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Transverse variation of dynamic effects on beam-and-slab medium span bridges

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ABSTRACT: The common approach used to determine the bridge traffic load consists of applying a dynamic amplification factor to the most critical static traffic case. This dynamic amplification factor ignores specific dynamic characteristics of bridge and traffic load. The transverse distribution of dynamic amplification across the bridge section also lacks consideration. In this paper, both issues are discussed using simulations of the passage of heavy traffic over an experimentally validated beam-and-slab bridge model. Strain variations across the bridge have been calculated for a number of scenarios involving different combinations of heavy trucks with varying velocities and directions. Results show a significant difference in transverse distribution of load effects and how certain longitudinal beams are more prone to dynamic excitation.

1 INTRODUCTION

Correct evaluation of the behaviour of highway bridges under heavy loading is extremely important both in the enhancing of design techniques, and also in the assessment of existing infrastructure. It is widely accepted that shortfalls exist in design codes due to inadequate consideration of the dynamic interaction between the bridge structure and the heavy vehicles crossing it (Gonzalez et al 2003).

Design load factors are necessarily conservative, as load underestimation may lead to catastrophic failure of the structure over its projected life span. However needless expense must be avoided if possible whilst ensuring that extreme load events do not cause failure. Cai et al (1998) have shown that, in many cases, the measured live load stresses from field load tests are much lower than those predicted by analytical methods, particularly in the case of prestressed concrete bridges. It has also been shown that measured load distribution factors are usually less than those predicted by design codes.

A number of studies on the dynamic loading of beam and slab (girder) bridges have been carried out previously (Huang 1993). However most of these previous studies have been limited to planar beam or grillage models. This paper uses a Finite Element Modelling (FEM) approach to obtain a more realistic bridge response.

Critical traffic loads are simulated using 3-dimensional finite element models. For the bridge under consideration these critical cases will comprise of the meeting of two heavy trucks on the bridge. The dynamic amplification due to the critical loading events derived from a Weigh-In-Motion (WIM) site is also examined; with the aim of showing that current design values may be overly conservative.

2 DYNAMIC INTERACTION

Chan and O’Connor (1990) define dynamic amplification as being “an increase in the design traffic load resulting from the interaction of moving vehicles and the bridge structure and is described in terms of the static equivalent of the dynamic and vibratory effects”. The Dynamic Amplification Factor (DAF) is given by the following equation:

$$DAF = \frac{\varepsilon_{Dyn}}{\varepsilon_{Stat}} \tag{1}$$

where ε_{Dyn} is the maximum dynamic load effect, and ε_{Stat} is the maximum static load effect. For the purposes of this study the load effect under consideration will be strain.

Among the factors which affect the level of dynamic excitation in a structure being loaded are bridge factors such as road profile, presence of bumps or potholes, support conditions, bridge damping and natural modes of vibration, etc. There also exist a number of variable dynamic properties associated with the crossing trucks, such as tyre stiffness, tyre damping, suspension stiffness, suspension damping, and truck velocity etc., which can greatly affect dynamic excitation. In this study bridge and truck dynamic finite element models are developed using the Msc/Nastran software (The MacNeal Schwendler Corporation 1997). Dynamic interaction between the truck models and the bridge model is performed using a Lagrange Multiplier technique developed by Gonzalez (2001).

3 BRIDGE MODEL

The bridge used for this study is the Mura River Bridge, Slovenia. The bridge is 32m long and has two lanes of bi-directional traffic flow. The bridge is of beam and slab construction, is simply supported and forms part of a larger structure. Five concrete longitudinal beams support a concrete slab, with a layer of asphalt acting as the road surface. Five concrete diaphragm beams are also present, in the transverse direction. The bridge has been previously modeled and experimentally validated by Brady (2004). Fig.1 shows the 1st mode shape of the bridge model.

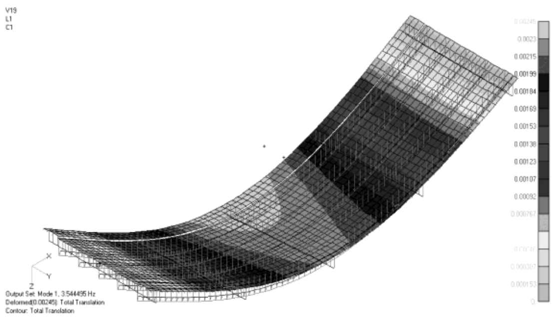

Figure 1. 1st mode shape of Bridge (3.5Hz)

The road surface profile is modeled as a random process described by a power spectral density function (Yang & Lin 1995). The classification system is based on values from the International Standards Organisation (ISO) (Wong 1993). In this case the bridge is taken to have a ‘good’ road surface and a minimum approach length of 100m is used for all traffic events.

Each truck may be passed down either side of the bridge. Trucks are run on the right-hand side of the road, based on continental European laws. A schematic layout of the bridge is shown in Fig.2. For direction 1 (D1 in Fig. 2) the trucks travel along a path 840mm from the centre-line of the bridge, between beam nos. 1 and 3.

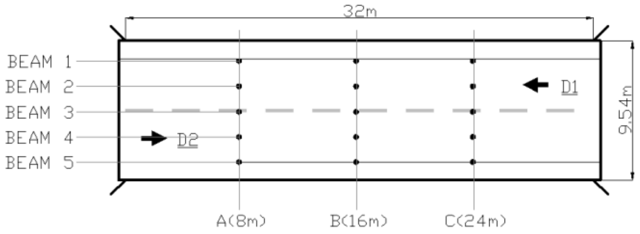

Figure 2. Schematic of bridge showing sensor location

For direction 2 the trucks travel along a path 840mm from the bridge centre-line, between beam nos. 3 and 5.

Strain is measured at five different transverse positions, representing each of the five longitudinal beams and for 3 different longitudinal positions; mid-span and both quarter spans. Table 1 below shows the location of the centerline of each beam relative to the bridge centerline. It is noted that the beam layout is not symmetrical about the centerline of the bridge.

Table 1. Distance from centerline of bridge to centerline of each longitudinal beam (metres)

Beam No.	1	2	3	4	5
Distance (m)	-3.349	-1.614	-0.137	1.862	3.579

4 TRUCK MODELS

The finite element truck models were modeled using rigid bodies supported by suspension and tyre systems. The trailer and tractor masses in the trucks are modeled as point loads distributed throughout the frame by rigid elements. The suspensions and tyres were modeled as spring dashpot systems, using typical stiffness and damping values (Kirkegaard 1997).

For single vehicle events a number of scenarios involving either a three-axle truck or a five-axle truck were simulated. Fig.3 shows the 5-axle truck model used in the simulations.

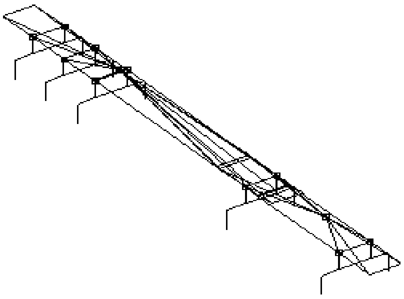

Figure 3. Five-axle truck model

For the single five-axle truck crossings a GVW of 62068kg is chosen, as based on WIM data this GVW value represents an extremely overloaded vehicle. Dimensions for the 3-axle truck used in the single vehicle loading events are based on the truck previously used to calibrate the bridge model (Brady 2004). The truck has a Gross Vehicle Weight (GVW) of 24484kg, typical of a fully loaded 3-axle truck. Table 2 contains details regarding the geometry and load distribution for the two trucks used in the single vehicle simulations. In table 2 WT represents the axle load for the 3rd axle in the 3-axle truck, and the total tridem (axles 3,4 and 5) load for the 5-axle truck.

Table 2. Axle-spacings and individual wheel loads for trucks used in single vehicle simulations.

Truck	Axle Spacings (m)				Axle Loads (kg)		
Type	A-S1	A-S2	A-S3	A-S4	W1	W2	WT
5-axle	3.6	5.8	1.2	1.2	7948	13360	40720
3-axle	3.22	1.37	-	-	6204	11112	7142

For the multiple vehicle events analysed in section 5.2 the axle-spacings and axle weights for the trucks vary from case to case, as do vehicle velocities and approach lengths.

5 RESULTS

5.1 Single Vehicle Events

For both the 3-axle and the 5-axle crossings the bridge response for three different truck velocities was analysed. These velocities were chosen as 60 km/h, 80km/h and 100kph. The bridge response at midspan due to a 5-axle truck crossing in direction 1 at 80km/h is shown in Fig. 4.

The variation in strain with the location of the truck front axle on the bridge can be seen in Fig.4. Maximum strain takes place when the front axle is located at close to 21 m from the start of the bridge.

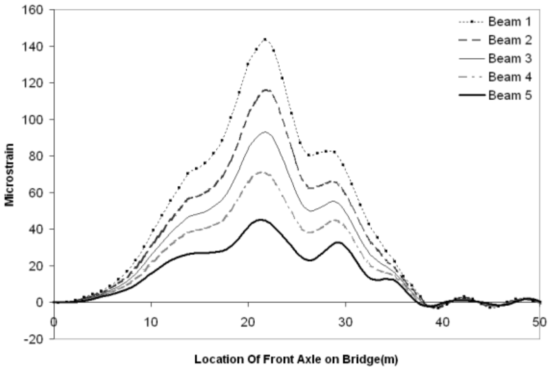

Figure 4. Dynamic response of bridge midspan to 5-axle truck loading (v=80km/h)

Fig.5 shows the variation of maximum bending strain across the bridge section for each beam and three longitudinal locations (8, 16 and 24 m from the bridge start). It can be seen that the edge beam (at about –3.5 m from the bridge centerline) under the lane where the traffic load runs over will experience greatest strain results. The maximum response (143.6 micro strains) will occur at the mid-span longitudinal location. For this midspan location, maximum strain varies transversely across the bridge, from 143.6 microstrain at beam 1 to 45.25 microstrain at beam 5. It can be seen that for a given longitudinal position, an approximately linear relationship exists between maximum bending strain and transverse position on bridge. The results are similar to those noted by Huang et al. (1993).

The trends encountered for simulations at different velocities and in direction 2, for both the three-axle and the five-axle truck are similar to those of Fig.5. The magnitude of the results will however vary, depending on, amongst other factors, vehicle weight.

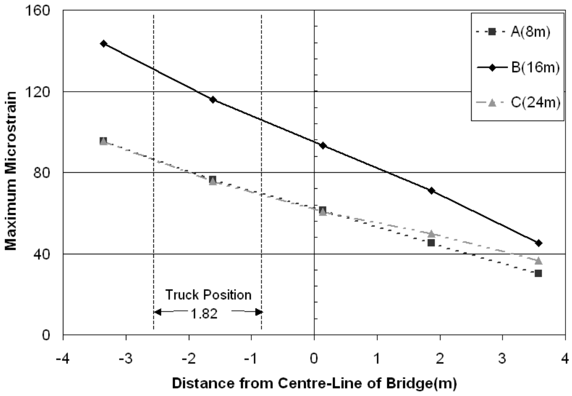

Figure 5. Variation of strain with transverse location for 5-axle truck loading (v=80km/h)

Fig.6 illustrates the variation of maximum static strain across the bridge section for the five-axle truck and three different velocities. For a given velocity, maximum strain varies approximately linearly across the bridge section.

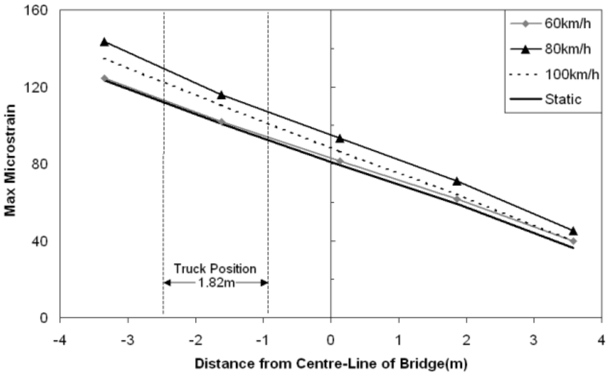

Figure 6. Max. microstrain Vs transverse position for five-axle truck traveling at different velocities.

Interestingly the intermediate velocity of 80km/h results in the largest values for maximum microstrain. It has been previously observed that different trucks have critical velocities at which the effect of dynamics is greatest depending on the dynamic characteristics of the specific truck (Brady 2004).

Figs.7 and 8 show the variation in DAF transversely for both three-axle and five-axle trucks traveling in direction1 at different velocities (60, 80 and 100km/h). The DAF for the range of velocities is calculated using Eqn.1 The approximate linearity between maximum strain and transverse position is again evident for the dynamic case, and varies from a maximum at the edge beam of the side on which the load is applied (beam 1) to a minimum on the extreme of the opposite side (beam 5).

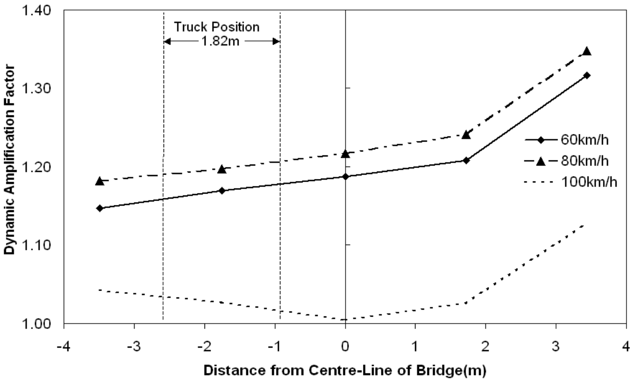

Figure 7. DAF Vs Transverse position for three-axle truck traveling at different velocities

It can be seen from Figs.7 and 8 that the effect of dynamics will be greater for the three-axle truck than for the five-axle truck. This is primarily as a result of greater GVW of the five-axle truck, which limits the excitation that can occur. The intermediate velocity of 80km/h results in greatest dynamic amplification for both trucks.

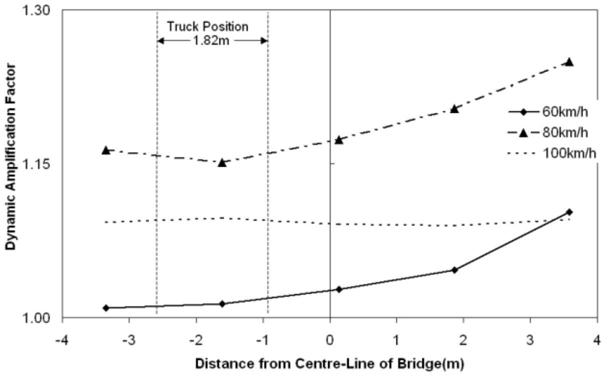

Figure 8. DAF Vs transverse position for five-axle truck traveling at different velocities

It is also evident from Fig.7 & 8 that the locations that experience greatest strain will have a low dynamic component and vice versa. In other words, the edge beam on the right-hand side in Fig.8, has the lowest maximum static strain in Fig. 6, returns the peak DAF values of 1.35 for the three-axle truck and 1.25 for the five-axle truck.

5.2 Multiple Vehicle Events

For the purposes of this study a number of meeting events were analysed. It is known that for a span length of 32m the critical load event is that of two five-axle trucks meeting at or near the mid-span location on the bridge.

The worst five meeting events for the bridge considered are obtained using Monte Carlo traffic simulations based on Weigh in Motion measurements collected on the A1 near Ressons (France). These worst cases are defined based on the maximum static bending moment on a one-dimensional beam model for the bridge lifetime. The traffic model defines the trucks axle spacings, axle-weights, and the meeting location on the bridge, for each of the meeting scenarios.

Truck velocities are randomly generated based on statistical distributions and approach length modified accordingly to ensure the correct meeting location is achieved. The trucks travel the same paths as outlined for the single vehicle events.

Table 3 contains some general information regarding the five-critical load cases being examined, with case 1 being the worst and case 5 being the 5th worst.

Table 3. General information regarding the trucks used in the critical meeting scenarios

	Truck 1			Truck 2		
CASE	GVW kN	L m	Vel km/h	GVW kN	L m	Vel km/h
1	506.5	11.5	89.8	541.9	12	79.1
2	537.8	12	85.4	435.4	11.9	82.2
3	525.7	11.1	84.6	411	10.3	82.1
4	439.9	9.5	84.0	432	12.4	90.6
5	410	11.5	87.1	491	11.8	79.8

It must be noted that the axle-spacings and axle weights vary for each truck, as does the exact position on the bridge at which the two trucks first meet.

Fig.9 shows a typical strain-time response for truck-meeting event (Case 1), where the maximum strain is experienced when both trucks are passing the mid-span of the bridge.

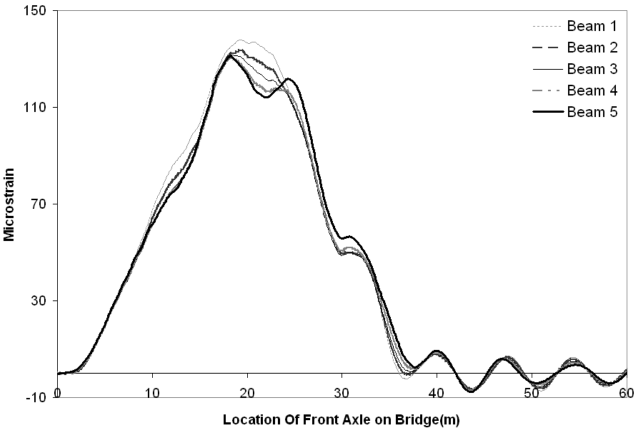

Figure 9. Dynamic response of bridge to 2xfive-axle truck meeting event

Fig.10 below shows the maximum strain experienced by each of the five longitudinal beams, for each of the five critical loading cases.

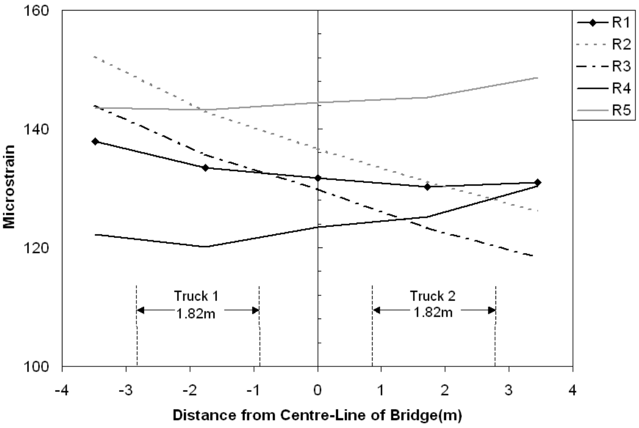

Figure 10. Transverse variation in dynamic response for 5 critical loading cases.

It is apparent that case 1 is no longer the most critical case, as the presence of dynamics and three-dimensional effects cause case 2 and case 5 to produce higher strain measurements at edge beams. Cases 2 and 3 cause high strain in the edge beam on the side of truck 1, as truck 1 is significantly heavier than truck 2 for each of these two cases.

Fig.11 shows the significance of the dynamic component in each of the meeting scenarios. Values for DAF of up to 1.17 occur for case 5, although the majority of the DAF values are located within the region between 1.0 and 1.1. This is as expected for a meeting event comprising of this kind.

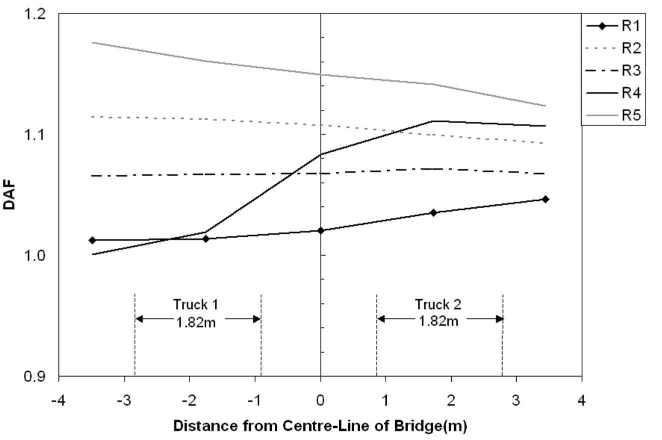

Figure 11. Transverse variation in dynamic amplification factor for 5 critical loading cases.

The effect of dynamics is approximately constant for each of the load cases, apart from case 4 where DAF values close to 1.1 are recorded at the beams on the side of truck 2, while DAF values close to 1.0 are recorded on the side of truck 1.

Fig.12 shows the variation of maximum dynamic strain and maximum static strain experienced by the bridge for each of the 5 cases.

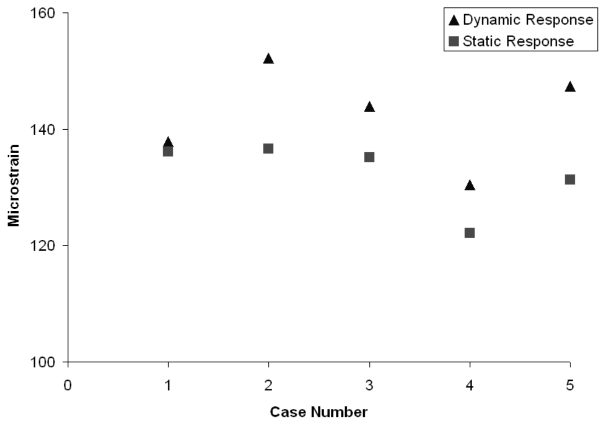

Figure 12. Comparison between dynamic and static response for 5 critical load events.

From Fig 12 it can be seen that case 2 will be the worst static case, and also the worst dynamic case. Case 1 will be the second worst static case, but when dynamics are incorporated case 5 becomes more critical. In other words, the ranking of the critical cases changes when these cases are analysed using the finite element approach

6 CONCLUSIONS

It has been shown that for the bridge considered single vehicle events result in the load effect being distributed approximately linearly across beam and slab bridges, even when the presence of dynamics is considered. The maximum load effect will be experienced at the extreme edge of the side of the bridge on which load is applied, with the minimum being experienced on the opposite side of the bridge. Also, it has been noted that the effect of dynamics is greatest at locations of low strain, with values of up to 1.35 times the static effect being recorded on edge beams. Although the study was restricted to just three different velocities the effects from the 80km/h analysis was greatest, which would imply that this velocity is closer to the critical velocities for both the three-axle and the five-axle trucks.

It has also been shown that for critical meeting events the importance of dynamics must be considered. While not recording a result in the region of the design value of 1.3 readings in edge beams reached values of up to 1.17 times the static equivalent. It is also noted that the worst static load case may not necessarily correspond to the maximum total (static + dynamic) load case. Complex analysis of critical loading events is thus of value in determining an adequate design load value.

7 ACKNOWLEDGEMENTS

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