13.3 BRIDGE CONDITION SURVEY AND IDENTIFICATION OF DAMAGES

13.3.1 Measurement of Shapes and Dimensions

(1) Objective

The purpose of this survey is to provide information on the overall dimensions of the bridge and section properties of members or elements consisting of the structures.

While a few available as-built drawings and repair of rehabilitation drawings were taken as reference for this activity, other important dimensions were measured in addition to the shapes and dimensions of elements or members. The data were collected, processed, and summarized, for the use in other activities, including modeling of structure and Load Rating.

(2) Inspection Teams

Several teams of inspectors were deployed on site to cover the whole Ayala Bridge structure within an eleven-day survey, to conduct hands-on verification of representative shapes and dimensions of elements or members of the Ayala Bridge. Two inspectors formed one team, which covered a specific area of the Ayala Bridge on a given schedule. Engineers coordinated the activities of all the inspection teams.

The inspectors were tasked to simultaneously perform another activity, Close-Up Visual Inspection of Damage.

(3) Coverage Areas

The bridge was divided into four general inspection areas: (a) Road/Deck level (RD), (b) Above Deck level (AD), (c) Below Deck level (BD), and (d) Substructure including Bearings (SB).

Items for verification on the road/deck level included deck road dimensions near midspan of both the South and North Spans, and approach road dimensions, taken approximately 1.0m from the face of the abutments. (See **Photo 13.3.1-1**)

Items for verification on the above deck level included shapes and dimensions of main truss members (excluding the bottom chord), sway brace members, and gusset plates. Spacing of main truss panels were also measured. (See **Photo 13.3.1-2**)

Photo 13.3.1-1 Road Deck Level - South Approach Road (Looking North East)

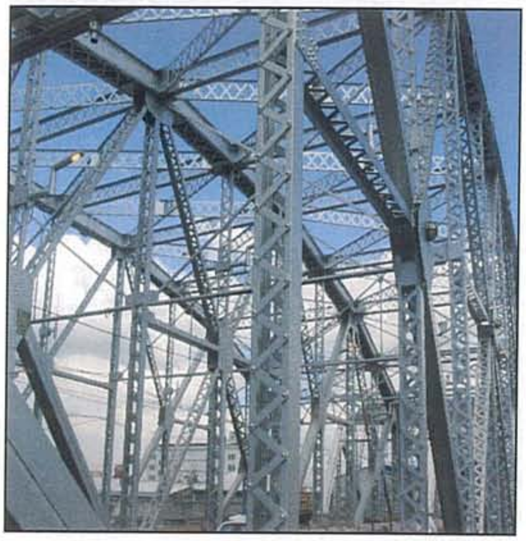


Photo 13.3.1-2 Above Deck Level - Main/Truss Members, Gusset Plates, and Sway Braces

Access to the connection of cross beams to bottom chords was very limited and no hands-on verifications could be made (See **Photo 13.3.1-3**). Items for verification on Substructures including Bearings included shapes and dimensions of bearings, including bearing shoe, at the substructures. The presence of the Pier concrete structure below water was verified with limited subsurface measurements around the Pier. Distances between bearings were also measured. (See **Photo 13.3.1-4**)

Photo 13.3.1-3 Below Deck Level - Bottom Chords, Cross Beams, Stringers, Deck Bracing and Gusset Plates. North Span shown (Looking South)

Photo 13.3.1-4 Substructures and Bearings - Pier Shown (Looking East)

(4) Reference Information

The Study Team was furnished with copies of a few as-built drawings, and repair and rehabilitation drawings of Ayala Bridge, and used them as references in planning this activity, and in designing the preliminary verification forms.

(5) Equipment and Procedure

Each team was equipped with safety gear (including hard hats, safety belts, safety goggles, safety shoes, and rain coat), measurement tools (8.0m tape measure, caliper) for verification, hammer for damage inspection, and digital video or still cameras, and verification forms and pencils for documentation.

Inspection on the road/deck level measured bridge section dimensions perpendicular to the bridge centerline. The dimension of approach road was measured at approximately 1 meter from the abutment.

Inspectors on the above deck level climbed up mobile scaffolds installed sidewalks to access the East and West Trusses, and mobile scaffolds installed on the median to access the Middle Trusses (See **Photo 13.3.1-5**). With the mobile scaffolds, verification of top chord members, gusset plates at the top chord, and sway braces were carried-out, (See **Photo 13.3.1-6**). Other members and gusset plates were accessed from the road/deck level, or were climbed up (see **Photo 13.3.1-7**). Verification on the below deck level was made possible by the use of a tugboat, with scaffolds installed on top, mainly at the South Span (see **Photo 13.3.1-8**), and by the use of a pontoon, mainly at the North Span (see **Photo 13.3.1-9**).

Photo 13.3.1-5 Use of Scaffolding at Main Truss

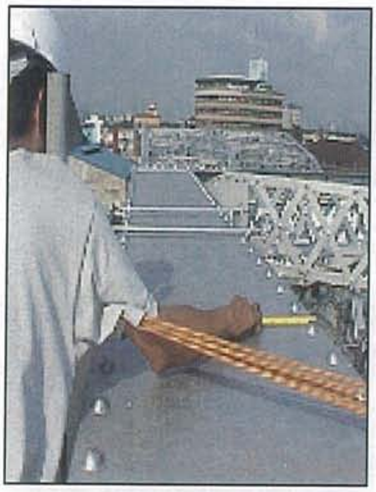


Photo 13.3.1-6 Measurement of Top Chord Member

Photo 13.3.1-7 Measurement of Mid Height Gusset Plates and Truss Members

Inspectors on the substructure including bearings climbed up telescoping ladders to perform verification on the bearings. The pier was accessed also via the use of the tugboat.

Verification on the below deck level was made possible by the use of a tugboat, with scaffolds installed on top, mainly at the South Span (See **Photo 13.3.1-8**), and by the use of a pontoon, mainly at the North Span. (See **Photo 13.3.1-9**).

Photo 13.3.1-8 Measurement of Below Deck Level Using Tugboat

Photo 13.3.1-9 Measurement of Below Deck Level Using Pontoon

Photo 13.3.1-10 Miscellaneous structures (Pipes)

Verification forms were based on reference documents. The inspector took note of the shapes of elements or members, and used a caliper to measure thickness, and a tape measure for other dimensions. Dimensions were measured at least two locations on the members, and averaged. The inspectors then filled up the inspection forms with this average value.

(6) Results

Table 13.3.1-1 lists drawings made based on the data presented in the inspection forms. Where dimensions that may be necessary for structure modeling and analysis could not be measured, information from the reference documents were reflected and marked with an asterisk (*) in drawings of Appendices shown in **Table 13.3.1-1**.

Table 13.3.1-1 List of Drawings

No.	Sheet No.	Title	Appendix
1	1	General Elevation, Plan Reflected Plan and Section	13.3.1-1
2	2	South Span Truss	13.3.1-2
3	3	North Span Truss	13.3.1-3
4	1	Cross Beam, Stringer, and Deck Bracing	13.3.1-4
5	2	Sway Brace	13.3.1-5
6	1	South Span Truss Gusset Plates at Top Chord	13.3.1-6
7	2	South Span Truss Gusset Plates at Mid-Height	13.3.1-7
8	3	South Span Truss Gusset Plates at Bottom Chord	13.3.1.8
9	4	North Span Truss Gusset Plates at Top Chord	13.3.1-9
10	5	North Span Truss Gusset Plates at Mid-Height	13.3.1-10
11	6	North Span Truss Gusset Plates and Splice Plate at Bottom Chord	13.3.1-11
12	7	Connection of Cross Beam to Bottom Chord	13.3.1-12
13	8	Deck Bracing Gusset Plates, and Connection of Stringer to Cross Beam	13.3.1-13
14	1	Pier	13.3.1-14
15	2	South Abutment	13.3.1-15
16	3	North Abutment	13.3.1-16

(7) Miscellaneous Structures

Miscellaneous structures, including non-structural elements, were noted, and those photographs were taken. These include pipes, lighting, and other architectural accessories. **Photo 13.3.1-10** shows an example of these miscellaneous structures.

13.3.2 Close-up Visual Inspection

(1) Objective

The purposes of this inspection are:

(a) To check the element/member condition and find the following structural damages and defects,

- Structural damages and defects such as deformation, missing members, deflection, etc.
- Material deficiencies and damages such as corrosion of steel members, loss or reduction of sections, cracks (width, length, depth), etc.

(b) To make judgment of damage degrees with damage rating, and

(c) To make detailed documentation including digital photos and videos.

The X-Y-Z method as described in the Manual, was used for the damage ratings. The data were collected, processed, and summarized, for the use of other activities, including: Non-Destructive Test of Material; Static Load Test and Microtremor Measurement Survey of Superstructure; Impact Vibration Test of Substructure; and Modelling of Structure and Load Rating.

(2) Inspection Teams

Several teams of inspectors were deployed on site to cover the whole Ayala Bridge structure within an eleven-day period to conduct close-up visual inspection of damages on the Ayala Bridge. Two inspectors formed one team, which covered a specific area of the Ayala Bridge on a given schedule. Engineers coordinated the activities of all the inspection teams.

The inspectors were tasked to simultaneously perform another activity, Verification of Shape and Dimension

(3) Coverage Areas

For inspection purposes, the bridge was divided into four general inspection areas or levels:

(a) Road/Deck Level (RD) / (b) Above Deck Level (AD) / (c) Below Deck Level (BD) / (d) Substructure including Bearings (SB).

(4) Reference Information

The Study Team was furnished with copies of a few as-built drawings, and repair and rehabilitation drawings of Ayala Bridge, and used them as references in planning this activity, and in designing the preliminary inspection forms.

(5) Equipment and Procedure

Equipment and access methods to the target areas were described in **Section 13.3.1** because the same team as the team for the shape and dimension survey conducted the close-up inspection.

The inspectors noted the damages found at each specific location, and the filenames of photographs or videos taken at that location. A damaged sheet containing one or two photographs, and type of damages inspected, assessed X, Y, Z levels, and damage rating, was prepared for each location where damage was found. Damage sheets were grouped according to inspection levels. A total of 9, 16, 160, and 16 damage sheets were each prepared for road/deck elements, above deck truss elements, below deck truss elements, and substructures and bearings, respectively.

The inspectors performed hammer test at each inspected member, to detect delamination of concrete or steel members, if any. A hammer was tapped along an area approximately 1.0 m^2.

(6) Criteria for Damage Rating

The X-Y-Z method for damage rating was used. In rating the damage for a specific member (steel, concrete, or miscellaneous structure), the type of damage was determined first. However, some members had two or more types of damages.

X was evaluated depending on the location or pattern of the damage; Y was evaluated depending on the depth or severity of the damage; and Z was evaluated depending on the scale or expanse of the damage. It was also determined if the member was a secondary or a main member.

(7) Results

Table 13.3.2-1 presents the results of the damage rating obtained from the Close-Up Visual Inspection; **Photo 13.3.2-1** shows typical damages by the structural component.

The inspection results are summarized as follows.

- Abnormal deflection is observed during vehicles passing at the north span was observed.
- A sway bracing at the south span and a stringer at the north span are missing.
- Stringers of the east side of north span are completely broken near the abutment. These stringers are presently supported by a temporary support structure.
- Serious section loss of a bottom chord is found at the joint.
- Cross beams are heavily corroded.
- Most of joint areas are heavily corroded.
- Steel bearings are not functioning properly because of cracks and heavy corrosion.
- Substructures are mostly sound, while having small cracks.
- The upper chords are mostly sound, while corrosion at joints are found.

From the serious damage conditions of members below the floor deck, a quantitative evaluation of the carrying capacity of superstructure is required since local failures are primary concerns.

Table 13.3.2-1 Damage Rating of Main Members by Close-Up Visual Inspection

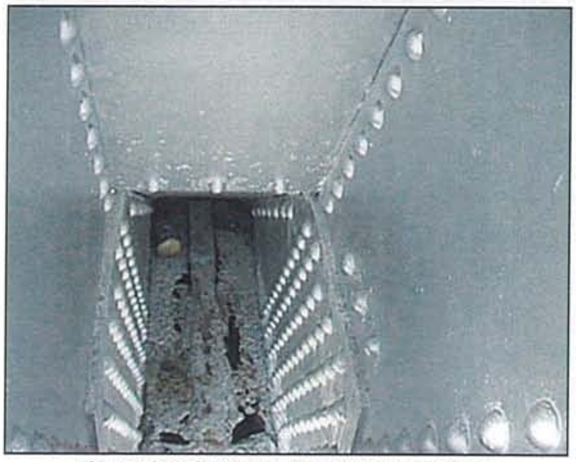
Component	Item	Member/Location	Damage Rating	Description
Superstructure	Shape / Dimension	Bot. Chord/South Span, East	III	Deformed
		Stringer/ North Span	I	Missing
		Deck Slab/North Span	I	Abnormal Deflection
		Sway Bracing / South Span	I	Missing
	Material Damage	Deck Slab / North Span	III	Crack
		Bot. Chord / South Span, East Truss	I	Heavily Corroded
		Gusset Plate / North Span, Mid Truss	I	Heavily Corroded
		Stringer / North Span	I	Crack / Broken
		Stringer / South Span	I	Reduced Section
		Bearing/All Location	I	Heavily Corroded/ Malfunction
Substructure	Material Damage	South Abutment / Main wall	III	Crack
		Bearings	I	Crack / Heavily Corroded
		North Abutment / Main wall	III	Crack

13.3.3 Non-Destructive Test of Material

(1) Objective

The purposes of this activity are to provide information on material properties and any damage that may not be visually observed based on the following tests conducted on steel and concrete members.

(a) Fractured Stringer and Heavily Corroded Cross Beams



(b) Reduced cross section of Bottom Chord due to corrosion

(c) Fractured Stringer at North East Portion

(d) Heavily Corroded Stringer at South East

(e) Damaged at Substructures and Bearings, Corrosion and (Abnormal) movement of Bearing at North Abutment. Roller most likely stuck-up

(f) Cracks and Concrete Spalling at Abutment (View South)

Photo 13.3.2-1 Typical Damages

(a) Steel Members

- Ultrasonic Thickness Gauging / Brinell Hardness Test / Ultrasonic Flaw Detection / Dye Penetrant Test

(b) Concrete Members

- Schmidt Rebound Hammer / Coring / Compression Test / Crack Width Comparator / Ultrasonic Pulse Velocity Test / Phenolphthalein Test / Chloride Test / Petrographic Analysis

To conduct this activity, the results of close-up visual inspection and the importance of the member/joint were considered to decide the locations of the non-destructive tests.

(2) Results

Table 13.3.3-1 shows the test results.

Table 13.3.3-1 Results of Non-Destructive Test

Test	Description and Results	Reference Appendices
Ultrasonic Thickness Gauging (UTG) (To measure the present thickness of the steel sections)	For the outer trusses, the measured side plate thickness ranged from 7 mm to 10 mm, and from 13 mm to 17 mm. For the middle trusses, the measured side plate thickness ranged from 8 mm to 9 mm, 15 mm to 18 mm, and 20 mm to 25 mm. Measured bottom plate thickness ranged from 7 mm to 10 mm.	Appendix 13.3.3-1
Brinell Hardness Test (To measure the hardness of the steel members)	Brinell Hardness Numbers ranged from 96 to 158, corresponding to equivalent tensile strengths of 328 MPa and 540 MPa respectively. in comparison, the standard tensile strength of A36 steel is in the range of 400 MPa to 550 MPa.	Appendix 13.3.3-2(1), (2)
Ultrasonic Flaw Detection Test (UFD) (To determine the presence of any internal defects or discontinuities in the steel sections)	The results showed no relevant indication of defects in any of the locations tested.	Appendix 13.3.3-3
Dye Penetrant Test (UFD) (To detect any surface - breaking defects)	Portions tested for DPT were as follows: 10 gusset plate; and 2) bottom flange of cross beam. The results of the DPT showed no relevant indication of defects in any of the locations tested.	Appendix 13.3.3-4(1), (2)
Schmidt Rebound Hammer Test (To determine the in-situ uniformity, surface hardness, and approximate compressive strength of concrete)	Measured rebound numbers ranged from 44 to 49 corresponding to calibrated compressive strengths ranging from 41 Mpa to 49 Mpa.	Appendix 13.3.3-5
Compression Test (To obtain the compressive strength of concrete)	Results of the two compression tests are 19MPa and 25MPa.	Appendix 13.3.3-5
Width Comparator (To eliminate surface crack widths)	Measured surface crack widths ranged from 0.8 mm to more than 1.5 mm.	Appendix 13.3.3-5
Ultrasonic Pulse Velocity (UPV) (To measure surface crack depths)	Estimated surface crack depth were: 107 mm (corresponding to crack width of > 1.5 mm); 126 mm (corresponding to crack width of > 1.5 mm); and 139 mm (corresponding to crack width of 0.8 mm).	Appendix 13.3.3-5
Phenolphthalein Test (To determine the depth of carbonation)	Measured depth of carbonation ranged from nil (0 mm) to 45 mm.	Appendix 13.3.3-5
Chloride Test (To assess the distribution of chlorides)	Results of the chloride test revealed low chloride levels ranging from 0.02% to 0.09%.	Appendix 13.3.3-5
Petrographic Analysis (To test for alkali-silica reaction)	Findings of the petrographic analysis showed no evidence of alkali-silica reaction.	Appendix 13.3.3-5

13.3.4 Static Load Test of Superstructure

(1) Objective

This activity aims to obtain the data for use in modeling of structures and Load Rating. During the static truck loading of the south span, the following in-situ measurements were taken: a) vertical deflections of selected points on the bridge deck; and b) corresponding strains in selected steel truss members. The static truck loading was conducted on the downstream side of the south span.

(2) Location of Deflection Survey Points and Strain Gauges

Figure 13.3.4-1 shows the locations of the deflection survey points and strain gauges. A total of nine (9) points were surveyed for deflections, while a total of twelve (12) truss members were measured for strains. Deflection survey points were distributed along the bridge deck, including three (3) near the west truss and three (3) near the middle truss. Strain gauges were attached on members located near the midspan.

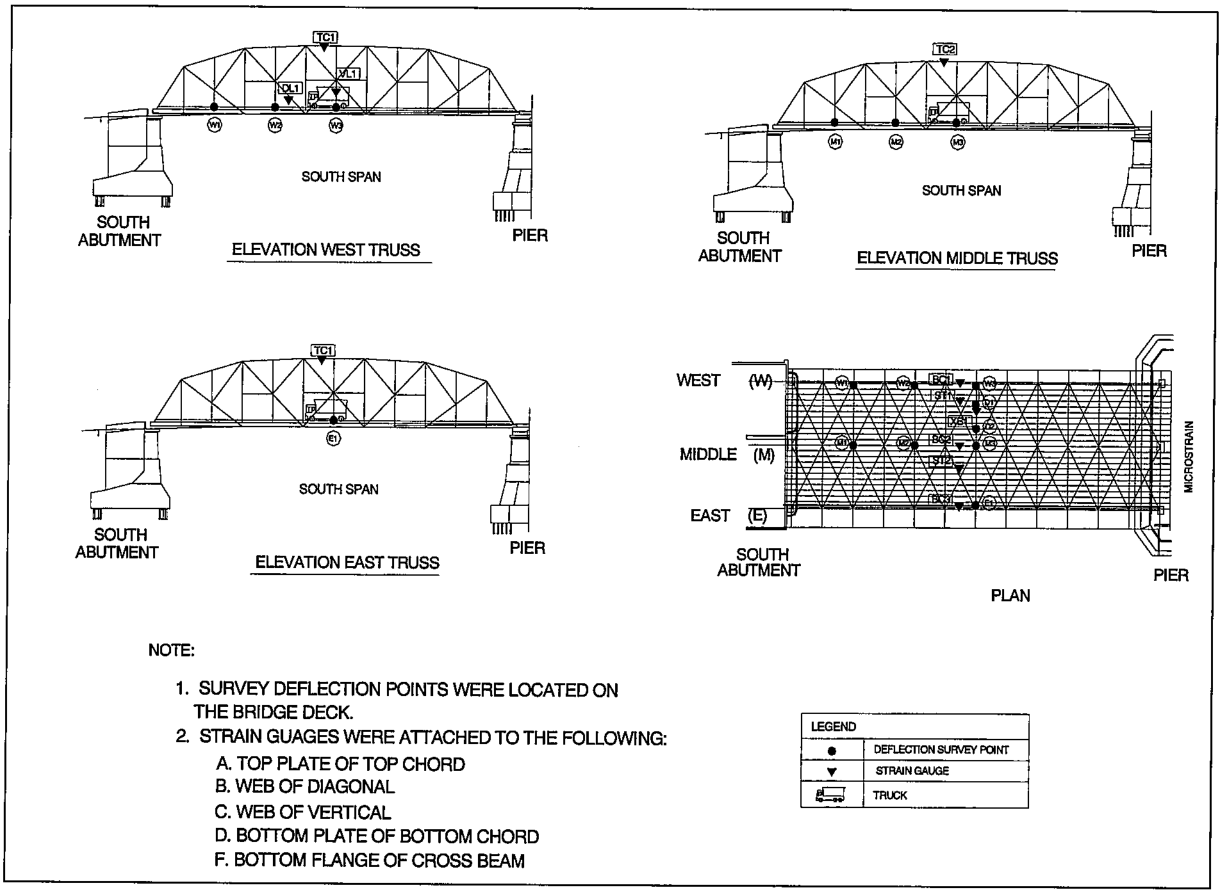


Figure 13.3.4-1 Location of Deflection Survey Points and Strain Gauges

(3) Step Load Pattern of Trucks

Photo 13.3.4-1 and **Figure 13.3.4-2** show the step load pattern of the trucks. Traffic was closed from midnight to 4:00AM on the day of testing. The structure was control-loaded using three (3) dump trucks. Truck Nos. 1 and 2 weighted 12.5 tons. Step Load No. 1 had Truck No. 1, Step Load No.2 had Truck Nos.1 and 2, and Step Load No.3 had Truck Nos. 1 to 3. The maximum truck load was 40.7 tons. After each step load, deflection and strain measurements were taken. The trends in measured deflection or strain were observed and instructions were then obtained from the Study Team whether to proceed to the next load case.

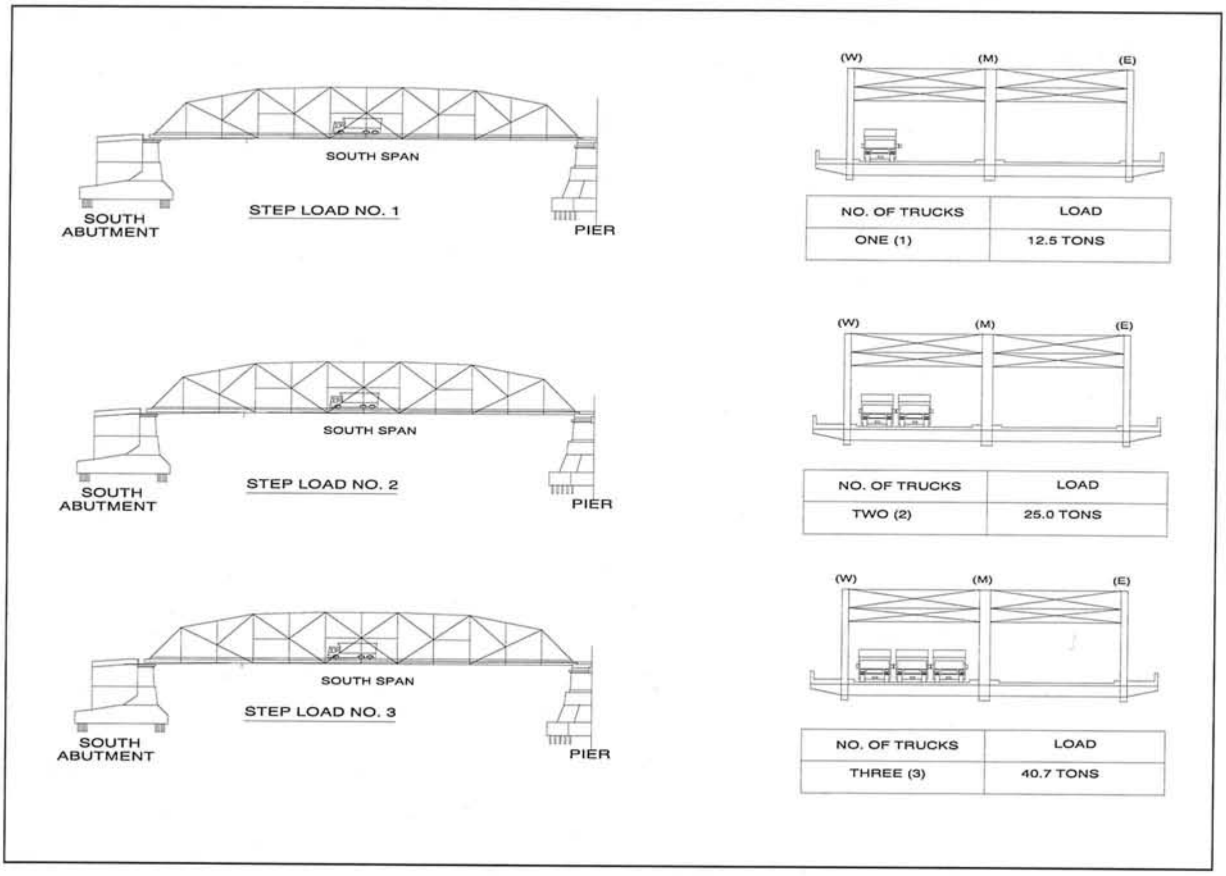
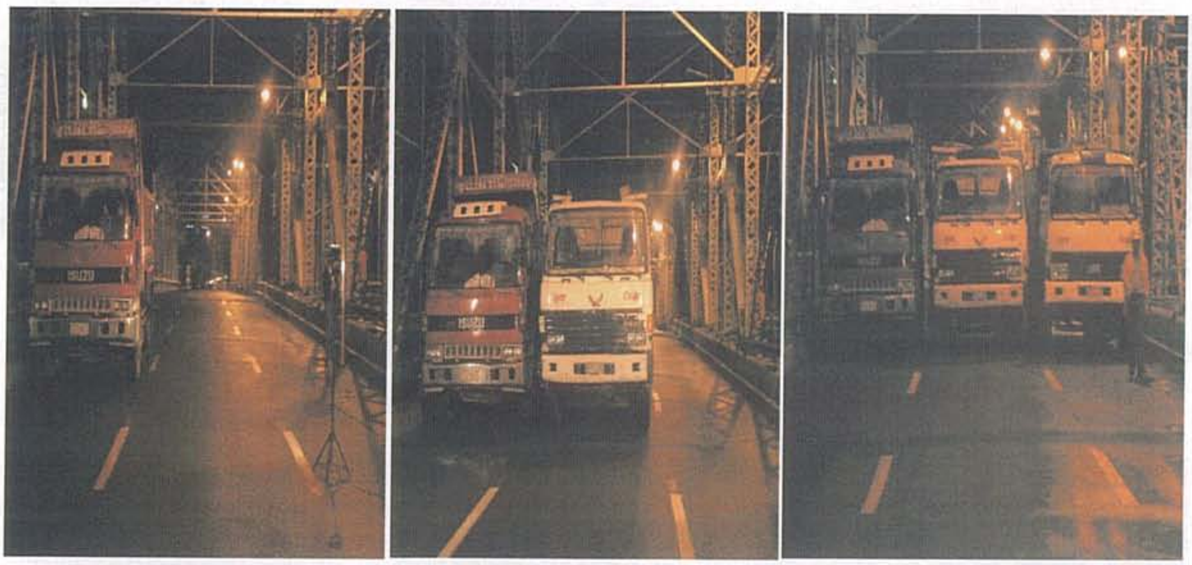


Figure 13.3.4-2 Step Load Pattern of Trucks



Step Load No. 1 Step Load No. 2 Step Load No. 3

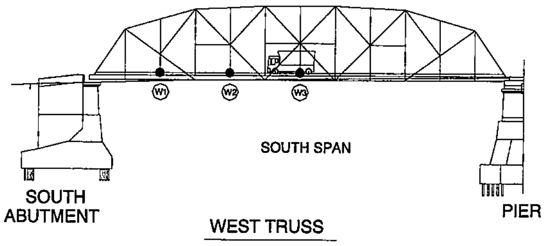
Photo 13.3.4-1 Step Load Pattern of Trucks

(4) Deflection Survey

Deflections were measured using Precise Level Wild NK – 2 complete with leveling rods and leveling bubbles, accuracy of the instrument was ±1 mm. Each deflection measurement was made thrice or twice and the average was recorded.

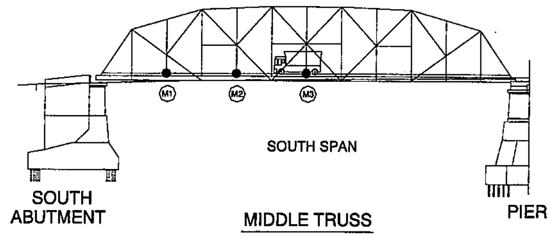
Measured deflections generally tended to increase with the addition of each truck load. Maximum deflection at the west truss) during step load 3, while maximum deflection was 7mm at deflection survey points D1 and D2 (near the middle of the roadway) also during step load 3.

Figure 13.3.4-3 summarizes the results of the deflection survey.



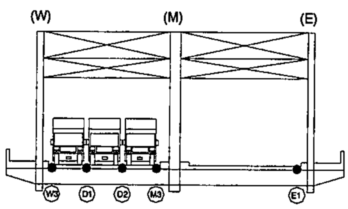
RECORDED DEFLECTION (mm)

STEP LOAD	W1	W2	W3
1 TRUCK	0	0	4
2 TRUCKS	1	2	5
3 TRUCKS	2	3	6



RECORDED DEFLECTION (mm)

STEP LOAD	M1	M2	M3
1 TRUCK	0	0	2
2 TRUCKS	0	0	2
3 TRUCKS	0	0	4



RECORDED DEFLECTION (mm)

STEP LOAD	W3	D1	D2	M3	E1
1 TRUCK	4	1	2	2	0
2 TRUCKS	5	5	4	2	-1
3 TRUCKS	6	7	7	4	0

Figure 13.3.4-3 Results of Deflection Survey

(5) Strain Measurement

Strain were measured using TML FLA-30-11 strain gauges. Strain data were collection thru a data acquisition system connected to a computer.

Figure 13.3.4-4 summarizes the results of the strain measurement. Measured strains were extremely small and ranged from 0.6 μ□ to 6.2 μ□.

13.3.5 Microtremor Measurement Survey

(1) Objective

This activity has two (2) specific objectives as follows:

(a) To identify and confirm the modes relevant to the deformations due to the dead load and governing live load cases (MS 18 lane loadings) considered in Modeling of Structure.

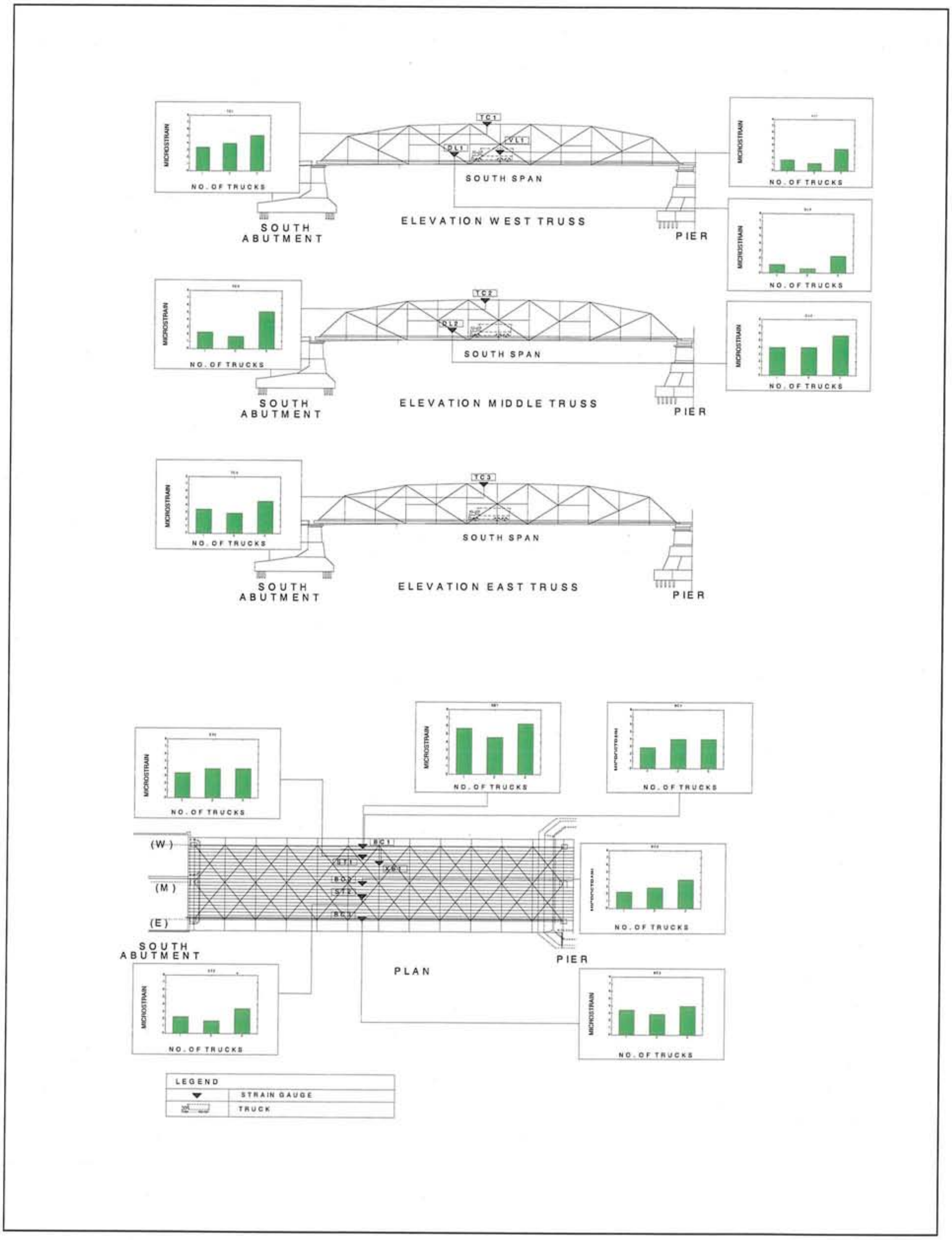


Figure 13.3.4-4 Results of Strain Measurement

(b) To confirm that the Impact Vibration Test of Substructure are associated with the vertical modes of the superstructure, not with the natural frequency of the bridge pier.

The microtremor measurements were conducted on both the South Span and the North Span with closing to traffic during the survey.

(2) Acceleration Sensors

Figure 13.3.5-1 shows the locations of the sensors on both spans during the survey.

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

The sensors were mounted on designated locations using beeswax, and were connected to a data acquisition system with 16-bit analog-to-digital converter using control cables.

For each span, twelve (12) measurement events were recorded. At each measurement event, accelerations were measured simultaneously at 1000 samples per second. Each event lasted around 8 minutes.

(3) Most Probable Natural Frequencies

The most probable natural frequencies are shown in **Table 13.3.5-1.**

- For the South Span, the most probable natural frequency of the 1st vertical mode is 3.2Hz, and that of the 1st torsional mode is 3.9Hz.
- For the North Span, the most probable natural frequency of the 1st vertical mode is 2.7Hz, and that of the 1st torsional mode is 3.4Hz

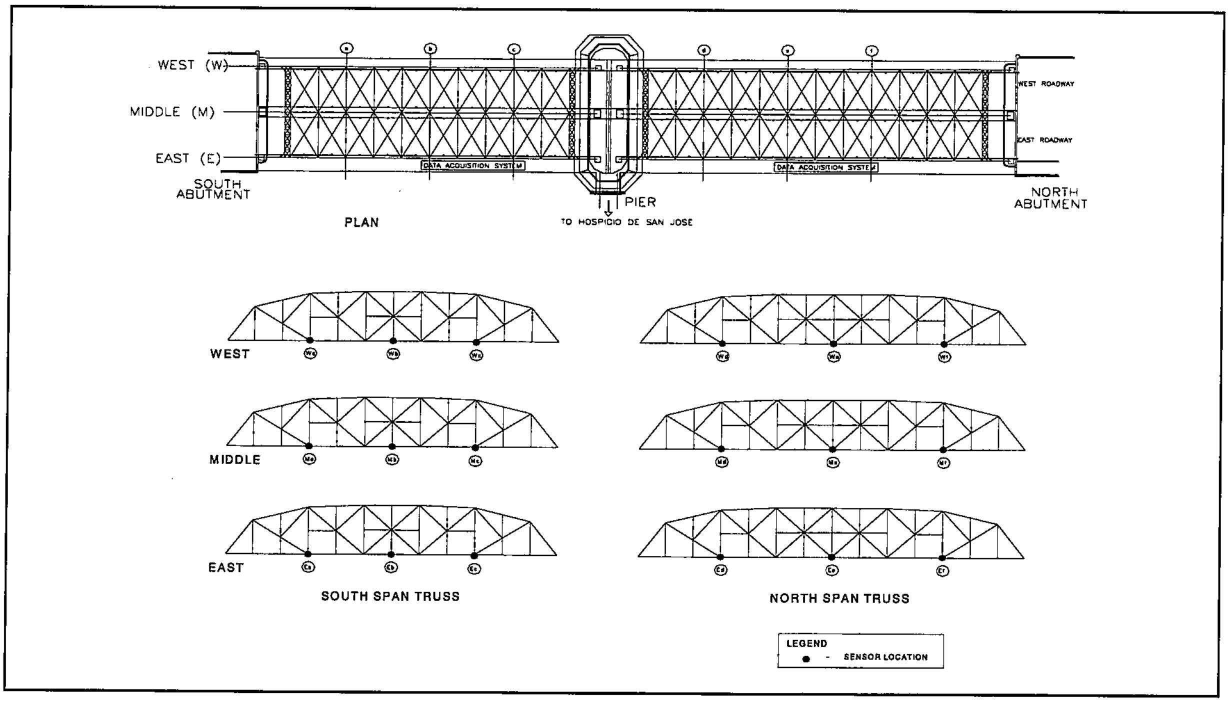


Figure 13.3.5-1 Location of Acceleration Sensors

Table 13.3.5-1 Most Probable Natural Frequencies

Span	1^{st} Vertical Mode	1^{st} Torsional Mode
South Span	3.2 Hz (3.3)	3.9 Hz (4.0)
North Span	2.7 Hz (2.8)	3.4 Hz (3.4)
( ) : Frequencies obtained from structural analysis model		

The 1^{st} vertical and 1^{st} torsional modes of the south were shown usually for reference in **Figure 13.3.5-2.**

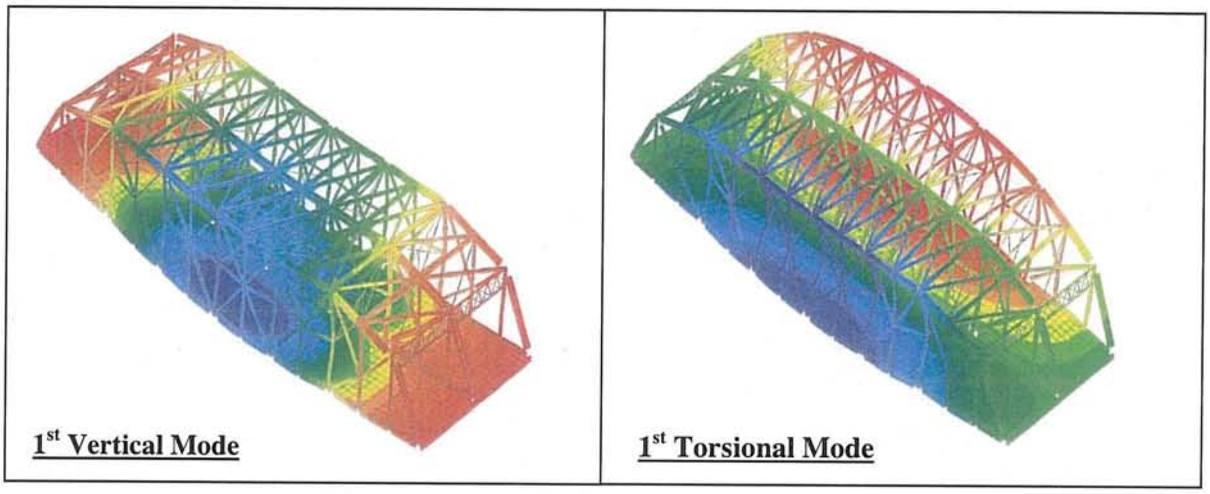


Figure 13.3.5-2 First Vertical and Torsional Modes

The results of the survey also confirm that the peaks around 2Hz and 5Hz in the spectra obtained for pier vibration[1] are associated to vertical modes of the superstructure, as would be evident in the succeeding discussion.

The above natural frequencies of the South Span and North Span have been identified as follows:

- Using the FFT algorithm, the amplitude spectra of the acceleration records of all the measurement events are plotted. The amplitude spectra of these records are shown in **Figures 13.3.5-3** and **13.3.5-4.**

- From the spectra of each measurement event, the resonant peaks are identified and their locations are noted. Where such peaks occurred clearly in two or more channels, a natural frequency is deemed indicated. Following this procedure, it can be inferred from **Figure 13.3.5-3** that the South Span has probable vertical vibration modes around 3.2 Hz, 3.8Hz, and 5.1 Hz. The North Span, on the other hand, has probable vertical vibration modes around 2.7Hz, 3.4Hz, and 4.7Hz, as can be observed from **Figures 13.3.5-4**.

- The amplitudes among the peaks shown in the spectra are compared to associate these peaks with the shapes of the vibration modes of interest.

- From **Figure 13.3.5-3** it can be observed that the spectral peaks around 3.2Hz have almost the same amplitudes that can be associated to the 1st vertical mode of the South Span (See **Figure 13.3.5-3**). The peaks at around 3.9Hz, on the other hand, can be associated the 1st torsional mode (see **Figure 13.3.5-3**), taking into account that the amplitudes of vibrations are maximum and almost the same at the mid-span of the East and West Trusses while the amplitude at the mid-span of the Middle Truss is considerably less. The spectral peak around 39.Hz in **Figure 13.3.5-3** at sensor location 'S', confirms that the mode associated with the 3.9Hz peak is torsional. The spectral peaks around 5.1Hz could be associated to a higher mode of vertical vibration.

- Similarly from **Figure 13.3.5-4,** it can be observed that the spectral peaks around 2.7Hz and5.1Hz and those of the North Span around 2.7Hz and 4.7Hz are associated to the observed peaks around 3Hz and 5Hz of the amplitude spectrum of the data gathered in the impact vibration test of the bridge pier (see **Figure 13.3.6-2** of **Section 13.3.6**).

- The results from the spectral analysis of all the records taken from other sensor locations are used to confirm the natural frequencies and the associated mode shapes that are identified in (2) and (3).

- The natural frequencies from the microtremor survey are closely correlated with the corresponding natural frequencies of the base models of both spans, as can be observed from **Tables 13.3.5-1**. The close correlation between survey and analysis results confirms that the engineering assumptions made in **Section 13.3.6** to take into account uncertain or missing data are valid.

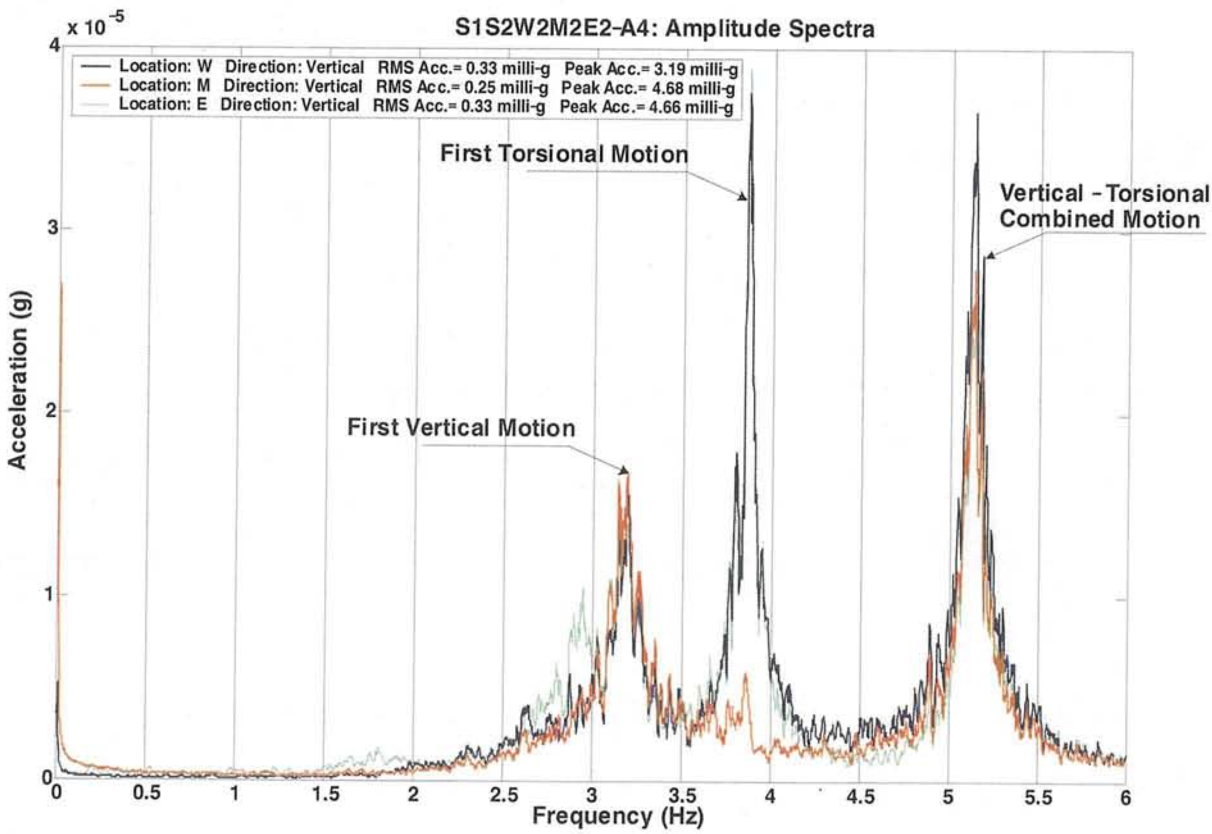


Figure 13.3.5-3 South Span Sample Amplitude Spectra: Vertical Direction

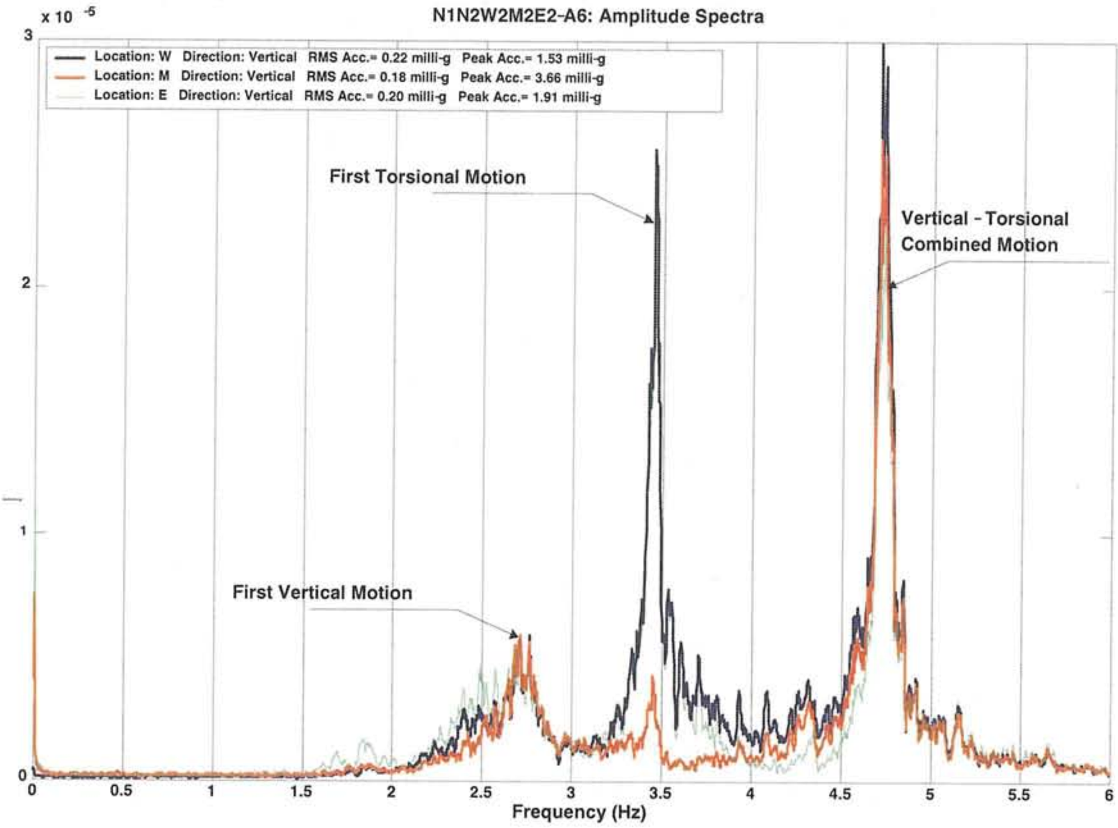


Figure 13.3.5.4 North Span Sample Amplitude Spectra: Vertical Direction

13.3.6 Impact Vibration Test of Substructure

(1) Objective

This Test was employed to evaluate the substructure soundness by focusing on the natural frequency of the pier.

(2) Procedure

Figure 13.3.6-1 shows the evaluation procedure for the soundness of the pier with the Impact Vibration Test. The feature of this test is as follows:

- The natural frequency can be obtained through several measurements of response waves with allowing normal vehicles passing on the bridge.

(3) Impact Pendulum

An impact pendulum was specially fabricated for this activity. The pendulum was positioned to impact the centerline of the Middle Pier of the bridge. (See **Photo 13.3.6-1**). The impacting head used had a mass of about 100 kg. (See **Photo 13.3.6-2**). The tip of the head was covered with rubber.

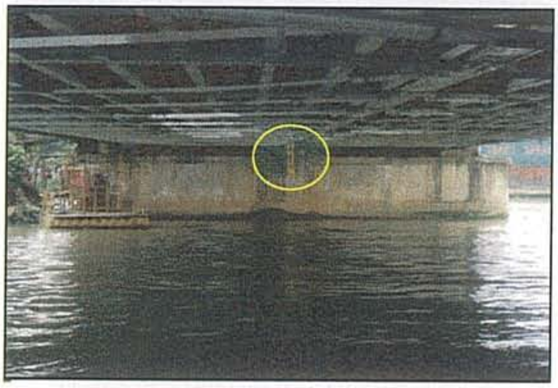


Photo 13.3-6-1 North Face of the Ayala Bridge Pier. Impact Pendulum highlighted (Looking South)

Photo 13.3.6-2 Impacting head of yellow-painted Pendulum, viewed from top of Pier (Looking West)

(4) Sensor and Locations

Five (5) force balance accelerometers (Kinemetrics ES-U) were calibrated to capture accelerations in the horizontal direction, and were positioned at Locations A, B, C, D and E on top of the Pier, beside the concrete bearing pads as shown in **Figure 13.3.6-2**

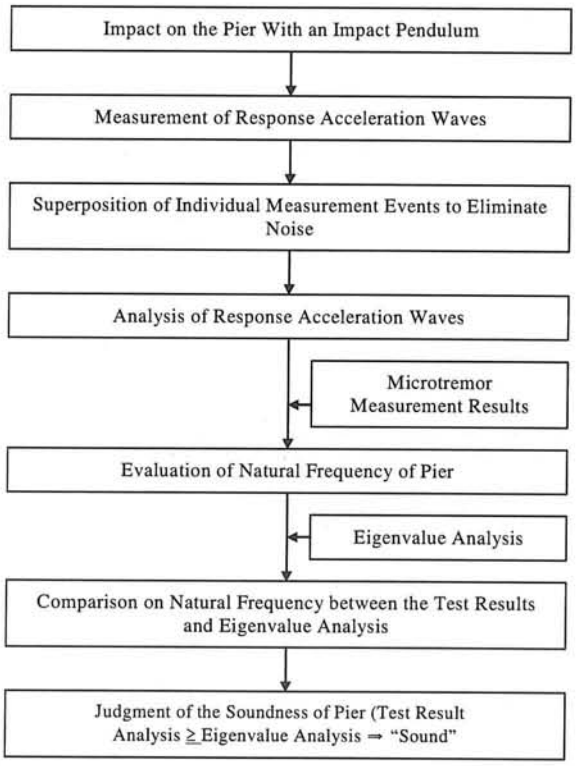


Figure 13.3.6-1 Evaluation Procedure for Soundness of Pier

(5) Data Processing

Vibration measurement data were processed using developed routines and algorithms in the MATLAB environment. For this processing, ten (10) individual measurement events that captured the impact of the pendulum on the Pier were superimposed; i.e., the location of the peak value in the time history of each event was determined, and each event was cropped as specific seconds before the peak and after the peak . The ten (10) events were then algebraically added, to form a superimposed measurement event. The total length of the superimposed event was about six (6) seconds.

Basically, processing consisted of plotting the recorded vibration acceleration against time (or time history), computing the root-mean-square (RSM) and peak values of the time-history as indicator of the level of vibration, and plotting the amplitude spectrum (acceleration vs. frequency), which was obtained using the Fast Fourier Transform algorithm. (See **Figure 13.3.6-2**).

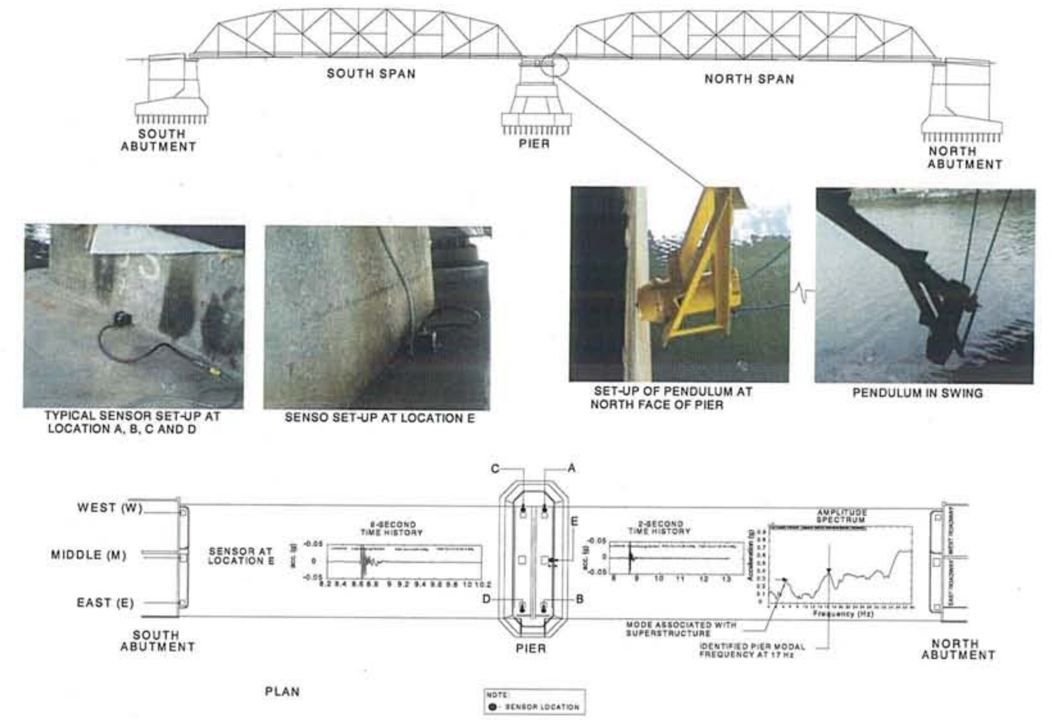


Figure 13.3.6-2 Sensor Location, Analysis and Response Acceleration at E Result of Fast Fourier

(6) Test Results

The peak and RMS acceleration at Location A and C, which are on the same side (north) or the Pier as Location E, but located beside the bearings of the outer truss are calculated. The computed peak and RMS acceleration at Location C was more than 4 times those at Location B and D, which are behind Locations A and C, respectively, on the outer south bearings on the Pier. (See **Table 13.3.6-1** for computed peak and RMS values acceleration. Refer to **Figure 13.3.6-2** for the relative locations of the sensors.

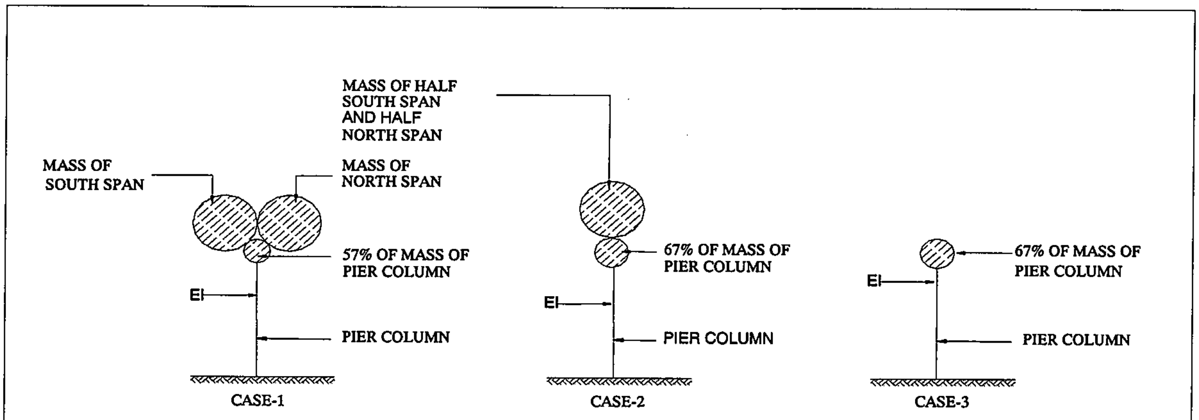
Table 13.3.6-1 Summary of Vibration Data

Sensor Location	Acceleration (gal)		Remarks
A	31.2	1.5	This data is result of the Super-Position of the ten (10) Events described below
C	35.6	1.6	
B	25.7	1.3	
D	25.5	1.3	
E	126.4	5.8	
E	8.2	0.7	Peak and RMS Values of Acceleration Shown only for Location E
E	19.2	1.1	
E	7.7	0.6	
E	18.0	1.0	
E	14.4	0.8	
E	8.8	0.6	
E	23.0	1.1	
E	24.4	1.1	
E	25.1	1.2	
E	38.8	1.8	

The natural frequency of the Pier was expected to be above 10 Hz. From the amplitude spectra generated from the superimposed measurement event, a peak occurring at around **17 Hz** was taken to be the fundamental natural frequency of the Pier in the longitudinal direction of the bridge. Peaks in the spectra around 3 Hz and around 5 Hz were presumed to be associated with predominantly vertical modes of vibration of the superstructure.

(7) Eigenvalue Analysis

The eigenvalue analysis of the pier was carried out for the following cases with consideration of the present bearing conditions.



CASE - 1 : FIXED SUPPORT CONDITION IS COMPLETELY EFFECTIVE AT THE BEARING.
CASE - 2 : FIXED SUPPORT CONDITION IS EFFECTIVE FOR HALF THE SOUTH SPAN AND HALF THE NORTH SPAN.
CASE - 3 : FIXED SUPPORT CONDITION IS NOT COMPLETELY EXPECTED.

Figure 13.3.6-3 Cases for Eigenvalue Analysis

The results obtained from Eigenvalue Analysis are given in **Table 13.3.6-2**

Table 13.3.6-2 Results of Eigenvalue Analysis

CASE	Frequency of the First Mode
Case - 1	15.2 Hz
Case - 2	17.0 Hz
Case - 3	20.5 Hz

(8) Comparison on Natural Frequency

According to the criteria given in **Table 13.3.6-3,** it was demonstrated that the pier was sound because of the following reasons:

- In original design both the spans were fixed at the pier and movable at the abutment. This means that case-1 is fit for the evaluation model in the ideal condition.

- According to the Microtremor Measurement Survey, motion associated with superstructures was observed. This means that bearing conditions are still effective under the small displacement or deflection, and case-3 is not adequate for the evaluation model.

- Movable conditions at both the abutment are not ideal states because of friction between the superstructure and substructure through bearings. The friction works even in conditions intended in the design under the quite small displacement or deflection.

- In case that interaction springs between foundation and surrounding ground are installed in the model, the calculated natural frequency will be smaller. This means that the natural frequencies indicated in **Table 13.3.6-2** become smaller in when taking into consideration the soil-structure interaction.

- From the above, the natural frequency obtained from the eigenvalve analysis is to be between case-1 and case-2. This natural frequency is smaller than that measured from the Impact Vibration Test.

In light of the definition of the soundness of the pier shown in **Figure 13.3.6-4** the pier is evaluated to be sound.

Table 13.3.6-3 Criteria for Substructure Soundness

Rating Index (k) = (Natural Frequency of Test) / (Natural Frequency of Analysis)

Rating Index	Rating		Action
0.7 or less	A	(A1)	In dangerous condition to abnormal external forces. Improvement work is required.
0.85 or less		(A2)	To conduct follow-up test to check the progress of deterioration.
0.86 or more	B		No problem at present

Table 13.3.6-4 Rating Index of Test Result

Case	Natural Frequency		Rating Index	Action*
	Analysis	Test		
Case-1	15.2 Hz	17 Hz	1.12	B
Case-2	17.0 Hz		1.00	B
Case-3	20.5 Hz		0.83	(A2)
* : Refer to **Table 13.3.6-2**				

13.3.7 Assessment of Critical Damages

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 **Figure 13.3.7-1.**

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

The kinds of damages are divided into thirty two (32) types by the material composition of the structure. Regarding damage type and evaluation method, the detailed and concrete explanations were described to inspectors in order to identify damages readily.

Damage Rating with X, Y, Z Method
↓ ← Non - Destructive Test of Material
Identification of Damage Rating I or II
↓
Categorization of Damage Rating
↓
Estimation of Section Loss of Main Members
↓
Evaluation of Damages
↓
Presumption of Original Design **(Section 13.4)**

Figure 13.3.7-1 Procedure of Diagnosis for the Soundness of Bridge

Figure 13.3.7-2 shows an example of the damage rating recorded in the study it has two (2) damages types with damage rating given in **Figure 13.3.7-3.**

DAMAGE

TYPE	CORROSION	
Evaluation	X	-
	Y	High
	Z	High
Rating	I	
Remarks		

DAMAGE

TYPE	BREAK/FRACTURE	
Evaluation	X	-
	Y	-
	Z	High
Rating	I	
Remarks		

Figure 13.3.7-2 Damage Rating Example Adopting the X, Y, Z Method

For members given a damage rating of II by the inspectors, the responsible engineer identified whether the members would be given of rating I or II through consultation with the DPWH.

(2) Categorization of Damage Rating

The damage ratings were categorized as discussed in **Chapter 6.**

(3) Evaluation of Damages

The results of damage evaluation for main members are summarized in **Table 13.3.7-1**. In the table heavily damaged main members are shown. Members given in the table can be identified by referring to the joint numbers shown in **Figure 13.3.7-4.**

Damage Type	(1) CORROSION

Corrosion is the representative damage of steel caused by oxidation of the member by the environment. Because of being progressive in nature but easy to be found, this is one of the important damages whose progress shall be protected by maintenance activities.

		Influence of Damage on Load-Carrying Capacity and Durability	
		High	Low
Location or Pattern (X)	Severeness	-	-
	Sample Description	-	-
Depth (Y)	Severeness	Section Loss	Surface Rust
	Sample Description	Swollenness of steel surface or reduction of cross section area due to missing of corrosion parts	Dotted surface rust areas
Expanse (Z)	Severeness	Entire	Local
	Sample Description	Entire expanse of corrosion or rust to members	Local expanse such as leakage area of corrosion or rust

Rating

Y	Z	Secondary Members	Main Members
High	High	II	II
	Low	III	II
Low	High	III	II
	Low	IV	III

Damage Type	(2) Crack

Almost all cracks generate at or near the welded connection area. Since cracks are hard to find, it is necessary to inspect carefully referring to figures or pictures of similar damages. When cracks are found, in-depth survey shall be conducted by qualified engineers.

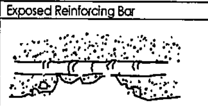
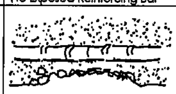
		Influence of Damage on Load-Carrying Capacity and Durability	
		High	Low
Location or Pattern (X)	Severeness	-	-
	Sample Description	-	-
Depth (Y)	Severeness	Cracks	-
	Sample Description		
Scale (Z)	Severeness		
	Sample Description		

Rating

Y	All Members
High (presence)	II

Damage Type	(10) Honeycomb

Honeycomb is mainly caused by insufficient compaction of concrete under construction. The extent of damage is determined by the presence of exposed reinforcing bars.

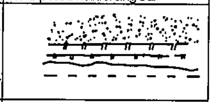
		Influence of Damage on Load-Carrying Capacity and Durability	
		High	Low
Location or Pattern (X)	Severeness	-	-
	Sample Description	-	-
Depth (Y)	Severeness	Exposed Reinforcing Bar	No Exposed Reinforcing Bar
	Sample Description		
Expanse (Z)	Severeness	Large area generated	Small area generated
	Sample Description	Superstructure: 0.1 m2 or more Substructure : 1.0 m2 or more	Superstructure: below 0.1 m2 Substructure : below 1.0 m2

Rate

Y	Z	Secondary Members	Main Members
High	High	II	II
	Low	IV	II
Low	High	IV	III
	Low	IV	IV

Damage Type	(11) Wear / Abrasion / Erosion

In this damage, the surface of concrete substructures is worn-out, abraded or eroded by water. The extent of the influence of the damage is determined from whether the damage has reached the reinforcing bar.

		Influence of Damage on Load-Carrying Capacity and Durability	
		High	Low
Location or Pattern (X)	Severeness	-	-
	Sample Description	-	-
Depth (Y)	Severeness	Exposed Reinforcing Bar	No Exposed Reinforcing Bar
	Sample Description		
Expanse (Z)	Severeness	Large Area Generated	Small Area Generated
	Sample Description	Substructure : 1.0 m2 or more	Substructure : below 1.0 m2

Rate

Y	Z	All Members
High	High	II
	Low	III
Low	High	III
	Low	IV

Figure 13.3.7-3 Examples for Damage Rating Reference Used in the Field Inspection

Table 13.3.7-1 Evaluation of Damages for Main Members of Superstructure

SOUTH SPAN						NORTH SPAN					
Location	Members Reference No.	Damage Type	Evaluation Based on Field Survey		Estimated Section	Location	Members Reference No.	Damage Type	Evaluation Based on Field Survey		Estimated Section
			Damage Rating	Diagnostic Category					Damage Rating	Diagnostic Category	
South Span, West Truss						North Span, West Truss					
Bottom	M109-112	CO	I	A	30%	Bottom	M139-142	CO	I	A	30%
Chord	M112-115	CO	I	A	30%	Chord	M142-145	CO	I	A	30%
	M115-121	CO	I	A	30%		M154-157	CO	I	A	40%
	M124-127	CO	I	A	30%	North Span, Middle Truss					
South Span, Middle Truss						Bottom	M234-238	CO	I	A	50%
Bottom	M206-209	CO	I	A	30%	Chord	M238-239	CO	I	A	30%
Chord	M209-212	CO	I	A	30%		M239-242	CO	I	A	30%
	M212-215	CO	I	A	30%		M242-245	CO	I	A	30%
	M215-218	CO	I	A	40%		M245-248	CO	I	A	30%
	M218-221	CO	I	A	40%		M248-251	CO	I	A	30%
	M221-224	CO	I	A	30%		M251-254	CO	I	A	40%
South Span, East Truss							M254-257	CO	I	A	40%
Bottom	M309-312	CO	I	A	30%		M257-260	CO	I	A	30%
Chord	M312-315	CR	I	A	30%		M260-263	CO	I	A	30%
	M315-318	CO	I	A	40%		M263-266	CO	I	A	30%
	M318-321	CR	I	A	40%		M266-269	CO	I	A	10%
	M321-324	CO	I	A	30%		M271-272	CO	I	A	50%
	M324-327	DE	I	A	30%	North Span, East Truss					
						Bottom	M342-345	CO	II	B	10%
						Chord	M351-354	CO	III	B	15%
							M357-360	CO	I	A	30%
							M360-363	CO	II	B	10%

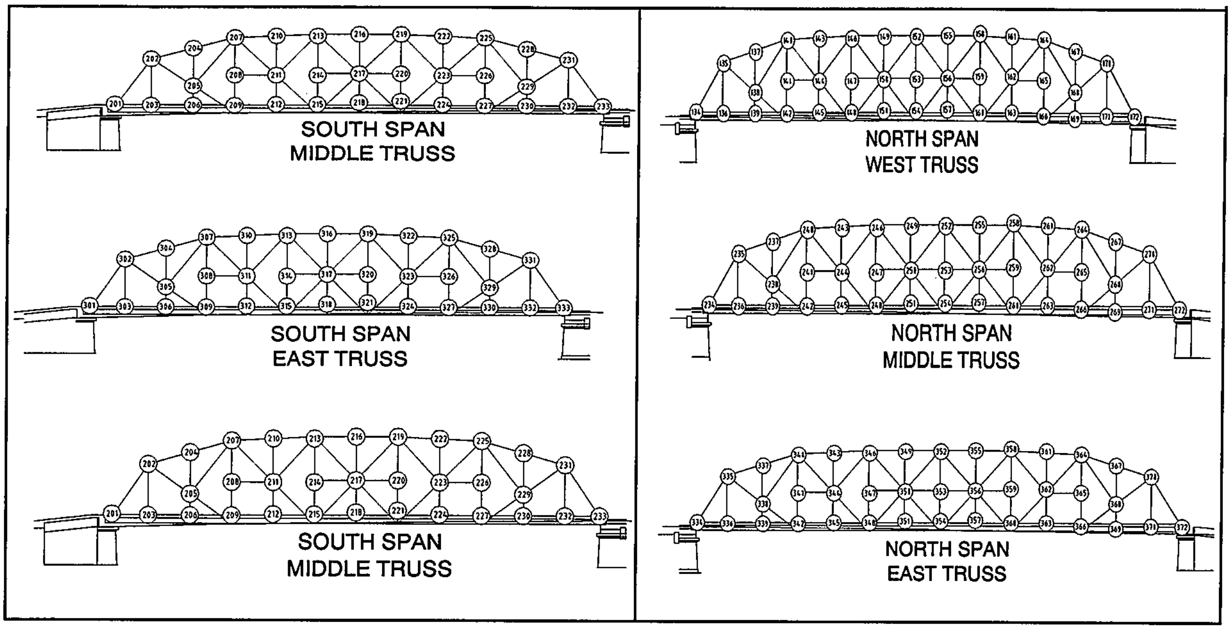


Figure 13.3.7-4 Node Numbers for Reference

Moreover, the damage evaluation of substructure is shown in **Table 13.5.1-1** of **Section 13.5.1.**

The detailed section and material properties are discussed in **Section 13.4.**

13.4 PRESUMPTION OF ORIGINAL DESIGN AND LOAD RATING

13.4.1 Objective

The purpose of the presumption of original design is to prepare the structural shapes, dimensions and material properties for the analysis model of the Load Rating as discussed in the **Section 13.4.3**. The following policy are used in the presumption of the original design:

- Inspection results are to be used for the determination of shapes and dimensions of visible portions such as superstructure, exposed parts of substructures.
- Material properties of members are to be basically determined from the non-destructive test. For the members for which the non-destructive test was not carried out, the material properties are to be estimated based on the relevant reference materials and as-built drawings, if indicated.
- The foundation type and its embedded depth are to be determined from as-built drawings, geotechnical survey results and relevant reference materials.
- The number of piles are to be fixed from the calculation based on the original design code.

13.4.2 Structural Shapes and Dimensions

(1) Superstructure

Most structural data of the superstructure does not need to be assumed because almost all the dimensions and details were measured. The shape and dimension measurement was used for structural analysis while the drawings are included in **Appendix 13.3.1.**

The material properties of each member use in the analysis are given in **Appendix 13.4.2-2.**

(2) Substructure

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

The type of foundation were found from the as-built drawing. The availability of materials and construction is known, thus the pile capacity of each pile could be estimated. **Appendix 13.4.2-3** shows the assumed shapes, dimensions and details of substructures.

Table 13.4.2-1 shows reactions estimated at the bearings.

Table 13.4.2-1 Reactions Estimated at Supporting Points

Unit: tons

		South Span		North Span	
		Abutment	Pier	Pier	Abutment
Dead Load	West Trust	201	200	238	235
	Middle Trust	348	352	413	422
	East Trust	215	213	262	257
	Sub-Total	765	765	914	914
Live Load	West Trust	68	68	78	78
	Middle Trust	102	102	117	117
	East Trust	68	68	78	78
	Sub-Total	238	238	274	274
Total Reaction		1003	1003	1187	1187
Total Dead Weight		1530		1828	

13.4.3 Load Rating

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

As far as the substructures are concerned, its capacity-demand ratio was discussed in "Vulnerability to Disaster" of **Section 13.6** because the safety of substructures are usually determined from seismic forces in this country.

(2) Procedure

The allowable stress method expressed by the following formula was used to evaluate the capacity of the members:

$$RF = \frac{\text{Capacity for Live Load Effect}}{\text{Vehicle Load Capacity}} = \frac{R - D}{L\,(1 + \imath)}$$

Where,

RF = Rating Factor (RF)

R = Allowable stress of member

- Inventory Level = 125 MPa
- Operating Level = 170 MPa
- Yield Stress = 228 MPa

D = Stress due to dead loads

$L(1+\imath)$ = Stress due to live loads

$\imath$ = Impact factor for live load effect

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 if the calculated deflection is comparatively equal with the conducted Static Load Test of the bridge and the analysis model was verified with the Microtremor Test whether or not its model represented the actual motion.

The bridge superstructure was modeled using the Strand7 Finite Elements Analysis System. An analysis of Load Effects was performed. A fully rigid joint as well as a pin joint assumption were both analyzed and investigated. MS 18 Load was used in the Live Load Analysis.

The Rating Factor (RF) can be used to determine the Load Rating (LR) of the superstructure members as follows:

$$LR = RF \bullet W$$

Where,

W = Weight of the Rating Vehicle in metric tons

(3) Analysis Results

Figure 13.4.3-1 and **13.4.3-2** show the members with RF less than 1.0,, which are highlighted, for the Inventory Level and the Operating Level, respectively.

The calculated numeric data of RF for each main member is given in **Appendix 13.4.3-1 (1/6) ~ (6/6)**

The results of minimum RF per location are shown in **Table 13.4.3-1**

Table 13.4.3-1 Minimum RF per Location

		South Span		North Span	
		Inventory Level	Operating Level	Inventory Level	Operating Level
West Truss	Bottom Chord	0.3 (10)	1.6 (51)	0.1 (3)	0.5 (15)
	Top Chord	2.4 (77)	4.4 (141)	1.2 (38)	2.7 (85)
	Main Vertical	2.1 (67)	3.3 (106)	4.2 (70)	3.4 (109)
	Main Diagonal	2.1 (109)	5.7 (182)	2.7 (86)	4.7 (150)
Middle Truss	Bottom Chord	- 0.2 (0)	0.9 (29)	**- 0.6 (0)**	**0.1** (3)
	Top Chord	2.3 (74)	4.3 (138)	1.2 (38)	3.0 (95)
	Main Vertical	1.2 (38)	2.1 (67)	0.6 (19)	2.0 (64)
	Main Diagonal	1.6 (51)	3.3 (106)	1.8 (58)	3.7 (118)
East Truss	Bottom Chord	0.0 (0)	1.2 (38)	0.1 (3)	1.1 (35)
	Top Chord	1.9 (61)	4.2 (134)	0.5 (16)	1.9 (61)
	Main Vertical	1.4 (45)	2..3 (74)	0.5 (16)	1.9 (61)
	Main Diagonal	1.6 (51)	3.3 (106)	1.4 (45)	3.1 (99)
Remarks	( ) : Equivalent Truck (tons)				

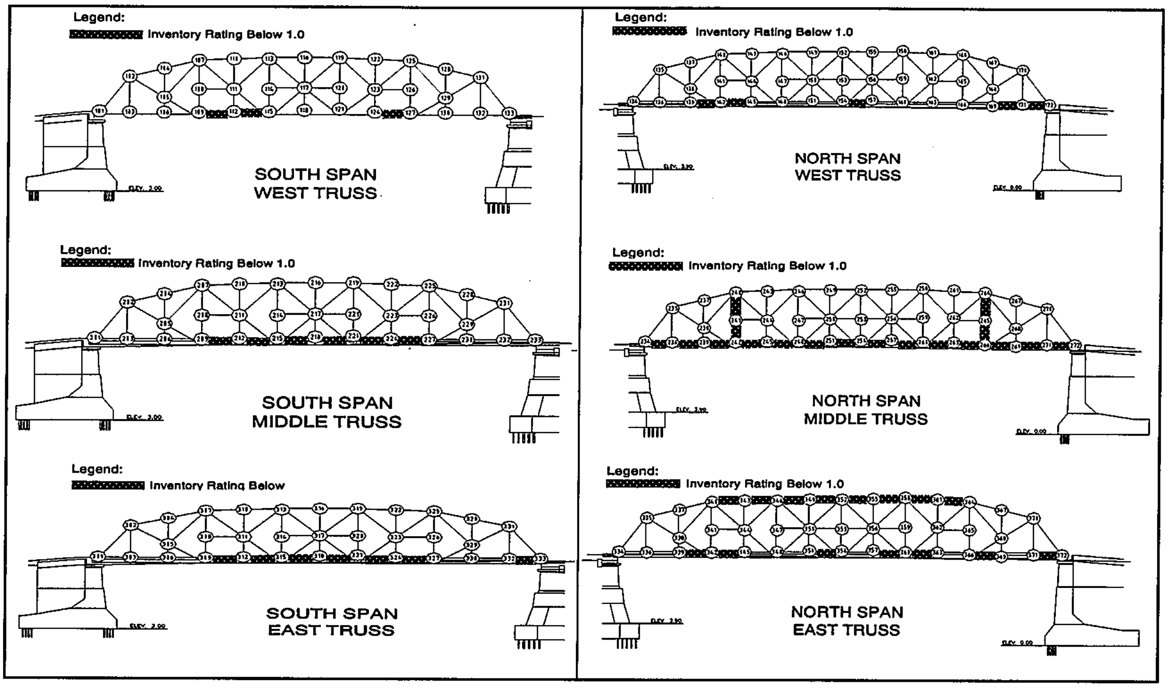


Figure 13.4.3-1 Member Below 1.0 of Rating Factor at Inventory Level

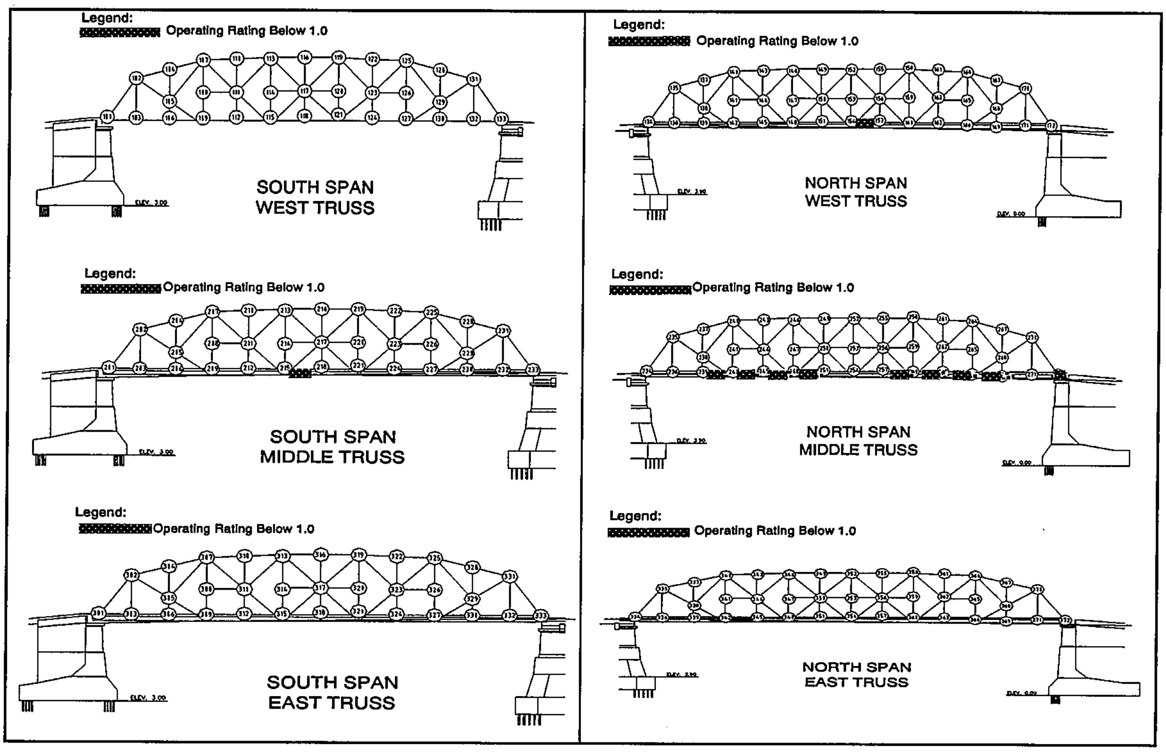


Figure 13.4.3-2 Member Below 1.0 Rating Factor at Operating Level

The analysis results suggest that the following counter measures shall be undertaken.

- Strict implementation of the regulation on the present vehicle load limit from 5 to 3 tons should be done, if improvement works are not carried out. However, the drastic improvement works of main members shall be required because RF_i of the inventory level is still insufficient for any vehicle.

- The drastic improvement works of the bridge should be conducted as soon as possible in order to secure safety of the facility and to prevent the potential partial or total collapse of the bridge.

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

The minimum Rating Factors were generated at the bottom chord of middle truss of the north span. This situation of the member can be understood by the following calculation (refer to **Figure 13.4.3-3** below).

RF_i = - 0.6 (RF of Inventory Level, Design Live Load being 32.7 tons)
RF_o = 0.1 (RF of Operating Level, Design Live Load being 32.7 tons)
S_e = 40% (Selection Loss of the Member)

The following equations are obtained for the particular member from the basic formula mentioned before.

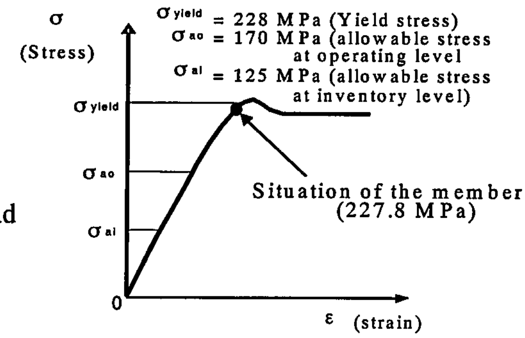
$\sigma_{DE} = \sigma_{ai} - (\sigma_{ao} - \sigma_{ai}) / (RF_0 - RF_i) \cdot S'_e \cdot RF_i$

$\sigma_{Le} + \sigma_i = (\sigma_{ao} - \sigma_{ai}) / (RF_0 - RFi) \cdot S'_e$

σ_{De} = Effective stress from the dead load

$\sigma_{Le} + \sigma_{ie}$ = Effective stress from live load and impact load

$S'_e = 1 - S_e/100$



From these equations, the stress from the dead load, live load and impact load are as follows:
$\sigma_{De} + (\sigma_{Le} + \sigma_{ie}) = 189.2 + (38.6) = 227.8$MPa (When a design level load of 32.7 tons being loaded)

Thus, if a design live load of 32.7 tons were loaded, the situation of the member which is most critical condition among all main members would be very close to the yield stress of 228 MPa.

Figure 13.4.3-3 Effective Stress for Members with Section Loss

13.5 TRAFFIC CONDITION

13.5.1 Traffic Condition on the Ayala Bridge

Figure 13.5.1-1 shows the hourly traffic volume variation of weekday on the Ayala Bridge by direction. Traffic volume bound for the south (Direction 2) changes considerably depending on the time zone: the most congested time is from 8:00am to 2:00pm. This suggests that the Ayala Bridge is mainly used for business activities.

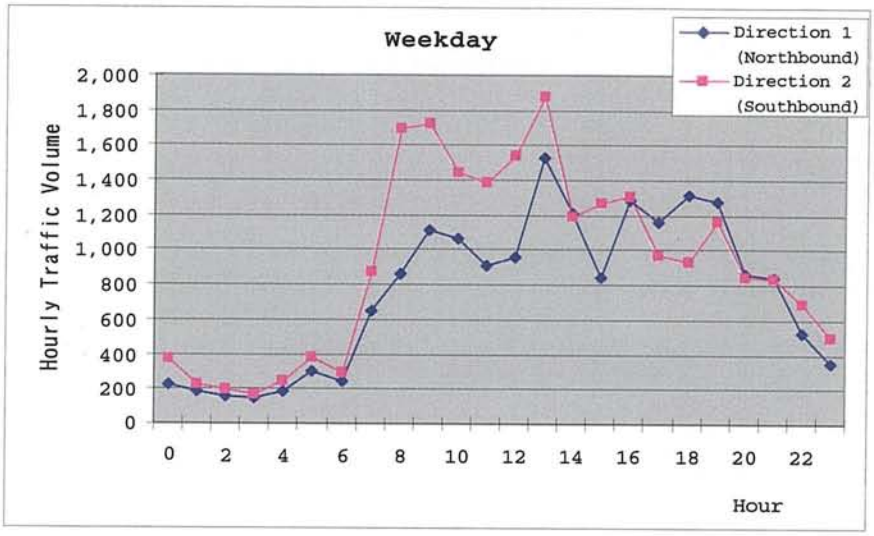


Figure 13.5.1-1 Hourly Traffic Volume Variation

The Level of Service (LOS) on the Ayala Bridge in 2002 was estimated to be D level (v/c ratio = 0.74) from the traffic count survey results conducted in the Study as discussed in **Section 5.4.2.** At this level, the drivers have little freedom to maneuver since traffic condition is approaching unstable flow.

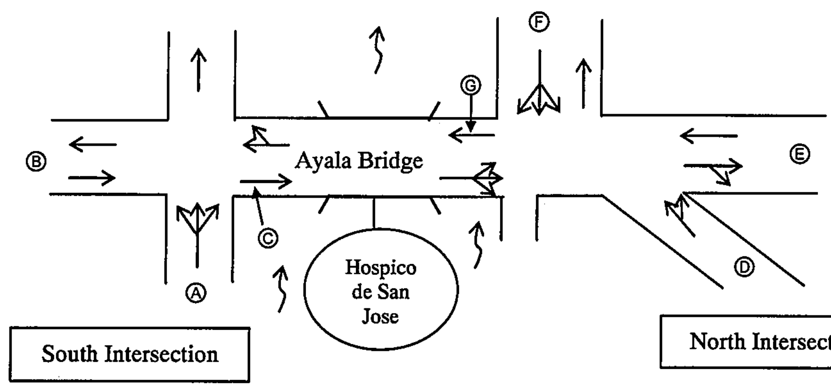
The counted traffic volume on the Ayala Bridge in December of 2002 was 40,390 PCU as AADT, with the vehicle type composition of: passenger car accounting for 92%, jeepney at 3%, large bus at 2% and trucks at 3%.

According to the Geometric Design of Highways and Streets (AASHTO 2001), the appropriate level of service for urban arterial area is required to be C level, that is, stable flow is required. In line with the policy on the geometric design requirements, the extent of traffic congestion is judged to be inappropriate.

13.5.2 Traffic Condition on Adjacent Intersections

Table 13.5.2-1 indicates the level of service of each approach road and intersection.

Table 13.5.2-1 LOS of Approach Roads and Intersections

	South Intersection			North Intersection	
	Delay Time (sec.)	LOS		Delay Time (sec.)	LOS
Approach Ⓐ	97.8	F	Approach Ⓓ	59.2	E
Approach Ⓑ	16.5	B	Approach Ⓔ	29.2	C
Approach Ⓒ	23.2	C	Approach Ⓕ	81.1	F
-	-	-	Approach Ⓖ	24.5	C
Intersection	48.1 (average)	D	Intersection	39.1 (average)	D
Reference Figure					

For the above table, the following features of the intersections are seen:

- The levels of service of both intersections are D, and the delay time is within 60 seconds,
- Approach Ⓐ, Ⓓ and Ⓕ are estimated to be heavily crowded,
- One reason for the traffic congestion of approach road Ⓐ is due to the high composition rate of heavy vehicles: accounting for 14%, and
- There are large variations in the level of service among approaches, which suggests that traffic congestion at the intersections may be improved by leveling the traffic congestion of approach roads with control of traffic signal.

13.6 VULNERABILITY TO DISASTER

Ayala Bridge is located within the area prone to disasters including earthquakes, typhoons, wind and flood. An average of more than 20 typhoons is expected to pass over the Philippine area of responsibility annually. The Philippines is also within the region of high seismic activities.

13.6.1 Earthquake

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

(1) Bridge Site

Ayala Bridge is located 13km from the Marikina Valley Fault System (MVFS). As a rule, bridge structures less than 5km distance are considered highly vulnerable. The 13km distance of Ayala Bridge makes it moderately vulnerable to earthquake.

(2) Date of Construction

Ayala Bridge was originally constructed on 1935 and reconstructed on 1950. Before and during those times, AASHTO has no recommendations with regards to seismic design. Therefore Ayala Bridge could be prone to seismic forces.

(3) Capacity – Demand Ratio under Earthquakes

The safety of the bridge under earthquakes is determined from the capacity and stability of substructures. Vulnerability of the bridge to earthquakes shall be represented by the ratio of capacity-demand under earthquakes.

The existing dimensions and structural data of substructures for the analysis were determined from the field survey results, as-built plans and "Presumption of Original Design" using the old code.

According to the Impact Vibration Test results, the pier may be sound. Calculation results using the old code is given in **Table 13.6.1-1**, which shows that the existing bridge may meet requirements specified in the old code. However, when considering the requirements specified in the latest code, the bridge has high vulnerability to earthquakes as shown in **Table 13.6.1-2.** This means the substructures of Ayala Bridge may not meet the latest seismic code requirements.

Table 13.6.1-1 Capacity / Demand Ratio of the Substructure Using the Old Code

(Using the Old Code)					
Substructure	Pile Foundation	Footing	Main Wall	Column	Wingwall
Abutment–A (South)	1.0 Safe	1.00 Safe	1.00 Safe	---	1.00 Safe
Abutment-B (North)	1.00 Safe	1.00 Safe	1.00 Safe	---	1.00 Safe
Pier	1.13 Safe	---	---	1.27 Safe	---

Table 13.6.1-2 Capacity/Demand Ratio of the Substructure Using the Latest Code

(Using the New Code)					
Substructure	Pile Foundation	Footing	Main Wall	Column	Wingwall
Abutment–A (South)	0.69 Fail	0.61 Fail	0.60 Fail	---	1.00 Safe
Abutment-B (North)	0.57 Fail	0.61 Fail	0.60 Fail	---	0.98 Fail
Pier	0.41 Fail	---	---	0.32 Fail	---

Appendix 13.6.1-1 shows the capacity/demand ratio for each structural component of substructures in detail.

13.6.2 Wind

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

13.6.3 Flood

The pier is located very close to the island being at down stream side. According to the scour survey carried out in the Study, there is no evidence of scouring observed. 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 substructures.

However, considering the present condition and configuration of the Ayala Bridge, especially the pier, the bridge is judged to be not vulnerable to flood action.

13.7 SPECIAL ISSUES

13.7.1 Vessel Collision

(1) Vessel Collision with Girders

The vertical clearance of Ayala Bridge is 3.50m less than a regulatory clearance of 3.75 m.

(2) Vessel Collision with Piers

The navigational space between pier and abutment of Ayala Bridge is 60.2 meters more than the preferable space of 43 meters.

13.7.2 Utilities

The existing utility lines of Ayala Bridge were as follows:

1) Two (2) Ø 400mm Water Pipes
2) Sixteen (16) Electricity Lines
3) Eight (8) Telecommunication Lines

13.7.3 Informal Settlers

There is one informal settler along Ayala Bridge according to the Social Condition Survey and the Environmental Impact Assessment.

Therefore, there will be few problems during the implementation of the project.

13.8 OVERALL ASSESSMENT OF EXISTING CONDITION

The present state of Ayala Bridge condition was summarized as shown in **Table 13.8-1**, and assessed as follows:

(1) Superstructure

- As shown in **Figures 13.4.3-1** and **13.4.3-2**, most of the lower chords show the rating factor, either inventory or operating level, less than 1.0, which means the structural capacity of bridge is less than 32.7 tons (design load).
- Floor system including cross beams and stringers are heavily corroded or fractured, while requiries very urgent replacement.
- The corrosion of members are observed to be heavy due to water leak from the deck slab further and aggravated by a bigger number of vehicles and present load of heavy vehicle than expected.

(2) Substructure

- As shown in **Table 13.5.1-1** the existing pier is sound to carry original design load, with minor damages.
- However, the stability and capacity of the pier and abutment are not sufficient to carry the load of the present design code with 0.4 ~ 0.7 capacity/demand ratios (C/D ratio).
- The insufficient stability of the substructure is due to the change of design code and requirements.
- The foundation is taken to consist of timber piles, which may not meet the latest seismic design code requirements.

(3) Traffic Function and Social Environment

- Large deflection of deck slab occurs during passage of vehicles.
- Traffic load limit is recommended to be set to 3.0 tons.
- As mentioned in **Section 13.1.3**, the National Historical Institute declared that the Ayala Bridge is one of the historical bridge, and strongly endorse the policy that the configuration of historical structures should be preserve.
- The present vertical clearance of 3.50m shall be improved at least up to the regulatory vertical clearance of 3.75m in order to prevent vessel collision with girders.
- No serious dislocation of people affected by the improvement works were observed.

Table 13.8-1 Overall Assessment of Existing Condition of Ayala Bridge

Item			Member/ Location	Rating	Remark	Main Survey Method
Structural Soundness	Superstructure	Shape/Dimension	Bot.Chord/South Span-East Truss	II	Deformed	Shapes/Dimension Survey
			Stringer/ North Span	I	Missing	
			Stringer/South Span	I	Loss End Section	
			Deck Bracing/South Span-East Truss	I	Missing	
			Stringer/South Span-East	I	Reduced Section	
		Material/Damage	Deck Slab/North Span	II	Crack	Damage Survey Material Survey
			Bottom Chord	I	Heavily Corroded	
			Gusset Plate/North Span-Mid Truss	I	Heavily Corroded	
			Cross Beam/North Span-East Truss	I	Heavily Corroded	
			Stringer/North Span	I	Crack (broken)	
			Bearing /All Locations	I	Heavily Corroded	
			130 test locations for UTG	o.k.	7mm to25mm	NDT
			40 test locations for Brinell Hardness	o.k.	328MPa-540MPa	
			37 test locations for UFD	o.k.	No Defects	
		Load-Carrying Capacity	West Truss South Span	o.k.	Deflection (7mm)	Static Load Test
			West Truss South Span	o.k.	Strain (0.6-6.2 me)	
		Operating Level	North Span=3 tons	not o.k.	Equiv. Truck=32.7 tons	Structural Calculations
			South Span = 29 tons	not o.k.	Equiv. Truck=32.7 tons	
		Assessment of Superstructure	**Not O.K. for 32 ton truck & heavy corrosions @ bottom chords & floor system**			
	Substructure	Shape/Dimension	North Abutment	o.k.	-	Shapes/Dimension Survey
			Pier	o.k.	-	
			South Abutment	o.k.	-	
		Material/Damage	North Abutment/Mainwall	III	Crack	Damage Survey Material Survey
			Bearing Pedestal/Pier	I	Crack	
			South Abutment/Mainwall	III	Crack	
			3 test locations for Schmidt Hammer	o.k.	41MPa to 49MPa	NDT
			6 test locations for Coring & Comp.	o.k.	19MPa to 25MPa	
		Stability	North Abutment	-	-	Impact Vibration Test
			Pier	o.k.	17 Hz (Rigid)	
			South Abutment	-	-	
			North Abutment/Mainwall,Wingwall	not o.k.	Bending Moment	Structural Calculations using latest code
			Pier Column	not o.k.	Spiral Requirement	
			South Abutment//Mainwall,Wingwall	not o.k.	Bending Moment	
		Assessment of Substructure	**Minor damage on existing substructure but design can not comply with the latest code**			
	Foundation	Structure/Shape	North Abutment/Ø300 -Timber Piles	not o.k.	665 Pcs.L=15m	Shape/Dimension
			Pier/Ø300 -Timber Piles	not o.k.	523 Pcs.L=15m	
			South Abutment/Ø300 -Timber Piles	not o.k.	576 Pcs.L=15m	
		Scouring	No exposed foundation	o.k.	-	Echo Sound Survey
		Bearing Capacity/Stability	Pier	-	-	Impact Vibration Test
			North Abutment/compression piles	not o.k.	C/D ratio = 0.72	Structural Calculations using new code
			Pier/compression piles	not o.k.	C/D ratio = 0.65	
			South Abutment/compression piles	not o.k.	C/D ratio = 0.67	
		Assessment of Foundation	**Existing Pile Foundation can not comply with the latest AASHTO codes**			
Assessment of Structural Soundness			**Existing Bridge structure needs strengthening to meet the present required strength for a bridge.**			
Traffic Function	Vehicle Weight Limitation Use		5 tons limitation during rehabilitation			
	LOS		D (V/C=0.70, V=42,000, C=59,000 PCU)			
	Geometrical Features		Fair (including access road to Hospicio de San Jose)			
	Safety of Vessel Transport		An existing vertical clearance of 3.50 m is insufficient for a regulatory clearance of 3.75 m.			
	Assessment of Traffic Function		**Insufficient**			
Social Environment	Utilities Hanged at the Bridge		2 water pipes (d=400mm), 16 electricity lines and 8 telecommunication lines.			
	Squatters		Four families live around the south abutment.			
	Historical Aspects		National Historical Institute lists Ayala Bridge as a historical structure.			
	Assessment of Social Aspects		**Historical situation shall be considered.**			
Overall Assessment			**Superstructure is necessary for the improvement work. Substructure shall be strengthened. Vertical clearance shall be improved at least up to regulatory vertical clearance of 3.75 m.**			