A COMPREHENSIVE STUDY ON THE VIBRATION OF HIGHWAY BRIDGES

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Final Report

Volume I

EXECUTIVE SUMMARY

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A COMPREHENSIVE STUDY ON THE VIBRATION FOR HIGHWAY BRIDGES

FINAL REPORT

VOLUME I:
EXECUTIVE SUMMARY

ABSTRACT

There had been public complaints of "excessive" vibration felt on our highway bridges which was reported in the local newspapers recently. The concerns were targeted at the MRR II viaducts at Ampang (in front of Flamingo Hotel hereinafter known as the Ampang Bridge) and at Kepong (near Desa Jaya Shopping complex known hereinafter as the Kepong Bridge). JKR had viewed the public complaints of bridge vibration seriously and is desirous in carrying out a comprehensive study to address this public concern with respect to the safety and serviceability of the bridges*.

Bridge vibration may sometimes indicate a loss in structural integrity of the bridge due to presence of damage or defects in the bridge. The opposite is also true whereby excessive vibration may sometimes result in damage to the bridge components. In either case, a comprehensive inspection of the bridge is necessary to ensure safety. It is also important to note that although the bridge integrity is intact, the level of vibration may still be sufficiently high as to cause discomfort or annoyance to the motorists. Thus, the level of vibration caused by the traffic must be measured and compared with the acceptable threshold value provided by established codes or standards. Should the vibrations exceed this value then some mitigation measures or control may be necessary to improve the level of service.

This is the approach to the vibration study as outlined in the JKR TOR. Local consulting firm Evenfit Consult Sdn. Bhd. (the Consultant) was commissioned by JKR to carry out this study. The study commenced on 1st October, 2012 and was completed on 31st March, 2013.

Access to the bridge for inspection was made using the skylift, ladder and scaffolding depending on the site conditions. Inspection was carried out visually, following the

* JKR had since decided that the Kepong Bridge be removed from the study in view of the fact that repair and strengthening work is now in progress and any assessment of the bridge could only be in the interim.

procedures outlined in the REAM (Road Engineering Association of Malaysia) guide on bridge inspection. Each bridge component of the 24-span bridge was checked visually for any sign of damage or defect. The severity and extent of the damage or defects found were further assessed and reported.

For bridge vibration monitoring, vibration caused by the ambient traffic was measured using an *accelerometer* installed under the bridge deck. The vibration measurement was carried out by a team of researchers from Universiti Teknologi Petronas Sdn. Bhd. (UTP). The data collected was analysed by the UTP team to obtain the statistical distributions of the data for comparisons with the acceptable value and to determine the dynamic characteristics of the bridge. The dynamic characteristics of the bridge in the forms of *natural frequencies*, *mode shapes* and *damping ratios* represent the inherent property of the bridge in vibration.

An analytical model based on the 3D Finite Element (FE) Method was also developed to assess the dynamic response of the bridge on a theoretical basis. Due to inevitable deviation in the structure's construction details from the design, and uncertainties associated with time-dependent material properties, support conditions etc., it is difficult to establish an FE model to represent the actual structure. Besides, it is well accepted that computer modelling alone could not determine completely the dynamic behaviour of the structures because certain properties such as damping and nonlinearity do not conform to traditional modelling treatment. As a standard engineering practice, the analytical model was "tuned" and calibrated against the modal data from field measurements. The analytical model thus calibrated could then be used to compute the vibration that would be caused by legal loads permitted under the Weight Restriction Order (WRO) 2003. These computed vibrations were then compared with the acceptable threshold value to determine whether the bridge needs to be strengthened or modified.

To ensure safety of the bridge during the study, deflections of two of the bridge spans and pier cross beams were monitored continuously for a period of 5 months. A *total station* was employed to take measurements of the deflections. Should the deflection readings suggest a progressive increase in these deflections then an alert would be triggered to initiate a closer surveillance of the bridge.

From the study, it was concluded that the vibration levels did not exceed the acceptable "High vibration" limit as to require any mitigation measure, although they were at the level that could be perceived by road users. No major damage or defects were detected in the bridge inspection. The few minor defects observed were considered not severe or extensive as to reduce the strength of the bridge or to contribute to the vibration of the bridge. They are indeed defects that were commonly found in concrete bridges in the country. As a good maintenance practice these minor defects may be repaired.

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Notation

AAR	Alkali Aggregate Reaction
AASHTO	American Association of State Highway and Transportation Officials
Ab	Abutment
a	Acceleration
a_{lim}	Limiting value for acceptable level of acceleration
a_{mes}	95th percentile of measured acceleration
a_{WRO}	Computed acceleration due to WRO Vehicle
a_x	Acceleration in the x direction
a_y	Acceleration in the y direction
a_z	Acceleration in the z direction
AV	Articulated Vehicle
DEF	Delayed Ettringite Formation
d_{LTAL}	Deflection due to LTAL
d_{MTAL}	Deflection due to MTAL
d_{mes}	Deflection measured
d_{WRO}	Deflection due to WRO vehicle
E_c	Young's Modulus for Concrete
f_i	Natural frequencies for mode $_i$ (from measured data)
f_i'	Natural frequencies for mode $_i$ (from analytical model)
f_o	Natural Frequency
FE	Finite Element
GVW	Gross Vehicle Weight
Hz	Hertz
K_b	Elastomeric Bearing Stiffness
K_f	Foundation Stiffness
KPH	Kilometer Per Hour
LHS	Left Hand Side

LTAL	Long-Term Axle Load
L_{LTAL}	Static Load effect due to LTAL
L'_{WRO}	Dynamic Load effect due to WRO Vehicle
*m*g	Milli g (gravity)
MTAL	Medium-Term Axle Load
M_{LTAL}	Moment due to LTAL
M_{WRO}	Moment due to WRO vehicle
N	Number of truck in a lane
NB	North Bound
Φ	Diameter
$\Phi(y)_i$	Mode shapes for mode $_i$ (from measured data)
$\Phi(y)_i'$	Mode shapes for mode $_i$ (from analytical model)
ρ_c	Deck mass density
RV	Rigid Vehicle
SB	South Bound
V_t	Travelling Velocity
WRO	Weight Restriction Order
ζ	Damping ratio
ζ_i	Damping ratio for mode $_i$ (from measured data)
ζ_i'	Damping ratio for mode $_i$ (from analytical model)

1 INTRODUCTION

1.1 General

Vibration on highway bridges is not uncommon. Public complaints on the vibration felt on highway bridges are commonplace. Changes in the modal parameters such as natural frequencies, mode shape and modal damping may be caused by changes in stiffness, mass and damping in a particular structure [1]. It is also well known among structural engineers that the presence of damage or defect in a bridge would result in changes in its structural stiffness. As such, bridge vibration may sometimes indicate a loss in structural integrity of the bridge due to presence of damage or defects in the bridge. On the other hand, excessive vibration may sometimes result in some damage to the bridge components. In either case, a comprehensive inspection of the bridge is necessary to ensure safety.

Bridges designed today require them to be lighter, more flexible and yet strong. This may lead to bridges that while fulfilling their *serviceability* limit state requirement as stipulated in the current codes, may give rise to new problem never experienced before [2]. Assessment of the dynamic performance of an existing bridge is thus important even if the bridge integrity is intact and the design complies with the design specifications.

1.2 Background

There had been public complaints of "excessive" vibration felt on our highway bridges which was reported in the local newspapers recently (Please refer to Appendix A and Appendix B). The concerns were targeted at the MRR II viaducts at Ampang (in front of the Flamingo Hotel hereinafter known as the Ampang Bridge) and at Kepong (near Desa Jaya Shopping complex known henceforth as the Kepong Bridge). JKR had viewed the public complaints of bridge vibration seriously and is desirous in carrying out a comprehensive study to address this public concern with respect to the safety and serviceability of the bridges.

Evenfit Consult Sdn. Bhd. (the Consultant) was commissioned by JKR to carry out a comprehensive study on the vibration for highway bridges. The study on bridge vibration is indeed a very specialized field of bridge engineering, requiring a large team of personnel and professionals with expertise in apparently diverse areas such as structural design and modal testing. As a result of this requirement and in fulfilling the needs specified in the TOR, the Consultant has invited the participation of a few specialist consultants in this study. They are: Ir. Dr. Mohd. Shahir Liew, Ir. Amir Ismail and Ir. Koo Chuan Seng from Universiti Teknologi PETRONAS Sdn. Bhd, ZECA Consult, and SCE Consultants Sdn. Bhd., respectively. The roles of each of these consultants are listed in Table 1 below:-

Table 1: Key personnel of the Study Team

No.	Names	Organisations	Positions	Responsibilities
1.	Ir. Dr. Ng See King	K & N Consulting Engineers Sdn. Bhd.	Project Manager	Overall management and administration of the study
2.	Ir. Amir Ismail	Zeca Consult	Head of Vibration Assessment	Management and Coordination of Vibration Assessment
3.	Ir. Ku Mohd.Sani	Evenfit Consult Sdn. Bhd.	Head of Bridge Inspection and Deflection Monitoring	Management and Coordination of Bridge inspection and deflection monitoring
4.	Ir. Dr. Mohd. Shahir Liew	Universiti Teknologi Petronas Sdn. Bhd	Vibration Measurement Expert	Vibration Monitoring and User Sensitivity Study
5.	Ir. Koo Chuan Seng	SCE	Senior	Development of the

		Consultants Sdn. Bhd.	Structural Engineer	Analytical Model
6.	Ir. Munning Jamaluddin	Materials Testing Laboratory Sdn. Bhd.	Deflection Monitoring Engineer	Deflection Monitoring
7.	Engr. Jamil Abdullah	Associate of Evenfit Consult Sdn. Bhd.	Senior Bridge Inspector	Coordination of Bridge Inspection

1.3 The Objective and Scope of Study

The main objective of the study, as stated in the JKR TOR, is to obtain a better understanding of the dynamic performance of the highway bridges and the vibration sensed by bridge users in order to establish the acceptable level of comfort and/or compliance with the Code of Practice.

This objective shall be achieved through the completion of the following tasks, as stated in the JKR TOR:-

(i) To carry out a comprehensive inspection of the bridge superstructures and substructures at MRR II viaducts at Kepong and Ampang sites as described previously.

(ii) To carry out field vibration measurements and monitoring of the bridge superstructure under actual traffic conditions including interpretation of results for compliance with Code of Practice or best practices.

(iii) To carry out analytical computation of bridge dynamic performance under actual traffic conditions including interpretation of results with respect to the measured vibration from (ii).

(iv) To establish or identify reasonable dynamic criteria for user sensitivity to bridge vibrations.

(v) To propose vibration mitigation measures, when necessary.

JKR had, in the ensuing meetings, decided that the focus of the study should be on the Ampang Bridge and that the Kepong Bridge would be excluded from the study. This decision was made based on the observation that the Kepong Bridge is currently under repair and strengthening; and any assessment now could thus be only in the interim.

1.4 Organisation of Report

The Final Report is a collection of works carried out in the study. It is presented in five volumes, comprising:-

Volume I: Executive Summary

Volume II: Inspection of the Ampang Bridge

Volume III: Deflection Monitoring

Volume IV: Bridge Vibration Monitoring Study and User Sensitivity Study

Volume V: Analytical Modelling

Volume I of the Final Report is intended as an executive summary of the report giving an overview of the works carried out and presenting the main findings. Since it is a condensation of the works already covered in the other volumes it is focused only on major findings. Volume II reports on the detailed inspection of the Ampang Bridge concluding with some recommendations for maintenance and repair. Volume III covers the survey work to monitor the deflection of the bridge for signs of failure. A big collection of raw data was compiled in its Appendix.

Volume IV is divided into two main sections: Section A deliberates on bridge vibration monitoring study and Section B on user sensitivity study. Volume V is devoted to a detailed description and discussion of works in calibrating the analytical model and its application in response prediction once calibrated.

Chapter 1 of the Executive Summary Report contains an introduction of the study, stating the background, main objectives and scope of the study. Chapter 2 presents a description of the bridge and the referencing system to the bridge components.

Familiarity of these terminologies and jargons may be crucial to the understanding the remaining part of the report. Chapter 3 covers the approach adopted by the Consultant in carrying out the study. Results of bridge inspection are covered under Chapter 4. Each of the defects observed is discussed and recommendations of repair provided. Chapter 5 covers the vibration study, which comprises three main activities: identification of user sensitivity criteria, vibration measurement and analytical modelling. Comparisons of the measured vibration and computed acceleration from the analytical model with the acceptable limiting value are made and presented toward the end. The last chapter of Volume I, Chapter 6 gives an overall discussion of main findings of the study and presents the conclusions as well as some recommendations.

2 DESCRIPTION OF THE BRIDGE AND REFERENCING SYSTEM

2.1 Description of the Bridge

The Ampang Bridge, located in front of the Flamingo Hotel in Ampang, is part of the Middle Ring Road II road network (MRR II). It was constructed in 1995 and comprises 24 spans with a total length of 960m (see Figure 1). The bridge was constructed by M/s IREKA Construction Berhad and designed by Perunding Hashim & Neh Sdn Bhd which was engaged by the Contractor to perform the alternative bridge design.

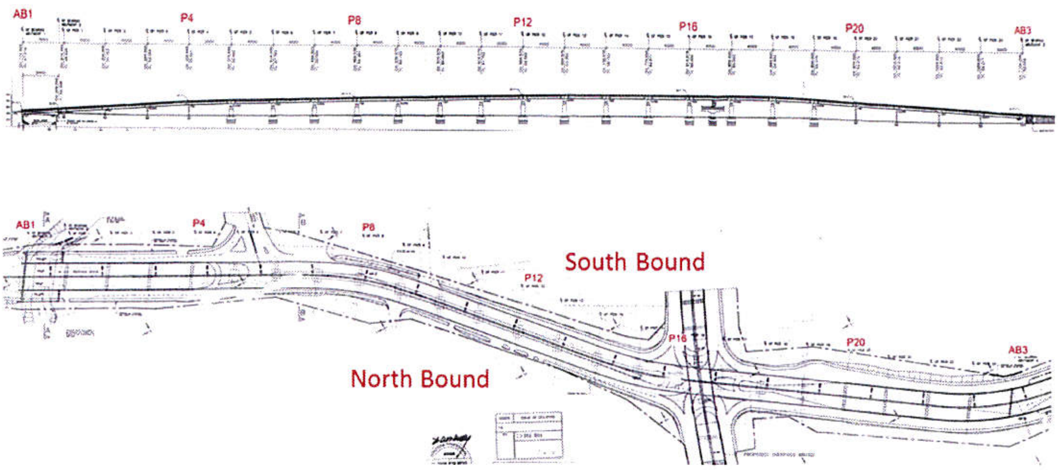

Figure 1: Elevation and plan of Ampang Bridge

The superstructure comprises two independent decks; carrying south and north bound traffic, which are supported by common substructures. Each deck is made up of an 180mm thick RC deck slab on 6 nos. of 1.985m deep precast post-tensioned tee girders spaced at 2.0m centres with an overall deck width of 12.7m. The bridge deck only has end diaphragms provided at both girder ends, and no intermediate diaphragm was provided. The typical cross section of the bridge deck is as shown in Figure 2.

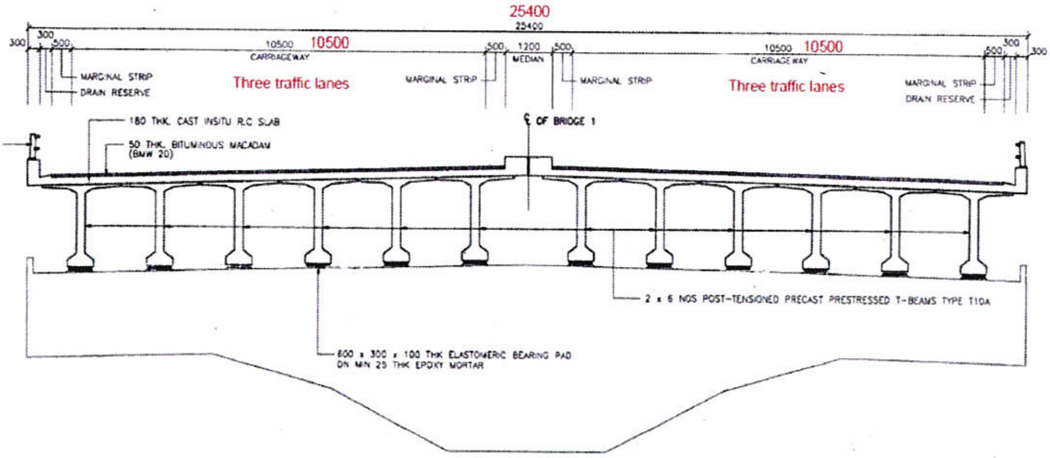

Figure 2: Cross section of the Ampang Bridge

In general, the Ampang Bridge can be divided into six independent frames:-

(i) *Frame 1: Ab* A/P*1/P2/P3/P4 (4 spans).*

(ii) *Frame 2: P4/P5/P6/P7/P8 (4 spans).*

(iii) *Frame 3: P8/P9/P10/P11/P12 (4 spans).*

(iv) *Frame 4: P12/P13/P14/P15/P16 (4 spans).*

(v) *Frame 5: P16/P17/P18/P19/P20 (4 spans).*

(vi) *Frame 6: P20/P21/P22/P23/Ab B (4 spans).*

Each frame comprises a typical continuous deck over 4 spans with joints provided at both ends. Abutment A, P4, P8, P12, P16, P20 and Abutment B are thus locations where expansion joints were installed. There are two types of expansion joints provided, i.e., elastomeric load bearing type and asphaltic plug type. The beams are supported by elastomeric bearings (600mmx300mmx100mm thick with the fixity provided with high yield strength Φ32 galvanised dowel bars at P2, P6, P10, P14, P18 & P22, i.e., at the middle pier of each continuous frame to restrain the top deck from longitudinal and transverse movements.

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* *Ab* denotes Abutment, *P* denotes Pier

The two bridge decks of the Ampang Bridge are supported by RC wall type abutments and two types of pier, namely "T" pier and multi-column pier. The bridge deck which has an arched profile is supported by multi-column piers at the end spans (approaches), i.e., P1 to P4 and P20 to P23 while the 'T' piers from P5 to P19 support the higher deck in the intermediate spans.

The multi-column piers were constructed using conventional cast-insitu construction method. The pier consists of 5 nos. of RC Φ0.9m columns at 5.26m spacing and with 1.7mx1.7m RC crosshead. The length of crosshead is about 25.4m long (see Figure 3).

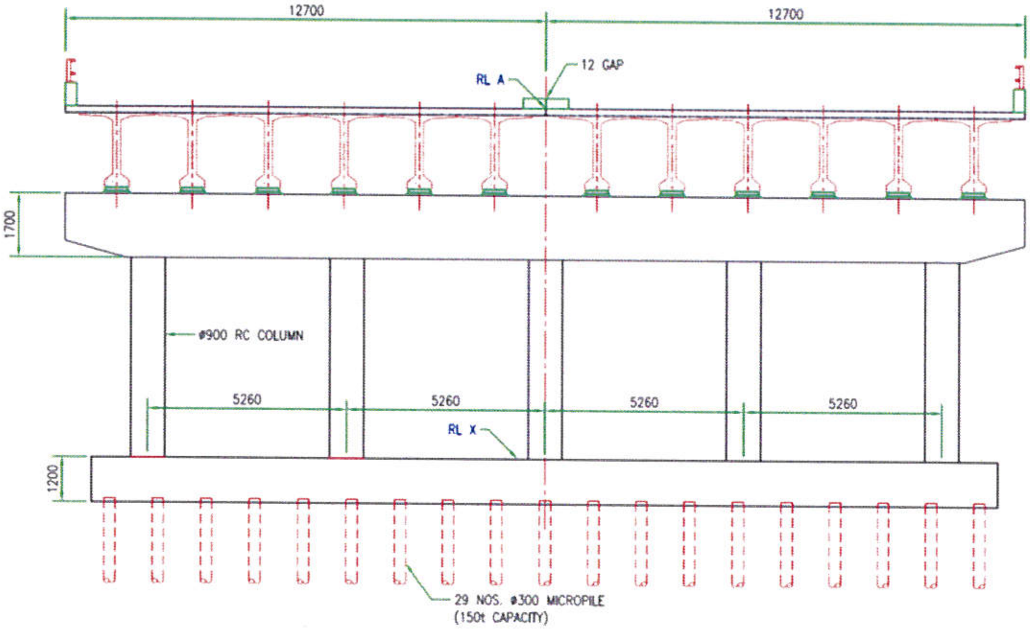


Figure 3: Typical multi-column pier for piers P1 to P4 and P20 to 23

The "T" piers were constructed using a unique proprietary construction method called "SOSROBAHU". Initially pier column of Φ4.0m was constructed and when completed, the prestressed crosshead (cross beam) with depth varies from 4.25m to 2.0m was temporary cast in the direction parallel to the existing road. Once the concrete had achieved its design strength, the crosshead was rotated to its final design position using the special "Sosrobahu" device which was cast in the pier between column top and crosshead. The typical elevation of the 'T' pier is shown in Figure 4. The width of the prestressed crosshead is typically 3.0m. The maximum column height of the 'T' pier is 10.945m (from top of pile cap to top of crosshead) at Piers P16 and P17, i.e., at the highest level of the bridge.

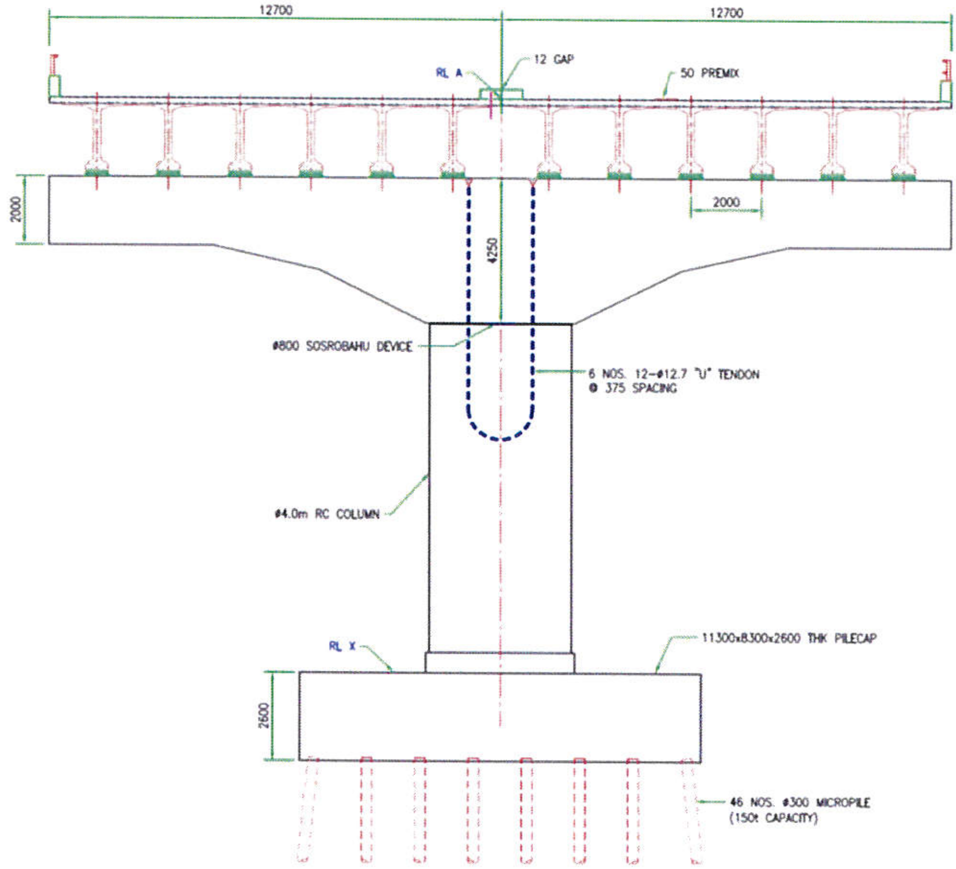


Figure 4: Typical "T" pier for piers P5 until P19

2.2 Bridge Referencing System

Devising a referencing system that is easy to apply and understand was an important task done prior to starting the study. This is to ensure a systematic recording and reporting of the conditions of the bridge components; common understanding of component identification among the Study Team members; and for ease of JKR in understanding the reports.

The referencing system for the Ampang Bridge generally adopted the reference indicated in the as-built drawings and where this was not available the referencing of the components was assigned following the guideline stated in the REAM's publication on 'A Guide to Bridge Inspection' [3]. The referencing system for Ampang Bridge is as described below:-

i. Spans and piers are numbered 1, 2, 3 etc., in ascending order starting from Keramat (see Figure 5).
ii. Abutment A is the abutment near to Keramat and Abutment B is the abutment near to Pandan Indah (see Figure 5).
iii. Beams are numbered 1, 2, 3, etc., in ascending order from left to right while facing towards Pandan Indah (see Figure 6).
iv. The referencing for the surfaces of the beams is based on where the beam surface is facing as shown in Figure 7.
v. The referencing for the surfaces of the piers is based on where the pier surface is facing as shown in Figure 8.
vi. The bearings are referenced as the bearing number followed with the pier number, span number and the traffic flow. For example for a bearing at Pier 16 supporting Beam 1 of Span 16 on a northbound traffic, the reference number shall be BG 1/P 16/S 16/NB (see Figure 9).

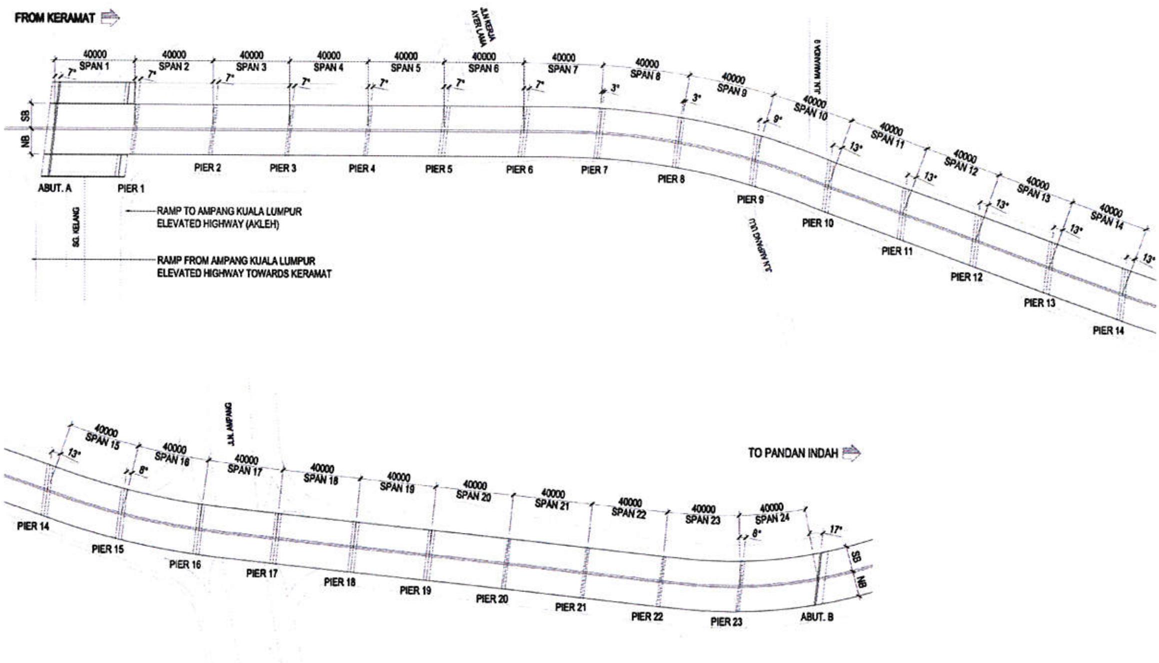


Figure 5: Naming of spans and substructures

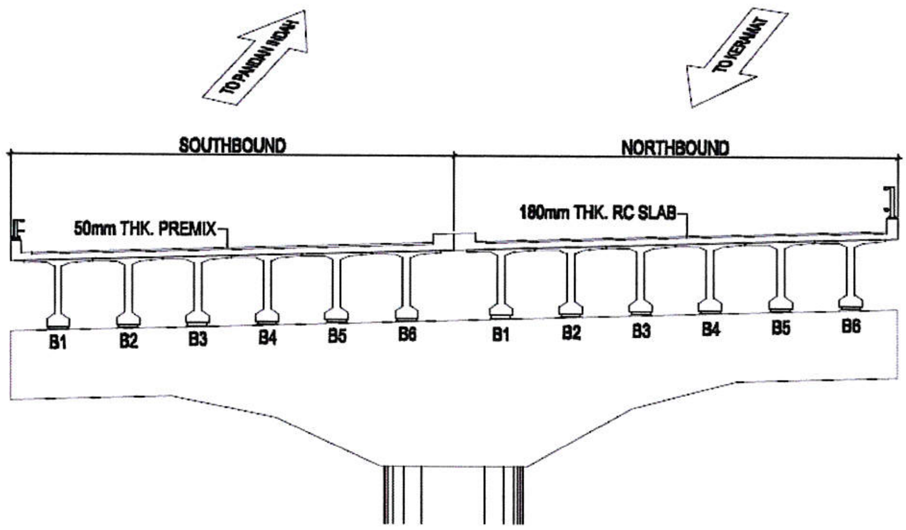


Figure 6: Naming of bridge beams

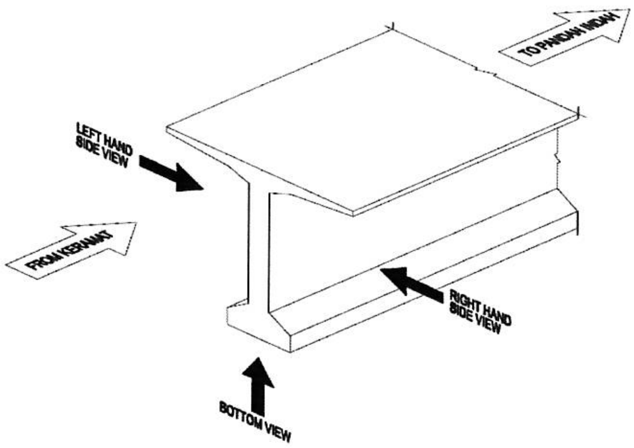
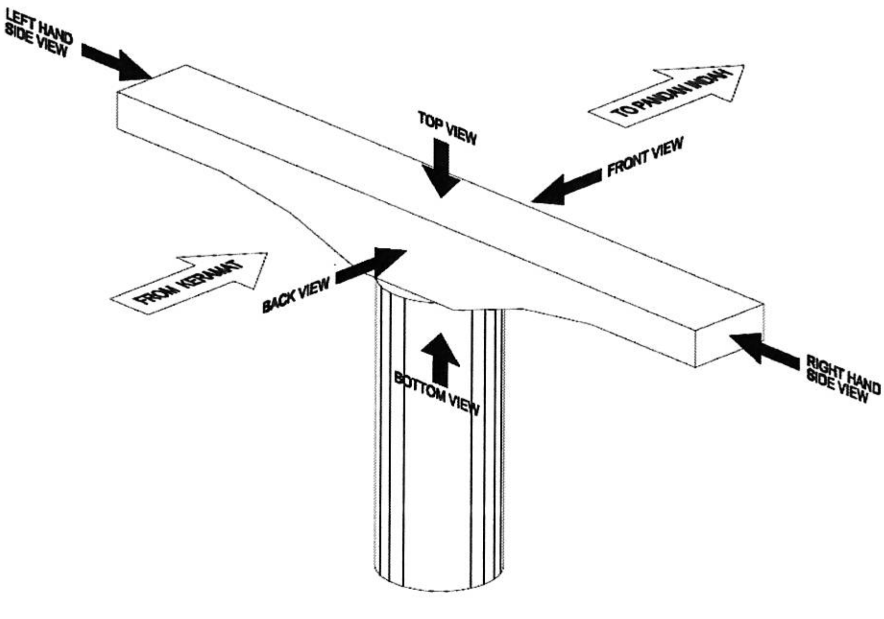
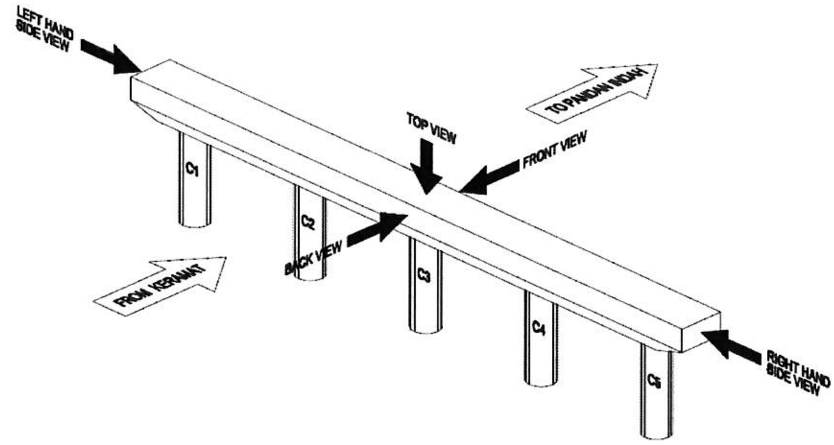


Figure 7: Referencing system for the sides of the beam



a) Referencing system for surfaces of 'T' pier



b) Referencing system for surfaces of multi-column pier

Figure 8: Referencing system for surfaces of piers

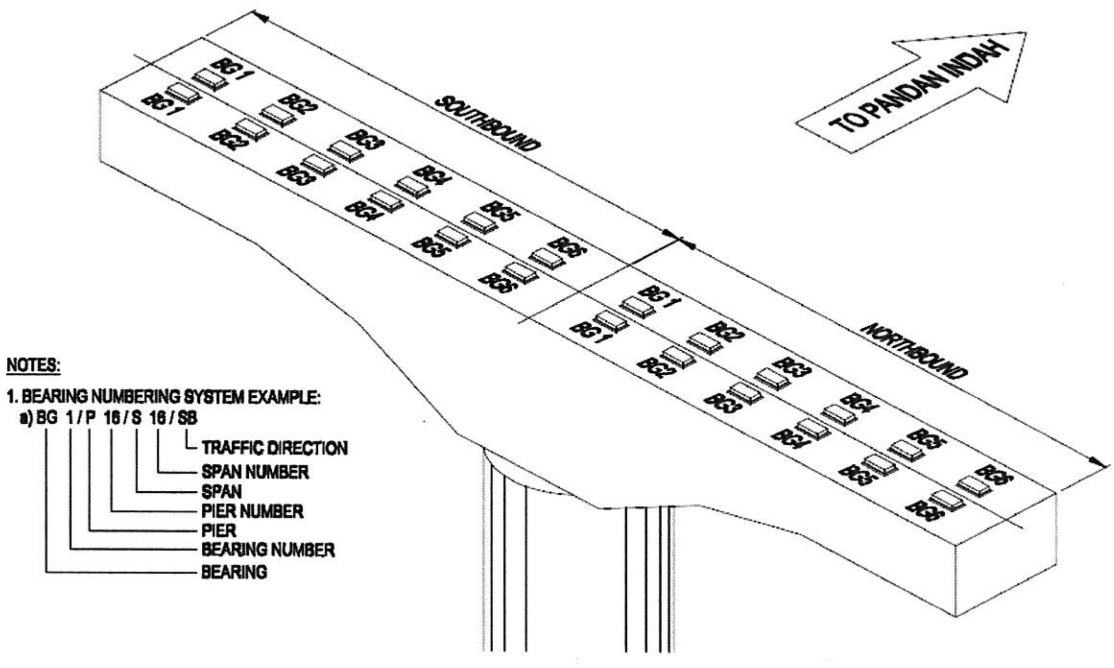


Figure 9: Referencing system for bearings

3 STUDY APPROACH

3.1 Introduction

The main objective of the study was stated in the JKR TOR as "... to obtain a better understanding of the dynamic performance of highway bridges and the vibration sensed by bridge users in order to establish the acceptable level of comfort and/or compliance with the Code of Practice". We can discern from the scope of study stipulated in the TOR that the JKR's immediate concerns are really more specific to the safety and serviceability of the two bridges at Ampang and Kepong.

The mandate given to the Consultant is thus conceived as one to address the vibration problem from two aspects: 1) structural safety and 2) serviceability in terms of bridge vibrations. With the completion of the tasks in addressing these two issues the objective as mentioned in the TOR shall be achieved.

3.1.1 Issue on structural safety

It was known, from desk study of the as-built drawings, that the bridge was designed to the Long-term Axle Load (LTAL) standard (which also involved checking against 20 units of Special vehicle (SV) loads). The current Weight Restriction Order (WRO 2003) [4], however, restricts the vehicles to the Medium-term Axle Load (MTAL) standard, a lower load intensity standard.

To ensure safety of the bridge during the study, deflections of two of the bridge spans and pier cross beams were monitored continuously for a period of 5 months. A *total station* was employed to take measurements of the deflections. Should the deflection readings suggest a progressive increase in these deflections then an alert would be triggered to initiate a closer surveillance of the bridge. This exercise, besides serving the purpose as a warning system for a bridge failure, would also provide some "bench-marked" values for the theoretical deflections to be compared. Measured deflection is almost always much smaller than the theoretical value and if the situation proved otherwise the integrity of the bridge is questionable.

3.1.2 Issue on serviceability in terms of vibration

Another aspect of the problem is now discussed: the issue of discomfort or annoyance felt by motorists due to the "excessive" vibration (serviceability limit state). Now, "how excessive is excessive" is the question! First, the Consultant has to identify, through literature review, the recommended criteria or measure that best indicate the level of vibration. The Consultant would next, through literature review, identify and recommend the critical value of acceptance for decision makings. In this respect, the value would serve the function of permitting a check of the vibration level to determine whether a corrective action is needed.

The Consultant had proposed that vibration measurement be made for a period of 2-8 hours under normal traffic condition. This duration of data collection is generally sufficient for the "stationary" assumption central to the technique used in the analysis of the data. The measured response from ambient excitation is regarded as the lower bound level of the actual traffic, which explains why an analytical model is needed.

As is customary in engineering, an analytical model is often used to predict the response under a situation that is not practical or too expensive for direct measurements. The present study thus requires that an analytical model be established in order that the response other than those measurable could be predicted. These may include the situation when the bridge is subjected to the loading from the WRO vehicles (vehicles complied with WRO 2003) and/or when the bridge had undergone some structural modifications. This model is to be based on a 3D Finite Element (FE) tuned and calibrated against the *modal model (experimental model parameters)* obtained from the *modal analysis* of the vibration data.

3.2 Study Approach

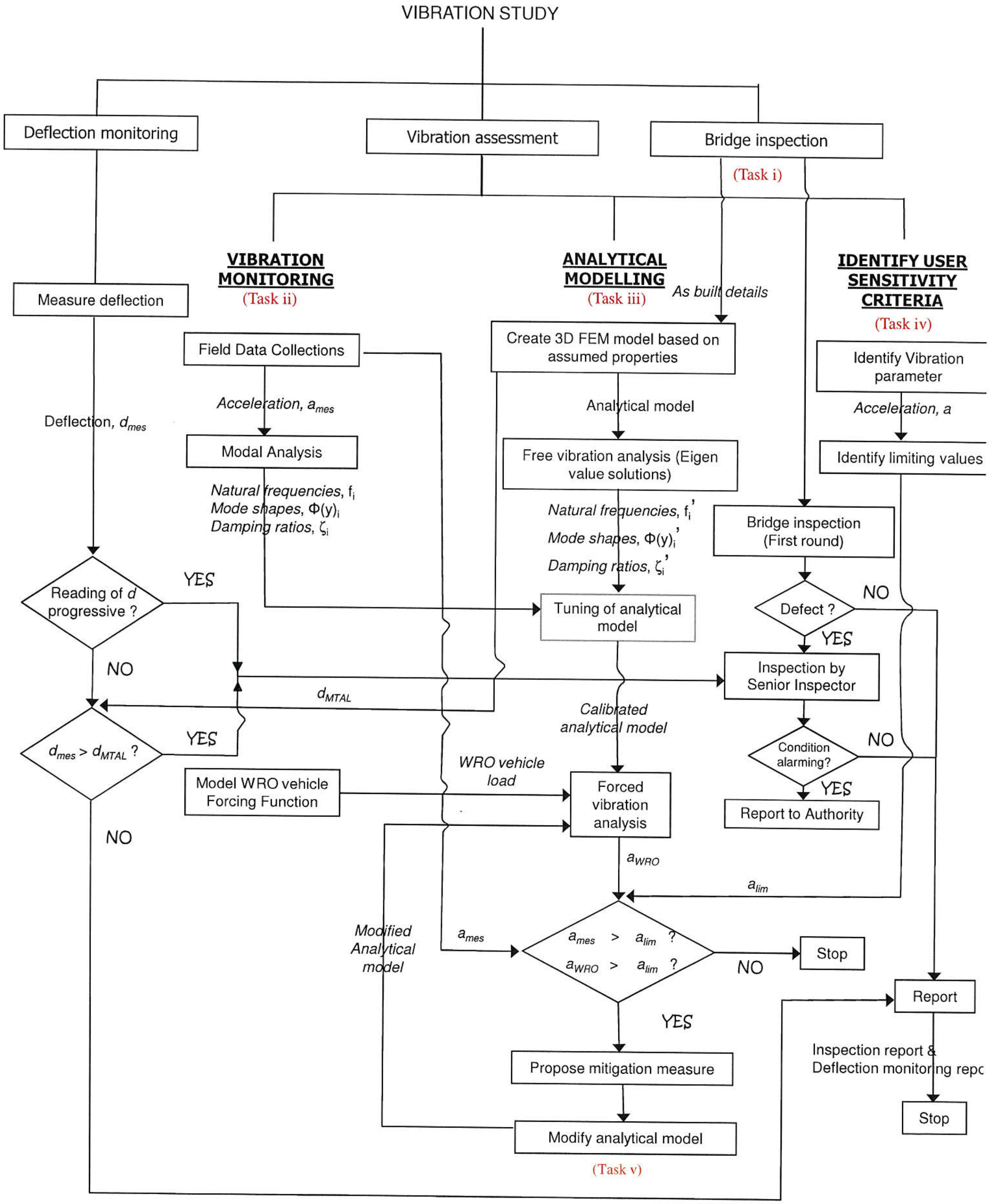
The approach adopted by the Consultant followed closely the tasks stated in the JKR TOR. The interaction of these tasks is depicted in Figure 10. The methodology undertaken for each of these tasks is now discussed.

3.2.1 Bridge inspection (Task i)

The TOR has required that the Consultant assess the condition of the bridge superstructures and substructures by a thorough inspection of the exposed bridge components in order to identify if there is any damage/defect to the structural members that may contribute to the vibration of the bridge.

Bridge inspection of the Ampang Bridge was carried out by the Consultant between the months of October 2012 and February 2013. The inspection was conducted by two inspection teams, each of which comprising an engineer inspector and two technician inspectors in order to complete the work within the stipulated time. The engineer inspector took photographic records of the damage/defects and recorded them in a photo list. The engineer inspector would then collate the photographs back in the office and prepare the report on the conditions of the bridge in the form of pictorial reports.

The technicians assisted the engineers in locating the damage/defects and recording them in the checklists adapted from the REAM Guide. The conditions of the bridge components in the span were recorded in one checklist. These ratings were then summarised with the highest ratings for each component in a Summary Report back at the office. Besides the condition ratings the Summary Report also gives some brief descriptions of the damage with brief recommended actions for the components in question.



Note : For notations and abbreviations refer to List of Notations and Abbreviations on Page xvi.

Figure 10: Study Approach

The conditions of the bridge were rated following the REAM Guide. The procedures involved a systematic inspection of every major bridge components and assigning condition ratings as shown in Table 2 below. Each bridge component would then be rated based on the worst condition ratings of the damage observed in the particular component.

Table 2: Definition of condition rating for Structures

RATING	DEFINITION
1	No damage found and no maintenance required as a result of the inspection.
2	Damage detected and it is necessary to record the condition for observation purposes.
3	Damage detected is slightly critical and thus it is necessary to monitor or implement repair work / routine maintenance work.
4	Damage detected is critical and thus it is necessary to implement repair work or to carry out a detailed inspection to determine whether any rehabilitation works are required.
5	Being heavily and critically damaged and possibly affecting the safety of traffic, it is necessary to implement emergency temporary repair work immediately or rehabilitation work without delay after the provision of a load limitation traffic sign.

The works by the two inspection teams were planned, scheduled and coordinated by the Senior Bridge Inspector. He was also responsible for reviewing and checking the reports for accuracy. He also went for further investigation of any damage reported by the bridge inspectors.

Access during Inspection

The Ampang Bridge has an arched profile, i.e., a low clearance (headroom) at the approaches and increasing in height towards the middle portion of the bridge, with the maximum clearance of about 10m at Pier 16 and 17. Considering the bridge configuration, the access for inspection was done in the following manners by the Consultant:-

a. By foot for inspection of the approach spans, namely Spans 1 to 4 and Span 24. Some of the higher piers and bearings were accessed by the use of a ladder and some of the higher deck was inspected by using binoculars.

b. Using a skylift for inspection of Spans 5 to 23. For the inspection of these spans, the skylift was positioned on the central median near the two piers of that span in order to have the closest access to the pier crossbeams, bearings, beams and end diaphragms (See Figure 11).

Figure 11: Inspection using skylift

During the early stage of the study, the Consultant had used an unmanned aerial vehicle (UAV) with attached camera for inspection of the bridge deck and bearings (see Figure 12). The pictures collected at this early stage were useful as a reconnaissance survey and presented in the preliminary inspection reports.

Figure 12: Remote controlled UAV used for inspection

Safety during Inspection

The Consultant had placed great emphasis on safety during inspection. The inspectors were required to put on all the safety attire such as safety jackets, helmets and boots during the inspection. Inspection was always carried out in a team of at least two inspectors to make sure that the inspectors could look after one another in case of untoward accidents to any one of them.

In the event that the skylift was positioned on the road and disrupting the traffic flow, adequate safety cones and flagmen were stationed to ensure a smooth and safe control of the traffic flow (see Figure 13).

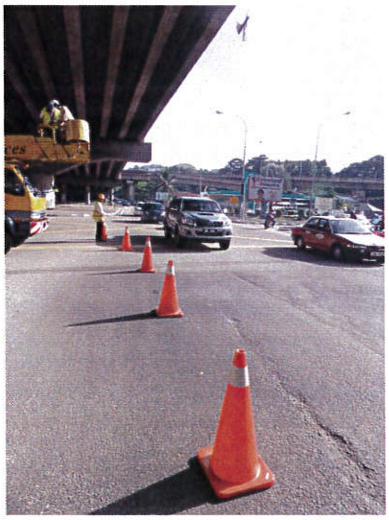


Figure 13: Traffic control during bridge inspection

3.2.2 Deflection monitoring

<u>The Equipment</u>

The deflection monitoring was conducted by means of a survey instrument taking deflection readings by placing the monitoring prisms and referenced to static control prisms installed at some buildings nearby. The taking of the deflection readings was done by Global-trak Systems Sdn. Bhd. (the Surveyor), a leading provider of advanced positioning solutions.

The Surveyor used the Trimble Real-time Monitoring to monitor the deflection of the beams and pier. This system consists of a Trimble S8 1" Robotic HP with Finelock Total Station (Figure 14) for target measurement; Trimble Access Software for field data collection, data computation, analysis of target displacement and review; Trimble Monitoring Prisms (Figure 15) for target (monitoring point) and as references (control points); an internal barometer in total station for automatic PPM correction of EDM distances measurement.

Figure 14: Trimble S8 1" robotic total station

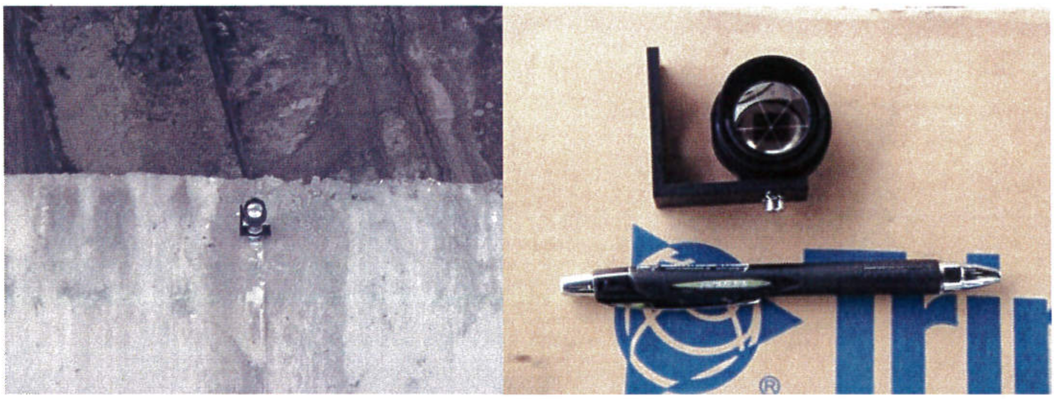


Figure 15: The monitoring prism to be installed on the bridge

<u>Monitoring Points</u>

Monitoring points of interest were the end of the pier cross beam (cantilever) and in the span. From 2-10-2012 till 5-10-2012 Pier 11 and Span 12, thought to be the tallest pier then, were monitored.

i. 7 prisms along the crossbeam of Pier 11 – one each at the ends and the rest equally spaced in between the prisms. This is to monitor the deflection/movement of the cantilever arm (see Figure 16).

ii. 5 prisms each along Beam 1 and Beam 3 of the northbound bridge deck – one at the mid-span two at the quarter span and two at span end (see Figure 17).

iii. 4 control prisms on buildings/structures nearby.

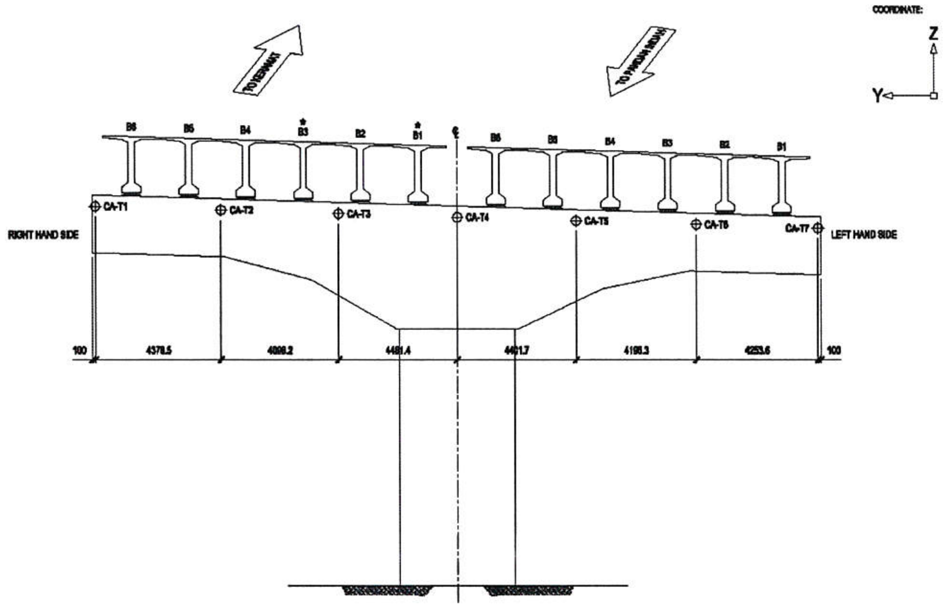


Figure 16: Location of prisms at Pier 11

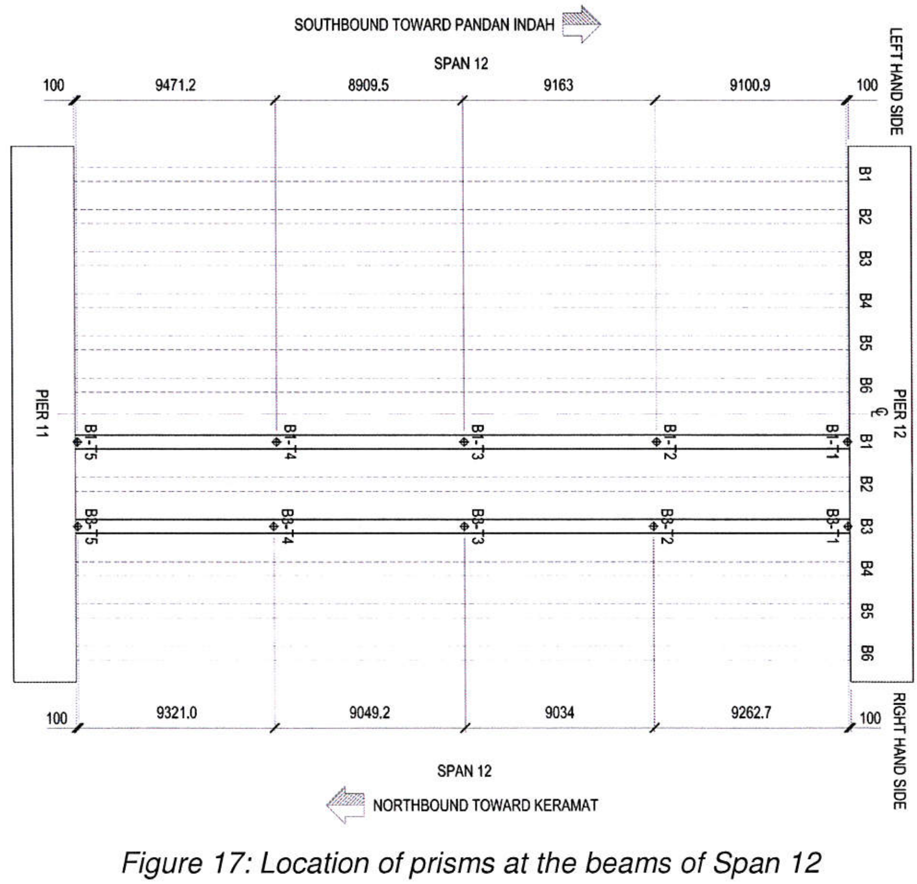


Figure 17: Location of prisms at the beams of Span 12

When the drawings became available from JKR it was noted that Piers 15, 16 and 17 were much taller than Pier 11 and the monitoring points were moved to Pier 15 and Span 16. A slight modification was also made to include the study of deflections at the edge and intermediate beams for both southbound and northbound bridges. The installation of the devices was as follows:-

i. 5 prisms along the crossbeam of Pier 15 – directly below B1/SB, B3/SB, B4/NB, B6/NB and in the middle of the pier crossbeam (see Figure 18).

ii. 3 prisms each along B1/SB, B3/SB, B4/NB and B6/NB – one at the mid - span and two at the quarter span (see Figure 19).

iii. 4 control prisms on some buildings/structures nearby.

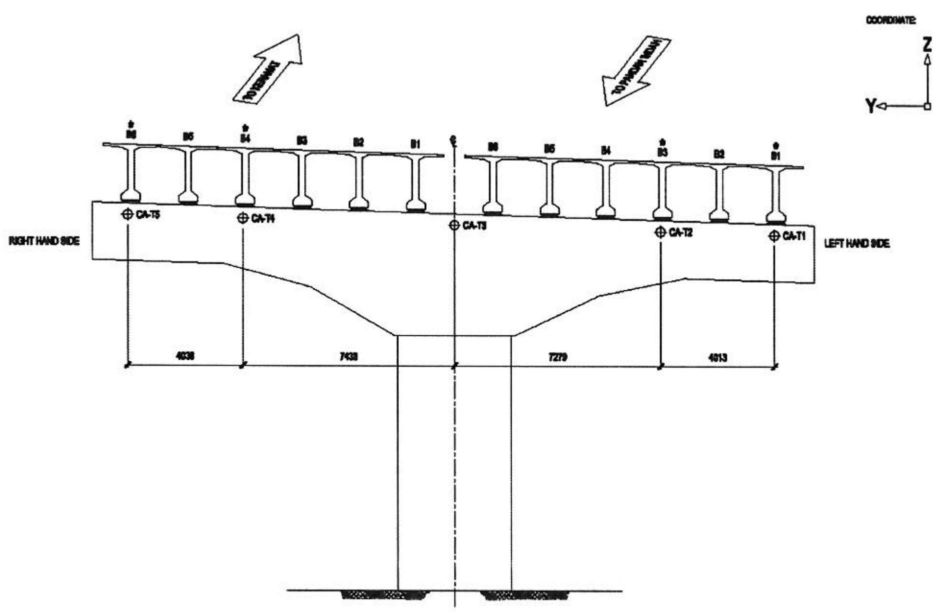


Figure 18: Location of prisms at Pier 15

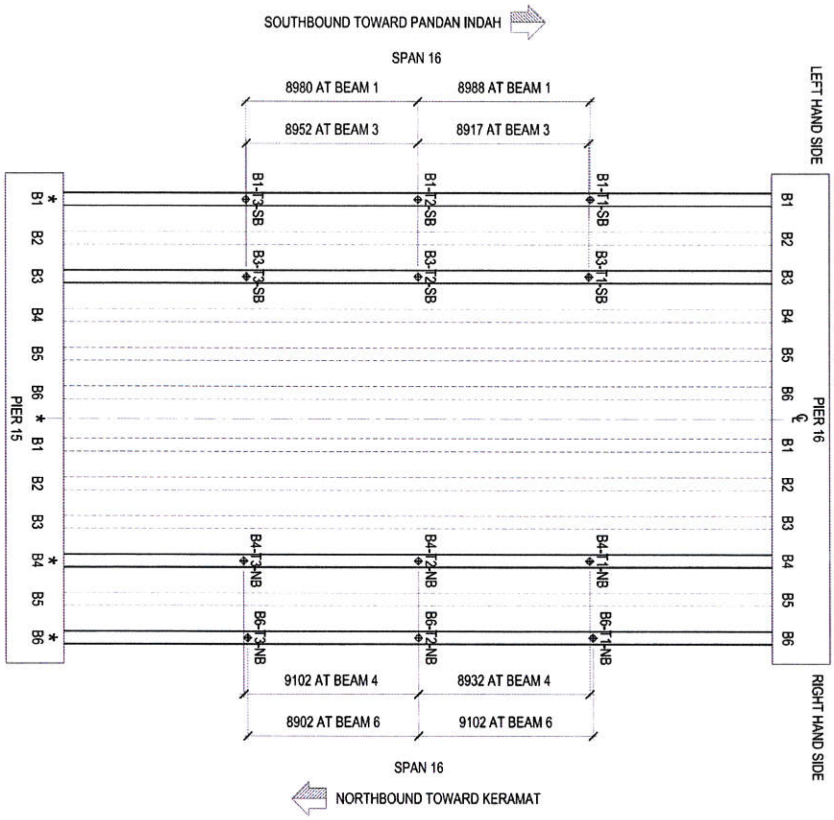


Figure 19: Location of prisms at the beams of Span 16

The dates of the deflection monitoring are given in Table 3 below. For the 12-hour continuous monitoring, the robotic total station was set to monitor continuously (about 15 minutes per round of data collection) from 6:00am till 6:00pm.

Table 3: Dates of taking deflection readings

Date	Monitoring period (hrs)	Components monitored	Remarks
2 Oct 2012	1	P11 & S12	Base Reading.
3 Oct 2012	12	P11 & S12	-
4 Oct 2012	12	P11 & S12	-

5 Oct 2012	12	P11 & S12	-
10 Oct 2012	12	P15 & S16	Base Reading. Abnormal displacement values at CA-T5 due to problem in the based reading.
11 Oct 2012	12	P15 & S16	New base reading was set for CA-T5.
12 Oct 2012	12	P15 & S16	-
18 Oct 2012	1	P15 & S16	-
25 Oct 2012	1	P15 & S16	-
1 Nov 2012	1	P15 & S16	-
8 Nov 2012	1	P15 & S16	-
9 Nov 2012	12	P15 & S16	Vibration measurement also in progress.
16 Nov 2012	1	P15 & S16	-
22 Nov 2012	1	P15 & S16	-
29 Nov 2012	1	P15 & S16	-
6 Dec 2012	1	P15 & S16	-
13 Dec 2012	1	P15 & S16	-
20 Dec 2012	1	P15 & S16	-

27 Dec 2012	1	P15 & S16	-
3 Jan 2013	1	P15 & S16	Rotated the base reading into relation of 90° to beam and the pier crossbeam.
10 Jan 2013	1	P15 & S16	-
17 Jan 2013	1	P15 & S16	-
25 Jan 2013	1	P15 & S16	-
31 Jan 2013	1	P15 & S16	-
7 Feb 2013	1	P15 & S16	-
14 Feb 2013	1	P15 & S16	-
21 Feb 2013	1	P15 & S16	-
1 Mar 2013	1	P15 & S16	Final reading.

3.3 Identification of User Sensitivity Criteria (Task iv)

A good deal of research has been done in studying the effect of exposure to vibration on man, especially in his working environment. There are research works on vibration study of bridges but none gives a specific definition of the reasonable dynamic criteria and the limiting values. Many international codes refer to criteria related to human sensitivity for people in buildings. Those on bridges were mainly directed to control of vibration in pedestrian bridges rather than road bridges.

3.4 Vibration Monitoring (Task ii)

3.4.1 Data collection

The methodology discussed herein is a part of the modal test which is used to monitor the vibration of the bridges through the installation of the SYSCOM equipment by SGS (Malaysia) Sdn. Bhd. in order to obtain detailed results of the bridge behaviour during traffic loading. Signal processing was later carried out on the results in order to obtain comprehensible results in the frequency domain which in turn would be utilised for model calibration described in Task iii).

The methodology addresses the vibration measurements, vibration data processing and analysis, interpretation of the results derived from the analyses. The vibration monitoring was based on actual traffic condition in which the bridge is subject to excitation from ambient loads. This methodology is utilised in view of the fact that:-

a) It is not practical and expensive to excite the bridge using mechanical shakers due to the size of the structure [5].

b) It is not practical to carry out a control test given the level of importance of the bridge access to the public on a daily basis.

All instrumented and monitored points were measured for responses in the lateral (y-axis), longitudinal (x-axis), and vertical (z-axis) directions using an accelerometer. These axis are local and to be made with respect to the directional motion of the particular structural component (i.e. transverse motion of pier, longitudinal motion of tee-girder). Measurement was carried out under both peak and non-peak hours for a period of 2-8 hours.

<u>The Equipment</u>

All on-site measurements were carried out using the following equipment:-

- Syscom instruments MR2002-SM24 – Seismic Recorder Digitizer
  Syscom MS2004 4 – Triaxial, high dynamic range accelerometers are as shown in the picture below (Figure 20).

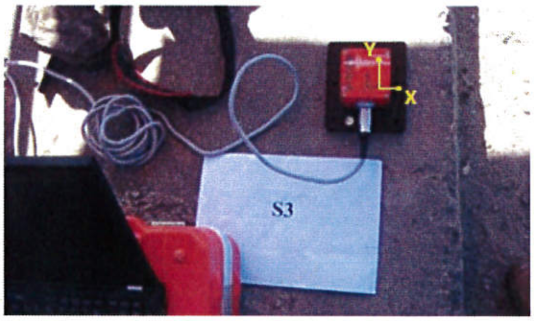


Figure 20: Typical Accelerometer

The main characteristic of the instrumentation system is the high accuracy needed to measure very low vibrations at very low frequencies. The responses or vibrations were quantified in the unit of acceleration. The accelerometers should be accurate enough to measure $10^{-8}g$, where g is the acceleration of gravity. Table 4 summarises the specifications of the instruments which was used in the study.

Sampling rate of the data was performed at 200Hz with a threshold of 0.5m*g*. The frequency pass-band of the digital signal processor was set to the sampling rate. The signal was cut off at 80 % of *Nyquist frequency*, e.g., 80 Hz for 200Hz. This filter is also called anti-aliasing filter or decimation filter. It was anticipated that this would be more than sufficient as bridge major modes tend to be in the region of less than 10Hz. 0.5m*g* was chosen as the threshold value based on human perception threshold normally recommended at 1.0m*g*.

Table 4: Characteristics of the instruments

Characteristics	Values
Accelerometer range	0 to 1 g
Sensitivity of the sensors and data acquisition system	Minimum of 10^{-8} g
Direction of vibrations	Vertical and horizontal
Data acquisition system analogue-digital conversion (A/D)	minimum 16 bits
Gain	Variable: 1, 10, 100 and 1 000 times
Low pass *anti-aliasing* filter	10 Hz
Frequency range	0 to 10 Hz
Frequency resolution	Better than 0.0001 Hz
Sampling rate	30 to 400 Hz
Digital filtering	54 dB by octave (*roll-off*)
Modal frequency separation	0.05 Hz

Measurement Points

Instrumentation of Frame #4 (Span 14 and 15)

There were 23 vibration sensors placed on the pier and beams of the structure. This process covered 3 piers: Pier 14, Pier 15 and Pier 16 corresponding to Span 15 and 16. Fifteen (15) vibration sensors were located on the piers whereas eight (8) vibration sensors were placed on the mid-span of the beams. The vibration sensor locations on Pier 14, Pier 15 and Pier 16 are shown in Figure 21, Figure 22 and Figure 23, respectively.

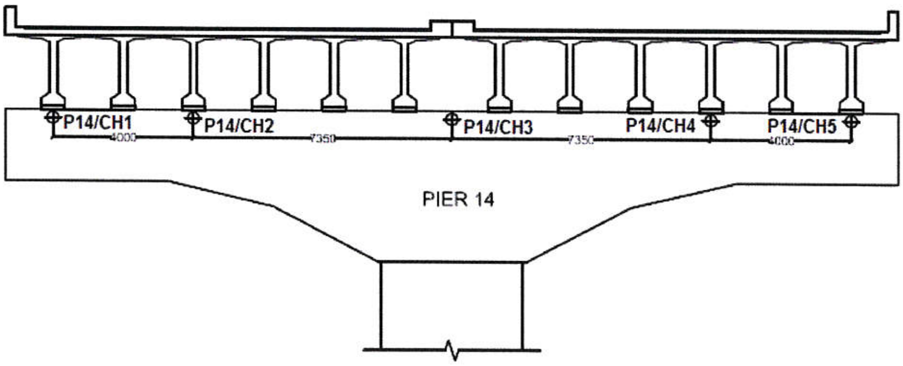


Figure 21: Pier 14 and the location of vibration sensors

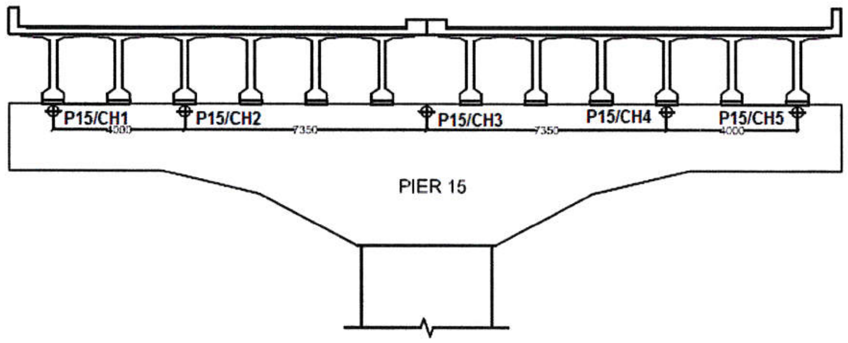


Figure 22: Pier 15 and the location of vibration sensors

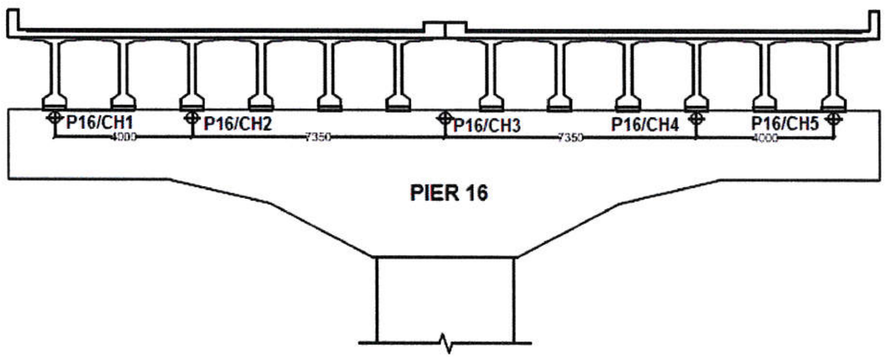


Figure 23: Pier 16 and the location of vibration sensors

In addition, there were 8 vibration sensor locations allocated at the mid spans of Span 15 and Span 16 on the left hand side and at the right hand side. The total number of sensors on the mid-spans was 8 as illustrated in Figure 24 and Figure 25.

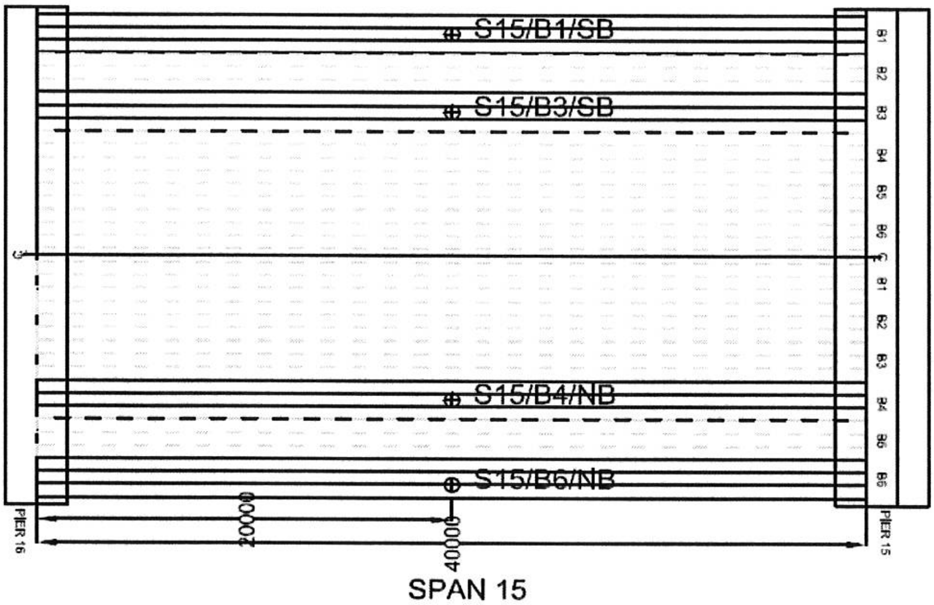


Figure 24: Span 15 and the location of vibration sensor at mid-span

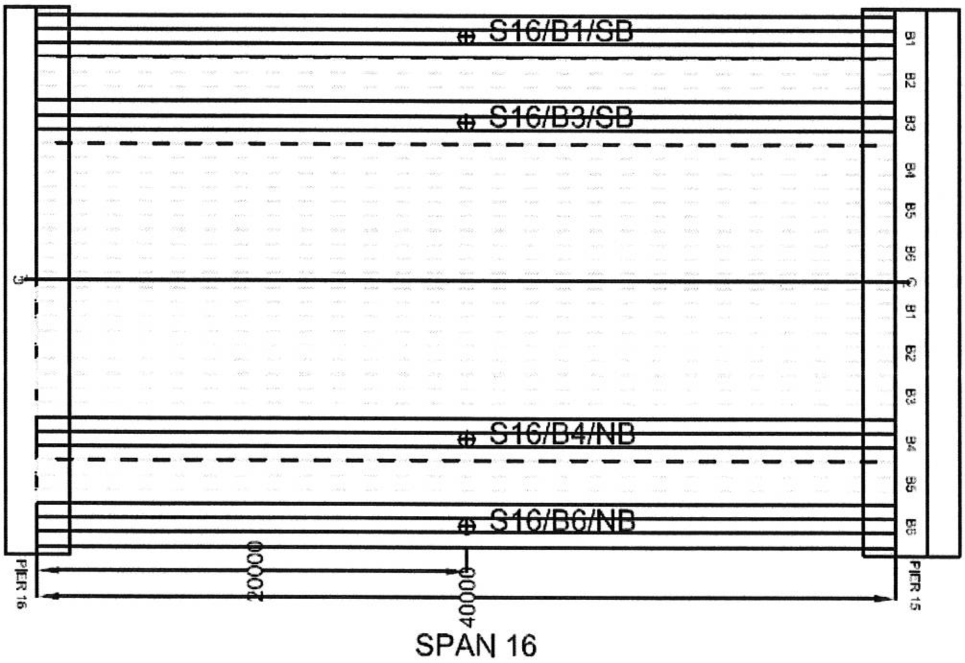


Figure 25: Span 16 and the location of vibration sensor at mid-span

The vibration sensors at Pier 14, 15 and 16 were monitored for a period of 2 hours whereas the sensors at mid-spans were monitored for 8 hours continuously.

Instrumentation of Frame #1 (Span 1 to 4)

The study was also extended to monitor performance of Frame #1 (corresponding to Ab A – Pier 4) based on the observation that some of the rubber bearings at Pier 2 were not fully seated on the bearing plinth. In fact, the pier was of the multi-column type which was different from the T-pier in Frame #4.

There were 5 measurement points for every pier and 4 measurement points for each span as shown in Figure 26 below. The monitoring at the pier was selected for the following purposes:-

- To verify that there is no significant modal parameter change from central pier to 1/3 distance to end of pier (even though the vibration values are largely varying). This applies to both sides of the carriageway.
- To verify that there is no significant modal parameter change from 1/3 distance to end span positions for both carriageways.

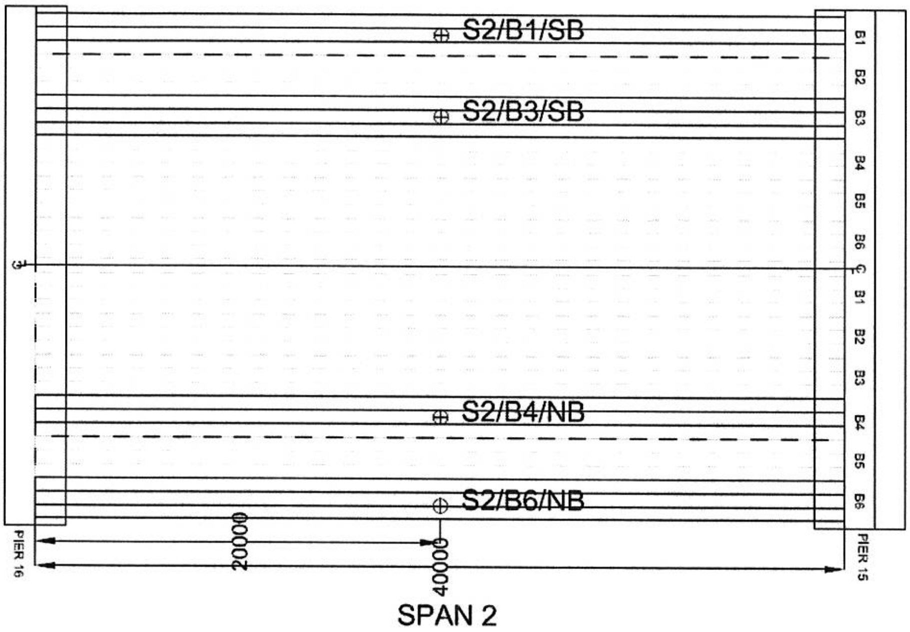


Figure 26: Partial plan view of the monitoring points for Frame #1

3.4.2 Data analysis (signal processing)

Data Integrity Check

Data integrity check was performed to assess the quality of the data procured from the SYSCOM vibration monitoring sensor. Zero tests were performed in a controlled environment where ambient vibration effects were at minimum levels and hence will be set as the benchmark against readings that are obtained from the Ampang Bridge. Tests were run over a span 15 minutes at a sample of 200Hz with a high-pass of 0.5m*g* to preclude all possible vibrations perceived by the human body.

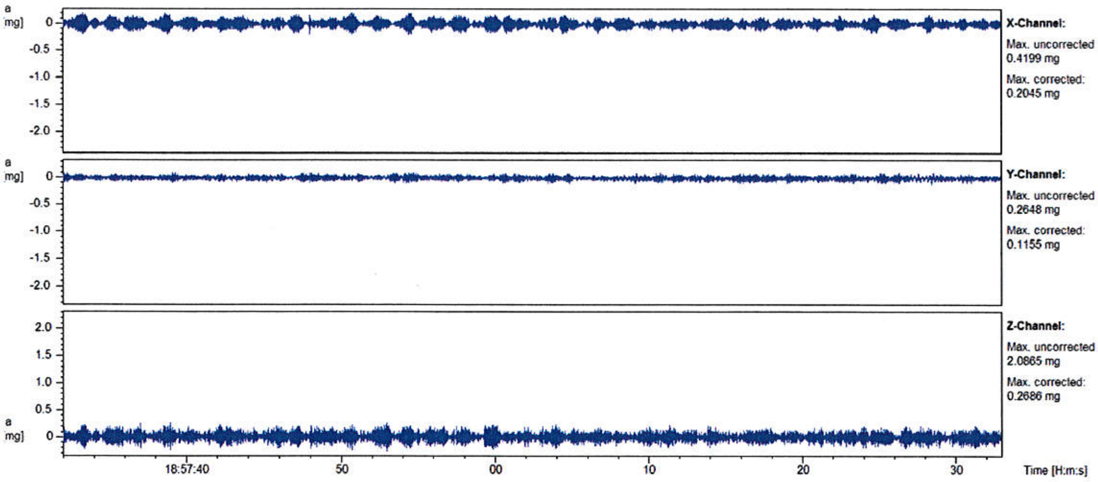


Figure 27: Time series from zero test for x,y and z directions

Figure 27 generally indicates that the instrument picks up some ambient vibration from the environment at levels far below human perception of 10m*g*. The results of the zero tests revolve below 0.5m*g* levels with a mean value of near zero. This is generally assumed to originate from ambient vibration or electrical hum in the readings.

Statistical Analysis (Time Domain)

Time domain is a common representation of the amplitude of the signal in milli-gravity acceleration (m*g*) as a function of time (time history). The following shows the typical time histories over short periods at position of P15/CH5 as a sample. Figure 28 shows a sample of vibration measurements in the time domain (signal) for x (longitudinal), y (transverse) and z (vertical) directions.

One time history represents a *sample function* of the *random process* and in order that the statistical properties derived from this single sample function could represent the random process, (which, in this case is the measured signal), we have to make the assumption that the vibration signal is produced by an *ergodic* random process.

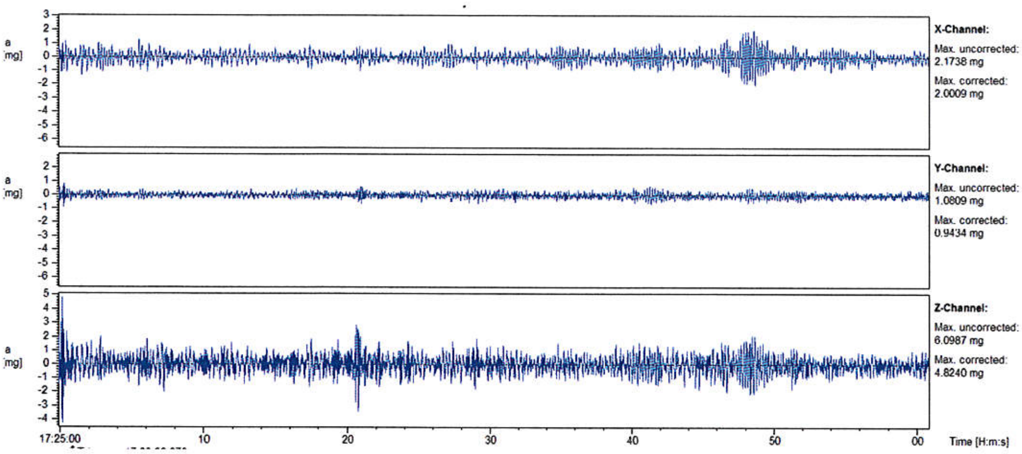


Figure 28: Time domain vibration measurements of P15/CH5 in the x, y and z directions

As a routine signal processing procedure, various temporal averages were computed to validate this assumption. This, of course, was preceded by other statistical techniques to filter and clean up the raw data. The reader should not bother himself too much in this explanation (as those who are interested may refer to Volume IV of the Final Report for greater details) but merely to note that the measured vibration is not a single value that could be compared with the critical value discussed in Section 3.3 above. For this purpose, a single characteristic value based on the 95th percentile, rather than the peak value would be used. The 95th percentile represents that 95% of the records would fall below the value. This requires that a cumulative distribution function (cdf) be derived from the time history.

<u>Modal Analysis</u>

Besides providing a characteristic value representing the level of measured vibration, the data was further analysed to determine the inherent dynamic characteristics of the bridge (as a vibratory system) in the forms of natural frequencies, mode shapes and damping ratios. This *modal data* would be used to tune the analytical model. The modal analysis is mathematically involved but standard algorithms such as Fast Fourier Transformation are available and were used by the vibration experts from UTP.

3.5 Analytical Modelling (Task iii)

FE Model

Three-dimensional models were established for Frames #1 and #4 in accordance with the information provided in the design drawings which were made available for this study. In the 3D FE models, all bridge components such as columns, pier crossheads, end diaphragms, girders and slabs are simulated by uniaxial 3-D elastic beam elements with tension, compression, torsion and bending capabilities. The element has six degrees of freedom at each node, i.e., three translations in x, y and z directions and three rotations about x, y and z axes of the nodal. Special elastic link elements which are capable to simulate the compression and shear stiffness of the elastomeric bearing are adopted (see Figure 29 and Figure 30).

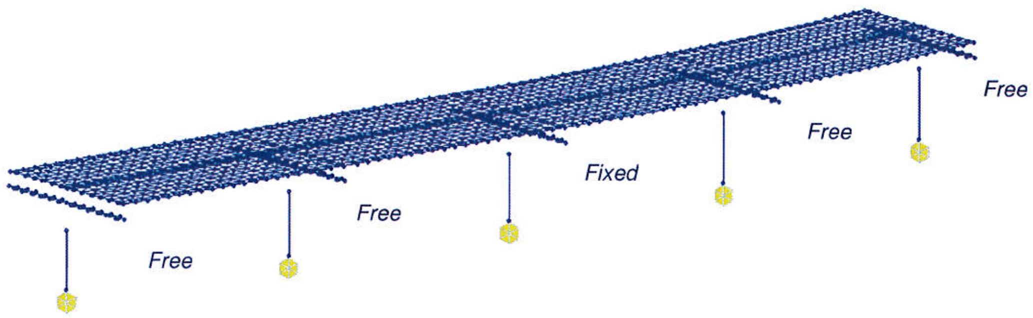


Figure 29: 3D grillage model for Frame #4

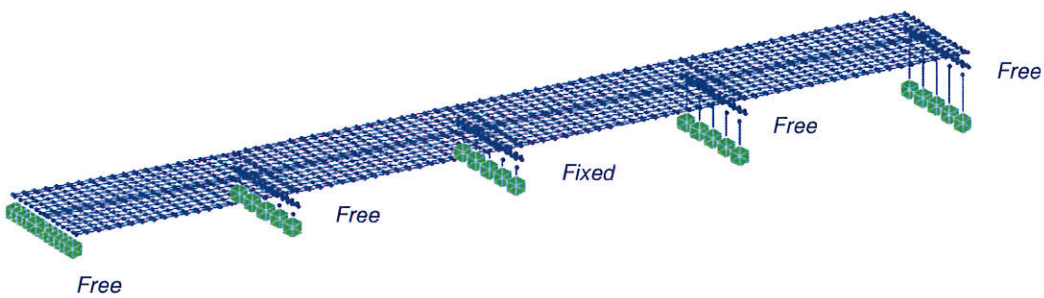


Figure 30: 3D grillage model for Frame #1

Modelling of the deck structure as grillage generally follows the recommendations of Hambly [6] and West [7]. Equivalent point springs to simulate pile-soil interactions of pile group are adopted as support boundary conditions for the piled foundation in the global analysis. All member sections, material properties, real constants such as E_c, and *poisson ratios* that reflect effectively the properties of individual structure members are listed in the following sections respectively.

Linear elastic analysis with small deflection was performed in *Midas Civil 2013*. The geometric change of the structure is always assumed to be small and can be neglected so that all forces and deformations are determined by the original configuration of the structure, i.e., the overall stiffness of the structure before and after deformed configuration are assumed to be identical.

Parametric Studies in Preparation for model calibration

Due to inevitable deviation in the structure's construction details from the design, and uncertainties associated with time-dependent material properties, support conditions etc., it is difficult to establish an FE model to represent the actual structure. Besides, it is known that computer modelling alone could not determine completely the dynamic behaviour of the structures because certain properties such as damping and nonlinearity do not conform to traditional modelling treatment [2]. Therefore, the FE model which was built based on design information has to be updated or calibrated using field testing results in order to approximate the current actual conditions of the structure.

In order to calibrate the FE model with field vibration results in terms of modal parameters i.e., natural frequencies and mode shapes, those parameters such as stiffness of supports, structural and material properties that may affect the modal properties of the bridge can be identified by the parametric studies. Hence, the model calibration can be conducted by adjusting these parameters to match the frequencies and mode shapes between testing and modelling.

Several parameters that would affect the modal behaviours of the bridge are identified for the parametric studies:-

(i) Foundation stiffness K_f

(ii) Structural Stiffness - Material elastic modulus of E_c

(iii) Deck mass density ρ_c

(iv) Elastomeric bearing stiffness K_b

(v) Damping Ratio ζ

Free Vibration Analysis & Calibration of the Analytical Model

Since the established model for Frame #4 is a 3D finite element model, a free vibration analysis is capable to extract all possible vibration modes of the structure. Vibration modes are inherent properties of a structure. Each mode is defined by a natural frequency, modal damping and mode shape. Any change of material properties or the boundary conditions of the structure, its modes will change accordingly. The fundamental modes with low frequencies are usually in vertical, longitudinal, transverse and torsion shapes. However, higher frequency modes are usually more complex in appearance and coupled with the fundamental modes.

All modes of a structural vibration can be extracted using the analytical FE model. Unlike the theoretical modelling, in the case of field measurements, modes which are not excited by random traffic vibrations will not be identified in the frequency domain. Generally, the overall response of a structure is a summation of responses due to each of its modes. The modal participation mass (MPM) distribution percentage demonstrates the importance of that mode. Modes which have higher MPM will dominate the structure responses. Higher modes always come with low MPM. When the excitation frequency is close to one of the natural frequencies, the response of that mode will dominate the response of the structure, i.e., resonance occurs – a small amount of input force causes a huge response. Eigen value and extraction technique used by *Midas Civil 2013* is Lanczos method.

Bridge Assessment using the Analytical Model

Once calibrated the Analytical Model could be used for different types of analysis to assess the performance of the Ampang Bridge:-

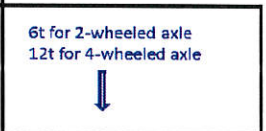
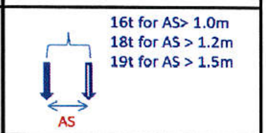
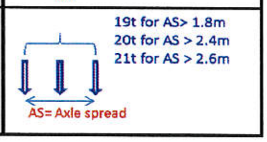
i. Static analysis of the bridge under various live loads.

ii. Dynamic analysis of the bridge under the WRO vehicles.

WRO vehicles used in this study represents vehicles that comply with the current Transport Act implemented vide Weight Restriction Order (WRO 2003) [4]. Since the WRO 2003 had been based on implementation of the Medium-term Axle Load (MTAL) Policy, it is deemed that a WRO vehicle with axle load limited by the First Schedule of WRO 2003 (see Table 5) would give a loading effect quite equivalent to that from the MTAL.

A few different types of WRO vehicles were identified with configurations and dimensions based on data collected during the Axle Load Study (1988). They can be divided into two categories: The rigid vehicle (RV) (Figure 31) and the articulated vehicle (AV) (see Figure 32 and Table 6).

Table 5: WRO-2003 permissible axle loads

Axle Configuration	Axle Spread AS (m)	Maxium Axle Load (t)	
(i) Wheel Load Single wheel	-	3	
(ii) Single Axle 2-Wheeled Axle 4-Wheeled Axle	 - -	 6 12	
(iii) 2-Axle (Tandem axle)	*AS* > 1.0 *AS* > 1.2 *AS* > 1.5	16 18 19	
(iii) 3-Axle (Tri-axle)	*AS* > 1.8 *AS* > 2.4 *AS* > 2.6	19 20 21	

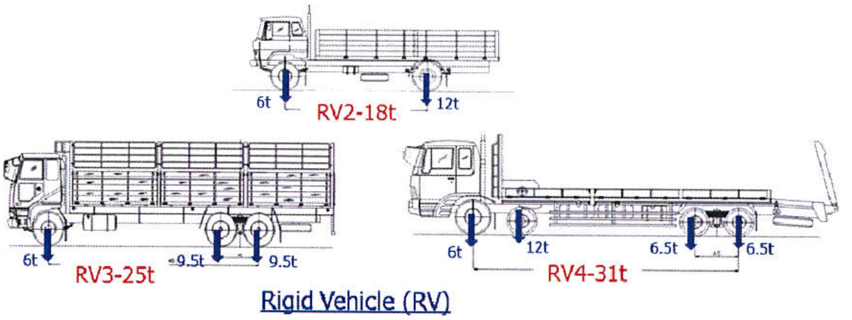


Figure 31: WRO-2003 rigid vehicles (RV)

Table 6: WRO-2003 truck configurations

Truck TYPE	Nos. Axle	WB	GVW
RV2-18t	2	2.60	18
RV3-25t	3	6.60	25
RV4-31t	4	8.00	31
AV3-30t	3	8.50	30
AV4-37t	4	10.70	37
AV5-39t	5	11.10	39
AV5-40t	5	11.10	40
AV6-44t	6	13.10	44
AV7-53t	7	13.10	53

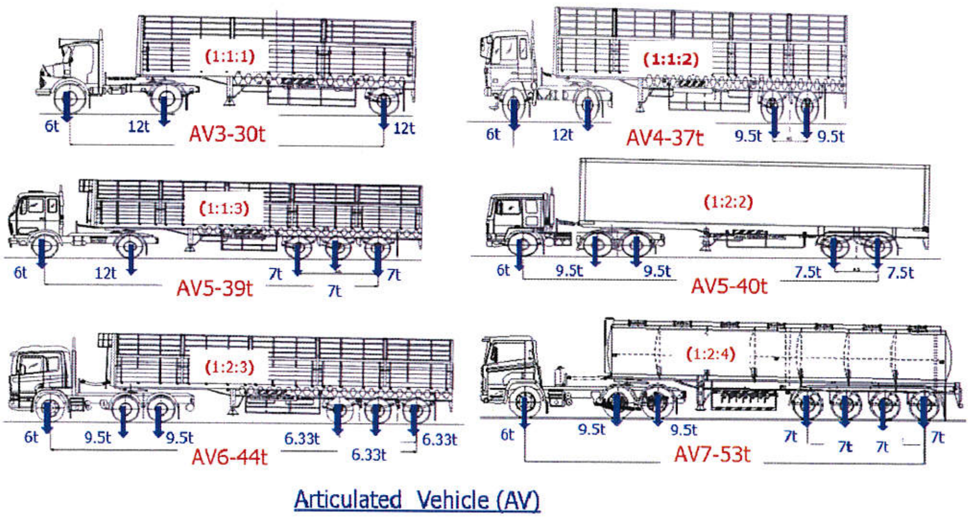


Figure 32: WRO-2003 articulated vehicles (RV)

Four (4) types of WRO truck; 2 for each RV and AV vehicles, were selected for the dynamic analysis:-

- *Types of WRO truck : RV2-18t, RV3-25t, AV3-30t and AV7-53t*
- *Travelling velocity (Vt) : 60, 90 and 120 KPH*
- *Number of trucks per traffic lane (N) : 1, 2, 3*

Truck spacing for vehicles in tandem is varied with travelling velocity and derived based on driver response time of 1s. To simulate the actual traffic patterns on the Ampang Bridge, the following four (4) cases were considered (see Figure 33, Figure 34, Figure 35 and Figure 36) :-

Case (i): WRO trucks traversing on outer traffic lane of south-bound deck.

Case (ii): WRO trucks traversing on outer traffic lane of south-bound deck whereas north-bound deck is loaded with MTAL to simulate traffic jammed condition.

Case (iii): WRO trucks traversing on outer two traffic lanes of south-bound deck, i.e. two trucks traversing side by side.

Case (iv): WRO trucks traversing on outer two traffic lanes of south-bound deck whereas north-bound deck is loaded with MTAL to simulate traffic jammed condition.

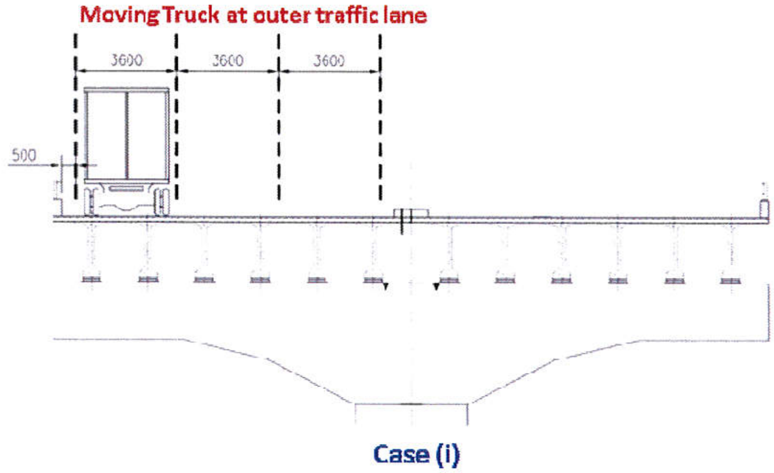


Figure 33: Frame #4- Case (i)

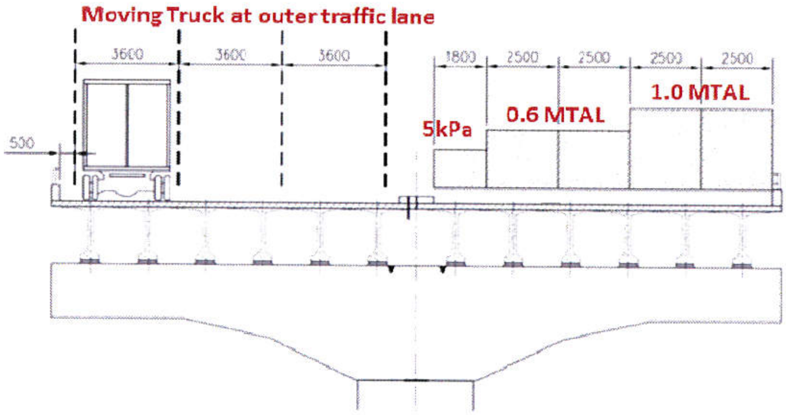


Figure 34: Frame #4- Case (ii)

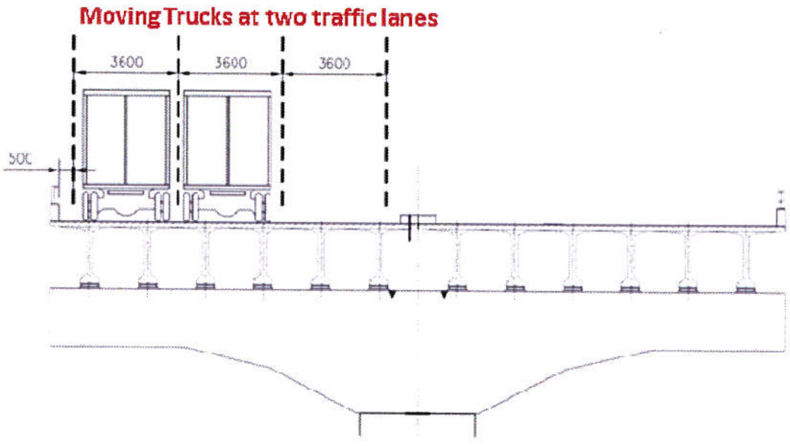


Figure 35: Frame #4- Case (iii)

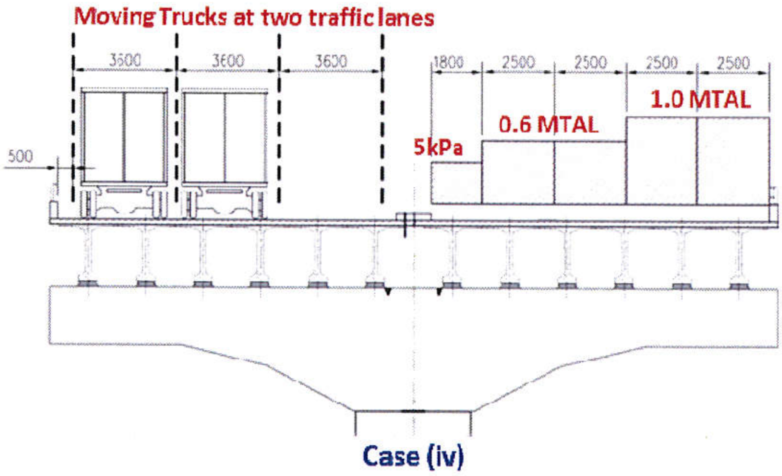


Figure 36: Frame #4- Case (iv)

Axle loads were simulated as triangular impulse loads and apply at girder nodes at pre-defined arrival times which were computed based on truck's axle spacing and travelling velocity. Various travelling velocities and different units of truck on each traffic lane (trucks in convoy) simulating the actual conditions were used to predict responses of girder under various conditions. The travelling velocities considered were 60, 90 and 120 KPH.

3.6 Vibration Control and Mitigation Measure (Task v)

The JKR TOR requires the assessment of dynamic performance of the bridge under the actual traffic condition. Measured response based on data collected in the study represents the lower bound condition of the traffic. Response from the WRO vehicles would be the upper bound condition. Both responses would be checked against the limiting value, as follows:-

a_{mes} Vs a_{lim}

a_{WRO} Vs a_{lim}

L'_{WRO} Vs L_{LTAL}

L_{WRO} Vs L_{LTAL}

where a_{mes} = 95th percentile of measured acceleration,

a_{WRO} = Computed acceleration due to WRO vehicles,

a_{lim} = Limiting value for acceptable level of acceleration,

L'_{WRO} = Dynamic load effects due to WRO vehicles,

L_{LTAL} = Static load effects due to LTAL representing the resistance.

Should any of the load effects exceed the threshold value or resistance a mitigation measure or vibration control would be required.

4 VISUAL INSPECTION OF THE AMPANG BRIDGE

4.1 Introduction

The primary concern for a bridge with "excessive" vibration is its structural safety. This concern could be alleviated by physically checking the condition of the bridge components for any sign of defects or deterioration that may reduce the capacity of the bridge. This is achieved through a comprehensive inspection of the bridge.

4.2 Bridge Condition from Bridge Inspection

The Consultant had completed inspection of all 24 spans of the 960m-long bridge in February 2013. The conditions of the bridge are reported in the forms of standard inspection checklists and pictorial reports and are presented in Volume II of the Final Report.

4.2.1 Discussions on common damage/defects

Inspection of the Ampang Bridge revealed a few common damage/defects associated with concrete bridges and a few more serious damage/defects that require attention from JKR. The damage/defects that were observed are:-

- i. Failed sheet piles bank protection at downstream of Abutment A.
- ii. Minor localised spalling of the beams.
- iii. Poor concrete at top flange of beam ends.
- iv. Honeycomb at the end diaphragms.
- v. Various types of cracks at the abutment and piers:-
  - a. Fine vertical cracks at pier columns.
  - b. Fine vertical cracks at base of crossbeams.
  - c. Fine vertical and horizontal cracks at pier columns.
  - d. Multiple cracks resembling DEF or AAR.
- vi. Improper contact of a few bearings.
- vii. Damaged expansion joints.
- viii. Cracks at central parapet directly above Pier 12.

These types of damage/defects are commonly observed on bridges and they are discussed in the following sub-sections.

1. Damaged sheet piles bank protection at downstream of Abutment A

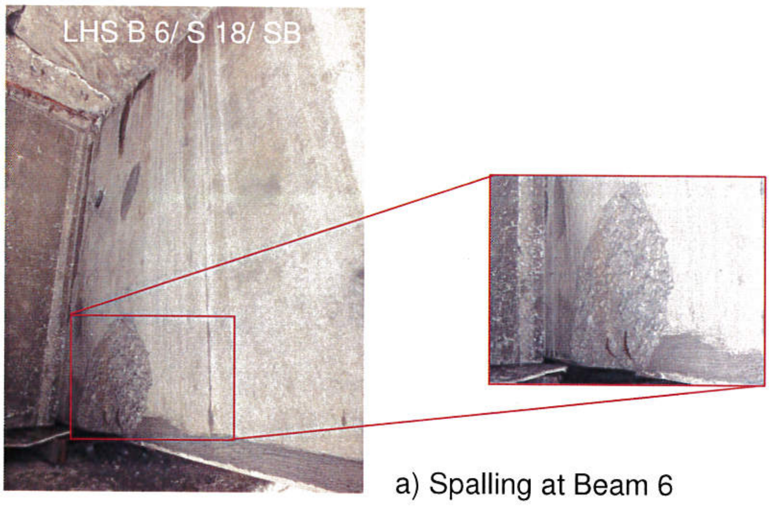
The sheet piles bank protection at downstream of the Abutment A of the slip road bridge had tilted and deformed resulting in exposure of the soil behind them and below the RC slab. See Figure 37. The failures could have been caused by the water from the roadside drain being channelled directly at the sheet piles (bottom picture). During the time of inspection, some of the exposed soil had been washed away and some of the sheet piles in front of Abutment A had tilted slightly. It is recommended that immediate remedial action be taken to rectify the damage. Even though the damage is slightly downstream from Abutment A, early intervention is important as the exposed soil is susceptible to further erosion/scouring and failure may propagate further and affect the integrity of Abutment A.

Figure 37: Damaged sheet piles

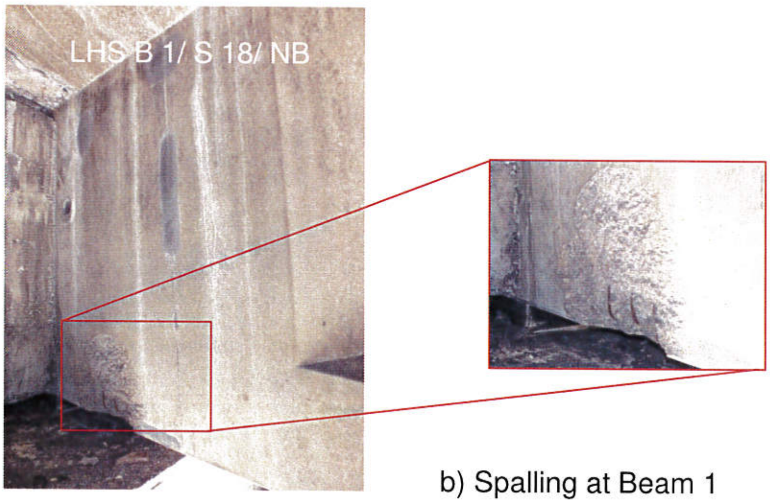
2. Minor localised spalling of the beams

A few of the beams were observed to have suffered minor localised spalling at the sides of the bottom flanges near the beam ends. See Figure 38. Some of the spalled areas exposed lightly rusted reinforcement. It appeared that the damage could have occurred during construction likely due to the impact during launching of the beams. This is minor damage that does not affect the structural integrity of the beams.

Nonetheless, they should be repaired whenever possible in order to protect the reinforcement from further corrosion.



a) Spalling at Beam 6



b) Spalling at Beam 1

Figure 38: Localised spalling at a few beams

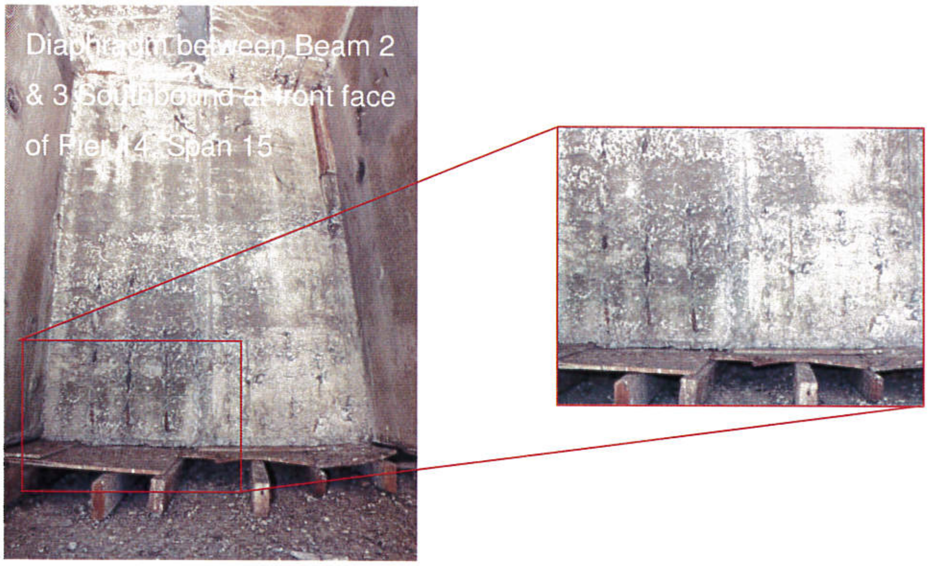
3. Poor Concrete at Top Flange of Beam Ends

Few of the top flanges at the beam ends suffered from poor concrete. See Figure 39. During the bridge construction, the precast beams were cast without casting the top flanges at the ends of the beams ('blockout' sections) to facilitate the lifting and launching of the beams. These 'blockout' sections were then cast during casting of the deck slabs. Quite a number of these locations were not concreted properly resulting in poor concrete finish with some having exposed reinforcement. Even with these defects, no structural damage was detected. Notwithstanding, it is recommended that the poor concrete be repaired to ensure that there is no future durability problem such as corrosion and spalling of reinforced concrete members.

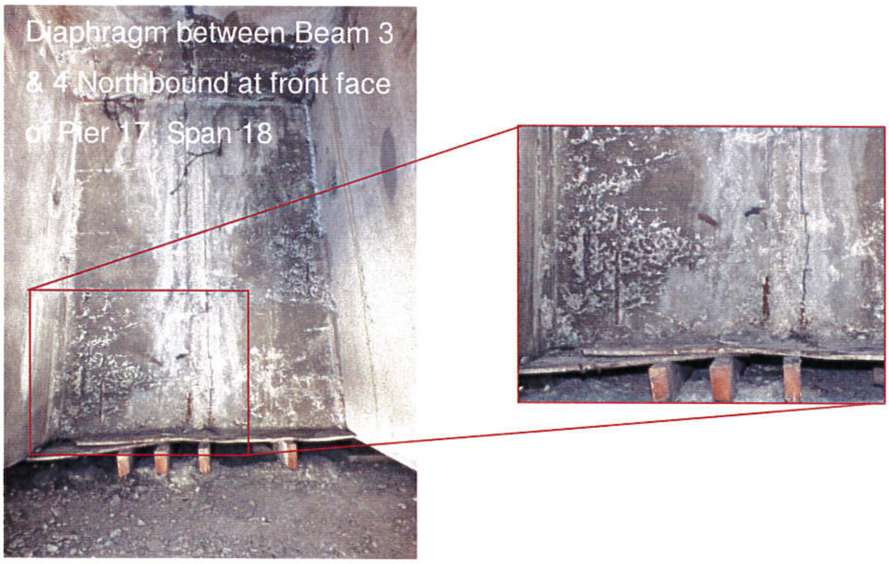
Figure 39: Some top flanges at beam ends that suffered from poor concreting

4. Honeycomb at the end diaphragms

Honeycomb in the diaphragms is the most common defect observed at the Ampang Bridge. See Figure 40. The defect ranges from minor to slightly severe in which the reinforcement was also exposed. Some of the exposed reinforcement was also due to insufficient cover. In this situation, there is a cause for concern as the exposed reinforcement will eventually lead to long term durability problem such as corrosion of the reinforcement and spalling of concrete. Therefore, it is recommended that this defect be repaired either by patching or formwork repair technique depending on the extent of the defects.



a) *Honeycomb at end diaphragm between Beams 2 & 3 Southbound at front face of Pier 14 facing Span 15*



b) *Honeycomb at end diaphragm between Beams 3 & 4 Northbound at front face of Pier 17 facing Span 18*

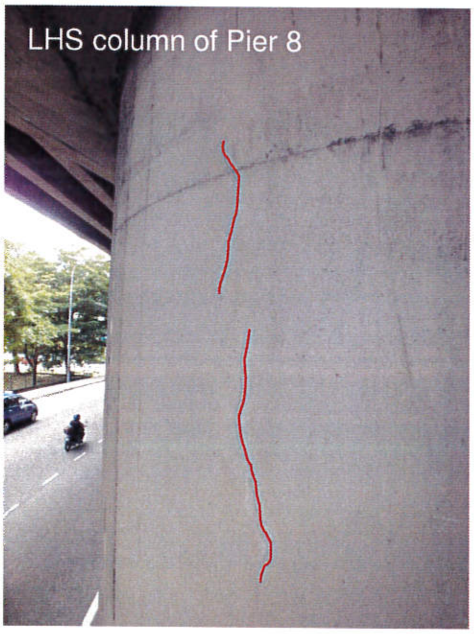
Figure 40: Honeycomb with exposed reinforcement was observed at many of the end diaphragms

5. Various Types of Cracks at the Abutment and Piers

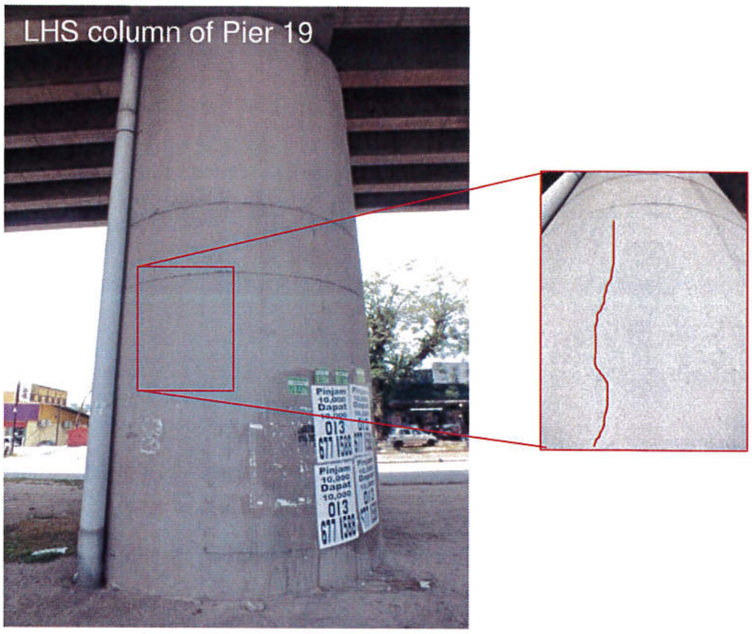
A few of the piers and abutments were observed to have suffered from various types of cracks. These cracks were mostly fine, i.e., 0.2mm or less and most can only be seen when viewed closely. The types of cracks observed are described here below.

a) Fine vertical cracks at pier columns

Fine vertical cracks were observed at the columns of Pier P2, P8, P9, P12, P13, P16 and P 19. See Figure 41. These cracks were fine with width of not more than 0.2mm and were located randomly on the surfaces of the pier columns. These cracks were likely to be caused by shrinkage based on their width, location and pattern of the cracks, i.e., triggered by tensile stresses induced in the surfaces of the columns due to volumetric reduction of concrete during the hydration process. Nonetheless, they should be monitored to check whether they are dormant or active.



a) Fine vertical cracks at left hand side of Pier 8 column with maximum width of 0.2mm (red line drawn adjacent to crack)

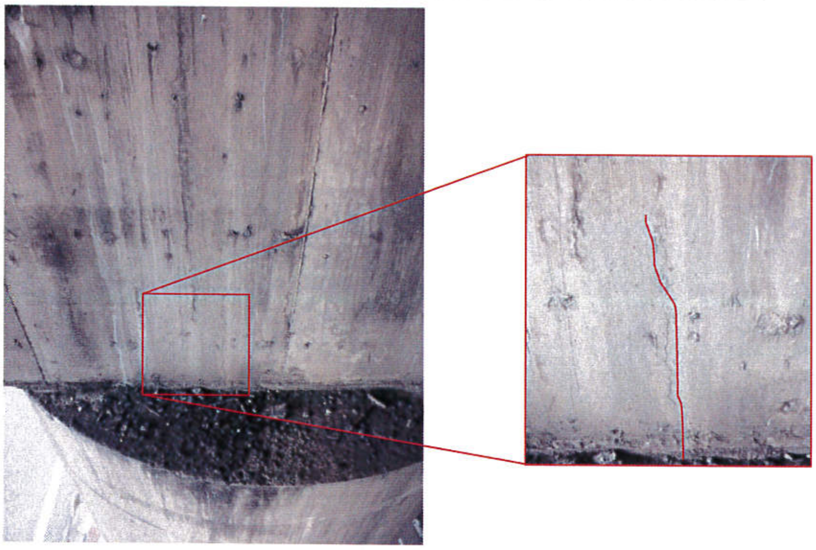


b) Vertical cracks at left hand side of Pier 19 column with maximum width of 0.2mm (red line drawn adjacent to crack).

Figure 41: Some of the vertical cracks observed at the pier columns

b) <u>*Fine vertical cracks at base of pier crossbeam*</u>

Fine vertical cracks were observed at the base of pier crossbeam of Pier P7 and P14. See Figure 42. These cracks with surface width lesser than 0.1mm occurred at the base of the crossbeams directly above the pier column and propagated vertically up. The crack at Pier 7 occurred only on the back face. The crack at Pier 14 occurred at both the front face and back face of the pier crossbeam but it is unlikely that they are through cracks as their locations and patterns did not match one another. These cracks did not appear to be structural and could have occurred during construction when the crossbeams were rotated into positions or when the crossbeams were stressed into place. Nonetheless, JKR should monitor them periodically to verify that they are dormant.



a) *Fine vertical crack with width smaller than 0.1mm at the back face of the Pier 7 crossbeam (red line drawn adjacent to crack)*

b) *Fine vertical crack with width smaller than 0.1mm at the front face of the Pier 14 crossbeam*

Figure 42: Fine vertical cracks at the base of pier crossbeam

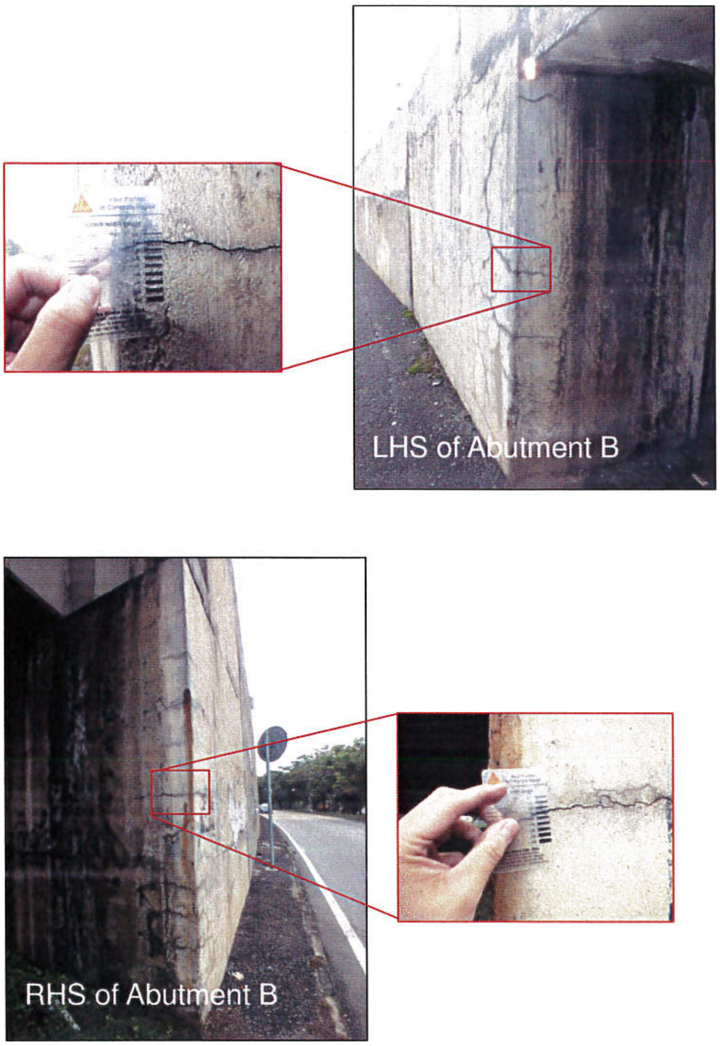
c) <u>*Fine vertical and transverse cracks in the column*</u>

Fine vertical and transverse cracks were observed in the columns of Pier P16. See Figure 43. These cracks especially on the right hand side of the column were quite apparent even though the measured maximum width was only 0.2mm. From the observation of the crack it is difficult to determine the cause of the cracks but based on the crack widths, their locations and patterns they were unlikely to be caused by structural deficiency. JKR could confirm this by monitoring them periodically to check whether they are active or dormant.

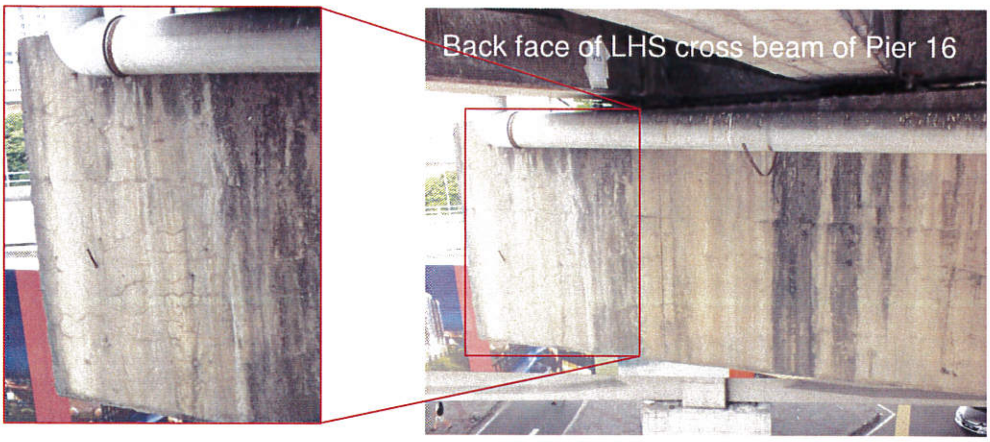
Figure 43: Fine vertical and transverse cracks with maximum width of 0.2mm at right hand side of column Pier 16

d) <u>*Multiple cracks at Abutment B and P16*</u>

Multiple cracks were observed at left and right hand side of Abutment B and right hand side back face of Pier 16. See Figure 44. The maximum crack width at Abutment B was 1.40mm while the maximum crack width at Pier 16 was 0.2mm. Besides exhibiting similar multiple crack pattern, they also occur at the sides only, i.e., at locations where they are exposed to rain. Therefore, based on these observations, it is suspected that the cracks might be caused by either delayed ettringite formation (DEF) or alkali aggregate reaction (AAR) as DEF and AAR will only occur when concrete is exposed to water. One way to confirm this diagnosis is to conduct petrographic examination of the affected concrete via thin slices of the cored samples. In the meantime, it is recommended these cracks should be monitored periodically as the cracks may widen with time.



a) Multiple cracks at Abutment B



b) Multiple cracks at left hand side of cross beam back face of Pier 16

Figure 44: Multiple cracks resembling DEF or AAR crack pattern

6. Improper Contact of a Few Bearings

Few of the bearings were observed that the plan area of the pads were not in full contact with the beams and bottom plinths. See Figure 45. This condition was detected on most bearings at Pier 2 facing Span 3; BG 4/P 6/S 6/SB; BG 1/P 23/S 23/NB; and BG 3/P 23/S 23/NB. The lack of contact caused additional stresses to be transferred to the bearings from the super structure. Despite this defect, the bearings appeared to be in good condition with no visible sign of deformation or displacement. However, they should be monitored periodically for any signs of deformation or displacement as they are experiencing uneven forces from the beams.

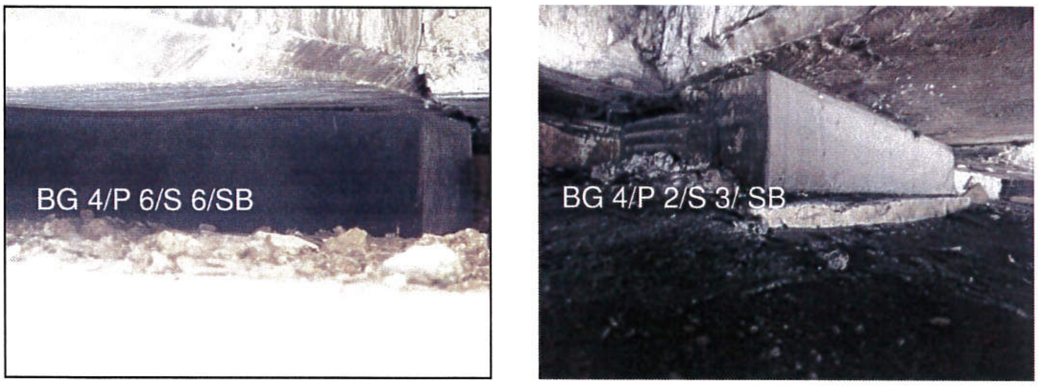


Figure 45: Some bearings that were not in proper contact with the beams

7. Damaged Expansion Joints

Two types of joints were observed at the Ampang Bridge, namely asphaltic plug type and elastomeric type. The asphaltic plug joints at P16 SB and P20NB were observed to have longitudinal cracks in the rubberised bituminous materials. See Figure 46. The occurrence of these cracks gave indication that the bituminous material was not flexible enough to accommodate the movements at the joints. It should be noted that this type of joint is not suitable for the Ampang Bridge as it has an arch vertical profile and large horizontal movement which is outside the capacity of the joint. The cracks do not cause any detrimental effect to the structural integrity of the bridge, but they would allow water and debris from the deck to leak to the underside of the bridge and disrupt smooth flow of traffic.

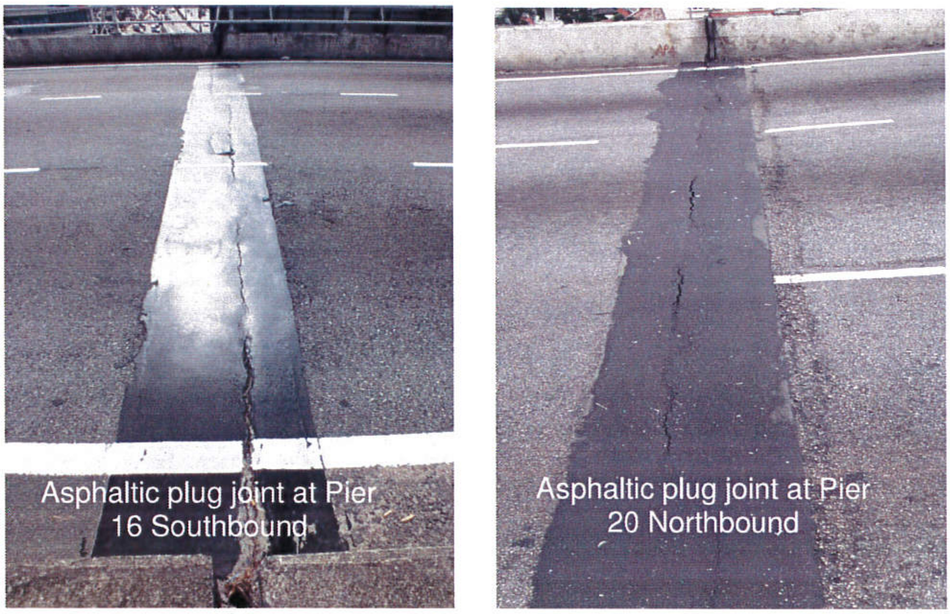
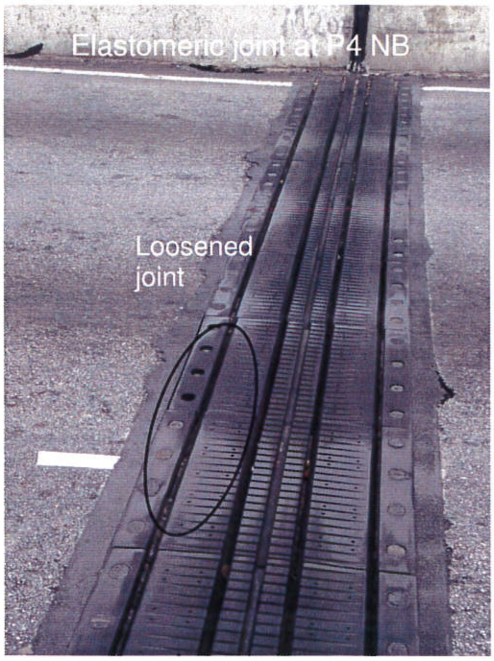


Figure 46: Cracks in asphaltic plug expansion joints

The elastomeric expansion joint in the northbound deck was generally in fairly good condition with minor wearing out of the elastomeric and loss of some anchor bolt covers. However, a few panels at centre lane of Pier 4 Southbound and at the slow and fast lane of Abutment B Northbound had loosened and were uplifted from their positions whenever vehicles travelled over them. See Figure 47. It was noted that the nuts for the anchor bolts at these locations were either loose or missing. The constant uplifting of these panels may worsen and endanger passing vehicles. Therefore it is recommended that the loosened panels be secured back to the deck as the joints were still in good condition.



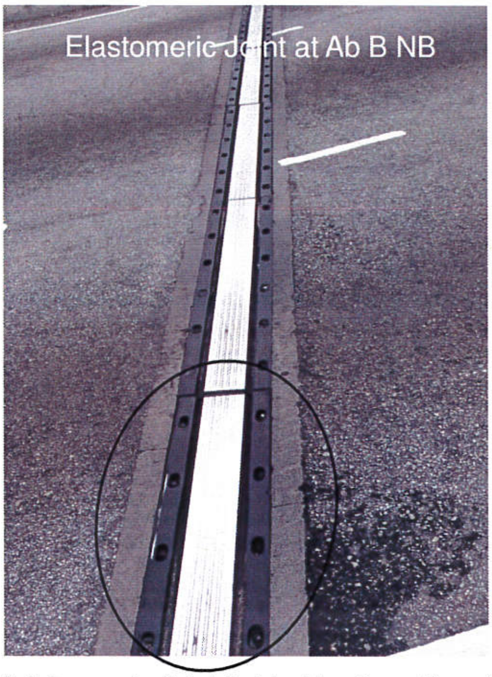


Figure 47: Loosened elastomeric joint panels (highlighted by the ellipse)

8. Cracks at central parapet directly above Pier 12

Cracks were observed at the central parapet directly above Pier 12. See Figure 48. The cracks were wide and went through the entire width of the parapet. The cracks occurred because the parapet was constructed continuous of the expansion gap between Span 12 and 13 and as the spans expand and contract (daily cyclic movement) high stresses were induced to the parapet and the cracks occurred when the stresses exceeded the concrete tensile stress capacity. As a good maintenance practice, it is recommended to hack the concrete parapet directly above the joints and introduce an expansion joint by providing proper gap and sealant at the parapet.

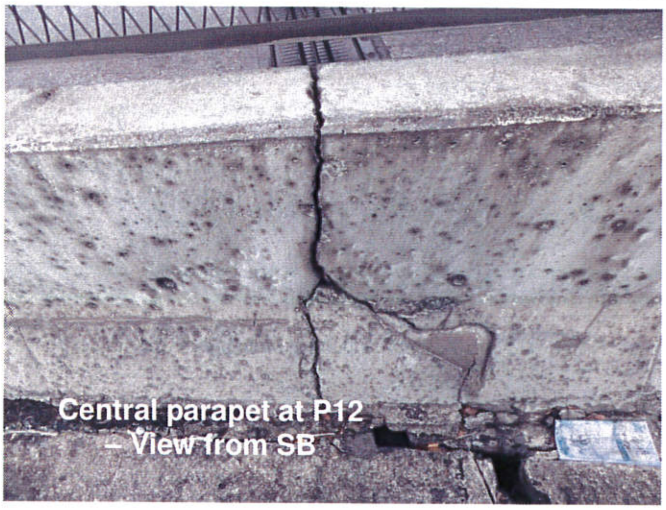


Figure 48: Cracks in central parapet at Pier 12

4.2.2 Findings in bridge inspection

The inspection has shown that the bridge was generally in fairly good condition with only some minor damage/defects. A complete discussion of these damages is presented in Volume II of the report. These damage/defects were mainly caused by poor workmanship during construction and are not uncommon in this type of construction. Their presence would not have impaired the structural integrity of the bridge and thus are not likely the source of vibration that had instigated this study. Neither are these damages likely to be caused by the vibration.

It is a good maintenance practice to have these damages repaired. The actions proposed to be taken on the observed damage/defects are as given in Table 7 below.

Table 7: Proposed actions on damage/defects

No.	Type of Damage/Defect	Proposed Actions	Remark
1.	Failed sheet piles bank protection downstream of Abutment A.	Further Investigation.	-
2.	Minor localised spalling of the beams.	Patched repair.	Specification & Drawing given.
3.	Poor concrete at top flange of beam ends.	Patch repair or formwork repair.	Specification & Drawing given.
4.	Honeycomb at the end diaphragms.	Patch repair or formwork repair.	Specification & Drawing given.
5.	Fine vertical cracks at pier columns.	Crack sealing or epoxy injection.	Specification & Drawing given.
6.	Fine vertical cracks at base of crossbeams.	Crack sealing or epoxy injection.	Specification & Drawing given.
7.	Fine vertical and horizontal cracks at pier columns.	Crack sealing or epoxy injection.	Specification & Drawing given.
8.	Multiple cracks resembling DEF or AAR.	Further Investigation/ formwork repair.	Specification & Drawing given.
9.	Improper contact of a few bearings.	Regular monitoring/Jack beam slightly up and fill gap with epoxy grout.	-
10.	Damaged expansion joints.	Replace damaged asphaltic plug joint.	Replace by same expansion joint as per original design.
11.	Cracks at central parapet directly above Pier 12.	Provide gap by formwork repair technique.	Specification & Drawing given.

The specifications and drawings for the proposed repair are given in Appendix C and Appendix D of Volume II respectively.

4.3 Structural Strength of the Ampang Bridge

Having calibrated the analytical model and with the convenience provided by the software *Midas Civil*, the load effects due to various live loads were determined. The results obtained from the analysis are tabulated in Table 8. The figures are maximum envelope of load effects at the mid-span in the structure.

Table 8: Load effects due to various live loads

Structure	d_{LTAL}	d_{WRO}	d_{WRO} / d_{LTAL}	M_{LTAL}	M_{WRO}	M_{WRO}/M_{LTAL}
Frame #4	36.1mm	15.98mm	0.44	3388kNm	1,343kNm	0.43
Frame #1	25.7mm	10.11mm	0.39	3279kNm	1307kNm	0.40

Note: *d* is deflection due to LTAL or WRO vehicles and *M* is bending moment due to LTAL or WRO vehicles.

The deflection monitoring of the Ampang Bridge for 5 months showed that the deflections of the monitored beams and piers fluctuated consistently and there was no sign of increase in the deflection readings. Although the main purpose of deflection monitoring was to serve as a mechanism of warning for an untoward bridge failure, the measured deflection could be used to check the stiffness of the bridge deck by comparing with the theoretical deflection.

The maximum vertical deflection of the monitored beams in Frame #4 was 8.50mm downward as compared to maximum theoretical deflection under WRO vehicle loading of 15.98mm downward. The deck had been designed to LTAL Loading to cater for a deflection of 36.1mm. Only 44% efficiency of the capacity was used up by the present traffic (see Table 8).

The ratio for the bending moment shows that the load effect due to the actual dynamic traffic load (represented by WRO vehicles) is only 43% and 40% of the flexural capacity for Frame #4 and Frame #1 respectively. These results more than show that the bridge is structurally sound.

5 VIBRATION STUDY OF THE AMPANG BRIDGE

5.1 Introduction

We recall from earlier discussion in Chapter 3 that the appraisal for satisfactory level of service in vibration involves a number of tasks, which are:-

- Identification of the user sensitivity criteria and the limiting values.
- Vibration monitoring under actual traffic condition.
- Analytical modelling and prediction of the vibration response from WRO vehicles.
- Vibration Assessment involving comparisons of the measured response and predicted response with the limiting values.

5.2 Identification of the User Sensitivity Criteria

5.2.1 General

An extensive literature review was conducted by the Consultant as reported in Volume IV Section B of the Final Report. From the literature search there was no finding related to standards pertaining to the vibration criteria of road bridges. Vibration standards relating to bridges have been mostly for pedestrian bridges where the human perception is heightened by the exposure to the environment as opposed to being in the safety of a vehicle (BD37/01 [17] and Mackenzie Report [13]). Other standards not related to road bridges are also benchmarked to compare ride-ability criteria (Japanese National Railway Association Standards [16]). Guidelines on perception of human response for occupants in a building are many. The criteria from all useful guidelines are summarised in Table 9 and Table 10.

Table 9: Summary of Human Perception Criteria

Code/ Guideline	Criteria of Perception (m*g*)	Criteria of Annoyance (m*g*)	Remarks
ISO2631-1:1997 [8] Building Criteria	1.5	31.5	Freq.band 2-6Hz
Simiu Wind Study [9]	5.0 – 15.0	15.0 – 50.0	-
VDI2057 [10]	1.5	31.5	Freq.band 2-6Hz
Wyatt TAW [11]	15.0	35.0	Freq.band 1-10Hz
Boggs Independent Study [12]	2.0	50.0	Freq.band <1Hz

Table 10: Summary of Riding/User Comfort Criteria

Code/ Guideline	Comfort Criteria (m*g*)	Remarks
Mackenzie Report [13]	72.0 – 175.0	Freq.band 2-6Hz
EN1990: Eurocode 1: Part 2 [14]	70.0	Freq.band <5Hz
Bro 2004 (Sweden) [15]	50.0	Freq.band <3.5Hz
Japanese Railway Standard [16]	50.0 – 100.0	Freq.band 2-6Hz and 6-20Hz
BD37/01 [17]	81.0	-

There tends to be a common agreement among the researchers (Table 9) with regards to perceptible levels and at what levels humans define vibration as intrusive and annoying to routine.

5.2.2 Recommendations of the limiting values a_{lim}

Based on Clause 1 in Appendix B of BD37/01 [17], for superstructure which has a natural frequency of vibration that exceeds 5Hz for an unloaded bridge in the vertical direction and 1.5Hz for the loaded bridge in the horizontal direction, the vibration serviceability requirement is deemed to be satisfied.

For superstructures where the fundamental frequency, f_o is equal to or less than 5 Hz, the maximum vertical acceleration of any part of the superstructure shall be limited to $0.5\sqrt{f_o}$m/s^2. Therefore at the fundamental superstructure frequency at 2.6-2.9Hz, the allowable vertical vibration criteria would be = 81- 86m*g*. It must be noted however that this clause in BD37/01 [17] is with particular regards to the vibration limit for pedestrian bridges, thus may not be suitable in being adopted as a standard for road bridges. Therefore other references which include ISO2631-1:1997 [8] and the Mackenzie Report [13] were studied.

ISO2631-1:1997 [8] is most commonly used in countries where there is absence of any clear cut vibration criteria. In this study it is taken as the benchmark for human perception and annoyance criteria as it presents the most stringent criteria compared to the other standards.

As for the riding/user comfort criteria, recommendations from the Mackenzie Report [13] were adopted for the following reasons:-

a) Although designed for pedestrian bridge limits, the contributing factors to the criteria could be easily demarcated between pedestrian perception and bridge factors. To adopt this standard, the bridge factors were only taken into account for the vibration criteria formulation.

b) The report is based on the BD37/01 [17] for baseline acceleration values with frequency weightings based on the internationally accepted ISO2631-1:1997 [8] (not present in the BD37/01 [17]).

c) The report is based on a very recent study in 2005, indicating the relevancy and optimization of factors taken into account for present day conditions.

d) Allows for flexibility between the value of ambient vibration operational values and exceptional loading values.

e) It is based on ISO2631 for Vibration Exposure Control.

The limiting criteria for user/riding comfort criteria are determined based on the Mackenzie Report [13] formula, yielding 175m*g* as the maximum threshold value. Accelerations below 72m*g* are deemed to be negligible under the user comfort criteria and are categorised as Low vibration range. Values exceeding 175m*g* are categorised as High vibration range. Operating values in an urban environment that occur within the range of 72m*g* – 175m*g* are defined as the Normal vibration range. The proposed bridge vibration criterion is shown in Figure 49.

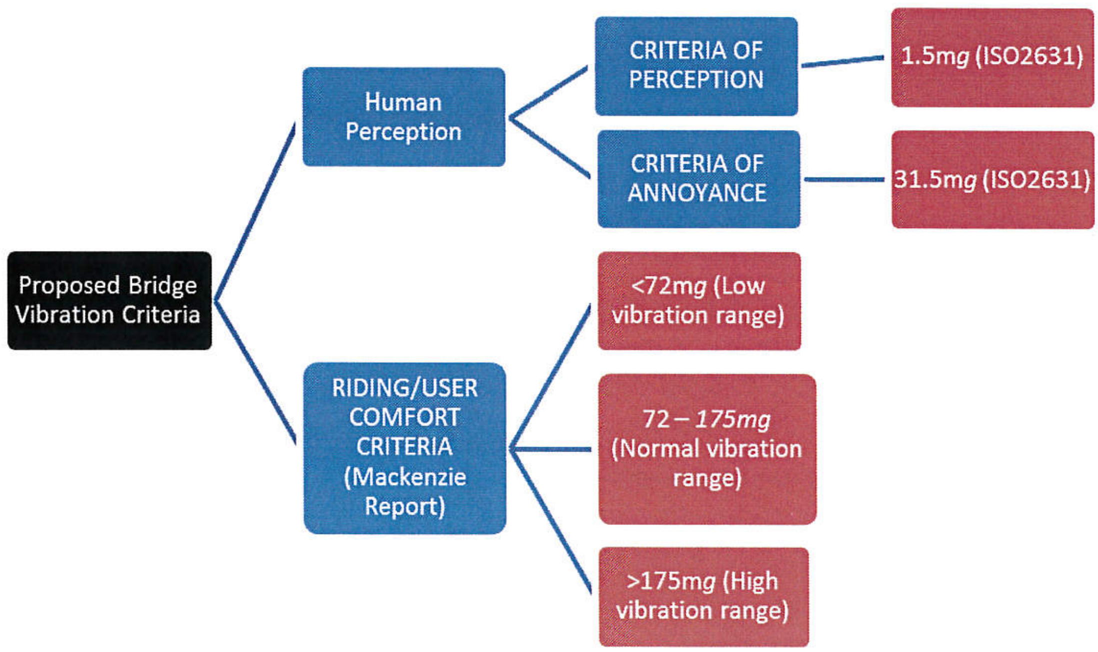


Figure 49: Proposed Human Perception and Riding/User Comfort Criteria

5.3 Vibration Monitoring Under the Actual Traffic

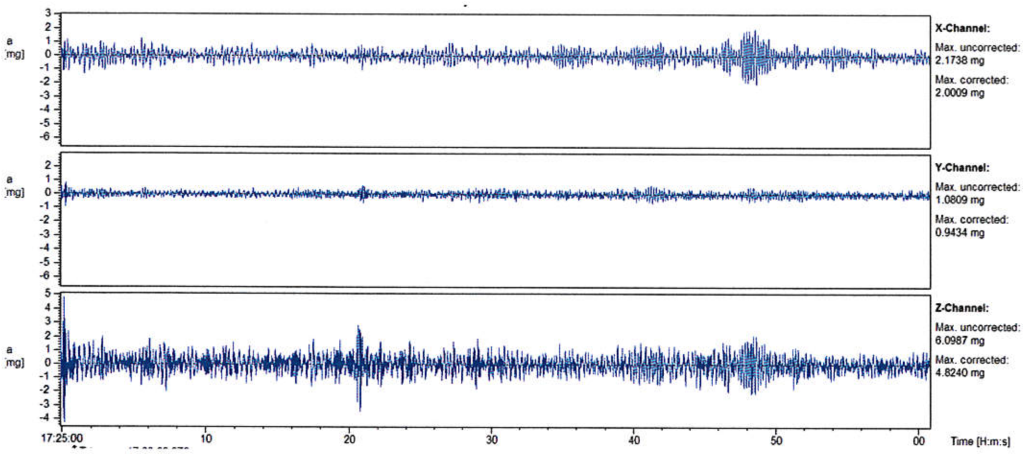


Figure 50: Time domain vibration measurements of P15/CH5 in the x, y and z directions

Figure 50 indicates that under continuous traffic loading on the Ampang Bridge, there are multiple impulse loads occurring in succession with overlapping decaying vibrations. Some impulse loads are larger than others, indicating possible excitation due to heavy traffic loading at expansion joints or potholes if any on the pavement, braking effects and acceleration effects.

The measured acceleration a_{mes} data (for each structure Frame #4 and Frame #1) was analysed as discussed in Chapter 3 and the cumulative distribution function (cdf) determined and presented in Table 11 and Table 12.

Table 11: Cumulative distribution function of Frame #4 span response

a (m*g*)	x	y	z
0	0	0	0
2	37.36%	35.20%	10.61%
4	57.35%	50.71%	20.87%
6	67.42%	58.22%	30.50%
8	73.77%	63.58%	39.28%
10	86.41%	78.74%	51.79%
15	92.83%	86.84%	67.20%
20	95.75%	91.91%	77.77%
30	97.85%	96.74%	89.44%
40	98.46%	98.47%	94.60%
50	98.71%	99.16%	96.98%
60	99.84%	99.48%	98.17%
70	99.90%	99.66%	98.82%
80	99.93%	99.76%	99.20%
90	100.00%	100.00%	100.00%

Table 12: Cumulative distribution function of Frame #1 span response

a (m*g*)	x	y	z
0	0.00%	0.00%	0.00%
2	45.53%	9.94%	8.21%
4	72.61%	19.63%	16.24%
6	85.82%	28.80%	23.87%
8	92.19%	37.30%	30.99%
10	95.43%	45.08%	37.59%
15	98.06%	61.19%	51.89%
20	98.82%	72.39%	62.65%
30	99.24%	85.16%	76.67%

40	99.36%	91.36%	84.62%
50	99.40%	94.61%	89.56%
60	99.98%	96.45%	92.69%
70	99.99%	97.57%	94.75%
80	100.00%	98.28%	96.16%
90	100.00%	100.00%	100.00%

There tends to be a common agreement in Table 9 with regards to perceptible levels and at what levels humans define vibration as intrusive and annoying. Based on Table 11 and Table 12, there is a trend indicating that 63-90% of the data during the monitoring period belongs to records that exceed human perception (1.5m*g* based on ISO2631-1:1997 [8] in Table 9). This also holds true for records belonging to Frame #1 where 55%-92% of the records exceed the prescription of ISO2631-1:1997 [8] for human perception. This indicates that for the duration of the monitoring period, perception of vibration was felt throughout the superstructure by the road users.

For duration of monitoring period at Frame #4, the 95th percentile of records in longitudinal (<20m*g*), transverse (<30m*g*) and vertical (<50m*g*) are taken. For duration of monitoring period at Frame #1, 95th percentile of records in longitudinal <10m*g*), transverse (<60m*g*) and vertical (<80m*g*) are taken. The details of these records can be seen in Table 11 and Table 12. These values are much smaller than the limiting value 175 m*g* established earlier.

There is an anomaly observed in Span 4 (Northbound B4, B6) which has caused the vibration readings to exceed 80m*g* for approximately 2% of all span records. The spans over Frame #1 are on a negative gradient and have transverse bars on the pavement (see Figure 51). The combined effect of acceleration/deceleration from low/high speeds coupled with impulse induced when travelling over the transverse bars are suspected to be the source of this anomaly. The effect of surface roughness on bridge vibration is a common observation and have been a subject of study for example in Ref.[18].

a) Northern approach *b) Southern approach*

Figure 51: Transverse bars at approach spans

5.4 WRO Truck-Induced Vibration from Analytical Model

To simulate the actual traffic patterns on the Ampang Bridge, the following cases were considered:-

a. Case (i): WRO trucks traversing on outer traffic lane of south-bound deck.

b. Case (ii): WRO trucks transverse on outer traffic lane of south-bound deck whereas north-bound deck is loaded with MTAL to simulate traffic jammed condition.

c. Case (iii): WRO trucks traversing on outer two traffic lanes of south-bound deck, i.e. two trucks side by side.

d. Case (iv): WRO truck transverse on outer two traffic lanes of south-bound deck whereas north-bound deck is loaded with MTAL to simulate traffic jammed condition.

Frame #4

Maximum responses under WRO truck-induced vibrations for various traffic cases are summarised and presented below for Frame #4. As the legal speed limit for trucks in Malaysia's highway is about 80-90 KPH, the tabulated values are for V_t less than 90 KPH. The edge girders were more severe than the internal girders. Induced accelerations in transverse (y) and longitudinal (x) directions are much lower than the vertical (z) accelerations.

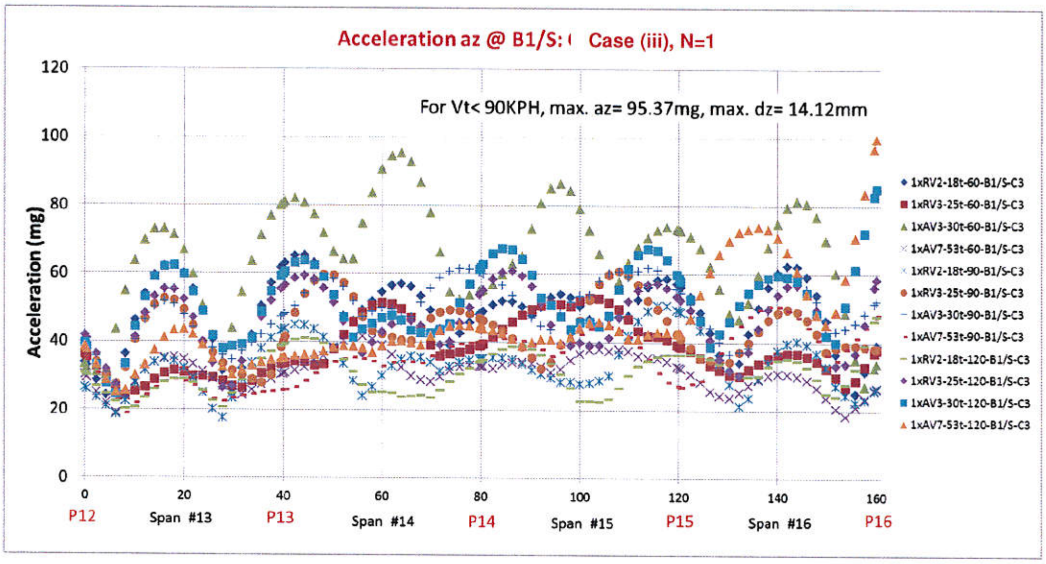
For the case of one truck (N=1) traversing on outer lane of deck, the maximum induced acceleration is 55.55m*g* under Case (ii). This is the most likely scenario which occurs daily on the Ampang Bridge during the peak hours. Human beings are perceptible to vibrations when they stay stationary. When traffic jam occurred on one deck during peak hours, a fast moving truck traversing on the adjacent deck will induce vibrations on both decks. Those road users who are trapped inside the non-moving vehicles will perceive the vibrations. The maximum dynamic vertical deflection is 9.5mm.

For two trucks traversing side by side i.e., one truck per traffic lane (N=1), the maximum induced acceleration is 99.35m*g* under Case (iv). This case may probably happen but not a daily event. The maximum dynamic vertical deflection is 14.12mm.

For the case of trucks traversing in convoy (N > 1) and on outer traffic lane, maximum acceleration of 65.68m*g* is obtained. This case may probably happen but not a daily event. The maximum dynamic vertical deflection is 10.45mm.

For the case of trucks traversing side by side and in convoy (N > 1), a maximum acceleration of 117.23m*g* is obtained. This case shall be considered as an extreme case and it is not recommended for serviceability limit state consideration; it could be recommended for ultimate limit check. It is unlikely that a convoy of trucks traversing side by side over the Ampang Bridge at such high travelling velocity. As highlighted, any change of axle configuration or truck spacing may affect the response. If any truck move slightly faster or slower than others, then re-analyse is required to obtain the responses.

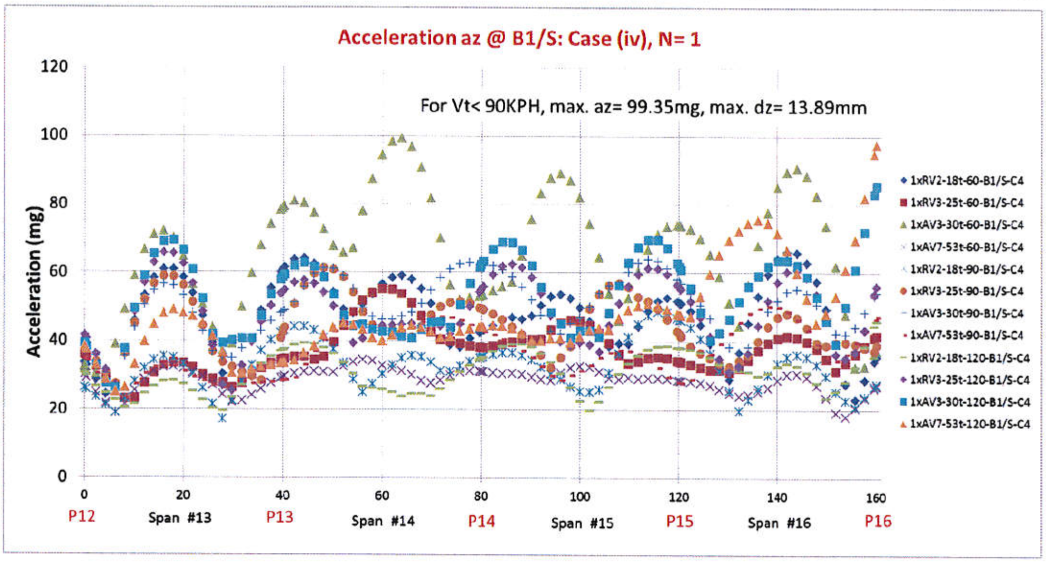
Figure 52 and Figure 53 present the maximum induced accelerations along edge girder B1/S for Case (iii) & (iv) as more severe cases. The maximum accelerations are 95.37m*g* and 99.35m*g* for Case (iii) and (iv) respectively. Both are induced by AV3-30t* traversing at 60 KPH. The maximum vertical deflection of 14.12mm for Case (iii) is induced by AV7-53t at 60 KPH.



Note: 1xRV2-18t-60-B1/S-C3 means the response at beam B1 due to one truck of RV2-18t traversing south bound at 60KPH under load Case (iii)

Figure 52: a_z @ B1/S: Case (iii), N=1

* Different types of vehicles are discussed in Section 3.5 under the heading of Bridge Assessment Using the Analytical Model



Note: 1xRV2-18t-60-B1/S-C4 means the response at beam B1 due to one truck of RV2-18t traversing south bound at 60KPH under load Case (iv)

Figure 53: a_z @ B1/S: Case (iv), N=1

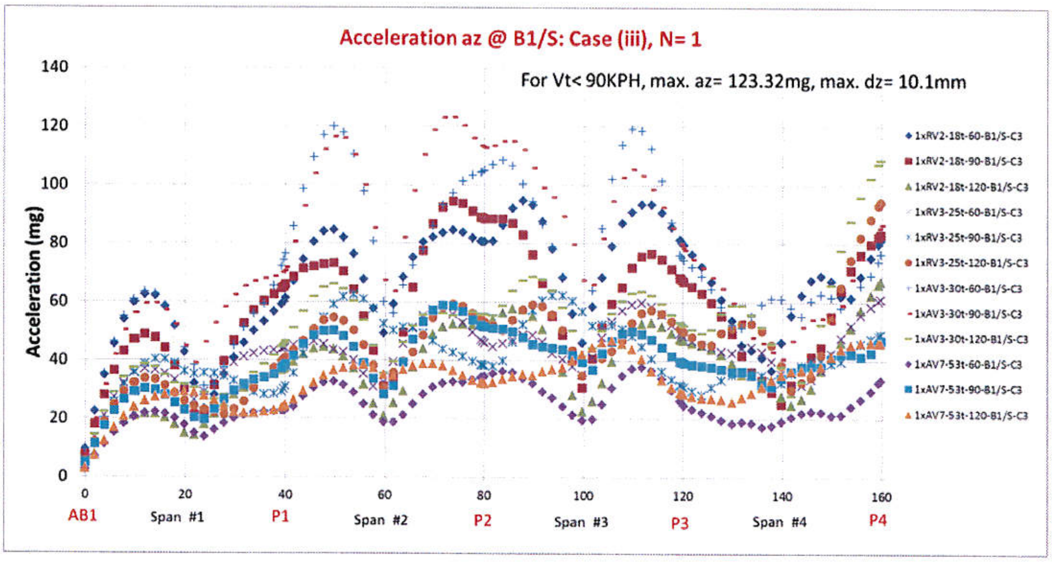
Maximum acceleration of 117.23m*g* is observed for Case (iii) when total 6 nos. of RV3-25t *traversing* side by side and in convoy at 90KPH, i.e. 3 nos. of RV3-25t per traffic lane. This case is considered an extreme condition as it is unlikely that the trucks would traverse side by side and in a convoy passing the bridge at such high velocity. Maximum displacement of 15.98mm occurs at edge girder when a total of 4 AV7-53t *traversing* side by side at 60KPH crossing the bridge.

Frame #1

There are so many variables that affect the performances of bridges due to truck-induced vibrations. That may explain the reason why no conclusive guidelines have been published internationally for the truck-induced vibrations of bridges.

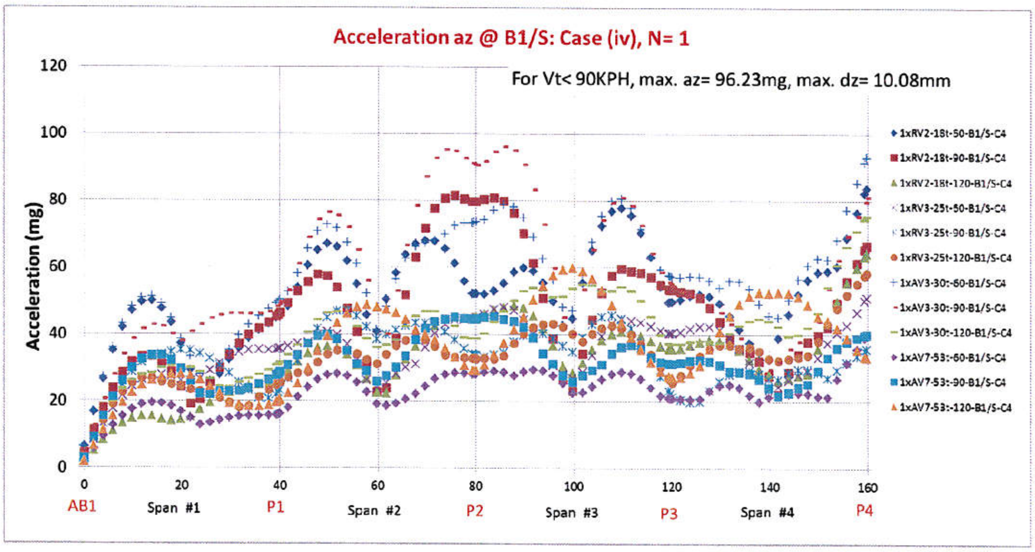
The following section presents the induced accelerations under various cases along bridge span of Frame #1, those result has been summarized, presented and commented in the following paragraph.

The following Figure 54 and Figure 55 present the maximum induced accelerations along edge girder B1/S for Case (iii) and (iv). Only one truck per traffic lane is considered i.e. N=1. The maximum dynamic vertical deflections are also reported.



Note: 1xRV2-18t-60-B1/S-C3 means the response at beam B1 due to one truck of RV2-18t traversing south bound at 60KPH under load Case (iii)

Figure 54: a_z @ B1/S: Case (iii), N=1

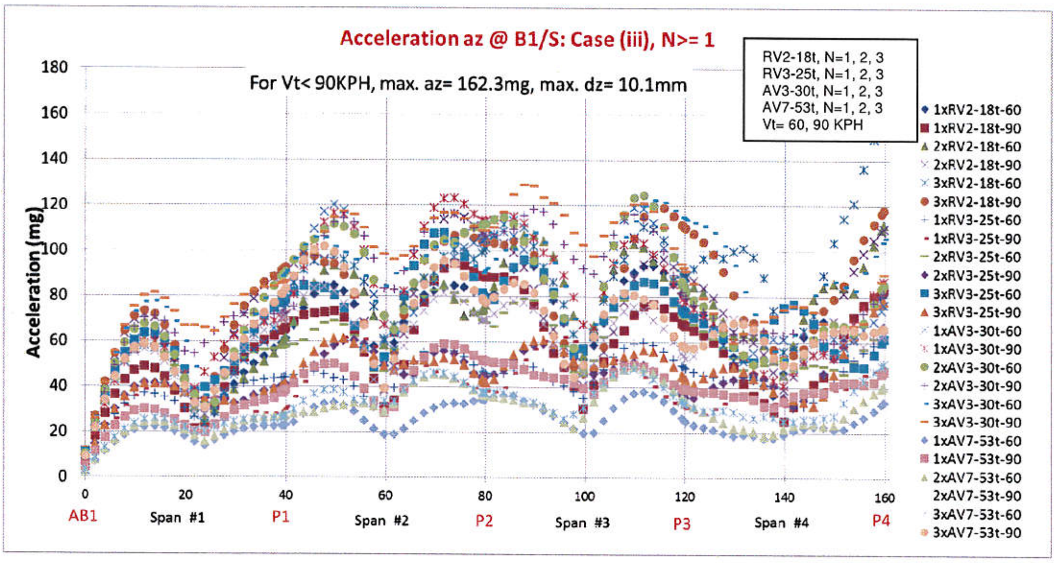


Note: 1xRV2-18t-60-B1/S-C4 means the response at beam B1 due to one truck of RV2-18t traversing south bound at 60KPH under load Case (iv)

Figure 55: a_z @ B1/S: Case (iv), N=1

The maximum accelerations are 123.32m*g* and 96.23m*g* for Case (iii) and (iv) respectively. Both are induced by AV3-30t traversing at 90 KPH. The maximum vertical deflection of 10.1mm for Case (iii) is induced by AV7-53t at 90 KPH.

For cases considering more than one truck per traffic lane i.e. N > 1, Figure 56 presents the accelerations along edge girder B1/S for Case (iii). The spacing of vehicle in convoy adopted is derived base on driver response time of 1s.



Note: 1xRV2-18t-60 means the response due to one truck of RV2-18t traversing at 60KPH

Figure 56: a_z @ B1/S: Case (iii), N >=1

Maximum acceleration of 162.3m*g* is observed for Case (iii) when total 6 nos. of RV2-18t traversing side by side and in convoy at 60KPH, i.e. 3 nos. of RV2-18t in convoy on each traffic lane. This case is considered an extreme condition as it is unlikely that trucks will be traversing side by side and in convoy passing the bridge at high velocity. The Maximum induced vertical displacement is 10.1mm.

5.5 Vibration Assessment

5.5.1 Comparisons between measured vibration results and user sensitivity criteria

Vibration monitoring was carried out for Frame #4 and Frame #1 of the Ampang Bridge. The results are presented in Table 11 and Table 12 of this report. From the total acceleration records, it was observed that:-

a) In the case of Frame #4, 90% exceeded vibration criteria for Perception, 11% exceeded vibration criteria for Annoyance; and none exceeded the High limit of vibration criteria for Comfort of 175m*g*. The 95th percentile value of acceleration was 42m*g*, which is far below the vibration criteria for Comfort (Lower vibration limit) of 72m*g* established earlier.

b) For Frame #1, 92% of the records exceeded vibration criteria for Perception, 25% of exceeded the vibration criteria for Annoyance and the 95th percentile of acceleration was 78m*g*, which is within the normal range of vibration for Comfort Criteria. None of the records exceeded the High vibration limit for Comfort Criteria.

5.5.2 Comparisons between analytical model results and user sensitivity criteria

Table 13 presents the results from the analytical model and the acceptable values from user sensitivity criteria and their comparisons.

Table 13: Comparison between analytical model (WRO 2003) and vibration criteria

		Human Criteria		Riding/User Comfort Criteria		
	WRO 2003 (m*g*)	**Criteria of Perception* <1.5m*g***	**Criteria of Annoyance* <31.5m*g***	**Low Vibration Range** <72m*g***	**Normal Vibration Range** 72-175m*g***	**High Vibration Range** >175m*g***
Frame #1	123.3	Exceed	Exceed	Exceed	Pass	Pass
Frame #4	99.4	Exceed	Exceed	Exceed	Pass	Pass

* ISO2631-1

** Mackenzie

From the table the following observations can be made:-

a) The acceleration for both Frame #1 and Frame #4 exceed the human criteria for Perception and Annoyance. This indicates that the vibration due to traffic could be perceived by the passenger in a stationary vehicle.

b) For both Frame #1 and Frame #4, the Comfort criteria falls within the category of Normal vibration range of 72 – 175m*g*.

5.6 Other Observations

In the course of the study the Consultant had carried out a number of separate studies to address specific doubts encountered along the way. Findings from these studies though do not directly answer questions on the safety or serviceability of the bridge, are nonetheless useful information. They are presented in this section under the following topics:-

i. Lack of contact of bridge bearings.

ii. Influence of travelling velocity on bridge vibration.

iii. Damping as the most significant parameter affecting bridge vibration.

iv. Lateral distribution properties of acceleration response.

v. Comparisons of measured acceleration and WRO truck-induced acceleration.

5.6.1 Lack of contact of bridge bearings

A few of the rubber bearings at Pier 2 were observed to have less than full contact with the beams and bottom plinths (see Figure 57). From the site inspection, area of contact was estimated to be about 75% of the existing bearing size. The bearings were nonetheless found to be in good condition with no visible sign of deformation or displacement. Apparently, the defect in seating was too minor to have caused the bridge to vibrate.

Figure 57: Some bearing that was not in proper contact with the beams

To further satisfy itself the Consultant had proceeded with a study using the calibrated FE model. In the model, elastomeric bearings are modelled as a special "link" element which takes into consideration the compressive and lateral shear stiffness of elastomeric bearings. By assigning values of stiffness to cater for two extreme cases of the bearing seating conditions, namely, "Fully Contact" and "Partially Contact". The results are plotted in Figure 58.

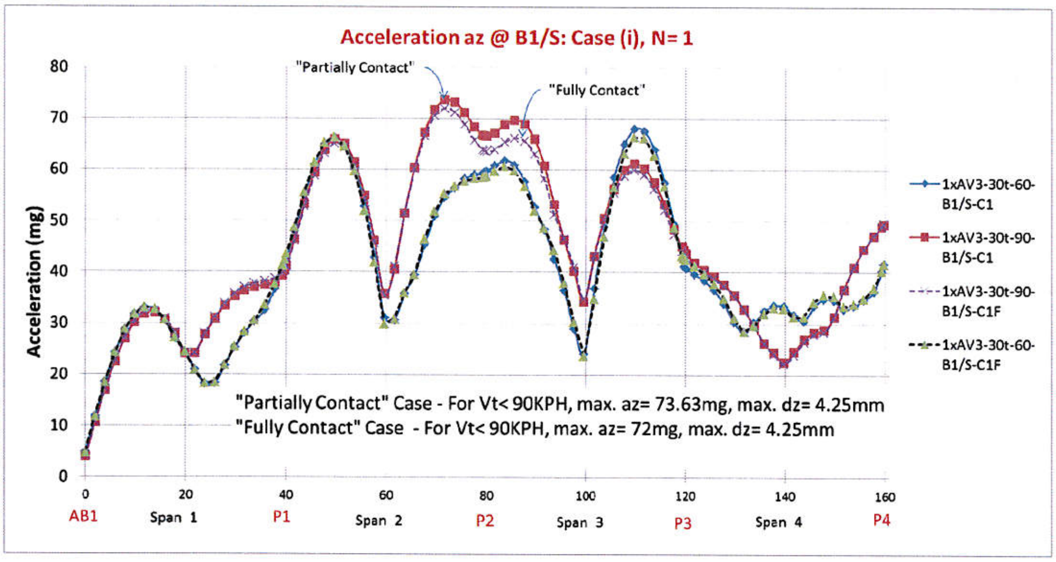


Figure 58: Comparison of a_z for "Fully Contact" & "Partially Contact" bearings

The difference of induced acceleration between "Partially" and "Fully" contacted bearings was only 2.26% which was considered as negligible. Also, from field measurements no anomalous results were recorded for Pier 2 and the data displayed similar vibration distributions as for Piers 1, 3 and 4. Thus, concern of improperly seated bridge bearings at Pier 2 inducing excessive vibration to the Ampang Bridge was not justifiable.

5.6.2 Influence of travelling velocity on bridge vibration

The Consultant had carried out a study to investigate the influence of travelling speed of a vehicle on the bridge on bridge vibration. The calibrated FE model was input with loading from a WRO truck travelling at different velocities. Four different types of WRO vehicles were investigated. The result corresponding to the acceleration response in the edge girder for Span 16 is typical in pattern and shown in Figure 59.

The figure presents the induced accelerations at mid-span of the girder for various travelling velocities. Heavier Gross Vehicle Weight (GVW) and higher velocities induce higher acceleration when travelling velocity is more than 150 KPH. For travelling velocity less than 150 KPH, there appeared to have no specific pattern. Anyway all accelerations induced are less than 40 m*g* i.e., within the low vibration range.

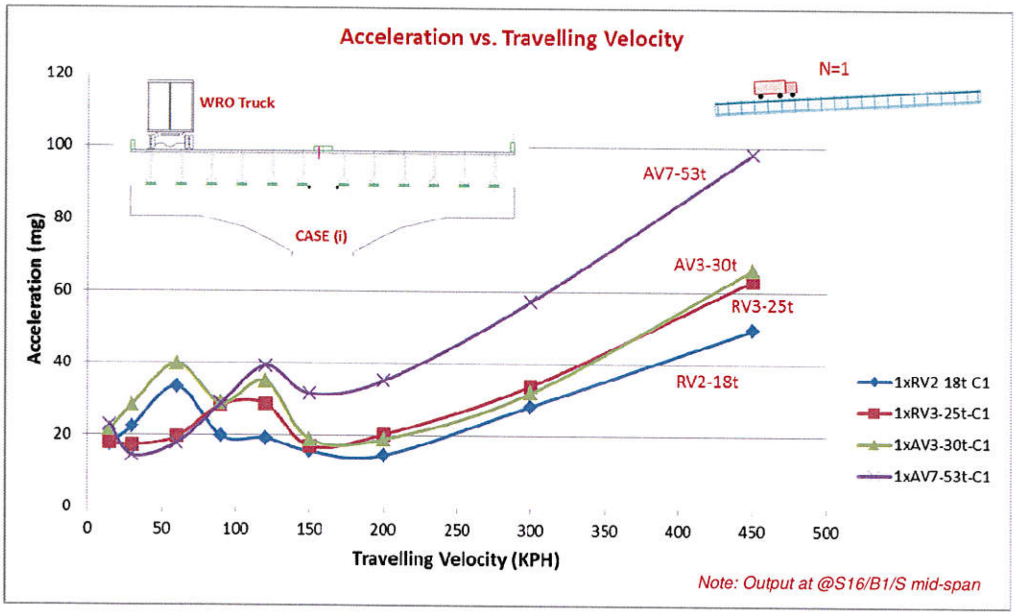


Figure 59: Case (i) az at S16 edge girder B1/S

The maximum response occurs when all the axles are still within the bridge spans. This is because at low velocity, the rear axles of truck can interfere with the response due to the front axles. Different truck axle configurations and travelling velocity define different arrival times of load into the bridge system. It is a complicated problem as it has an unpredictable trend at low travelling velocity. Any slight change in the axle configuration or travelling velocity would affect the vibration; therefore it is advisable to analyse and study on a case-to-case basis.

At high travelling velocity, the maximum response always occurs after all truck axles are out of the bridge structure. This is expected as there is less interference of response from front and rear axles. When departure time of truck axles is short, fewer oscillations of bridge will occur during the passage. The forced vibration in the structure will eventually be subjected to damping effects, thus reducing the amplitude of the response.

5.6.3 Damping as the most important modelling parameters in vibration

In developing an analytical model a base FE model with assumed values of parameters such as stiffness, boundary condition, material properties and damping ratio must first be constructed. This base model would subsequently be tuned and calibrated with the measured response. The Consultant had carried out a time-history analysis to compare the dynamic responses for the calibrated and un-calibrated models induced by one WRO vehicle (AV3-30t) traversing the outer lane of south-bound deck at a velocity, V_t of 60 KPH. The damping ratios, ζ of 0.0%, 2.05% and 5.0% were used as the variable.

Maximum accelerations a_z along the B1/S edge girder are tabulated in Figure 60 for comparison. By comparing the responses for calibrated and un-calibrated with various damping ratio, it can be observed that damping ratio ζ is an important parameter that needs to be determined accurately as it significantly affects the responses of structure.

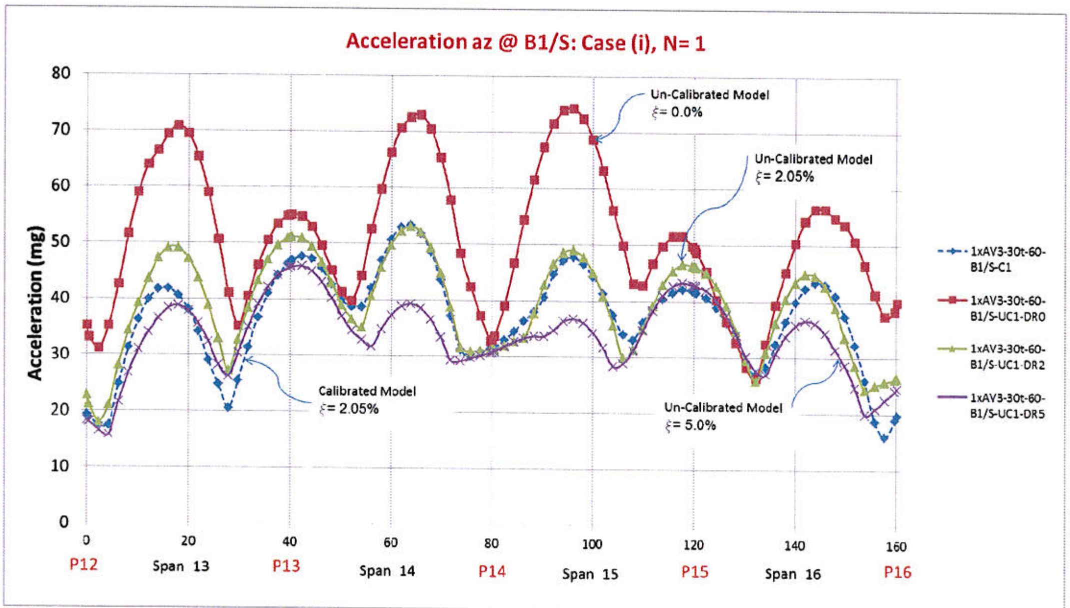


Figure 60: Comparison of response between calibrated and un-calibrated models

The following observations and comments can be made:-

(i) Dynamic response of structure is highly affected by damping ratio ζ used. Low damping ratio of bridge structure will yield higher acceleration response. Therefore, damping ratio is an important parameter that shall be determined from field vibration testing to ensure accuracy of dynamic response prediction. To obtain an upper bound solution, $\zeta = 0\%$ can be used.

(ii) If the same damping ratio is adopted for both calibrated and un-calibrated models, the difference of acceleration induced for Frame #4 is insignificant.

(iii) FE model calibrations by tuning of stiffness, boundary conditions and material properties have less prominent effects on dynamic response of structure as compared with changing structure's damping ratio.

5.6.4 Lateral distribution of dynamic response

Due to the fact that in the case of Frame #4 there are two independent decks supported by a single "T" pier, the lateral dynamic response distribution properties of the deck is quite interesting.

The following figures (Figure 61 and Figure 62) present the lateral distribution of the dynamic response across the deck's girders. It is observed that edge girders usually have higher acceleration response as compared with other internal girders. This is mainly due to more flexibility at the tip of cantilevered crosshead.

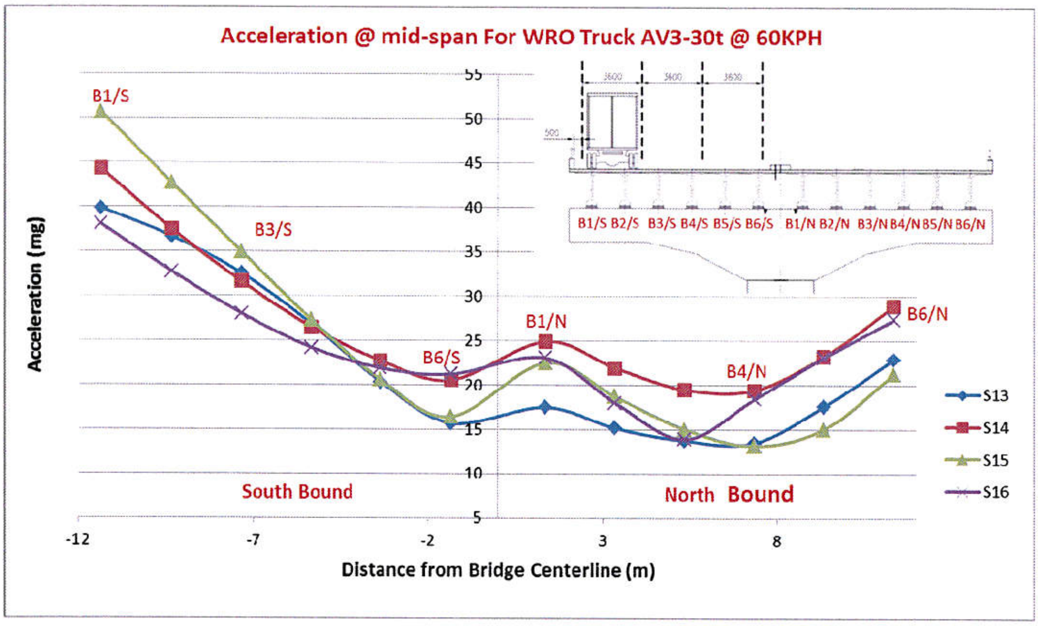


Figure 61: Case (i) az at different girder positions (mid-span)

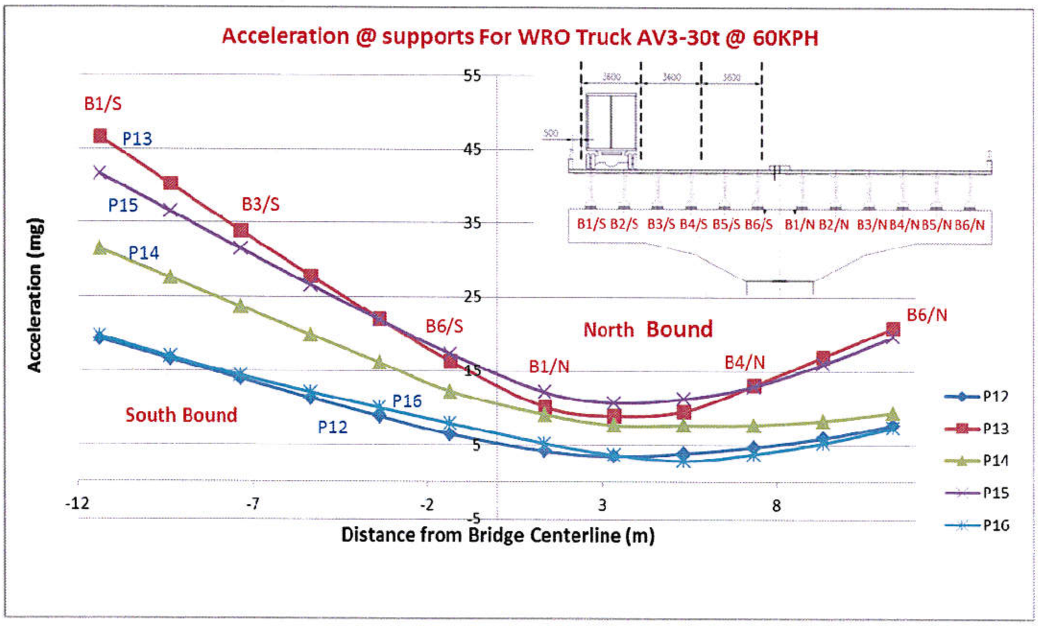


Figure 62: Case (i) az at different girder positions (supports)

5.6.5 Comparison of WRO truck-induced response with field measured response

It was discussed in the earlier chapter that the response measured in the field may represent the lower bound of the actual traffic condition while the predicted response from WRO vehicle the upper bound. Comparison of the two results would offer us some knowledge of how close these two bounds are. Dynamic responses for WRO truck-induced vibrations at eight (8) field measured locations are extracted from analysis output and compared with the 95th percentile of the measured results.

Frame #4

For N = 1, meaning only one truck in the lane, the maximum accelerations of 44.3m*g* and 79.1m*g* for Case (i) and (iii) are at the internal span B1/S and are indicated in Table 14. Both accelerations are induced by the WRO truck AV3-30t travelling at 60KPH. Based on 95th percentile of the measured values, the measured acceleration is about 42m*g*. This value is approximately equivalent to one AV3-30t vehicle traversing at 60KPH.

Table 14: Frame #4- Comparison of analytical and field measured results for N=1

FOR N = 1

Girder Location	a_x			a_y			a_z		
	Case (i)	*Case (iii)*	*95%*	*Case (i)*	*Case (iii)*	*95%*	*Case (i)*	*Case (iii)*	*95%*
S15/B1/S	2.4	4.2	3.6	4.0	5.6	19.7	44.3	79.1	30.7
S15/B3/S	2.0	3.7	4.6	3.9	5.5	24.7	31.6	59.9	30.2
S15/B4/N	1.6	2.5	3.6	4.3	7.0	26.0	22.2	40.5	36.3
S15/B6/N	1.4	2.4	5.2	4.3	6.9	31.1	28.9	48.8	42.1
S16/B1/S	3.9	6.5	12.4	4.3	7.2	2.7	39.8	74.8	19.0
S16/B3/S	3.1	5.3	12.3	4.5	7.4	2.9	32.5	54.2	27.6
S16/B4/N	1.9	2.7	19.6	4.1	6.7	3.8	14.7	26.9	22.2
S16/B6/N	2.3	3.1	27.0	4.2	6.8	5.3	22.9	37.9	41.0

Note: 95% is the 95th percentile of the measured values

For N > 1, the maximum accelerations of 56.8m*g* and 100.*4mg* for Case (i) and (iii) are at the internal span B1/S and are indicated in Table 15. Case (iii) has a much higher value than measured from site. This shows that the ambient traffic does not include vehicles represented in Case (iii).

Table 15: Frame #4- Comparison of analytical and field measured results for N>1

FOR N >= 1

Girder Location	a_x			a_y			a_z		
	Case (i)	*Case (iii)*	*95%*	*Case (i)*	*Case (iii)*	*95%*	*Case (i)*	*Case (iii)*	*95%*
S15/B1/S	4.3	7.6	3.6	7.0	9.9	19.7	56.8	100.4	30.7
S15/B3/S	3.9	6.8	4.6	6.8	9.9	24.7	44.0	82.1	30.2
S15/B4/N	2.7	4.2	3.6	7.0	10.7	26.0	39.9	72.4	36.3
S15/B6/N	2.9	4.7	5.2	7.0	10.7	31.1	51.2	89.7	42.1
S16/B1/S	5.5	9.2	12.4	7.8	12.7	2.7	51.7	93.1	19.0
S16/B3/S	4.3	7.2	12.3	8.0	13.2	2.9	39.6	69.4	27.6
S16/B4/N	3.1	4.9	19.6	7.5	12.3	3.8	24.1	42.7	22.2
S16/B6/N	3.1	4.7	27.0	7.6	12.6	5.3	32.3	55.8	41.0

Note: 95% is the 95th percentile of the measured values

Frame #1

For N = 1, the maximum accelerations of 35.8m*g* and 67.7m*g* for Case (i) and (iii) are at the internal span B1/S and are indicated in Table 16. Both are induced by the WRO truck AV3-30t travelling at 90KPH. Based on 95th percentile of the measured values, the measured acceleration is about 59m*g* except an exceptional case of 99.9m*g* at S4/B6/N during period of site measurement. This value is approximately equivalent to two units AV3-30t traversing side by side at 90KPH.

Table 16: Frame #1- Comparison of analytical and field measured results for N=1

FOR N = 1

Girder Location	a_x			a_y			a_z		
	Case (i)	*Case (iii)*	*95%*	*Case (i)*	*Case (iii)*	*95%*	*Case (i)*	*Case (iii)*	*95%*
S2/B1/S	5.5	8.9	6.0	5.4	8.5	24.2	35.8	67.4	48.7
S2/B3/S	4.5	7.9	7.2	5.0	7.8	30.8	25.7	50.8	32.9
S2/B4/N	3.7	6.8	13.5	3.9	5.8	24.2	23.1	43.0	41.7
S2/B6/N	5.0	8.2	5.8	3.7	5.6	31.5	26.0	46.2	35.7
S3/B1/S	5.8	10.0	7.3	4.1	6.3	44.3	34.4	67.7	43.5
S3/B3/S	4.8	8.2	6.7	4.3	6.6	39.7	27.5	55.7	50.4
S3/B4/N	3.4	5.6	4.8	4.6	6.9	36.8	17.7	32.4	40.1
S3/B6/N	4.6	7.3	4.4	4.9	7.4	24.2	23.2	39.0	29.0
S4/B1/S	3.8	6.2	6.1	6.2	10.1	30.9	33.4	61.1	58.6
S4/B3/S	2.7	4.7	3.8	6.5	10.6	18.2	20.9	32.7	29.6
S4/B4/N	2.1	3.5	7.2	6.9	11.2	42.5	19.6	37.1	52.0
S4/B6/N	4.3	6.9	8.0	7.3	11.9	73.2	22.8	43.0	99.9

Note: 95% is the 95th percentile of the measured values

For N > 1, the maximum accelerations of 56.0m*g* and 102.9m*g* for Case (i) and (iii) are at the internal span B1/S and are indicated in Table 17. Case (iii) has much higher value than the site measured value. This shows that the ambient traffic does not include vehicles represented in Case (iii).

Table 17: Frame #1- Comparison of analytical and field measured results for N >1

FOR N >= 1

Girder Location	a_x			a_y			a_z		
	Case (i)	*Case (iii)*	*95%*	*Case (i)*	*Case (iii)*	*95%*	*Case (i)*	*Case (iii)*	*95%*
S2/B1/S	6.0	10.3	6.0	5.6	8.8	24.2	56.0	98.6	48.7
S2/B3/S	5.5	10.2	7.2	5.3	8.2	30.8	50.5	102.3	32.9
S2/B4/N	4.9	9.2	13.5	4.7	6.9	24.2	41.6	80.1	41.7
S2/B6/N	5.5	9.6	5.8	4.7	6.7	31.5	46.9	90.5	35.7
S3/B1/S	7.2	12.0	7.3	4.8	7.5	44.3	53.2	102.9	43.5
S3/B3/S	5.6	10.5	6.7	5.1	7.9	39.7	43.1	88.9	50.4
S3/B4/N	4.3	7.9	4.8	4.7	7.6	36.8	39.8	75.9	40.1
S3/B6/N	5.6	9.6	4.4	5.0	8.1	24.2	47.3	85.9	29.0
S4/B1/S	6.0	10.3	6.1	7.9	12.7	30.9	40.8	75.2	58.6
S4/B3/S	3.9	7.2	3.8	8.3	13.3	18.2	31.3	52.2	29.6
S4/B4/N	3.6	6.0	7.2	8.4	13.7	42.5	24.4	45.9	52.0
S4/B6/N	6.1	10.1	8.0	8.9	14.6	73.2	30.5	54.9	99.9

Note: 95% is the 95th percentile of the measured values

6 CONCLUSIONS AND RECOMMENDATIONS

6.1 Introduction

The tasks as outlined in the JKR TOR aim to address the public complaints in the news report with regard to the vibration in the Ampang Bridge. They are:-

(i) To carry out a comprehensive inspection of the bridge superstructures and substructures at MRR II viaducts at Ampang.

(ii) To carry out field vibration measurements and monitoring of the bridge superstructure under actual traffic conditions including interpretation of results for compliance with Code of Practice or best practices.

(iii) To carry out analytical computation of bridge dynamic performance under actual traffic conditions including interpretation of results with respect to the measured vibration from (ii).

(iv) To establish or identify reasonable dynamic criteria for user sensitivity to bridge vibrations.

(v) To propose vibration mitigation measures, when necessary.

The overall relationships and interactions of the tasks are depicted in a flow chart/data flow diagram in Figure 10. This chapter summarises the principal findings from these tasks and presents an overall discussion of the works done; concluding with recommendations.

6.2 Principal Findings from Bridge Inspection & Deflection Monitoring (Task i)

Chapter 4 of this report presents the Consultant's work on bridge inspection and deflection monitoring. All major bridge components had been inspected and no major damage or defects had been observed. The few minor defects detected are indeed common defects found in concrete bridges in the country. They were not critical in terms of severity and extents and do not affect the structural integrity of the bridge. They are not likely to have contributed to the vibration of the bridge.

Measurements of the deflections were carried out at the mid-spans of beams in Span 12 and Span 16 as well as at endpoints of pier cross beams of Pier 11 and Pier 15. The results for a period of 5 months show fairly consistent fluctuations of deflections. The maximum downward deflection at the mid-span, which occurred in Span 16, was found to be 8.5mm. This value was much smaller than the theoretical value of 33.9mm due to MTAL obtained from the analytical model. The maximum downward deflections at the end of pier cross beam occurred in Pier 15 and was recorded as 5.9mm compared to the value of 21.5mm due to MTAL. An important finding from this monitoring work was that there was no sign of progressive increase in the deflection readings and as such did not trigger an alert during the study.

6.3 Principal Findings from Bridge Vibration Monitoring Study (Tasks ii – v)

Chapter 5 of this report covers literature research on the user sensitivity criteria, measurement and analysis of the vibration response from actual traffics; and development of an analytical model for predicting the vibration response caused by WRO vehicles. Important findings were as follows:-

<u>User sensitivity study</u>

The literature covers a large amount of works on vibration study on bridges but none gives a specific definition of the reasonable dynamic criteria and the limiting values. Many international codes refer to criteria related to human sensitivity for people in buildings. Those on bridges were mainly directed to control of vibration in pedestrian bridges rather than road bridges. Based on reviews of the literature, the Consultant defines two criteria to be adopted in this study:-

i. Human Perception Criteria (Perception and Annoyance)
ii. Riding/User Comfort Criteria

Perception relates to a reasonable level of vibration that most ordinary people would perceive. *Annoyance*, on the other hand, suggests discomfort reaction with respect to vibration magnitude [8]. The Human Perception Criteria proposed (see Figure 49) were obtained by referring to recommendations from ISO2631-1:1997 [8].

The Riding/User Comfort criteria were developed based on the work by Mackenzie [13] and three categories were specified: the Low vibration range, Normal vibration range and the High vibration range (see Figure 49).

Vibration monitoring

Vibration monitoring was carried out for Frame #4 and Frame #1 of the Ampang Bridge. The results are presented in Table 11 and Table 12 of this report. From the total acceleration records, it was observed that in the case of Frame #4, 90% exceeded vibration criteria for Perception, 11% exceeded vibration criteria for Annoyance; and none exceeded the High limit of vibration criteria for Comfort of 175m*g*. The 95th percentile value of acceleration was 42m*g*, which is far below the vibration criteria for Comfort (Lower vibration limit) of 72m*g* established earlier.

For Frame #1, 92% of the records exceeded vibration criteria for Perception, 25% of exceeded the vibration criteria for Annoyance and the 95th percentile of acceleration was 78m*g*, which is within the normal range of vibration for Comfort Criteria. None of the records exceeded the High vibration limit for Comfort Criteria.

Analytical modelling

An analytical model was developed for Frame #4 and Frame #1 of the bridge, as discussed in Chapter 5 of this report. The model was used to predict the vibration response (in terms of acceleration) caused by WRO vehicles as well as the load effects due to WRO vehicles and MTAL.

Maximum induced vertical acceleration for Frame #4 is 99.35m*g* at edge girder when two trucks traversing side by side and the maximum dynamic vertical deflection is 14.12mm which is much lesser than the deflection under static MTAL i.e. 33.9mm. Thus, it can be concluded that if Frame #4 was adequately designed, it is safe to cater for all dynamic effects due to current WRO trucks.

Further, as discussed above, the 95th percentile of the measured acceleration is about 42m*g*. This value is approximately equivalent to one unit AV3-30t traversing at 60KPH.

For Frame #1, the maximum induced vertical acceleration is 123.32m*g* at edge girder when two trucks are traversing side by side and the maximum dynamic vertical deflection is 10.1mm. The induced deflection is much smaller than the deflection due to static MTAL i.e., 23.6mm. It can be concluded that if Frame #1 was adequately designed to cater for static MTAL, it is adequate to cater for all dynamic effects due to truck-induced vibrations.

Based on 95th percentile of the measured values, the measured acceleration is about 59m*g* except for a single case of 99.9m*g* at S4/B6/N. This value is approximately equivalent to two units of AV3-30t WRO trucks traversing side by side at 90KPH.

6.4 Conclusions

From the summary and discussions above it can be concluded that:-

i. The bridge is without any major defects and is structurally safe.

ii. Two user sensitivity criteria were defined: one for human perception (Perception and Annoyance) and another for Riding/User Comfort. The limiting value for Perception is 1.5m*g*. The limiting value for Annoyance is 31.5m*g*. In the case of Riding/User Comfort Criteria the limiting value for Low vibration range is 72m*g* and for High vibration range is 175m*g*. The range between these two values is the Normal vibration range.

iii. For both Frame #4 and Frame #1, the vibrations from both measured (under actual traffic) and predicted (under WRO vehicles) were at a level as to be perceived and felt annoying. They are however in the acceptable Normal vibration range.

iv. The deflections monitored at the spans and piers indicated that there was no sign of progressive increase in the deflection readings and as such did not trigger an alert during the study.

6.5 Recommendations

i. Despite the conclusions made above that the bridge is structural safe, the Consultant had proposed repair of defects detected as a good maintenance practice. The recommendations including specifications and drawings of the proposed repair are given in Volume II of the Final report.

ii. The very high vibration measured in Frame #1 has raised an issue as to the impact of transverse bars on bridge vibration. Future study on impact of surface roughness on bridge vibration is recommended. A lot of studies have been done on the effect of surface irregularities on bridge vibration, for example, Ref. [18].

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APPENDIX A:

"The Star" Report on MRR2 Vibration

Published: Friday August 17, 2012 MYT 9:56:00 PM

Probe 'swaying' MRR2, say motorists

KUALA LUMPUR: The level of safety at the Middle Ring Road (MRR2) here is being questioned again as several users have expressed their anxiety, claiming two viaducts on the 35km expressway 'swayed' during heavy traffic.

They alleged that the situation at the expressway, which was regarded as extraordinary, could be clearly felt when trapped on the viaducts in Kepong and the Flamingo Hotel junction in Ampang, during heavy traffic.

Former chairman of the Malaysian Press Institute, Datuk Seri Azman Ujang, regarded the incident as abnormal and needed the immediate attention of the authorities concerned. He said the experience he went through last Wednesday night was quite disturbing because it felt as though the viaduct was moving.

"I was really shocked, as if there was a small earthquake in Kuala Lumpur, and it went on for more that 10 seconds. I observed that there were no heavy vehicles near me.

"I definitely am aware of this problem in MRR2. I thought the problem of the viaduct shaking was solved after repairs were made but it is still happening and the situation is worse," said Azman, who was on his way to a breaking-the-fast function in Damansara.

Azman, who is former general manager of the Malaysian National News Organisation (Bernama), said he hoped the situation could be monitored immediately.

He said the Public Works Department and Kuala Lumpur City Hall, which were responsible for the expressway, must review the strength of the viaducts.

According to Mohd Rizal Azman, 30, the swayings on the viaduct in Kepong were repeatedly felt especially when the number of vehicles increased, and during rain.

Mohd Rizal, who stays in Flora Damansara and often uses the route to send his wife to her place of work in Selayang, said the swaying incident was extraordinary compared to the shaking he often felt prior to this.

"I use the route almost everyday, other that sending my wife to work, I also use the expressway to go to my father's house in Batu Caves every weekend.

"I admit the viaduct will shake during heavy traffic, that is normal, but the swaying felt lately is very different and it is like a mini earthquake which makes us feel like we are swinging," he told Bernama, here.

Prior to this, the Kepong viaduct was closed three times from 2004 when Kepong residents revealed that 7,000 cracks were found on 31 of the 33 pillars of the viaduct.

The route was fully closed down following the detection of more serious damage at the viaduct, two years later.

On Aug 3, 2008, the viaduct was closed for the third time when cracks were discovered at the 28th pillar forcing the government to allocate RM70 million to repair the viaduct, which can accommodate 5,000 vehicles at any one time.

Meanwhile, a woman, who also felt the same experience when she was on the viaduct at the Flamingo Hotel junction, said 'the swinging or swaying' during the incident was extraordinary compared to the shaking she felt when using the route previously.

The woman, who declined to be identified, said the situation could be caused by the engineering technology used on the viaducts, but if otherwise, the incident must be given attention to avert any untoward incidents.

"Prior to this, the situation was not too obvious, but when I really stopped on the viaduct because of the congestion, I felt as if I was swaying," she said.

A total 130,000 vehicles, including trailers, use the MRR2 route, which connects the Kepong and Selayang areas to Kuala Lumpur and Petaling Jaya, daily since it opened in 2002. - Bernama

APPENDIX B:

"The New Straits Times" Report on MRR2 Vibration

Tuesday May 21, 2013 11:23 AM

NEW STRAITS TIMES

HOME NATION STREETS WORLD BUSINESS SPORTS LIFE & T

Hot Topics: Kuala Lumpur Malaysia

GENERAL

28 August 2012| last updated at 08:43AM Email Print

'No need to fear MRR2 vibrations'

Google + 0 comments

KUALA LUMPUR: The vibrations felt at the Middle Ring Road 2 stretch in Kepong and near the Flamingo Hotel in Ampang are being monitored closely by the Public Works Department (PWD).

Its director-general Datuk Seri Mohd Noor Yaacob said his team was getting equipment to measure the vibrations.

"There is vibration definitely, but we are very sure that there is nothing for MRR2 users to worry about. We are monitoring the vibrations now and going to acquire equipment so that we can know the measurement," said Mohd Noor, who is also the president for Board of Engineers Malaysia.

The safety of MRR2 was also vouched by the PWD senior director for civil engineering, structure and bridge branch Datuk Dr Abdul Aziz Arshad.

"Technically and engineering-wise, I'm very confident of the safety as I was also involved in MRR2's construction.

"We design all bridges in Malaysia according to the British standard -- BS 5400. The British standard is very conservative and there is no way that the safety of the public is compromised."

He also said that the frequently sighted workers on the bridge were involved in normal repair works which had nothing to do with the current vibrations felt.

"The maintenance work on MRR2 is round the clock to detect any damage on the road. So there is nothing to worry now, we are keeping it in shape."

He added that the small magnitude vibration on Aug 17 might have alarmed road users. Hence, the many complaints on the viaducts along the 35km road which connects Kepong and Selayang to here and Petaling Jaya.

"We take public complaints very seriously. We have our people monitoring the situation and, as far as we are concerned, there shouldn't

It was reported that the department's road facilities maintenance division assistant director-general, Leow Choon Heng, said no abnormal vibrations had been detected when heavy vehicles passed through the viaducts. The Kepong viaduct was previously closed three times from 2004 when residents there revealed that 7,000 cracks were found on 31 of its 33 pillars.

The stretch was fully shut down two years later following the detection of more serious damage at the viaduct, which could accommodate up to 5,000 vehicles at any one time.

APPENDIX C:

Final Values of Parameters of Calibrated Analytical Model

After numerous fine-tuning of the parameters, the updated set of structural and material properties as presented below were derived and implemented in the FE model.

Frame #4:

(i) Foundation stiffness K_f

Table1: Final values of foundation spring stiffness for Frame #4

PIER	SDx	SDy	SDz	SRx	SRy	SRz	Node
P12	2.84E+05	2.66E+05	1.20E+06	6.14E+07	4.20E+07	6.56E+06	*2003*
P13	2.84E+05	2.66E+05	1.20E+06	6.14E+07	4.20E+07	6.56E+06	*2,006*
P14	2.84E+05	2.66E+05	1.20E+06	6.14E+07	4.20E+07	6.56E+06	*2,009*
P15	2.84E+05	2.66E+05	1.20E+06	6.14E+07	4.20E+07	6.56E+06	*2,012*
P16	2.84E+05	2.66E+05	1.20E+06	6.14E+07	4.20E+07	6.56E+06	*2,015*

(ii) Structural Stiffness-Material elastic modulus of E_c

E_c= 34.86E6 kN/m^2 for C50 concrete

28.7E6 kN/m^2 for C30 concrete

(iii) Deck mass density ρ_c

ρ_c= 25.0kN/m^3 for precast girder with C50 concrete

24.0kN/m^3 for cast-insitu RC with C30 concrete

(iv) Elastomeric bearing stiffness K_b

$K_{b,h}$= 2,219kN/m

$K_{b,v}$= 583,000kN/m

(v) Damping Ratio

ζ = 2.05%

Frame #1

(i) Foundation stiffness K_f

Table 1: Final foundation stiffness for Frame #1

Pier No	SDx	SDy	SDz	SRx	SRy	SRz	Node
AB1	1.09E+06	4.27E+05	6.11E+06	1.02E+09	1.31E+08	9.30E+07	*3*
P1	4.23E+05	2.82E+05	1.90E+06	2.35E+08	9.92E+06	2.67E+07	*13*
P2	4.23E+05	2.82E+05	1.90E+06	2.35E+08	9.92E+06	2.67E+07	*18*
P3	4.23E+05	2.82E+05	1.90E+06	2.35E+08	9.92E+06	2.67E+07	*23*
P4	4.23E+05	2.82E+05	1.90E+06	2.35E+08	9.92E+06	2.67E+07	*1,829*

(ii) Structural Stiffness-Material elastic modulus of E_c

E_c= 35.7E6 kN/m^2 for C50 concrete

29.4E6 kN/m^2 for C30 concrete

(iii) Deck mass density ρ_c

ρ_c= 25.0kN/m^3 for precast girder with C50 concrete

24.0kN/m^3 for cast-insitu RC with C30 concrete

(iv) Elastomeric bearing stiffness K_b

$K_{b,h}$= 2,219kN/m

$K_{b,v}$= 583,000kN/m

Except a reduction factor of 0.75 is applied for stiffness values for bearings at P2 for Span 3:-

$K_{b,h}$= 1,664kN/m

$K_{b,v}$= 437,250kN/m

(v) Damping Ratio

ζ = 1.02%

In either case, the final set of parameters for material and boundary properties used in calibrated model is within the acceptable engineering ranges. Also, a good agreement of frequencies has been found between calibrated FE model and field vibration results for the targeting modes.

APPENDIX D:

Derivation of user sensitivity criteria for User/Riding Comfort

In 2005, the UK Highways Agency has commissioned two companion studies for the review of the dynamic sensitivity of footbridges. The studies examine the dynamic sensitivity from two vantage points. The research reported within this paper and undertaken by Flint & Neill Partnership, is to determine the tolerance of pedestrians to the dynamic motion of footbridges (D.Mackenzie, C.Barker, N.MacFadyen, & B.Allison, 2005 [13]).

The report proposed to retain a value similar to the current standard of BD37/01 [17] for acceptable vibrations as a base level, and to factor this up and down in recognition of the need to provide the designer and owner the means to modify the response of the bridge for site specific and user sensitivity criteria.

$$a_{limit} = 1.0.k_1.k_2.k_3.k_4$$

Where

k_1 = site usage factor

k_2 = route redundancy factor

k_3 = height of structure factor

k_4 = exposure design factor

Figure 1: Proposed derivation of maximum acceleration based on Mackenzie study

The factors k_i are defined as follows, a) k_1 is the factor related to the relative importance or criticality of maintaining vibration standards as a function of the surrounding location (hospitals would have a lower tolerance than industrial zones), b) k_2 is the factor related to the redundancy level of a structure or the means of access to the bridge (a bridge in a rural setting should not attract as stringent a requirement as a bridge located on an arterial route travelled by many users), c) k_3 is the factor related to the user perception as a function of the bridge height (this relates to degrees of vertigo which alters the perception of safety and vibration, d) k_4

is the factor that is related to the exposure of the bridge parapet to the bridge user (a pedestrian bridge with glazed parapets would expose the user to the view of the traffic below, altering perceptions towards vibration.

It can be seen from the factors above that k_1 and k_2 factors relate directly to the nature of the bridge whereas k_3 and k_4 relate directly to pedestrian perception. It is proposed that as a modification to the a_{limit} formula proposed by the Mackenzie Report [13] to better represent the vibration limits of road bridges rather than pedestrian bridges, the factors of k_3 and $k_4$are not included in the calculations of vibration limits. As such the modified formula to represent road bridge conditions are proposed as such:

$$a_{limit} = 1.0(k_1 k_2)$$

The factor weighting related to the formula above is given in the Mackenzie Report [13] as such in Figure 2.

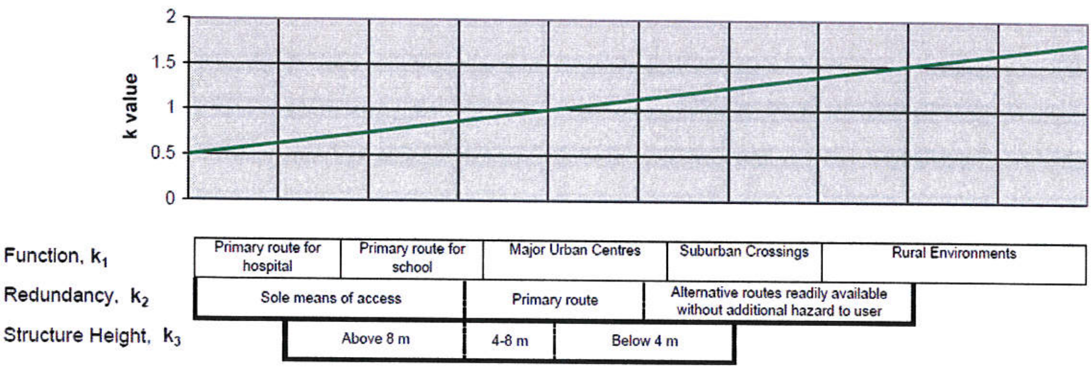

Figure 2: Response modifiers as proposed by the Mackenzie Report [13] as addendum to the base level acceleration limit of 1.0m/s² (100mg)

In order to develop the baseline values based on the modified formula of the Mackenzie report [13], k_1 and k_2 factors were only taken into account for the adjustment factor. Two (2) values are proposed from the Mackenzie report [13], a) lower boundary conditions and b) upper boundary conditions. These conditions were based on the k_1 factor being that of a 'major urban centre' and k_2 being that of a 'primary route' category. The lower and upper bounds would be based on the ranges of these categories.

Lower boundary: 1.0 x 0.9 x 0.8 = 72m*g*

Upper boundary: 1.0 x 1.4 x 1.25 = 175m*g*

The Normal vibration range as per recommended based on the modified formula of the Mackenzie Report [13] to reflect the criteria of road bridges would be from 72 – 175m*g*.