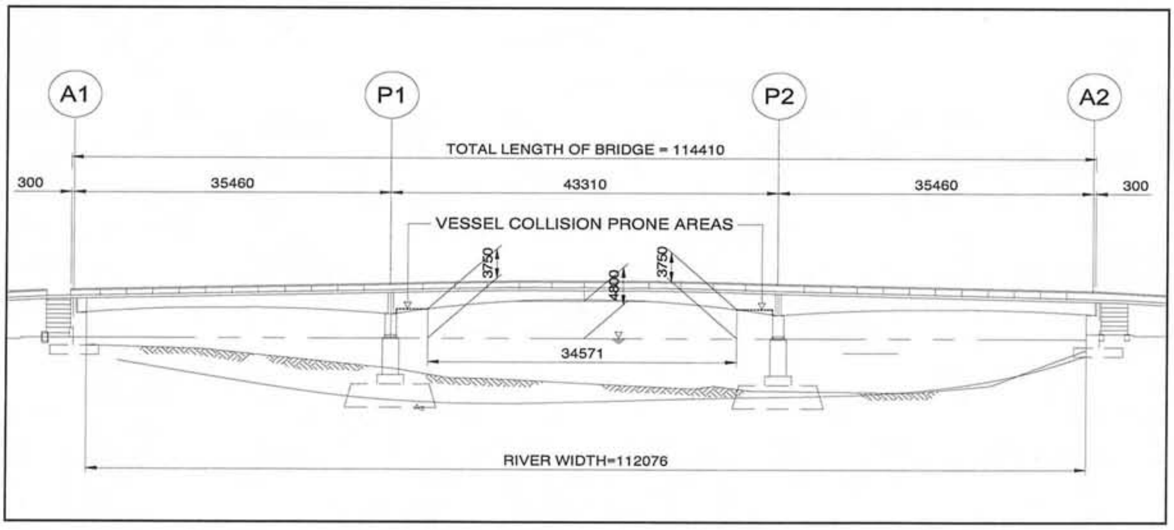


Figure 20.1.1-1 Navigational Clearance of Jones Bridge

- From the above configurations, it can be seen that the exterior girders are prone to vessel collision as proven by the rupture of existing girder on the upstream side and the out-of plane deformation of the exterior girder on the downstream side.
- It should be noted that no previous rehabilitation was done on Pier walls and existing foundation connection. This connection is insufficient as far as the requirement of the latest code is concerned.

(3) Historical Background

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

20.1.2 Natural Condition Survey

(1) Topographic Survey

(a) Control Monument

Two (2) GPS Stations were established as control points for Jones Bridge as shown in **Table 20.1.2-1.**

Table 20.1.2-1 GPS Stationing and Coordinates

STATION	GPS COORDINATES		
	NORTHING	EASTING	ELEVATION
GPS-J1	1614130.267	497479.562	12.779
GPS-J2	1614228.548	497433.738	16.200

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 20.1.2-2 shows the scope of works of topographic survey. Topographic plan is shown in **Appendix 20.1.2-1.**

Table 20.1.2-2 Scope of Work of Topographic Survey

Description	Original Scope	Actual Work
CONTROL POINT SURVEY (GPS)	1	2
PROFILE SURVEY	115 m Bridge Section + 200 m Each of Both Approach Roads (200 x 2) Total = 515 m	115 m Bridge Section + 243.81 m + 222.5 m at each approach roads Total = 581.31 m
ROAD CROSS-SECTION SURVEY	Bridge Section (115m) : @ 10m Interval Approach Roads (400m) : @ 20 m Interval Width: Bridge 25m + 50m each at both sides = 125m Total = 35 Sections	Bridge Section (115m): 11 sections Approach Roads (466.31m): 22 sections Width: 125 m Total = 33 Sections
TOPOGRAPHIC SURVEY	515 m (Length) x 125 m (Width) = 64,375 sq. m	581.31 m (Length) x 125 m (Width) = 72,663.75 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

Two boreholes were drilled at the bridge site. Borehole No.1 (BH-1) was drilled at the back of the abutment at the foot of the stair located at the side of Binondo area. The other borehole, BH-2, was drilled at the corner side of Pier P2.

Borehole No. 1 (BH-1) consists of clayey silt, elastic silt and silty clay. The layers vary in consistencies from soft to very stiff (N=3 to 27) from the ground level down to 20.0 meters. The bearing layer occurs starting from 22.0 meter depth extending down to the bottom of the borehole at 33.0 meters.

Borehole No. 2 (BH-2) is characterized by a generally granular formation in the uppermost 5.0 meters. A thick cohesive formation that extends from 5.0-meter depth down to 16.0-meter. Consistencies vary from medium stiff to very stiff with corresponding N-values of 6 to 22. It is generally dense with N-value of 38 to 40. From 18.0 to 30.0-meter depth, another cohesive formation is present. The portion from 18.0 to 27.0-meter is stiff to very stiff (N=10 to 27) while the lower portion down to 30.0-meter depth is hard to very hard (N=44 to refusal).

Results of the survey were used for estimation of seismic force and resistance of foundations. Survey results are shown in **Appendix 20.1.2-2** including borehole logs and locations.

(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 20.1.2-3 (1/2)** to **20.1.2-3 (2/2)**

As shown in **Figure 20.1.2-1**, scouring around substructures were not found.

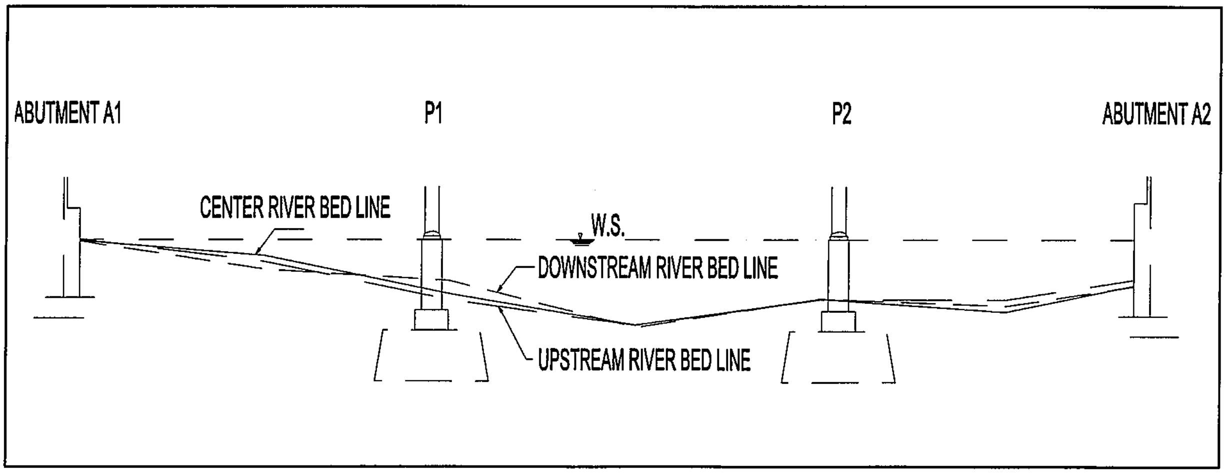


Figure 20.1.2-1 Result of Scour Survey Conducted in Jones Bridge

20.1.3 Bridge Condition Survey and Identification of Damages

(1) Measurement of Shapes and Dimensions

(a) Objective

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

(b) Inspection Teams

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

(c) Coverage Area

Verification of Road Deck Level includes road deck width, length of bridge, dimensions of post, railing, sidewalks and expansion of joint gap (see to Photo **20.1.3-1**).

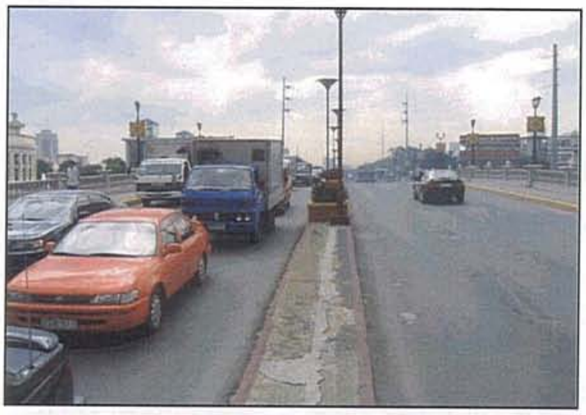


Photo 20.1.3-1 Road / Deck Level Inspection

Photo 20.1.3-2 Below Deck Level – Bottom Soffit of Steel Girders and Sway Bracings

Verification below deck level was taken at various locations of girders (see **Photo 20.1.3-2**). Since the section varies at different locations, several sections were measured, taking into considerations that the dimensions of external girders are different from internal girders. All girders were verified.

The substructure dimension for visible portions was also measured and verified. Unexposed portions of substructure were not able to be inspected. Shape and dimension measurement survey was conducted on bearings and the type of support was verified. Scour survey was also conducted under this activity.

(d) Reference Information

Basic drawings of Jones Bridge were furnished to the inspection teams and was used as reference in various activities and analysis of the structure.

(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 made easily using tape measure.

Verification of measurements below deck level required the use of tugboat with mounted scaffold (see **Photo 20.1.3-3**). Transferring of scaffolding was faster using this system. Each activity and inspected damages were supported with still photos and each dimension was recorded. Results of special test were also supplemented with photos. The bearings of piers and abutments were inspected with the aid of telescopic ladders and were measured using a caliper.

Photo 20.1.3-3 Tugboat with Scaffoldings, and Canoe - Equipment used in Below Deck Inspection

(f) Miscellaneous Structures

Non-structural elements such as pipes, lighting and other architectural accessories were noted and photographs were taken (see **Photo 20.1.3-4).**

Photo 20.1.3-4 Miscellaneous Utilities in Jones Bridge

(g) Results

Table 20.1.3-1 lists the drawings that summarize the data presented in the verification forms. **Figure 20.1.3-1** shows the lateral deformations of G1 and G8 girders. Dimensions that were shown in the drawings were utilized in structural model for improvement plan and presumption of original design. The detail drawings are shown in **Appendix 20.1.3-1 (6/8** to **7/8)**.

Maximum lateral deformation was follows:

Girder G1 = 115.0 mm (toward downstream side)
Girder G8 = 280.0 mm (toward downstream side)

Table 20.1.3-1 Lists of Drawings

Sheet No.	Title	Appendix
1	General Plan Elevation and Sections	20.1.3-1 1/8
2	Girder Elevation and Sections	20.1.3-1 2/8
3	Girder Elevation and Damage Details	20.1.3-1 3/8
4	Sway Bracing	20.1.3-1 4/8
5	Substructure with Bearings	20.1.3-1 5/8
6	Horizontal Deflection of Exterior Girders	20.1.3-1 6/8
7	Horizontal Deflection of Interior Girders (G2 & G6)	20.1.3-1 7/8
8	Main Girder Vertical Offset of Center Span (Span 2)	20.1.3-1 8/8

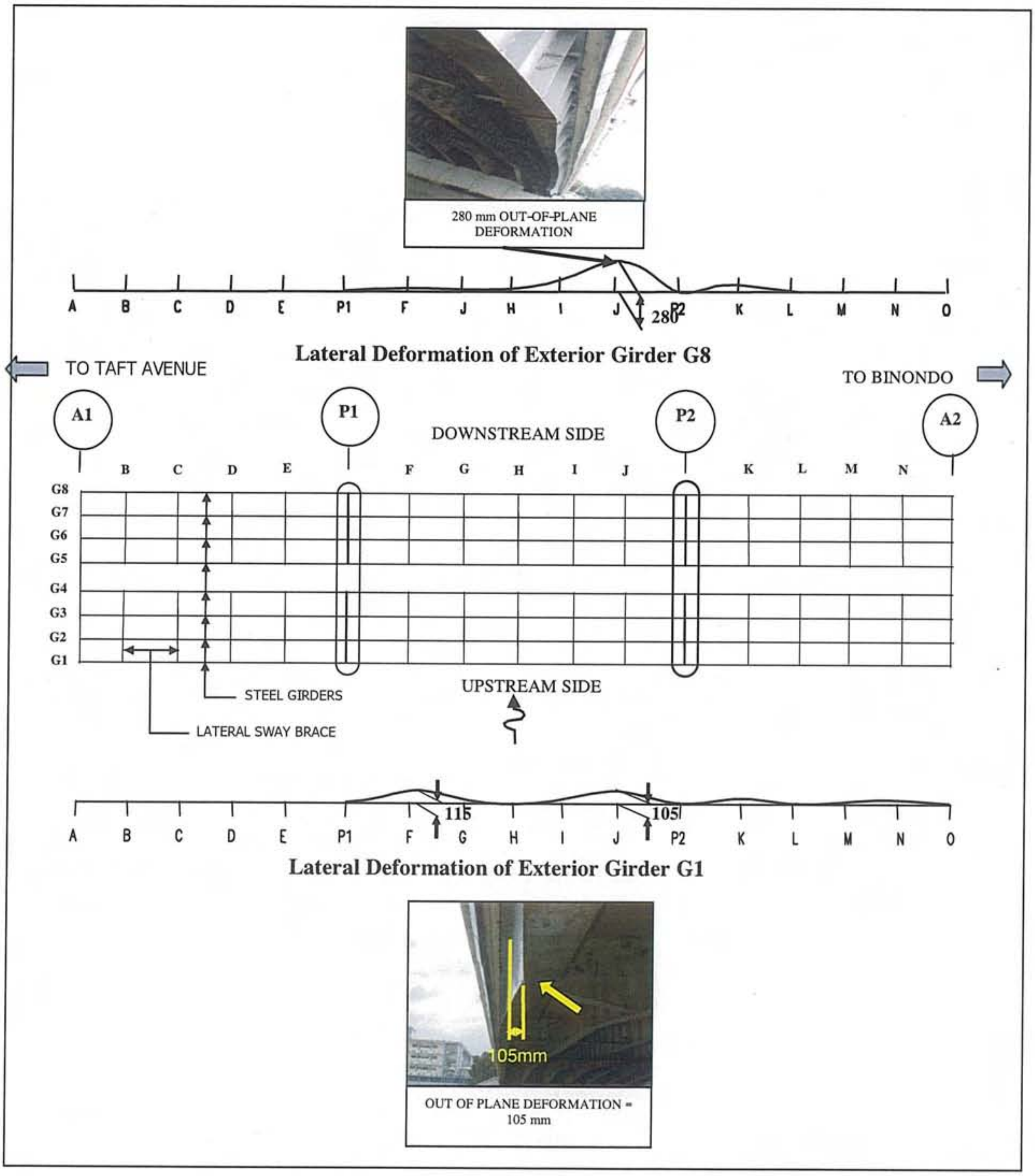


Figure 20.1.3-1 Lateral Deformation of Exterior Girders – Shapes and Dimensions

(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 inspection was conducted.

Available drawings were taken and used in planning for this activity. The X-Y-Z Method was used to obtain damage ratings. The damage ratings were used for determining the location of Non-Destructive Test of Material.

(b) Inspection Teams

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

(c) Coverage Area

The inspections cover the entire superstructure, road/deck, below deck level and substructure including bearings.

(d) Reference Information

The study team was furnished with copies of basic drawings of the bridge and used them as reference in various activities and analysis of the structure.

(e) Equipment and Procedure

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

(f) Criteria for Damage Rating

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

(g) Results

Survey results are as follows:

- There was no damage found on deck slab roadway of Jones Bridge except for some minor damages of concrete post and railing.
- Bearings on Abutment A1 and Pier P2 are heavily corroded.
- The exterior girders were heavily damaged or ruptured by collision of vessels.

- A ruptured girder on upstream side was once repaired but again damaged by vessel causing a 500 mm vertical rupture on web.
- Buckling of downstream exterior girder vertical stiffener are also noted.
- A large horizontal deformation on exterior girders was found as shown in **Figure 20.1.3-1**.
- The lower flange of exterior girder at point of bearing support has settled down by 50 mm, which has also revealed cracks after conducting non-destructive testing.
- Local buckling of sole plate and bottom flange at bearing positions are observed.
- The major damages of Jones Bridge are shown in **Figure 20.1.3-2**. The damage rating of main members based on Close-up Visual Inspection is shown on **Table 20.1.3-2.**

The damage sheets were documented in **Appendix 20.1.3-2 (1/11)** to **(11/11)**.

CORROSION OF BEARING SUPPORT
ABUTMENT A1 SIDE.
SECTION LOSS □ 10%

CRACK/FISSURE OF SWAY BRACING
G7-G8

GIRDER G8 OUT-OF-PLANE
DEFORMATION
DUE TO VESSEL COLLISION.

PIER 2 BEARING
CORROSION
SECTION LOSS<10%

GIRDER G1 RUPTURE DUE TO VESSEL
COLLISION. RUPTURED LENGTH=500mm,
HORIZONTAL DEFORMATION=105mm

BREAK/RUPTURE OF SWAY BRACING
(G1-G2)

50 mm OUT-OF-PLANE DEFORMATION
WITH CRACKS

10 mm OUT-OF-PLANE DEFORMATION

DEFORMATION WITH CRACK AT WELD
JOINT
CRACK LENGTH=460mm

Figure 20.1.3-2 Close-up Visual Inspection of Major Damages in Jones Bridge

Table 20.1.3-2 Damage Rating of Main Members by Close-up Visual Inspection

Component	Item	Member / Location	Damage Rating *1	Description
Superstructure	Shape and Dimension, Material Damage	Girder 1, Span 2	I	Girder 1 break rupture of bottom flange to 1/3 web depth and 105mm out-of-plane deformation due to vessel collision
	Shape and Dimension	Girder 8, Span 2	II	6 degrees (280mm) out-of-plane deformation due to vessel collision
	Material Damage	Bearings at A1 (G1-G8)	II	Corrosion of bearing support
	Shape and Dimension	Sway Bracing (G1-G2)	I	Break rupture of sway bracing
	Shape and Dimension	Sway Bracing (G7-G8)	I	Crack / break of sway bracing
	Shape and Dimension	Sway Bracing (G1-G2)	I	Break rupture of sway bracing
	Material Damage	Bearing at Pier 2	II	Corrosion of pier 2 bearing
	Material Damage	Girder 1 near A1	II	Corrosion of G1 bottom flange
	Material Damage	Girder 1, Span 2	I	Crack at welded joint of bottom flange
	Material Damage	P2 Bearing of G1	II	Crack on bearing shoe sole plate
	Shape and Dimension	Girder 8, Span 2	II	10 mm out-of-plane deformation
	Shape and Dimension	Girder 8, Pier 2	II	50 mm out-of-plane deformation
	Material Damage	Girder 8, Bottom Flange	I	Crack and deformation at welded connection of G8 bottom flange above P2 bearing
	Shape and Dimension	Girder 8, Span 3	II	50 mm web dent/deformation
Substructure	Material Damage	Pier Body P1	III	Vertical Crack
	Material Damage	Abutment Body A2	III	Horizontal Crack

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

(3) Non-Destructive Test of Material

(a) Objective

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

(b) Results

Table 20.1.3-3 shows the results of non-destructive test of material.

Table 20.1.3-3 Results of Non-Destructive Test

Test	Results	Reference Appendices
Ultrasonic Thickness Gauging (UTG) (To measure the present thickness of steel sections)	The measured bottom flange has thickness of 9mm to 15mm. The cover plate thickness ranges from 10mm to 16mm. The web of the girder measures 9mm to 11mm.	Appendix 20.1.3-3 (1/3 to 3/3)
Brinell Hardness Test (To Measure the hardness of the steel members)	Results of Brinell Hardness Numbers (BHN) ranged from 121 to 177. Corresponding to A370, the equivalent tensile strength is in the range of 438 Mpa to 580 Mpa.	Appendix 20.1.3-4
Dye Penetrant Test (DPT) (To detect any surface-breaking defects)	Results of Dye-penetrant test showed cracks in weld joint at main Girders G1, G7, and G8.	Appendix 20.1.3-5
Schmidt Rebound Hammer Test (To determine the in-situ uniformity, surface hardness, and approximate compressive strength of concrete)	Measured rebound numbers ranged from 20 Mpa to 40 Mpa.	Appendix 20.1.3-6
Compression Test (To obtain the compressive strength of concrete)	Results of the two samples were 25.3 Mpa and 23.0 Mpa.	Appendix 20.1.3-6
Phenolphthalein Test (To determine the depth of carbonation)	Two (2) cores were examined, value results on the depth of carbonation are 3mm and 15mm.	Appendix 20.1.3-6
Chloride Test (To assess the distribution of chlorides)	Results of the chloride test revealed chloride levels were not detected from the two (2) core samples.	Appendix 20.1.3-6
Petrographic Analysis (To test for alkali-silica reaction)	The findings of petrographic analysis have shown some evidence of cement-alkali reactions, including alkali-silica reaction. Additionally, ettringite (calcium sulfoaluminate hydrate) found in the core samples, could also be a source of distress to the concrete. Ettringite is a normal by-product of hydration of cement but some of it could be due to sulfate attack.	Appendix 20.1.3-6

(4) Special Test

(a) Microtremor Test

Objective

Microtremor measurement survey is conducted to determine the fundamental frequencies and natural dynamic characteristics of an existing bridge superstructure.

Acceleration Sensors

Microtremor surveys on the two (2) superstructures of Jones Bridge – the Lawton Bound lane and the Binondo Bound Lane – were conducted at sixteen (16) pre-selected locations on the bridge. The time histories and frequency spectra of the recorded events were analyzed to identify natural frequencies of each three (3) span superstructure and the corresponding mode shapes.

Figure 20.1.3-2 shows the location of the sensors positioned on the bridge.

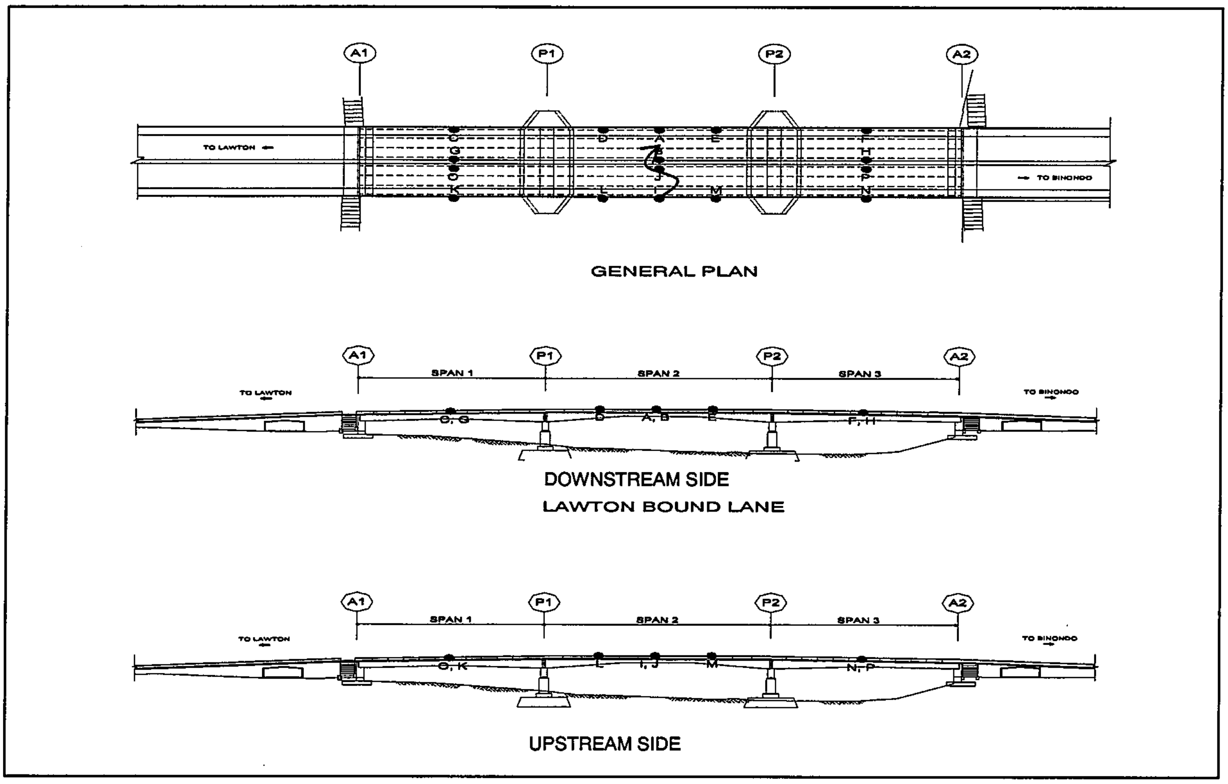


Figure 20.1.3-3 Location of Sensors on the Bridge during Conduction of Micro-tremor Survey

Most Probable Natural Frequencies of Superstructure

The most probable natural frequency of the 1st Vertical Mode is 2.40hz and the for the 1st Torsional Mode is 2.80hz. The results are compared with the analysis and tabulated **in Table 20.1.3-4.**

The result of analytical model using STRAND7 software and compared with the result above:

Table 20.1.3-4 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	2.40	1	2.30
Torsional	2.80	3	2.59

(b) Impact Vibration Test

Objective

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

Procedure

An impact pendulum on rigid arm was used to cause impact on Pier P2. The impact was positioned to excite the Pier at its north face, along the centerline of the median. The head of the pendulum had a mass of about 100kg and its tip was covered with rubber. Then, the pendulum was set-up to hit the pier approximately 1 meter from the top of pier.

Sensors and Locations

Vibration response of the pier due to each impact was recorded at two (2) pre-selected locations on top of the pier: one at the off center of the pier (see **Photo 20.1.3-4**), directly in front of the impact point, and the other, near one end of the pier. The primary mode of vibration of the pier in the longitudinal direction of the bridge was determined using the acceleration records at the center of the pier. Data from the sensors were acquired using sophisticated computer system as shown in **Photo 20.1.3-5**.

Photo 20.1.3-4 Sensor Location B, off center on top of Pier P2

Photo 20.1.3-5 Data acquisition system

Fourteen (14) individual data impact records were superimposed to form a stacked data for Pier P2. (See **Figure 20.1.3-4**). The natural frequency of the pier corresponds to the location of the spectral peak in the Amplitude Spectrum. From the amplitude spectrum and phase spectrum of the stacked data, the fundamental natural frequency of Pier P2 in the longitudinal direction is about **42 Hz.**

The results of analytical model from STRAND7 software analysis for Pier P2 and compared with the results from the impact test are tabulated in **Table 20.1.3-5**.

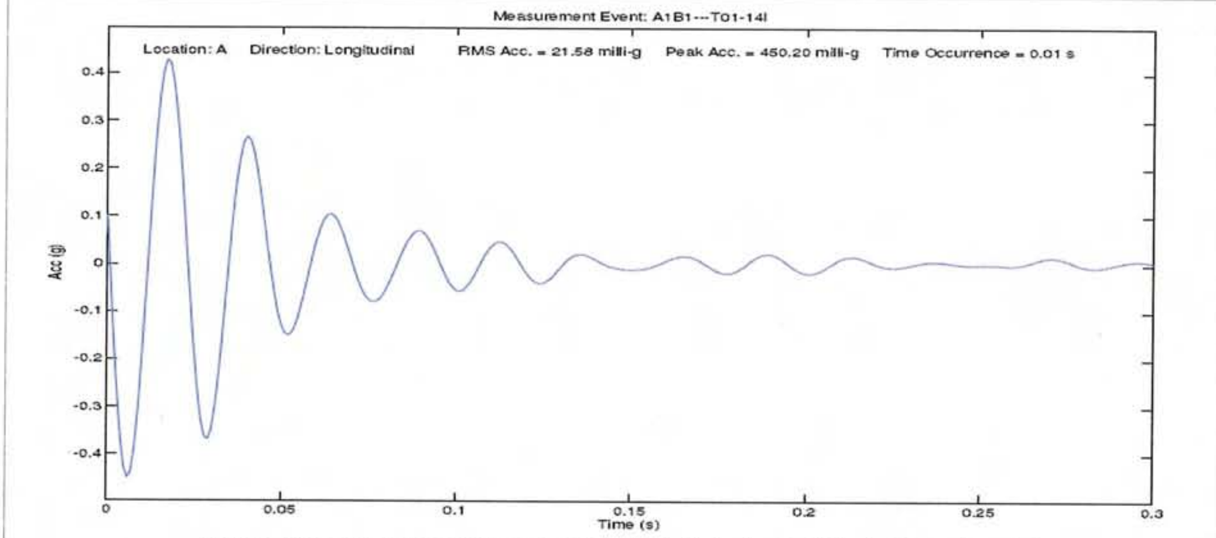


Figure 3. Time History (0.3-second segment) of longitudinal vibration for 14 Stacked Impact Records from Measurement Event A1B1---T01.

Figure 20.1.3-4 Time History of Longitudinal Vibration from Stacked Impact Records of Pier P2

Table 20.1.3-5 Structural Soundness of Pier P2

Mode Shape	Natural Frequency		Rating Index*	Remarks
	From Impact Test (Hz)	From STRAND7 Analysis (Hz)		
Longitudinal	42.00	41.517	1.012	(>0.85) OK

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

(5) Assessment of Critical Damages

(a) Evaluation Criteria

Damages of bridge members were inspected under the close-up visual inspection and non-destructive test of material were identified and evaluated in compliance with the procedure set forth in **Section 13.3.7, Chapter 13.**

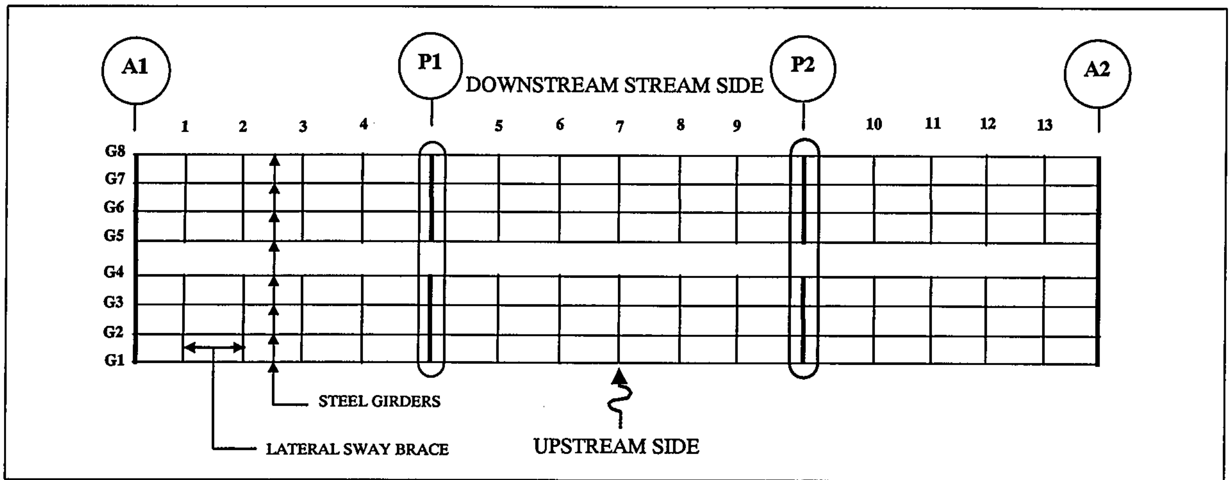
(b) Damage Rating with X, Y, Z Method

Damage rating follows the procedures and criteria set forth in **Section 13.3.7, Chapter 13.**

Evaluation of Damages

Evaluation results on damages of main members were summarized in **Table 20.1.3-6.**

Table 20.1.3-6 Evaluation of Major Damages on Jones Bridge



	Location		Evaluation Based on Field Survey				Evaluation Based on Non-Destructive Test	Evaluation Based on Special Test
	Member	Node	Damage Type	Damage Rating	Diagnostic	Estimated Section Loss		
Superstructure	G1, Span 2	9	B/R	I	A	40%	• Ultrasonic Thickness Gauging - **No Defects** • Brinell Hardness Test – **No Defects, Tensile Strength Conforms to A36 Materials** • Dye Penetrant Test – **Crack at Weld Joint of Main Girders G1 Length = 10.5", G7 Length = 9", G8 Length = 7"**	• Micro-tremor Test - **OK**
	G8, Span 2	8-9	DE	II	B	<10%		
	G1, near A1	1	CO	II	B	<10%		
	G1, Span 2	6-7	CR	II	B	<10%		
	G8, Span 2	6-7	DE	II	B	<10%		
	G8, Pier 2	9-P2	CR, DE	I	A	<10%		
	G8, Span 3	10	DE	II	B	<10%		
	Sway Brace, G1-G2	5, 9	B/R	I	A	50%		
	Sway Brace, G7-G8	7	CR, B	I	A	50%		
	Bearings at A1	A1	CO	II	B	<10%		
	Bearings at Pier P2	P2	CO	II	B	<10%		
	Bearing of G1 at Pier P2	P2	CR	II	B	<10%		
Substructure	Abutment A1		OK			-	• Schmidt Rebound Hammer Test – **OK** • Compression Test - **OK, fc' Range 260 to 410kg/cm² (25.5 – 40.21 Mpa)**	-
	Pier P1		CR	III	B	-		Impact Vibration Test **Rating Index = 1.012> 0.85 (OK)**
	Pier P2		OK			-		
	Abutment A2		CR	III	B	-		-

1 – See Item 4.10 of Manual for Diagnosis of Damages * See **Figure 20.1.3-5** For member location reference

20.1.4 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 for the analysis model of the Load Rating.

(2) Structural Shapes and Dimensions

<u>**Superstructure**</u>

The structural members of Jones Bridge are all measurable and can easily be inspected. The superstructure does not need to be assumed because all the dimension details can be measured. **Appendix 20.1.4-1 (1/17 to 10/17)** show shapes & dimensions and details of superstructure. **Appendix 20.1.4-1 (11/17 to 17/17)** show the section properties of steel girders used in the analysis.

<u>**Substructure**</u>

Pier bodies were also measurable except for the existing caisson foundation that supports the piers. The dimensions of exposed portion of the substructure were all measured. Size of existing caisson and reinforcement of the pier was obtained from the Construction Drawings furnished by the DPWH. From these, the structural member shapes and dimensions were verified and compared to the data gathered during visual inspection and measurement of the bridge.

Appendix 20.1.4-2 (1/8 to 8/8) shows the verified shapes, dimensions and detail of substructure.

(3) Structural Soundness (LOAD RATING)

The bridge superstructure was modeled using the STAADIII Finite Elements Analysis System. An analysis of Load Effects was performed. MS 18 Load was used in the Live Load Analysis.

Live load rating factors are calculated at two levels: inventory and operating levels. The allowable stress that was adopted for inventory level evaluation was 125 MPa. For operating level evaluation, it is 170 MPa.

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

<u>**Analysis Results**</u>

The load ratings calculated are tabulated on **Table 20.1.4-1.**

At section no. 11 of Exterior Girder G1, it shows that the section is no longer capable of carrying any live load and has a diminished capacity to dead load. However, since the bridge consists of multiple girders, the member redundancy leads to redistribution of force demand to the adjacent girders and sections. The force redistribution is taken in the load rating calculation for interior girder G2 which, as indicated in the Table, is still capable of supporting the live load forces redistributed by the lack of capacity of girder G1. Also, at section no. 15 of Exterior Girder G1, it shows a load rating of 0.88 which is less than 1.0. This is because of the redistribution of forces from the damage section at Section No. 11 to the adjacent span.

The exterior girder with an out-of-plane deformation is checked for load rating based on a 3D grid model of the bridge. Although most of the load carried by this girder is due to sidewalk, it is observed that vehicle live load on the carriage way distributes part of the demand forces on this girder. The load rating results that the exterior girder is still capable of supporting the AASHTO HS20-44 live load passing over the bridge. However, the out-of-plane deformation may cause girder instability (lateral buckling) if the cross/sway bracings are not fixed.

The complete calculation of load rating is presented in **Appendix 20.1.4-3 (1/24 to 24/24).**

(4) Vulnerability to Disaster

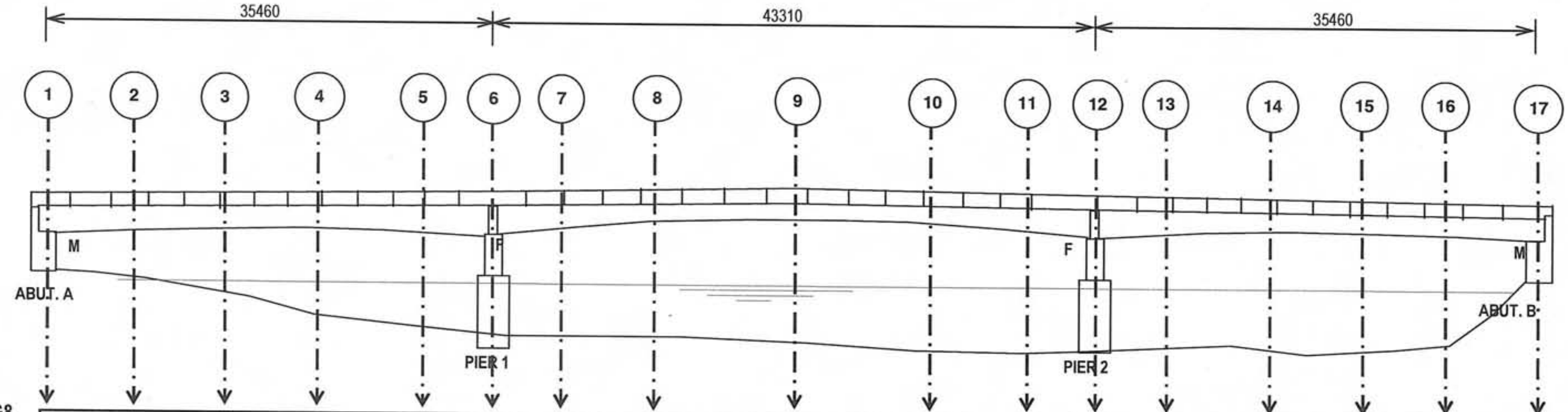
(a) Earthquake

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

Bridge Site

Jones Bridge is located 14.5km from the Marikina Fault Valley System (MVFS). As a rule, bridge structures less than 5km distance are considered highly vulnerable. The 14.5km distance of Jones Bridge makes it moderately vulnerable to earthquake. The type of soil and its response characteristics will have to be properly evaluated and considered in the design of strengthening.

Table 20.1.4-1 Load Rating of Superstructure – Inventory Level (Allowable Stress Rating)



Girder		1	2	3	4	5	6	7	8	9	10	11	12	13	14	15	16	17
G8	RF	6.3	3.153	1.13	2.71	3.649	4.70	5.251	6.196	2.56	7.474	3.986	4.835	5.26	2.70	1.21	3.089	6.19
	LL	206.5	103.1	37.1	88.6	119.3	153.7	171.7	202.6	83.7	244.4	130.3	158.1	172.0	88.3	39.5	101.0	202.4
G7	RF	3.0	2.849	2.32	2.02	3.068	3.08	3.408	3.853	2.231	3.817	3.358	3.41	2.968	1.99	2.29	2.67	3.13
	LL	99.2	93.2	75.9	66.2	100.3	100.8	111.4	126.0	73.0	124.8	109.8	111.5	97.0	65.2	74.9	87.3	102.4
G6	RF	2.7	2.382	2.11	1.49	2.7	2.51	2.861	3.261	2.37	3.286	3.321	2.492	2.688	1.48	2.09	2.246	2.71
	LL	87.3	77.9	68.9	48.8	88.3	82.0	93.5	106.6	77.5	107.4	108.6	81.5	87.9	48.5	68.5	73.4	88.6
G5	RF	2.3	1.623	1.51	1.17	2.079	2.16	2.34	2.327	1.743	2.322	2.324	2.156	2.071	1.15	1.51	1.385	2.17
	LL	75.9	53.1	49.3	38.2	68.0	70.6	76.5	76.1	57.0	75.9	76.0	70.5	67.7	37.5	49.4	45.3	70.9
G4	RF	2.0	1.541	1.51	1.10	1.87	2.17	2.344	2.263	1.735	2.341	2.45	2.289	2.069	1.17	1.47	1.583	2.28
	LL	66.5	50.4	49.5	35.9	61.1	71.0	76.7	74.0	56.8	76.5	80.1	74.8	67.6	38.2	48.1	51.8	74.5
G3	RF	2.6	2.326	2.11	1.50	2.691	2.49	2.835	3.24	2.35	3.183	2.775	2.44	2.608	1.43	2.07	2.336	2.84
	LL	83.9	76.0	69.1	49.0	88.0	81.4	92.7	106.0	76.8	104.1	90.7	79.7	85.3	46.7	67.8	76.4	92.8
G2	RF	2.9	3.006	2.23	1.99	2.909	3.13	3.449	3.824	2.057	2.922	2.563	2.35	2.79	1.81	2.09	2.985	3.09
	LL	94.0	98.3	72.9	65.2	95.1	102.4	112.8	125.0	67.2	95.6	83.8	76.7	91.2	59.1	68.3	97.6	101.0
G1	RF	8.2	5.267	1.03	2.47	3.117	4.14	4.83	9.481	2.214	10.19	0.0	7.156	4.80	2.16	0.88	2.631	7.99
	LL	267.3	172.2	33.7	80.7	101.9	135.5	157.9	310.0	72.4	333.1	0.0	234.0	156.9	70.7	28.6	86.0	261.3

Note: RF – Rating Factor
LL – Equivalent Truck Loading in tons

Construction Details

The existing superstructure is supported by wall-type pier. This type of support could easily attain collapse mechanism due to lack of support redundancy. Pier Wall is connected to the existing caisson foundation by 1" diameter dowel bars (see **Figure 20.1.4-1**). Large seismic forces cannot be resisted by this connection.

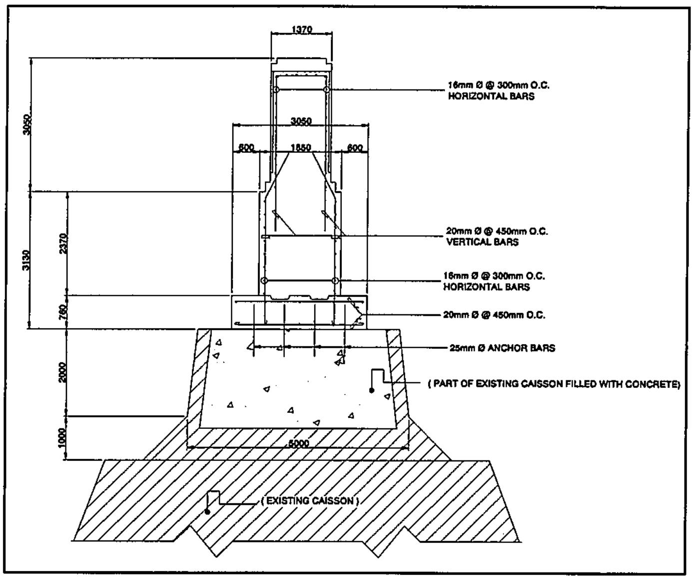


Figure 20.1.4-1 Detail of Existing Pier Wall

Structural Configuration

The regular configuration of Jones Bridge is structurally favorable.

Date of Construction

Jones Bridge was constructed on 1948. Before and during those times AASHTO have no recommendations with regards to seismic designs. Therefore, analysis should be made using the latest code requirements.

Analysis Results

Results of the structural analysis made for the substructure of Jones Bridge showed that the Piers are still well within the limit of their capacities based on the Original Code that was used on the design. However, when applying the Latest Code, it is very relevant that the capacities are well below the limit. Though the support condition at top of Pier P2 became fixed, this has not reduced the tendency of the bridge to attract seismic forces. Also, the existing piers were constructed on top of the caisson which was built on 1916. The detail connection of the existing caisson and pier is more to be treated as hinged in nature. This means that the abutment will be on its maximum capacity when seismic occurs as the piers will only attract not so large seismic forces. However, to consider abutments to attract more seismic forces is not justifiable. Thus, fix condition at the bottom of the piers and allowable springs at abutments were considered in the analysis.

A more significant change in the Latest Code has rendered the piers very weak under seismic forces. Based on the original code used in the design of the piers, a factor of 0.04W seismic forces were utilized as compared to the much sophisticated method used in latest code which uses Multi-mode Spectral Analysis. The results of analysis considering the original code and latest code are tabulated in **Table 20.1.4-2** below:

Table 20.1.4-2 Capacity-Demand Ratio of Substructures of Jones Bridge

Member	Capacity / Demand Ratio			
	Using Old Code	Remarks	Using Latest Code	Remarks
Pier P1	7.00	Safe	0.543	Fail
Pier P2	11.71	Safe	0.643	Fail
Pier P1 Caisson	1.56	Safe	0.900	Fail
Pier P2 Caisson	1.52	Safe	0.890	Fail

It is clearly seen from the above table that the substructure needs improvement to conform with the latest code requirements. The calculations for the above assessment are presented in **Appendix 20.1.4-4 (1/28 to 28/28)**

(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 Jones Bridge is located was recorded in 1995 with a gust velocity reaching to 255 kph. This indicates that Jones Bridge has been exposed to more than 200 kph basic design wind speed specified in the Philippine Code. Therefore Jones Bridge 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.

(d) Special Issues

Vessel Collision

As mentioned in Section 13.6, 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

  The regulated vertical clearance by PCG is 3.75m. The actual vertical clearances of Jones Bridge are 3.6m near the piers and 4.8m at center span. It was evident that the clearances near the piers are prone to vessel collisions as concluded in the evaluation.

- Vessel Collision with Pier

  Though the space of 43.4 meters between piers is more than the preferable distance of 43.0 meters, it is the vertical clearance near the piers that pose a continuous threat to passing vessels at Jones Bridge.

Utilities

Below are the existing utility lines in Jones Bridge:

1) 40 - ϕ 100mm Telecommunication PVC Pipe
2) 4 - ϕ 100mm GI Pipe Electrical Line
3) 1 - ϕ 340mm GI Pipe Water Line

Informal Settlers

There are no informal settlers under Jones Bridge. Therefore, there will be no problem in the implementation of the project.

20.1.5 Overall Assessment of Bridge Condition

The present state of Jones Bridge is assessed based on the following informations:

(1) Superstructure

Major Damage Description and Causes

- Rupture of bottom flange up to $^1/_3$ depth of web of Exterior Girder G1 (upstream) and sway bracing near Pier P2,
- Deformation and rupture of sway bracings due to vessel collision,
- Large lateral deformation of Exterior Girders at Upstream and downstream sides due to vessel collision,
- Cracks at well joint connections of main girders G7 and Exterior Girders G1 and G8 due to impact from frequent vessel collision,
- Corrosion of bearing shoes at Pier P2 and Abutment A1 due to poor maintenance and water leakage at expansion joints,
- Missing top members of sway braces due to the passage of telecom pipes and accessories,
- Missing bolts at girders due to frequent vessel collision.

As shown in **Table 20.1.4-1,** Exterior Girder G1 has no more live load structural capacity at portion on near Pier P2.

(2) Substructure

- As shown in **Table 20.1.4-2** measures against earthquake, the existing piers are not sound to carry the design loads based on latest code, Pier 1 with C/D = 0.543 and Pier 2 with C/D = 0.643.
- The stability of foundations are not enough to carry the design load based on latest code, Pier 1 caisson with C/D = 0.900 and Pier 2 caisson with C/D = 0.890.
- The lack of stability is caused due to the change of design code, especially the design requirement.

(3) Social Environment

- As mentioned earlier in Item 3, Section 20.1.1, the National Historical Institute declared Jones Bridge as one of the Historical Bridges and strongly endorses the policy that the configuration of the bridge should be preserved
- Vertical clearance to prevent vessel collision shall be improved at least up to regulatory vertical clearance of 3.75m near pier supports.
- No serious dislocation of people will be affected by the improvement works were observed.

(4) Conclusion

The improvement work of superstructure of Jones Bridge should be done as to prevent the frequent exterior girders from vessel collisions. The present damage at exterior girder G1 and G8 need a much improvement work immediately because it poses danger not only to the public but also to the vessels passing under the bridge.

The following measures were deemed necessary to improve the existing condition of the Jones Bridge:

- Additional girders should be installed adjacent to the existing exterior girders to take its structural functions.
- The existing ruptured (G1) and deformed (G8) exterior girders should be repaired and reinstalled to function as vessel collision protection of the superstructure. In addition, retention of the exterior girders is consistent with NHI's policy of preserving the bridge's original configuration.
- Restoration of ruptured sway braces are necessary.
- Missing bolts and sway brace members should be replaced.
- Corrosion of steel members particularly bearing shoes at top of Pier 2 and Abutment A1 should be eliminated to prevent further deterioration.
- Pier walls and foundation based on the structural analysis made does not conform with the latest code requirements. A retrofit measure is necessary to strengthen the substructure to meet the latest code requirement.

Table 20.1.5-1 summarized the overall assessment of existing bridge.

Table 20.1.5-1 Overall Assessment of Existing Condition of Jones Bridge

Items			Member/Location		Damage Rating	Damage Condition	Diagnosis (IW or FI)
Structural Soundness	Superstructure	Shape/Dimension	Span 1	Main Exterior Girder, G8 near Pier 1	II	Maximum horizontal deformation is 10mm, span length 43,620mm, 10mm < 43,620/125 = 350mm shall be FI.	FI
			Span 2	Main Exterior Girder, G1, Span 2	I	Broken of lower flange and 1/3 of web height, girder under side walk, dangerous to the third party.	IW
				Exterior Girder, G1, Span 2, Welded Connection	I	The improvement work shall be carried out even for minute cracks.	IW
				Main Exterior Girder, G8, near P2	II	Maximum horizontal deformation is 280mm span length 43,620mm, 200mm < 43,620/ 125 = 350mm shall be FI.	FI
				Main Exterior Girder, G8, near Pier 2	II	Maximum horizontal deformation is 50mm, span length 43,620, 50mm < 43,620/125 = 350mm shall be FI.	FI
				Main Exterior Girder, G6 near P2	II	Maximum horizontal deformation is 5mm, span length 43,620mm, 5mm < 43,620/125=350mm shall be FI.	FI
				Main Exterior Girder, G8	I	Steel cracks was found at welded portion. The improvement work shall be carried out even for minute cracks.	IW
				Sway Bracing/ G1-G2	I	Broken of sway bracing, improvement work shall be carried out.	IW
				Sway Bracing/ G7-G8	I	Steel cracks was found at welded potion.	IW
			Span 3	Main Exterior Girder, G8, near Span 3	I	Maximum horizontal deformation is 50mm, span length 35,585mm, 50mm < 35,585/125 = 285mm shall be FI.	FI
		Material/Damage	Span 1	Main Exterior Girder, G1, Bottom of flange web connection, near A1	II	Rust scattered and generated exclusively is observed but section loss is small, less than 10% of the thickness of plate.	FI
				Main Exterior Girder , Bottom flange of G1-G8 (Span 1, 2, 3)	II	Rust scattered and generated extensively is observed, section loss is small, < 10%.	FI
				Main Interior Girder/ G2 near A1	II	Rust scattered and generated extensively is observed, section less is small, < 10%.	FI
				Main Interior Girder/ G3 at A1	II	Rust scattered and generated extensively is observed, section less is small, < 10%.	FI
				Main Interior Girder G1 near A1	II	Rust scattered and generated extensively is observed, section less is small, < 10%.	FI
			Span 2	Main Exterior Girder , Bottom flange of G1-G8 (Span 1, 2, 3)	II	Rust scattered and generated extensively is observed, section loss is small, < 10%.	FI
			Span 3	Main Exterior Girder , Bottom flange of G1-G8 (Span 1, 2, 3)	II	Rust scattered and generated extensively is observed, section loss is small, < 10%.	FI
				Main Interior Girder G4 near A2	II	Rust scattered and generated extensively is observed, section less is small, < 10%.	FI
			90 Test locations for UTG		Ok	Flange thickness = 9mm to 15mm; Web thickness = 10mm to 16mm	-
			59 Test locations Brinell Hardness		Ok	BHN = 121 to 177; Tensile Strength = 438 Mpa to 580 Mpa	-
			5 Test locations for Dye Penetrant Test		I	Damaged members are found with cracks	IW
		Operating Level	Exterior Girder G1		0.76	Ruptured Girder	IW
		Assessment of Superstructure	**Exterior Girders should be repaired and improved. Also, corrosion should be eliminated and sway bracing be restored and rehabilitated.**				
	Substructure	Shape/ Dimension	Bearing Body/ A1		Ok	Honeycomb at abutment and corrosion of bearing found	-
			Pier 1 and P2		Ok	Vertical crack at pier and corrosion of being found	-
		Material/ Damage	Bearing Body at Abutment1		II	Rust scattered and generated extensively is observed, section loss is small, less than 10%.	FI
			Bearing at Pier 2		II	Rust scattered and generated extensively is observed, section loss is small, less than 10%.	FI
			27 Test locations for Schmidt Hammer		Ok	Compressive Strength = 20 Mpa to 40 Mpa	-
			2 Test locations for Coring and Compression Test		Ok	Core Samples = 340mm and 380mm; Comp. strength = 23.0Mpa and 25.3Mpa	-
			2 Test locations for Phenolphthalein Test		Ok	Depth of Carbonation = 3mm and 15mm	-
			2 Test locations for Petrographic Analysis		Ok	No evidence of alkali-silica reaction	-
		Strength of Pier Body	Pier P1 Column		Not Ok	Insufficient strength of pier body	IW
			Pier P2 Column		Not Ok	Insufficient strength of pier body	IW
		Assessment of Substructure	**Piers are not sound and should be retrofitted to conform with the requirements of latest code**				
	Foundation	Structure/Shape	Pier P1 Caisson of 31.8m x 10.2m		Not Ok	Length of caisson not clear in as-built drawings	FI
			Pier P2 Caisson of 31.8m x 10.2m		Not Ok	Length of caisson not clear in as-built drawings	FI
		Scouring	No exposed foundation		Ok	No scouring occurred	-
		Bearing Capacity/ Stability	Pier P1		Not Ok	Length of caisson foundation not clear	
			Pier P2		Not Ok	Length of caisson foundation not clear	
		Assessment of Foundation	**Footing is not sound and should be retrofitted to conform with the requirements of latest code**				
	Assessment of Structural Soundness		**Retrofit of Substructure is recommended to conform with the requirements of the latest code**				
Traffic Function	Vehicle Weight Limitation Use		RF = 0.0 (Ruptured Exterior Girder)				
	LOS		D (v/c = 0.723)				
	Geometrical Features		Fair including approach road				
	Safety of Vessel Transport		At Piers P1 and P2, 3.6m < 3.75 insufficient, Collision Protection for Girder and Pier needed				
	Assessment of Traffic Function		Insufficient				
Social Environment	Utilities Hanged at the Bridge		40- ϕ 100 mm Telecommunication Pipe, 40- ϕ 100 mm Electrical Line, 1- ϕ 340 mm Water Line				
	Squatters		No families live around the Jones Bridge				
	Historical Aspects		NHI declared Jones Bridge as one of the Historical Bridges.				
	Assessment of Social Aspects		**NHI poses no objection to rehabilitate Jones Bridge as long as the basic configuration is retained**				
Overall Assessment			**Repair of existing Exterior Girders and sway braces, Retrofit works for substructures are recommended**				

20.2 COMPARATIVE STUDY ON REHABILITATION METHOD

20.2.1 Proposition of Rehabilitation Method

Three (3) schemes were proposed and compared for the best possible rehabilitation scheme as per criteria set forth in **Chapter 14**. These 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

In this scheme, repair of damaged members and sections were given emphasis. Major works include:

- Cleaning and painting corroded sections
- Repair of ruptured sway/bracing
- Repair of ruptured girder (web and lower flange) by steel plate patching
- Repair of sole plate and girder bottom flange at bearing location by partial replacement

From the above scope of works, it will be noted that only immediate measure for damaged sections were given attention. Partial section replacement will only improve local area but the out-of-plane girder deformation will not be corrected. Thus, stability is not assumed.

Also, the following considerations were also noted:

- Least expensive
- Shortest construction period (6 months)
- Temporary closure is needed during repair of bearing supports and ruptured girder.

No improvement on the navigation clearance is rendered on this scheme.

(2) Medium-Scale Rehabilitation

Major works included in this scheme are almost similar to Small-Scale Rehabilitation except that it involves cutting and section replacement. Unlike Small-scale Rehabilitation which will only involve repair by patching local area, this scheme takes into consideration the entire damaged section.

Construction period on this scheme is longer than small-scale rehabilitation. The following items were considered in the formulation of this scheme:

- Structural stability is much better by full section replacement.
- Girder out-of-plane deformation is partially corrected.

- Existing traffic functionality is also maintained.

Construction cost is higher than scheme 1 because of much needed material replacement and labor force.

(3) Large-Scale Rehabilitation

This scheme considered the installation of additional steel girders at each side of the bridge to replace the structural function of the two existing girders. Major improvement works are involved in the formulation of this scheme. The following advantages were given credence and deemed very important as to the structural stability, navigation clearance and historical aspect consideration:

- Girder out-of-plane deformation is finally eliminated
- The most structurally stable among all schemes
- Existing exterior girders will serve as vessel collision protection
- The most durable among the three scheme.
- The exterior girders will preserve the original configuration of the bridge but its structural function will be taken by the new additional girders.

20.2.2 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.

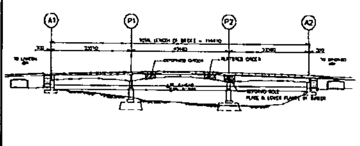
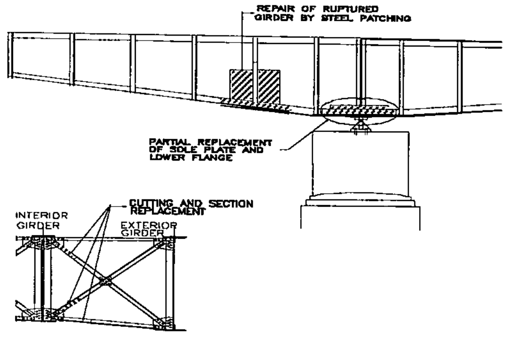
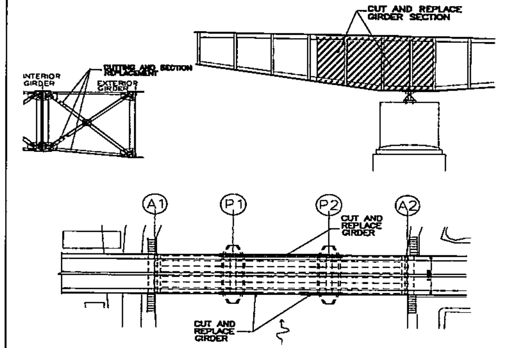
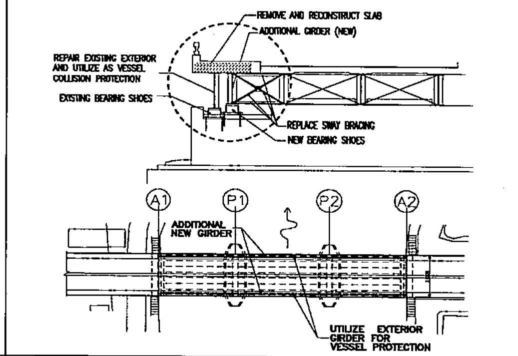
As the results compared in **Table 20.2.2-1**, the **large-scale rehabilitation scheme** was recommended as the best scheme in terms of engineering aspects.

20.2.3 Lifecycle Cost Analysis of the Bridge

(1) Procedure

Based on the bridge condition survey mentioned in **Section 20.1** and engineering study made in **Section 20.2.1**, the life cycle cost (LCC) analysis of the Jones 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.**

Table 20.2.2-1 Comparative Study of Possible Schemes for Jones Bridge

IMPROVEMENT TYPE		PROPOSED SCHEMES					
		SCHEME 1 : SMALL SCALE REHABILITATION REPAIR OF DAMAGED MEMBERS AND SECTIONS		SCHEME 2 : MEDIUM SCALE REHABILITATION REPAIR/REPLACEMENT OF DAMAGED GIRDER SECTIONS		SCHEME 3 : LARGE SCALE REHABILITATION ADDITIONAL GIRDERS	
FIGURES		 LOCATION OF SIGNIFICANT STRUCTURE DAMAGE					
		 REPAIR OF RUPTURED GIRDER BY STEEL PATCHING; PARTIAL REPLACEMENT OF SOLE PLATE AND LOWER FLANGE; CUTTING AND SECTION REPLACEMENT; INTERIOR GIRDER; EXTERIOR GIRDER		 CUT AND REPLACE GIRDER SECTION; CUTTING AND SECTION REPLACEMENT; INTERIOR GIRDER; EXTERIOR GIRDER; A1; P1; P2; A2; CUT AND REPLACE GIRDER		 REMOVE AND RECONSTRUCT SLAB; ADDITIONAL GIRDER (NEW); REPAIR EXISTING EXTERIOR AND UTILIZE AS VESSEL COLLISION PROTECTION; EXISTING BEARING SHOES; REPLACE SWAY BRACING; NEW BEARING SHOES; A1; P1; P2; A2; ADDITIONAL NEW GIRDER; UTILIZE EXTERIOR GIRDER FOR VESSEL PROTECTION	
MAJOR WORKS		• Cleaning and Painting Corroded Sections • Repair of Ruptured Sway Bracing • Repair of Ruptured Girder (Web and Lower Flange) by Steel Plate Patching • Repair of Sole Plate and Girder Bottom Flange at Bearing Location by Partial Replacement	RATING	• Cleaning and Painting Corroded Sections • Repair of Ruptured Sway Bracing • Repair of Ruptured Girder by Cutting and Section Replacement • Repair of Sole Plate and Girder Section at Bearing Location by Cutting and Section Replacement	RATING	• Cleaning and Painting the Steel Structures for the Whole Bridge • Replacement of Ruptured Sway Bracing • Provide Additional Girder with New Bearing Shoes Next to Existing Exterior Girder • Repair and Retain Existing Exterior Girder to Function as Vessel Collision Protection • Remove and reconstruct existing deck slab, sidewalk, railing and expansion joint	RATING
(1) STRUCTURAL ASPECT (30%)	STABILITY	• Basic structural repair and maintenance to provide immediate measure for damaged sections • Girder out-of-plane deformation is not corrected thus stability is not assured • Partial section replacement improves only local area	E	• Full section replacement is structurally better than Scheme 1 • Girder out-of-plane deformation is partially corrected at replaced sections • Needs partial reconstruction of sidewalk • Girder section repair/replacement at bearing area is structurally more stable than Scheme 1	D	• Structural capacity and stability is best among three schemes • Additional girder augments the capacity and stability of the superstructure and eliminates out-of-plane girder deformation • Existing exterior girder can be utilized as vessel collision protection without transmitting impact forces to the superstructure	B
	DURABILITY	• Least durable among schemes • Needs periodical painting works		• Full section replacement is more durable than Scheme 1 • Needs periodical painting works		• Highest degree of durability among the three schemes • Existing exterior girder will protect main bridge structure from vessel collision	
(2) CONSTRUCTION ASPECT (30%)	COST	• PhP 46 Million • Least Expensive	B	• PhP 103 Million	D	• PhP 161.80 Million • Cost is the most expensive	D
	DURATION	• Shortest at Six (6) months		• Twelve (12) months		• Longest at eighteen (18) months	
	METHOD / DIFFICULTY	• Difficult to execute, temporary works and temporary supports needed when cutting and replacing sections, especially replacement of bearing supports • Removal of deck and sidewalk not necessary		• Partial removal of deck and sidewalk necessary • Temporary supporting girders necessary during replacement of steel members/girders. Method of providing additional temporary support becomes very difficult for stability during construction		• Removal of existing deck and sidewalk necessary (about 3.4m)	
(3) TRAFFIC / NAVIGATION FUNCTIONALITY (20%)		• Existing condition for traffic functionality will be maintained • No improvement on navigation vertical clearance. Vessel collision with exterior girder still possible.	D	• Existing condition for traffic functionality will be maintained • No improvement on navigation vertical clearance. Vessel collision with exterior girder still possible	D	• Existing condition for traffic functionality will be maintained • Navigation clearance for bridge girders improved due to shallower depth of new girder • Additional protection for vessel collision provided by existing exterior girder	A
(4) IMPACT TO TRAFFIC DURING CONSTRUCTION (15%)	AFFECTED LANES	• Temporary closure necessary during repair of bearing supports • Outer lanes will be disturbed by staging works	C	• Outer lanes will be disturbed by staging works • Other 2 lanes provided by detour bridge	B	• Only 2 lanes can be used during construction • Other 2 lanes provided by detour bridge	B
	LOAD CAPACITY REDUCTION	• Outer lane vehicle load will be limited during repair of ruptured girder		• Vehicle load will be limited during repair of ruptured girder • Detour bridge can carry heavy vehicles		• Vehicle load will be limited during construction • Detour bridge can carry heavy vehicles	
	DETOUR	• Not necessary but needs temporary closure		• Necessary		• Necessary	
(5) SOCIAL / ENVIRONMENTAL IMPACT (5%)		• Painting works will have considerable impact to river • Cutting and welding works will have considerable impact to river	C	• Painting works will have considerable impact to river • Cutting and welding works will have considerable impact to river	C	• Painting works will have major impact to river • Minimal cutting and welding works has minimal impact to river	C
OVERALL EVALUATION AND RATING		2	2.50	3	2.35	1 (RECOMMENDED)	3.55

(2) Jones 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 Jones Bridge are as follows:

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

Many vessels had collisions with the superstructure and substructure of Jones Bridge

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

In 1992, rehabilitation of deck was made

The deterioration curve of the Jones Bridge is estimated and shown in **Figure 20.2.3-1**.

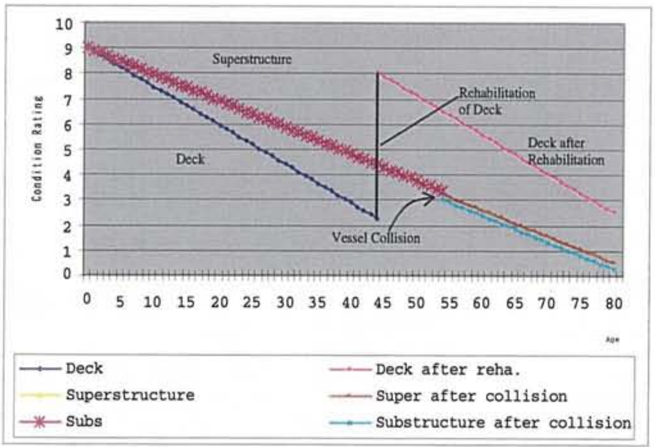


Figure 20.2.3-1 Deterioration Curve of Jones Bridge

(3) Rehabilitation Schemes and Cost Estimates

The engineering study proposed the possible rehabilitation schemes and cost estimates which were shown in **Table 20.2.2-1**

(4) Lifecycle Cost Analysis Model

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

a. Replacement case
b. Rehabilitation case

The explanation of the lifecycle cost analysis model for these two (2) cases were already described in **Section 14.3**. The same model is used for this analysis.

(5) Extended Service Life by Improvement Proposals

Using the deterioration curve in **Figure 20.2.3-1**, the relationship between investment cost and improvement condition rating shown in **Figure 14.3.5-1** in **Section 14.3.5**, the expected extended service life of Jones Bridge is calculated and shown in **Figure 20.2.3-2** and **20.2.3-3**. The service life of the Bridge is varied to extend by type of improvement.

If large scale rehabilitation is implemented, the service life of the bridge is expected to extend 31 years so total service life will be 36 years from 2007.

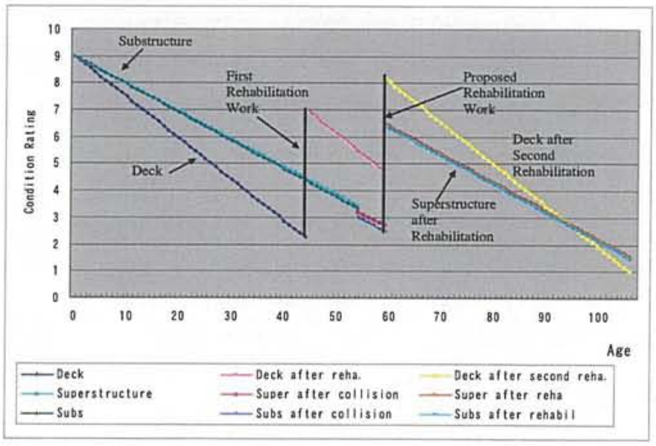
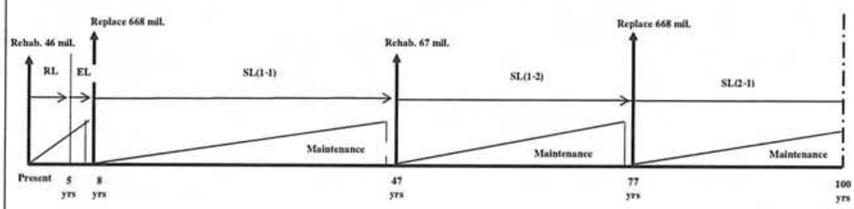
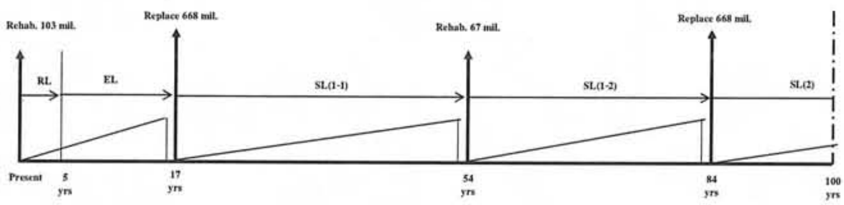
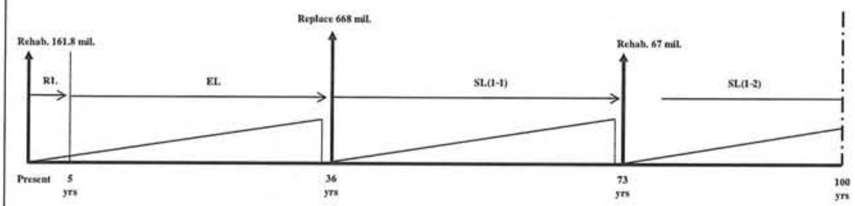
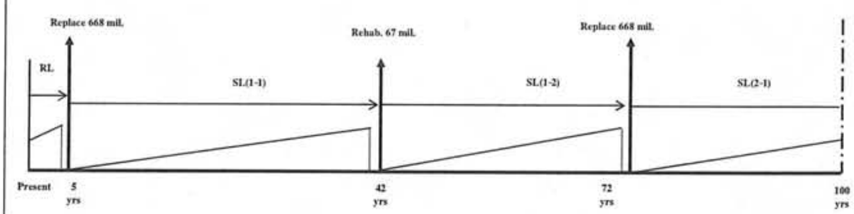


Figure 20.2.3-2 Deterioration Curve of Jones Bridge after Rehabilitation

Scheme	Life Cycle Cost Model	Service Life (Yrs)	LCC (Discounted rate 15 %)
Small Scale Rehabilitation		Type Year RL 5 EL 3 SL(1) 67 SL(2) 25 Total 100	305 million (0.81)
Medium Scale Rehabilitation		Year RL 5 EL 12 SL(1) 67 SL(2) 16 Total 100	261 million (0.71)
Large Scale Rehabilitation		Year RL 5 EL 31 SL(1) 64 Total 100	183 million (0.49)
Replacement		Year RL 5 EL 0 SL(1) 67 SL(2) 6 Total 78	370 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 20.2.3-3 Life Cycle Analysis of Jones Bridge

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

The lifecycle cost of the Jones Bridge is calculated and shown in **Table 20.2.3-1.**

Table 20.2.3-1 Evaluation of Rehabilitation Schemes by LCC Analysis

Unit: Million Pesos

	LCC at Discount Rate of 15%	Recommended Rehabilitation from LCC Analysis
Small Scale Rehabilitation	305 million (0.81)	3
Medium Scale Rehabilitation	261 million (0.71)	2
Large Scale Rehabilitation	183 million (0.49)	1
Replacement	370 million (1.00)	4

Notes: ( ) is ratio of LCC to replacement

(7) Recommendation based on Life-Cycle Cost Analysis

The result of the life cycle cost (LCC) analysis shows that the large scale rehabilitation scheme has the least cost among the alternatives. Judging from the LCC viewpoint of Jones Bridge. The large scale rehabilitation scheme is recommended.

20.2.4 Recommendations

Based on the comparative study aiming at engineering aspects and the life-cycle cost analysis, the most recommendable scheme is the **large-scale rehabilitation** because it is the most durable of all scheme at 42 years, as indicated in the analysis above.

Large-Scale Rehabilitation as mentioned in **Section 20.2.1**, with the additional girders at each side adjacent to the existing exterior girders will further enhance the structural stability of the structure. The existing exterior girders which will also be repaired and reinstalled will serve as vessel collision protection as their structural function will be carried by the additional girders. This measure will further improve the navigational clearance because the additional girders are shallower than the existing exterior girders.

The increase in dead weight by installing new girders is less than 2.0%, which is considered to be acceptable with minimal impact in terms of the stability of substructures.