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Experimental analysis of an asymmetric reinforced concrete bridge under vehicular loads

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Abstract. Dynamic response of a three span continuous bridge has been determined by full scale experiments on the bridge. In the experiments, a heavy vehicle was driven across the bridge at different speeds and along different lanes of travel and the strains were recorded at different locations. The bridge was made of reinforced concrete and was asymmetric in plan and in elevation. Frequencies and modes of vibration excited by the vehicle were determined. The dependence of the dynamic amplification on bridge location and vehicle speed was investigated and dynamic amplifications up to 1.5 were recorded, which was higher than values predicted by bridge design codes. It was evident that when this asymmetric bridge was loaded by an asymmetric forcing function, higher modes, which are lateral and/or torsional in nature, were excited. Dynamic modulus of elasticity and the support stiffness influenced the natural frequencies of the bridge, which in turn influenced the dynamic amplifications. Larger than anticipated dynamic amplification factors and the excitation of lateral and/or torsional modes should be of interest and concern to bridge engineers.

Key words: bridge; dynamic response; strains; dynamic amplification; experimental testing; vibration; frequency; mode; asymmetry; torsion; bridge code.

1. Introduction

The dynamic response of highway bridges under the passage of heavy vehicles is difficult to accurately establish, either experimentally or theoretically. This is due to the influence of a large number of parameters on the bridge response and hence the difficulty in generalising the results. Nevertheless, any experimental investigation will provide useful results and will enhance the knowledge base in this area. Vibration characteristics of a bridge and the relationship between dynamic increments prescribed in bridge design codes and those determined experimentally will be of interest to bridge design engineers. In the design of bridges, the static live load is increased by a dynamic load factor (formerly called the impact factor) to account for the dynamic effects of vehicle - bridge interaction.

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Since the 1920s, authorities in a number of countries have developed standard methods for calculating the dynamic amplifications and incorporated them into codes governing the design of bridges (Canteni 1992). Irrespective of the means of quantifying the dynamic effects in the different bridge codes, it is understood that there is usually an amplification in the response due to the dynamic effects. Some authorities initially based the calculation of the amplification on span length but, since the 1970s, an increasing number of codes have instead defined this dynamic amplification as a function of the first flexural frequency. Codes which used the first flexural frequency provision include the Ontario Highway Bridge Design Code (OHBDC 1983), the Swiss Highway Bridge Design Code (Cantieni 1992) and the Bridge Design Code of the Australian State Road Authorities (1992). In the later edition of the Ontario Highway Bridge Design Code (OHBDC 1993), a constant dynamic increment of 25% has been specified for all spans, except for very short ones.

Though it is widely acknowledged that the natural frequencies of a bridge, (in addition to vehicle dynamics and surface roughness), have an effect on the dynamic amplifications, the accuracy of codifying these amplifications as a function of the first flexural frequency has been the subject of considerable debate among bridge designers and researchers. In order to enhance the understanding of the dynamics of continuous asymmetric bridges and to explore the influence of natural frequencies on the dynamic response, full scale testing of such a bridge was undertaken. The dynamic behaviour of these types of bridges is less understood in comparison to that of simply supported bridges. The dynamic response of the chosen bridge was investigated by recording strains and by monitoring its vibration during and after, the passage of a test vehicle. The dynamic amplifications in the bridge response at certain specific locations were determined and compared with the provisions in the design codes. In this paper, the dynamic amplification is defined as the ratio of the dynamic (or total) response to the static response at the location and will be termed the Dynamic Amplification Factor (DAF). Testing also sought to measure the natural frequencies of vibration of the bridge and compare them with theoretical expectations, paying particular attention to issues such as higher order modes of vibration excited by the (test) vehicle, dynamic modulus of concrete and support stiffness.

This paper describes the experimental testing and discusses the results and matters arising therefrom. The results produced several conclusions concerning longitudinal asymmetry, plan asymmetry, vehicle speed, and the ability of code provisions to account accurately for dynamic vehicle - bridge interaction. Experimental results also raised questions about the application of current provisions for dynamic effects in continuous bridges and the excitation of lateral and torsional modes of vibration in a broad bridge under asymmetric conditions. Bridge design codes specifying the dynamic amplification factors assume that the first flexural mode of vibration is excited. From the present experiments, it is evident that in asymmetric bridges such as the one tested here in, higher modes of vibration, which are lateral/torsional in nature are also excited. Higher values of dynamic amplifications (up to 1.5), than those predicted by the design code, were recorded. Since the higher modes of vibration are lateral/torsional in nature, they will not only cause higher stresses but also asymmetric stresses which will differ from those caused by longitudinal flexure. Dynamic amplifications less than unity were recorded at locations outside the "range of influence of the source of excitation" as discussed by Cantieni (1983). Dependence of the dynamic amplifications on location and speed was demonstrated. Results indicate that the present code provisions may not be adequate for these types of bridges.

2. Procedure

2.1. Background

During 1956-57 the Canadian government conducted the first series of full scale tests on 52 bridges. Most modern heavy vehicles have their pitch-type natural frequencies between 2 and 5 Hz and these tests revealed that there is a tendency for the dynamic amplification factor to increase for bridges with an observed fundamental frequency in this range. During 1969-1971 a second series of tests was completed on continuous concrete bridges and the results again indicated that bridges with the natural frequency in the range 2-5 Hz were particularly vulnerable to dynamic loads (Billing 1982, Billing and Green 1984). Bakht and Pinjarkar (1989) reviewed these results and pointed out that most dynamic testing has been done with specific vehicles and the resulting dynamic amplifications may be different from those recorded during realistic traffic conditions.

In the mid 1970s the Swiss government (Cantieni 1983) initiated a project to systematically summarise and organise the data and results of 60 years of full scale bridge testing and investigated the effect of road roughness on the dynamic amplification. It was observed that the road roughness had an effect in reducing the critical frequencies to the range 1.5 Hz-3 Hz (from 2-5 Hz). It is thus evident that the dynamic response of simply supported bridges is well understood and there is a substantial amount of test data pertaining to them. On the contrary, dynamics of asymmetric and/or continuos bridges is less understood and more difficult to generalise, though there is some available research on them (Cantieni 1992). Thus, it was decided to investigate the dynamic behaviour of an asymmetric continuous bridge. A large proportion of highway bridges pass over other roads, which determine their pier configuration. This often necessitates skewed and longitudinally asymmetric structures. Therefore, a bridge of this kind was sought, so that testing could investigate the effect of these parameters on the dynamic response.

2.2. Description of bridge

The Fig Tree Pocket Bridge, which is a road bridge located in the west of Brisbane, Australia, met these criteria. Moreover, it is a continuous, three-span bridge with unequal span lengths of 12 m, 24 m and 22 m (see Fig. 1). To accommodate the Centenary Highway which passes beneath the structure, the bridge was constructed with a skew of 29°. The superstructure consists of a reinforced concrete deck, constructed over five standard pre-stressed continuous I girders (see Fig. 2), which have an average depth and width of 1425 mm and 540 mm respectively. The bridge includes a pedestrian footpath and, therefore, the centerline of the *road* does not coincide with the Center of stiffness of the *bridge*. That is, the bridge is asymmetric in plan as well as being longitudinally asymmetric. A cross-section of the bridge is shown in Fig. 3.

The bridge girders are supported on elastomeric bearings whose compression stiffness was supplied by the bearing manufacturer. The rotational stiffness of the bearings was determined by comparing the appropriate analytical results for the fundamental frequency with that observed during bridge testing, as discussed in section 6.4. These bearing stiffnesses were then incorporated into the frequency analysis of the bridge, using the grillage method and yielded an increase of about 2% in the values of the early frequencies.

The objective of the experiment was to study the vibration of this bridge and to evaluate the dynamic amplifications at the centres of each span for a range of vehicle speeds and lateral vehicle

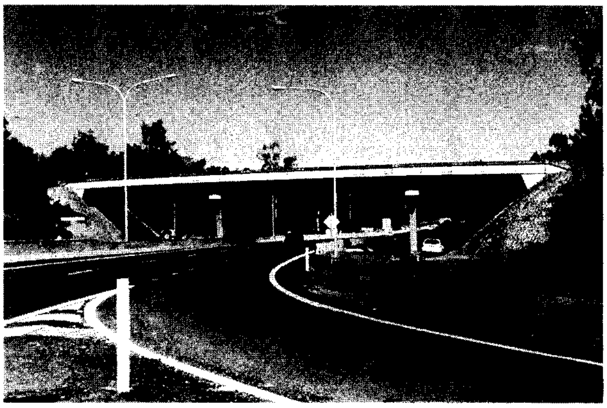


Fig. 1 Fig Tree Pocket Bridge

Fig. 2 Bridge girders

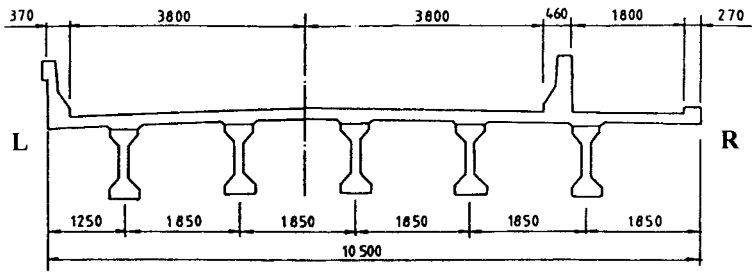


Fig. 3 Cross section of bridge

positions. Experiments were conducted on the bridge on two separate occasions, using two different vehicles.

2.3. Test vehicles

There were two requirements for a test vehicle: its mass had to be sufficient to induce an adequate

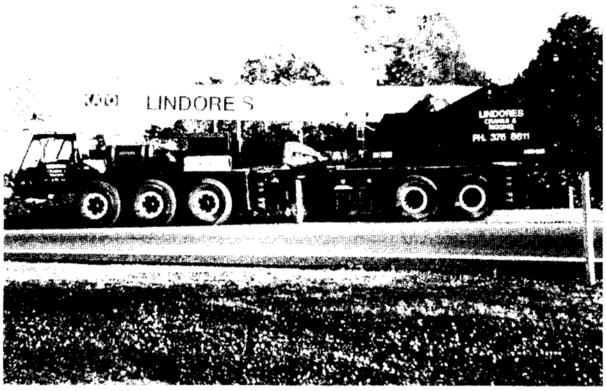


Fig. 4 Test vehicle in first series

Fig. 5 Test vehicle in second series

output vibration response, and its wheel base short enough to concentrate loads on one span. A 47 tonne crane with 3 front and 2 rear axles, which satisfied these requirements, was used for the first series of experiments (Fig. 4). In order to verify some of the unusual results in this asymmetric bridge, further (limited) experimentation was undertaken at a later date. A truck with 2 front and 3 rear axles, together with a dump truck loaded on top giving a gross load of 38 tonnes, was used in the second series (Fig. 5). This made it possible to study the bridge response for two different truck loads with different front and rear wheel configurations and to complement and supplement the results in each of the two series. Figs. 6 and 7 illustrate the wheel configurations and load distributions of the two test vehicles.

2.4. Instrumentation and data acquisition

On both occasions, the bridge was instrumented using standard 60 mm strain gauges which were glued to the bottom flanges of the main girders at their mid span positions as shown in Fig. 8. The following criteria governed the location of the strain gauges.

(1) The centre girders were instrumented in order to investigate the longitudinal flexural mode of

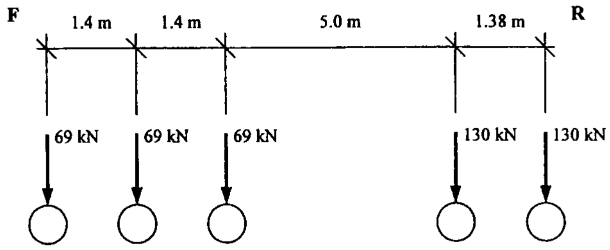


Fig. 6 Wheel loads in first test series

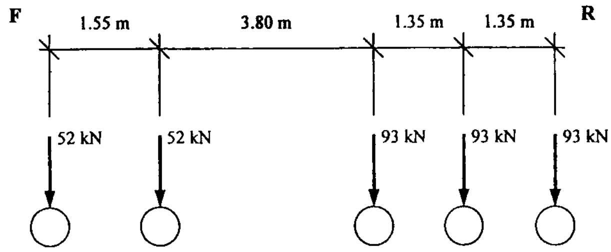


Fig. 7 Wheel loads in second test series

vibration, which is usually the fundamental mode.

(2) The centre girders in the end spans were instrumented to investigate the effect of unequal end span lengths.

(3) The outside girders of the centre span were instrumented in order to capture asymmetric stresses, (which will differ from those caused by longitudinal flexure), due to the excitation of torsional and/or lateral modes of vibration.

Due to the road centre line not corresponding with the stiffness centre line (i.e., the centre line of the superstructure), it is possible for lateral and/or torsional modes of vibration to be excited, when the bridge is subjected to asymmetric loading. Dummy gauges constituting one arm of the Wheatstone Bridge which are used for temperature compensation were considered unnecessary, as the test vehicle was expected to be on the bridge only for a short period of time, except for the "equivalent" static tests where the test vehicle traversed the spans at a very low speed (approx. 5 km/h). In any case, the gauges were balanced (ie., zeroed) before each experiment thus allowing for temperature

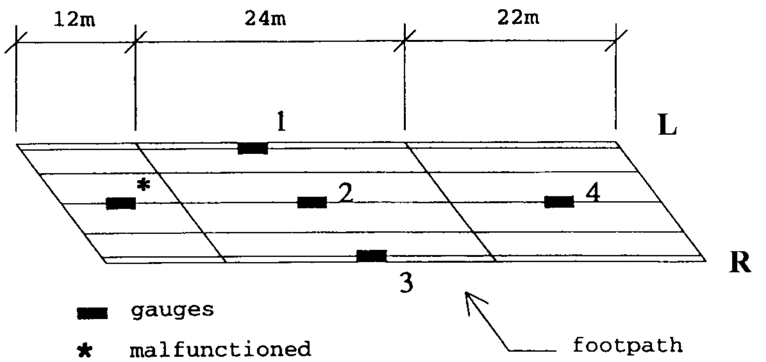


Fig. 8 Strain gauge locations

compensation.

Because the bridge is skewed, the instrumented points could not be arranged in a straight line across the middle span. Instead, each gauge was attached to the bottom flanges of a corresponding girder, in a crack-free zone. Gauges were placed parallel to the support lines (of the girders) to ensure that the maximum amount of strain was induced in each gauge. The numbers shown in Fig. 8 represent Channels 1, 2, 3 and 4. Unfortunately, in the first series of experiments, Channel* failed to function reliably and could not be replaced. Therefore, it was discarded in the first series of experiments and the results from it were ignored in the second series of experiments.

In the first series, the data acquisition system consisted of an analog to digital converter, strain amplifier and Asystant Plus, which is a powerful data acquisition program. The rate of sampling was dependent on the test run and ranged from 50 data points per second (Hz) to 300 Hz when the vehicle was travelling at 60 km/h. These rates are suitable for the range of frequencies of vibration of the bridge. While acquiring data, in order to achieve meaningful results, the raw signal was amplified five hundred times by the strain amplifiers and a further eight times using a software gain facility in Asystant Plus. In the second series, the data acquisition system supplied by Blastronics was used and there was no need for an analog to digital converter. The sampling rate was much higher on this occasion and ranged upto 800 Hz.

3. Bridge testing

3.1. Equivalent static tests

The "equivalent" static experiments involved the test vehicle traversing the spans along the left and right lanes at a speed of about 5 km/hr, so that dynamic effects would not be induced in the superstructure. It has been suggested convincingly that, at this speed, dynamic responses are not present and a static response is obtained (Bahkt and Pinjarkar 1989). The common method used to obtain a static result is to park the vehicle at the point being considered and measure the deflection. However, in this project where the response in all three spans is being investigated, it would have been necessary to park the vehicle at several points on the bridge. As this would have posed problems due to traffic disruption, this method was not pursued. The data collected during the equivalent static experiments will be used to calculate the dynamic amplifications in the spans.

3.2. Dynamic tests

The dynamic tests constituted four series of tests where the vehicle travelled at various speeds ranging from 10 km/h to 60 km/h. The four series of tests are as follows:

(1) Vehicle in left lane travelling in legal direction
(2) Vehicle in left lane travelling in illegal direction
(3) Vehicle in right lane travelling in legal direction
(4) Vehicle in right lane travelling in illegal direction

The data collected during the dynamic experiments, together with the data collected earlier from the static experiments, were used to determine the dynamic amplification factors. The legal and illegal directions of travel were included to investigate the effect of entering the span from a different direction and the consequent effects of asymmetric loading. The lateral vehicle position was varied to investigate the excitation of modes other than the longitudinal flexural mode, which is generally assumed to be the fundamental mode. The test vehicle speed was varied to investigate the effect of speed and the consequent effect of time taken to cross a span. Effect of road surface roughness was not considered in this investigation.

3.3. Test for natural frequencies of the bridge

A separate experiment was conducted to determine the natural frequencies of vibration of the structure. An accelerometer was attached to the concrete beam under the footpath, and records were taken immediately after a large vehicle, such as a bus or concrete truck, passed over the structure. Several such records were taken. The data acquisition rate, using Asystant Plus was 100 Hz, and the duration of samples was either 2.5 or 5 secs.

4. Results

4.1. Series I

Losing the signal in the electronic noise becomes a problem with some electronic data acquisition systems. Upon graphing the original data in this series of tests it was evident that there was substantial noise in the signal, even though every attempt was made to control the noise. In an attempt to minimise the noise component, an object-oriented C++ program was devised, incorporating a low pass filter package. The raw signal response was then transferred through this software, resulting in a much smoother output signal. The peak strains obtained from these graphs pertaining to the "equivalent static" and dynamic tests are used to evaluate the dynamic amplification factor DAF defined as:

$$\text{DAF} = \text{peak dynamic strain} / \text{peak static strain} \tag{1}$$

The results for the dynamic amplifications calculated from this series of experiments are presented in Table 1. For the purpose of interpreting the results, the right hand lane is the one close to the footpath, as shown in Fig. 3. Each test run is defined as follows in the Table:

xyyz

where "x" represents the lane in which the vehicle travelled (L = left, R = right), "yy" represent the vehicle speed, while "z" indicates the direction of travel along the right or left lane (legal = +, illegal = –). When the test vehicle travelled along the left lane, readings from channels 3 were ignored as the corresponding strain gauge was not within the range of influence and as DAF is treated as a local effect in this investigation. Similarly, when the vehicle travelled along the right lane, readings from channel 1 were ignored.

4.2. Series II

With the improved "Blastronics" data acquisition system used in this series of experiments, most of the problems encountered in the previous series were absent. In this series, it was observed that

Table 1 DAF in first test series

Test	Ch 1	Ch 2	Ch 3	Ch 4
L10+	0.9	1.1	*	0.7
L20+	0.9	1.3	*	0.9
L30+	1.0	1.2	*	1.0
L40+	1.0	1.1	*	0.9
L50+	1.5	1.5	*	1.1
L60+	1.3	1.5	*	1.3
R10+	*	1.0	1.0	1.0
R20+	*	1.1	1.0	1.1
R30+	*	1.1	1.2	0.9
R40+	*	1.1	1.0	1.1
R50+	*	1.2	1.2	1.0
R60+	*	1.6	1.4	1.5
R10-	*	1.1	1.0	0.9
R20-	*	1.1	1.1	0.9
R30-	*	1.1	1.0	0.9

*denotes the gauge was outside the direct zone of influence (L or R denotes vehicle in Left or Right lane respectively)

the DAF was not significantly influenced by the direction of travel (legal or illegal) of the vehicle in a lane. Hence, average peak strains have been used to calculate the DAF as defined in Eq. (1) and the results are presented in Table 2.

4.3. Natural frequencies of bridge

Fig. 9 illustrates a typical vibration response of the bridge after numerical smoothing with a Blackman-type filter. The commercial data acquisition package Asystant Plus was able to perform a Fast Fourier Transform (FFT) and plot the corresponding power spectrum. The FFT is a numerical process which resolves a given signal (excitation) into its constituent harmonic components. The power spectrum is a plot of the energy distributed into each harmonic, and provides a means of identifying the constituent frequencies corresponding to the modes of vibration of the system. This is an important feature, as most engineering structures rarely vibrate solely in a single mode after being subjected to an external excitation. The vibration of the Fig Tree Pocket Bridge is a typical example of this phenomenon. Fig. 10 shows the power spectrum corresponding to the bridge response. The natural frequencies evaluated from the FFT/power spectrum analysis have been summarised in Table 3. These are average values obtained from 9 records of the bridge vibration.

4.4. Grillage analysis

Though the main thrust of this paper is the experimental testing of a continuous bridge, with the intention of determining its dynamic response, it seems appropriate to carry out some limited analytical work to verify some of the experimental results. It has been widely accepted that the grillage method provides an efficient and accurate analysis of bridge structures. Tan *et al.* (1998)

Table 2 Average DAF in second test series

Test	Ch 1	Ch 2	Ch 3	Ch 4
L20	1.0	1.1	*	0.9
L40	1.0	1.0	*	1.0
L60	1.2	*	*	1.3
R20	*	1.0	1.0	1.0
R40	*	1.0	1.1	1.1
R60	*	1.0	1.2	1.0

*denotes the gauge was outside the direct zone of influence (L or R denotes vehicle in Left or Right lane respectively)

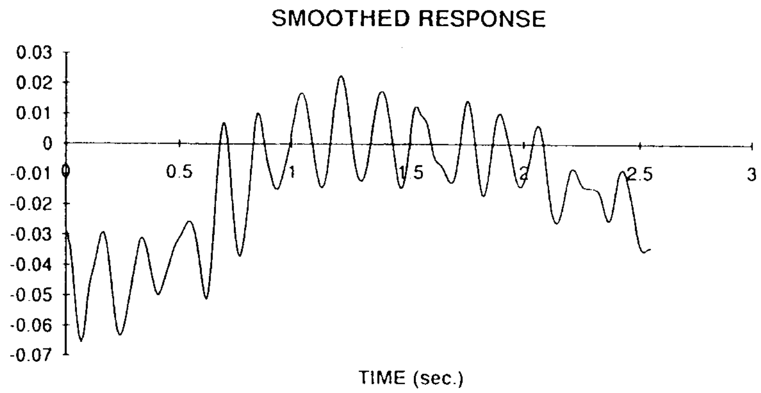


Fig. 9 Typical smoothed acceleration record

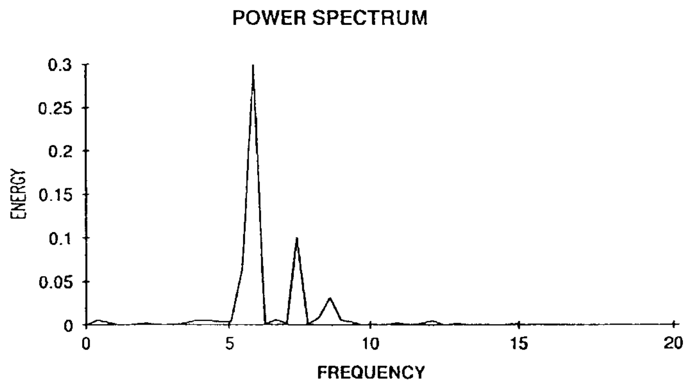


Fig. 10 Power spectrum analysis

investigated the dynamic amplifications of a single span symmetric bridge due to a moving load where the bridge was modelled as a grillage. In order to verify the experimental values of the natural frequencies of vibration of the bridge under investigation, and to determine the associated mode shapes, the superstructure was analysed as a grillage shown in Fig. 11. Table 4 summarises the material and section properties used for the girders in the grillage analysis, based on the

Table 3 Frequencies of excited modes

Mode	Frequency (Hz)
1	5.85
2	7.20
3	8.60
4	9.30

dynamic modulus of elasticity for concrete. Girders 2-6 support the bridge deck, while girders 1 and 7 are really slabs beyond the zone of influence of the edge girders (Fig. 3). The moments of inertia of the longitudinal girders were calculated using equivalent concrete areas for the steel reinforcement. The mass of the superstructure was distributed to the girders. The dynamic modulus of concrete is used in the analysis as it is more appropriate for treating dynamic problems. The significance of using the dynamic modulus in vibration analysis was discussed elsewhere (Memory *et al.* 1995). The results from the grillage analysis are shown in Fig. 12 and Table 5, from where it can be seen that the first five frequencies of vibration are in the range 5.73 Hz-10.28 Hz, which is outside the usual range of the natural frequencies of most modern day heavy vehicles.

The results from the power spectrum analysis (Table 3) gives the frequencies of the modes of vibration excited during the testing, while the grillage analysis gives the first five natural frequencies of vibration of the bridge (Table 5). By comparing these two sets of values, it is possible to determine the modes of vibration which were/were not excited during the testing. This will be discussed later.

5. Discussion of results

5.1. General observations

All strain gauges, for both series, experienced positive strains (downward deflection) when the

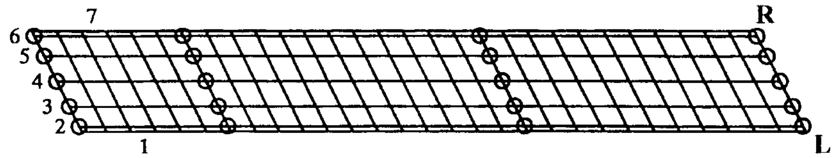


Fig. 11 Grillage model

Table 4 Details of grillage

Girder #	Area (m^2)	Ixx ($m^4 \times 10^{-4}$)	J ($m^4 \times 10^{-4}$)	Density (kg/m^3)	E (MPa)
1	0.166	6.36	3.18	2500	42370
2	0.603	1260	83.3	3531	48470
3, 4, 5	0.598	1210	372	2722	48470
6	0.776	1390	84.0	2754	48470
7	0.131	5.18	2.59	6536	42370

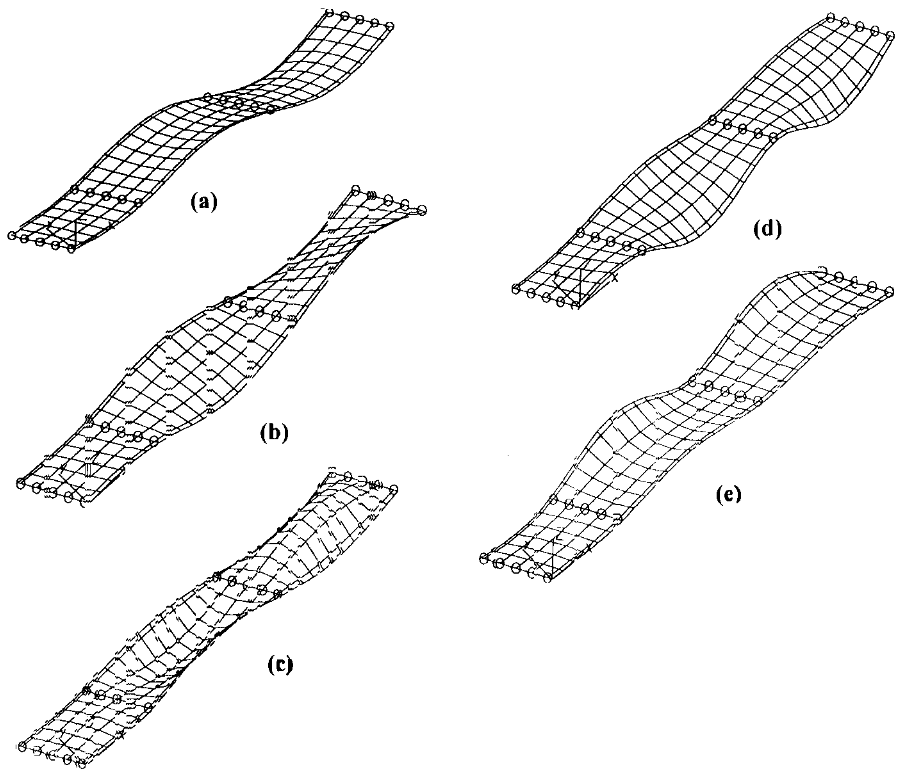


Fig. 12 Mode shapes from free vibration analysis; (a) First Mode, f_1 = 5.73 Hz, (b) Second Mode, f_2 = 6.53 Hz, (c) Third Mode, f_3 = 8.39 Hz, (d) Fourth Mode, f_4 = 8.74 Hz, (e) Fifth Mode f_5 = 10.28 Hz

Table 5 Results from free vibration analysis

Mode	Frequency (Hz)
1	5.73
2	6.53
3	8.39
4	8.74
5	10.28

vehicle was on the instrumented span and negative strains (upward deflection) when the vehicle was off the instrumented span. This indicates that the gauges were recording the positive/negative deflections correctly. Moreover, as anticipated, larger readings were recorded on the gauges closer to the load, indicating that the gauges were functioning properly.

5.2. Dynamic amplification factors (DAFs)

From Tables 1 and 2, it can be seen that the DAF varied from 0.7 to 1.6 and from 0.9 to 1.3 in the first and second series. Some of the low DAFs, such as 0.7 or 0.9 occurred when the test vehicle travelled at low velocities. This aspect is discussed in section 6.1. The values of DAF determined experimentally in the present investigation are similar to the theoretical values of DAF obtained by Tan *et al.* (1998), for moderate damping. Hwang and Novak (1991) evaluated the DAFs in single span, simply supported bridges, using computer simulations. In their simulations the bridge was modelled as a beam. Their results will not be applicable to the continuous and somewhat broad bridge tested herein. It is possible that the full dynamic vehicle - bridge interaction might not have developed at the low velocities due to the excitation of the body (or bounce) mode of vibration of the vehicle. This bouncing mode of vibration can cause a reduction in the forces transmitted to the bridge via the axles when they are at or near the strain gauges, resulting in low DAFs. The high DAF in the R60+ test run in the first series can be attributed to a large crack between the bridge and the approach road. When the test vehicle entered the bridge in the right lane, legal direction, it encountered the crack, which induced a significant bounce in the test vehicle and increased dynamic strain in the girders. This in turn caused a high DAF value. The crack was avoided in the second series. The maximum DAF, for this continuous and somewhat broad bridge, was higher in the first test series which employed a heavier test vehicle. Channels 1 and 3 gave unusually low or high values for the DAF, when the vehicle travelled along the right and left lanes respectively (Ranchigoda 1992). As stated earlier, DAF being a local effect, these abnormal values of DAF are not realistic and have not been included in the Tables.

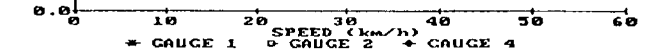
The reasonably consistent range of values for the DAF obtained on two separate occasions with different test vehicles is noteworthy. On both occasions higher values of DAF occurred when the vehicle travelled on the left lane and the direction of travel of the vehicle (legal or illegal) was not significant. On the basis of the first flexural frequency of 5.79 obtained as an average between the observed and calculated values, the dynamic load factor according to the AUSTROADS Bridge Design Code (1992) is about 1.27. It can be seen that in an asymmetric set up such as this bridge, the real dynamic amplification can be much higher at certain locations than the code specified value.

When the vehicle travelled along the left lane of this asymmetric bridge, it is possible that it excited a higher mode (probably the second) which is lateral and/or torsional in nature. For the bridge vehicle system considered here, it could be expected that the natural frequencies of the bridge (> 5.73 Hz) are higher than those of most modern day heavy vehicles (2-5 Hz). However, as the frequency spectrum of the dynamic wheel loads of these vehicles can vary with speed and pavement roughness, and the bridge frequencies can also be influenced by the mass of the moving vehicle (Cantieni 1992), it is not possible to be certain about this (frequency ratio). Nevertheless, in the given asymmetric set-up, higher modes of vibration of the bridge have been excited and these seem to have caused larger dynamic amplifications, in comparison with those when the fundamental mode is excited. A similar feature was observed in the Six Mile Creek Bridge, which is a short span, steel and concrete highway bridge located on the Cunningham Highway west of Brisbane, Australia (Memory *et al.* 1991). Huang *et al.* (1995) analytically treated the dynamic response of a box girder bridge and reported that higher modes of vibration caused torsion and distortion in the bridge. This phenomenon will be further discussed in the paper. Appropriate provisions for dynamic amplifications in asymmetric and/or continuous broad bridges cannot be therefore done in a simple manner as suggested in some bridge design codes. Even if a higher dynamic load factor is used in the design, the stress amplifications due to lateral and/or torsional modes of vibration in a bridge may not be accounted for.

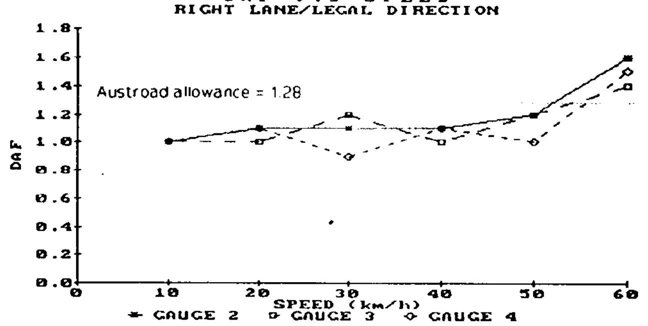
In addition to this, it is also possible that the variation of the DAF for the various runs could be explained by considering the effect of the vehicle body mode vibration excited as the vehicle approaches the bridge and the effect of the phase angle of excitation. These two factors could also be important in determining dynamic amplifications - but are outside the scope of the present investigation.

5.3. Effect of vehicle speed on DAF

Figs. 13a and 13b show the variation of DAF with speed of vehicle as it travelled along the left and right lanes respectively. These Figures indicate that the relationship between DAF and speed is not necessarily an increasing function. It seems to vary up and down. However the general trend is that a larger DAF is experienced as the vehicle speed increased. This is probably due to the vehicle excitation (usually, the body mode vibration) becoming more significant at higher speeds and leading to larger dynamic response. Since the static response is not influenced by vehicle excitation, the ratio of the dynamic response to the static response (DAF) is consequently larger. These Figures also indicate that the variation in DAF with speed is somewhat more pronounced when the vehicle travelled in the left lane. This is presumably because an asymmetric load would have loaded the bridge when the vehicle travelled along the left lane and would have excited higher modes of vibration, as explained earlier.



(a)



(b)

Fig. 13 Variation of DAF with speed: (a) Vehicle on left lane; (b) Vehicle on right lane

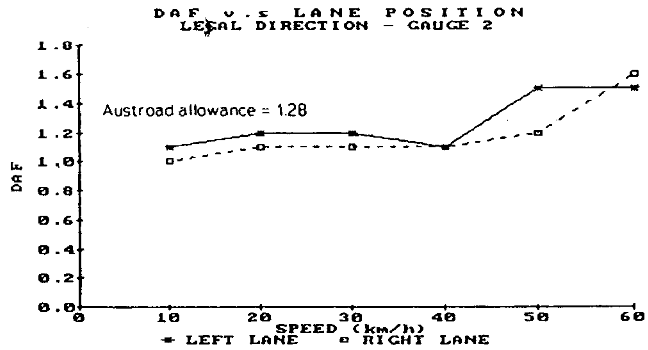


Fig. 14 Variation of DAF at location of strain gauge # 2

5.4. Effect of lateral position on DAF

Fig. 14 shows the variation of DAF with speed at gauge 2, which is located on the bridge centre line, when the vehicle travelled along the left and right lanes. It can be seen that higher values of DAF were obtained at all speeds when the vehicle travelled along the left lane. This could probably be explained as follows. When the vehicle travels along the left lane, the bridge is subjected to an asymmetric forcing function which has excited a higher mode, say the second mode (with f_2 = 6.53 Hz) of the bridge, while it excited the fundamental mode (with f_1 = 5.73 Hz) when it travelled along the right lane, which is very close to the bridge centre line. This higher mode which is lateral/torsional in nature has given rise to a higher DAF for the left lane when compared with the right lane.

5.5. Frequencies of modes excited by traffic

Table 3 shows the frequencies of the first four modes of vibration excited by traffic, while Table 5 shows the first five natural frequencies of vibration of the bridge obtained from a grillage analysis. The first mode excited by the traffic has a frequency of 5.85 Hz and compares well with the calculated first frequency of 5.73 Hz based on the dynamic modulus of elasticity and frictionless supports. The calculated frequency increased when the stiffness of the support was accounted for and approached 5.85 Hz, indicating that the first mode was excited by the vehicles.

Some of the higher modes, which have lateral and/or torsional components, do not correlate as well and this could be due to the restricted degrees of freedom in the two dimensional analytical (grillage) model. However, as evident from power spectrum analysis in Fig. 10, the energies associated with these modes were generally less than those corresponding to the fundamental mode. Consider the calculated second natural frequency of 6.53 Hz. The power spectrum suggests the next excited mode to have an average frequency of 7.20 Hz. Even if the support stiffness is accounted for, the calculated second natural frequency does not correlate very well with the second frequency obtained from the power spectrum analysis. This is probably because the grillage model lumps the stiffness of the bridge onto one plane and does not consider the out of plane stiffnesses of the girders and safety barriers. If these are included, the model will have a higher overall stiffness and will give a higher value for the calculated second natural frequency. The third calculated frequency is 8.39 Hz and the observed frequency is 8.6 Hz. This suggests that the third mode correlated quite well and this asymmetric mode was active during the response. Finally the fourth calculated frequency of 8.74 Hz and the observed frequency of 9.3 Hz, point out that the 4th natural mode might have been absent during the testing. However, the higher modes, associated with less energy, are less significant in the dynamic response of the bridge.

6. Implications of results

6.1. Can the DAF be less than unity?

When the test vehicle travelled in the left lane, at the low speed of 10 km/hr in the first test series, a DAF as low as 0.7 was obtained. It is suggested that, even if more sophisticated acquisition software had been used, this DAF would not have peaked above 1.0. This raises the question: can the DAF be less than unity? There might not be a simple answer to this question due to the complex nature of bridge-vehicle interaction, especially in an asymmetric bridge. There could be a number of reasons for this and the following discussion might be worthy of consideration.

A simple explanation in relating the DAF to the ratio of vehicle frequency/bridge frequency may not be directly applicable to the complicated vehicle - bridge response problem. It is therefore interesting and reasonable to look at other possibilities as well. The DAFs evaluated from several bridge tests tend not to exceed 1.5, even when resonance has occurred. This is because the duration of the forcing function is limited to the time that it takes the vehicle to traverse the bridge and, therefore, bridges only undergo quasi-resonance. It is therefore possible for a relatively short-span bridge to be subjected to vehicle induced vibration, but still have a DAF less than 1.0 because the duration of excitation is so brief and is out of phase with previously induced vibration. This could be a possible reason how bridge superstructures can actually have DAFs less than 1.0.

It is also possible that in this continuous bridge, inertial effects can cause values of DAF < 1. Moreover, this particular location where DAF was less than 1 was located near channel 4 when the vehicle travelled along the left lane and was outside the zone of influence as defined by Cantieni (1983). This was also the case with other locations where abnormal values of DAF were recorded.

In addition to the above reasoning, the complexity of bridge-vehicle interaction phenomenon should also be considered. This bridge response (and hence the DAF) depends on the vehicle vibration which in turn depends on the vehicles suspension and tyres and its speed and the condition of the approach surface. The vehicle body mode vibration, if excited, can cause reduced axial loads and result in DAF values < 1. It is also possible that the elastomeric bearings could have influenced the dynamic response. One of the above explanations or a combination of some of them could have resulted in values of DAF < 1.

6.2. Consequences of plan asymmetry and loading

Experiments conducted on two occasions revealed a distinct trend for the DAF to be significantly greater when the test vehicle travelled in the left lane of the bridge. There are at least two plausible explanations for this phenomenon. The first revolves around the longitudinal asymmetry of the structure. When the vehicle travels in the right lane, the order of span lengths traversed is 22 m, 24 m and 12 m. The order is reversed when it travels in the left lane: 12 m, 24 m and 22 m. Moreover, when on the right lane, the vehicle is close (0.75 m) to the centre line of the bridge, due to the footpath and therefore excites the fundamental mode. But when it is on the left lane, the loading is more asymmetric and excites a higher mode and a consequent larger DAF. This suggests that, if a bridge is asymmetric in plan, then the DAF may be different for each lane, depending on the degree of asymmetry.

6.3. Modulus of elasticity

It is well known that the modulus of elasticity of concrete depends on the rate of loading and it is appropriate to use this in dynamic analysis. Theoretical verification of the dynamic modulus of elasticity was not possible from the bridge testing carried out herein. Nevertheless, it was verified empirically that using the dynamic modulus of elasticity for concrete yielded a slightly more precise estimate of the natural frequencies of vibration.

6.4. Bearing stiffness

Using a finite element software package, together with the dynamic modulus of elasticity and estimated values of the rotational support stiffness, the fundamental frequency of the superstructure with elastomeric bearings, was evaluated. The structure was analysed for different values of the rotational support stiffnesses, while the compression stiffness was kept constant at a value of 475 kN/mm, which was supplied by the bearing manufacturer. From this investigation, the interpolated rotational support stiffness was 110 MN/rad, which corresponded to the fundamental frequency of 5.85 Hz.

7. Conclusions

The following conclusions may be established from the results of this investigation.

(1) An increase in vehicle speed usually resulted in an increase in the DAF, for this particular bridge and these phenomena were slightly more pronounced when the vehicle was in the left lane. In addition to vehicle speed, time of excitation and the condition of the approach surface also influence the DAF. As the vehicle speed increased and the time of excitation decreased, the DAF increased to a limit dependent on the maximum attainable vehicle speed for the bridge. However, bridge-vehicle interaction is complex and the DAFs are also dependent on the vehicle vibration mode excited.

(2) It was discovered that the longitudinal asymmetry of a continuous bridge has little effect on the centre span DAF and that the direction of travel on a span did not influence the results significantly.

(3) The most important observation was the significance of higher modes of bridge excitation. It was found that when the vehicle travelled in the left lane, it excited the higher modes of vibration which are lateral/torsional in nature. These higher modes resulted in higher DAFs compared with the DAFs when the vehicle travelled in the right lane where the fundamental mode was excited (symmetrical forcing function). Asymmetry of the bridge can therefore cause higher modes to be excited and result in higher dynamic amplifications than those provided in present bridge design codes.

(4) It has been observed that the DAFs are different for each lane in a bridge with an

asymmetrical plan. Moreover, DAF depends on the span. Hence, DAF in a continuous bridge is a local effect and as such must be accounted for in this manner. It is not possible to have a single DAF for the entire bridge unless a very conservative design is provided.

(5) As noted in the introduction, some bridge design codes require that the dynamic load factor be calculated using a first flexural frequency relationship. Many bridge designers have objected that the dynamic load factors are too high and, therefore, the method does not warrant the extra work involved in estimating the fundamental frequency of a proposed superstructure. With the recorded DAF of 1.6, the Ontario Highway Bridge Design Code (1983) and the AUSTROADS Bridge Design Code (1992), would underestimate the design dynamic load factor by approximately 18%. In this context, there are two points worthy of consideration: (i) The underestimation in dynamic load factor of 18% may seem large, but this design factor will be applied to a static load representing a number of design vehicles, equal to the number of lanes. It is unlikely that the bridge will be subjected to such a number of loads simultaneously and therefore, the design dynamic load factor may be acceptable; (ii) Even if the design dynamic load factor is conservative, it is based on the assumption that the first flexural mode is the dominant mode excited by vehicular traffic. As a result, dynamic amplifications in stresses due to excitation of lateral and/or torsional modes may not be adequately provided for in the design. This should be of interest and concern to bridge designers.

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