MOVING DYNAMIC LOADS CAUSED BY BRIDGE DECK JOINT UNEVENESS: A CASE STUDY

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ABSTRACT

It is well known that uneveness is one of the main causes for moving variable loads generated in vehicles. Such uneveness can be due to inadequate construction quality, deterioration of the pavement surface due to traffic loading and environmental influences or other isolated spots on the pavement surface.

Bridge deck joints are used to ensure that bridges can expand and contract as designed without causing failure due to rigid connections to a rigid pavement. It is assumed that these joints should not become a reason for generation of variable loads in vehicles due to uneveness with the adjacent bridge and pavement.

In a recent investigation it was found that areas of isolated rutting was identified at certain constant distances away from a major bridge on a highway in South Africa. On deeper investigation, it was found that the bridge deck joint was protruding above the adjacent road and bridge surface by approximately 10 mm. Closer inspection showed that definite moving variable loads were generated by the heavy vehicles that passed over the joint. The areas where the oscilation of the unsprung mass caused the loads to be at its maximum, co-incided with the areas of rut failure observed on the road.

Subsequently, the case was modeled using a vehicle dynamics simulation package to investigate whether such behaviour could be predicted. This paper focusses on the output from these simulations, and provide general guidelines regarding maximum uneveness from bridge deck joints for typical South African heavy vehicles, in order to minimise the generation of moving variable loads.

INTRODUCTION

It is well known that unevenness is on of the main causes for moving variable loads generated in vehicles (Cebon, 1999). Such unevenness can be due to inadequate construction quality, deterioration of the pavement surface due to traffic loading and environmental influences or other isolated spots on the pavement surface.

Bridge deck joints are used to ensure that bridges can expand and contract as designed without causing failure due to rigid connections to a rigid pavement. It is assumed that these joints should not become a reason for generation of variable loads in vehicles due to unevenness between the adjacent bridge and the pavement.

During investigations of bridge deck joints on the N3 between Pietermaritzburg and Durban (Umthlatuzana South Bridge - Kwa-Zulu Natal, South Africa), it was observed that heavy vehicles that drove over the bridge deck joint on the eastern (downhill) side of the viaduct generated excessive (compared to the road before and after this position) moving dynamic loads when travelling over the joint. On closer examination, it was observed that there were distinct ruts that formed downhill of the bridge deck joint.

In this paper, the generation of these moving variable loads is investigated using a simple vehicle-analysis program, and the predicted locations of maximum loads compared to those observed on the road. The practical value of this is that it indicates the ability to predict such deterioration on a road using relatively simple software (TFP), and the effect of discrete uneveness on a pavement's deterioration due to excessive moving dynamic loads generated. Finally, it is indicative of potential deterioration of a road due to unevenness and the lack of maintenance of such unevenness.

BACKGROUND

The initial observation of unusual deterioration of the pavement surfacing was made on the slow lane of the N3 South just after the Umthlatuzana South Bridge. In Figure 1 a view of the bridge, the bridge deck joint and the positions of the discrete ruts that developed are shown.
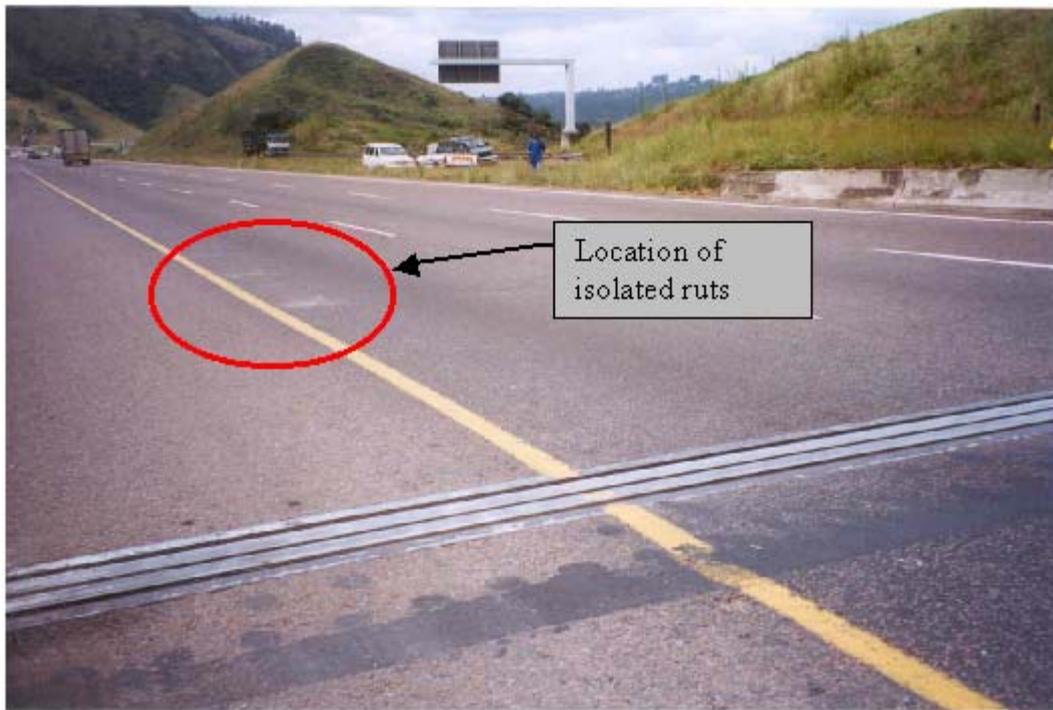
Figure 1. General view of pavement beyond Umthlatuzana South Bridge in southern direction showing location of bridge deck joint and location of discrete ruts.

During the investigation the distance between the bridge deck joint and the discrete ruts were measured. It was observed that the distances between the bridge deck joint and the first rut was approximately 8 m, while the distance between the remaining 3 ruts were approximately 2,5 m in each case.

The speeds at which heavy vehicles travel on the section of road was obtained from a Weigh-in-Motion site (Key Ridge 814) as approximately 50 km/h. As the road is sloping in the direction of travel at the location of the bridge, and it was observed that the heavy vehicles are using their brakes as they travel over the bridge, it was decided to use a speed of 40 km/h for the analyses.
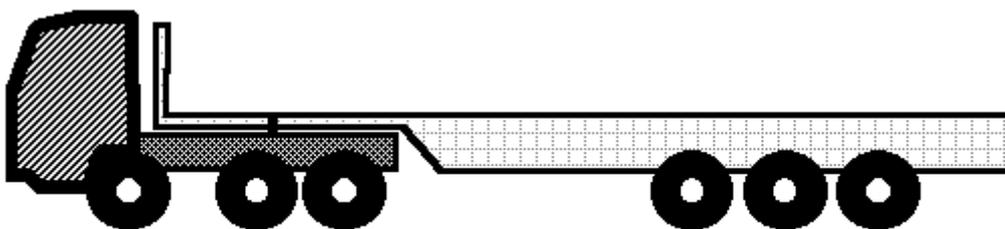
Figure 2. Schematic of vehicle used in TFP simulation.

Typically, the road is being used by articulated (truck-tractor and semi-trailer combination – 123 axles) and 1222 (interlink consisting of truck-tractor and two semi-trailers – 1222 axles) vehicles, and it was decided to model a typical 123 vehicle (Figure 2) as done previously in Steyn (2000). The main vehicle parameters used

for the analysis are shown in Table 1 while the road was modelled as a level section of road (400 m long) with a 10 mm unevenness simulating the bridge deck joint.

Table 1. Dimensional data for articulated vehicle used in vehicle simulation.

PARAMETER	VEHICLE IDENTIFICATION AND DATA
	Articulated (123)
Length [m]	16,41
Width [m]	2,49
Height [m]	3,32
Gross Combination Mass [kg]	48 900

The heavy vehicle data was simulated using the TFP software (TFP) to determine the moving variable load history of the vehicle. The Tyre Force Prediction Program (TFP) program is a typical program for the analysis of simple vehicle response simulations. TFP is an analytical model of a truck or truck-tractor and semi-trailer combination used to predict the forces that occur between the tyres of the vehicle and the road. In TFP, vehicle response is simulated at constant speed, and no turning or braking manoeuvres or roll effects can be simulated. It is only a two-dimensional vehicle model. TFP (and other similar programs) has been used for many studies available in the literature on vehicle-pavement interaction. Van Niekerk (1992) evaluated TFP for use in simulations of variable vehicle loading and it was found to provide satisfactory results. TFP results were compared with measured variable load data and it gave a reasonable indication of the overall trend of the dynamic axle masses. The output from this analysis is shown in Figures 3 and 4.

Each axle of the in suspension system in TFP is modelled as an independent 2-spring suspension connected to the unsprung mass of the axle, wheels, and wheel support hardware. Also included in the model are viscous damping and coulomb damping.
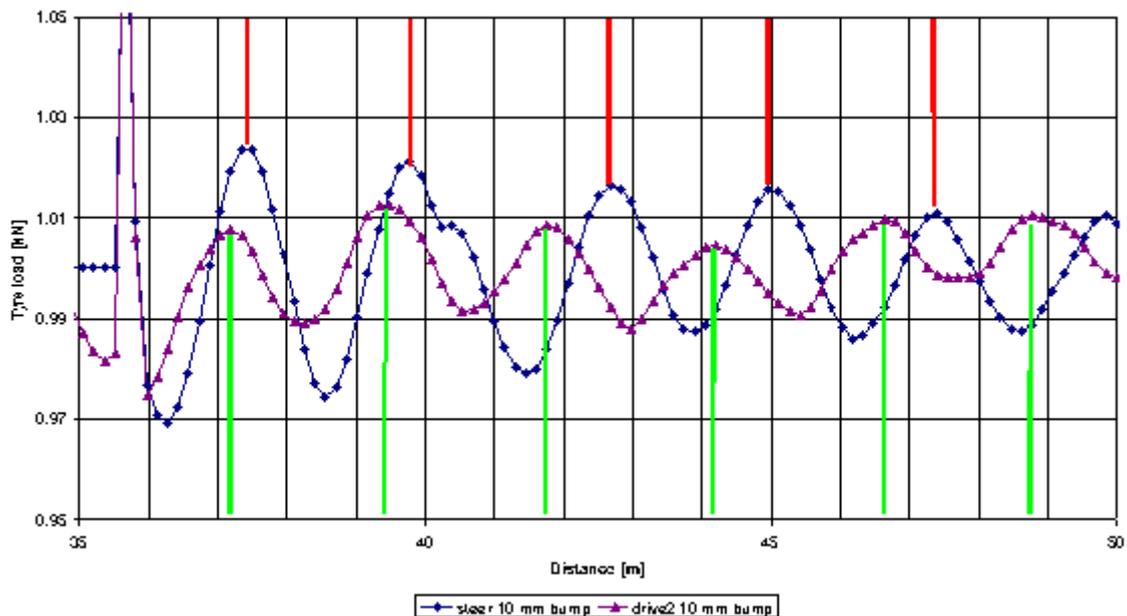
Figure 3. Variable loads generated in steer and drive axle of 123 vehicle after passing over a 10 mm bump at 40 km/h.
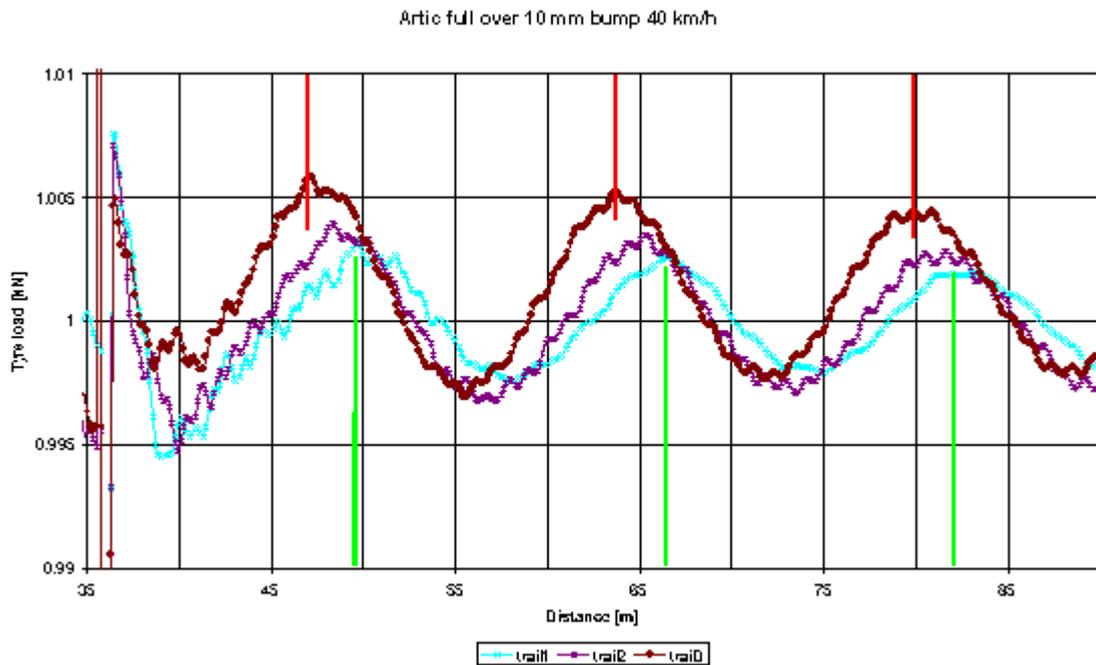
Figure 4. Variable load responses from trailer axles after passing over a 10 mm bump at a speed of 40 km/h.

DATA ANALYSIS

It should be well understood that the ruts that were observed on the road were caused by numerous vehicles running at different speeds and loads. All the analyses that were done are for a single typical truck, and it is to be expected that output from the analyses will not be directly correlated with the data observed on the road. The objective is to find similar trends in the simulation data and the observed data.

In Figure 3 the dynamic response from the steer and drive axles are shown. The interval between peak loads is between 2 m and 3 m. This relates comparatively well with the measured intervals between the ruts on the road of approximately 2,5 m. In Figure 4 the dynamic response from the trailer axles are shown. The interval between these peak loads is approximately 15 m. Due to the heavier loads carried on these axles the lower frequency may be expected. The 15 m peak loads may still correspond to some of the ruts observed on the road.

It is a well-published fact (Gillespie, 1992; Huhtala, 1995) that the dynamic loads imposed by heavy vehicles on roads occur at two distinct frequencies. These are the body bounce and axle hop frequencies. The body bounce frequency is associated with the sprung mass of the vehicle (approximately 90 per cent of the total mass of a loaded vehicle) while the axle hop is associated with the unsprung mass of the vehicle. The typical body bounce frequency is between 2 and 3 Hz, while the typical axle hop frequency is between 13 and 18 Hz. The body bounce frequency translates to peak loads at intervals of between 2.7 and 3.7 Hz for a vehicle running at 40 km/h (11.1 m/s). This would thus also coincide with the ruts observed on the road.

The output from the analysis indicates that the simulation software predicts the locations of the maximum moving dynamic loads for the steer and drive axles as being closely correlated with the locations of the discrete ruts on the road beyond the bridge deck joint. The fact that the initial maximum moving dynamic load is not seen on the road may be attributed to a concrete approach slab underneath the asphalt beyond the bridge deck joint.

A summary of the values of the Moving Variable Loads with the bump simulated at a height of 10 mm is provided in Table 2. All the data are shown as percentages of the average for the specific axle. It can be seen that the Moving Variable Loads varied between 95.43 per cent of the average up to 112.14 per cent of the average value during the simulation. This translates to a potential "overloaded" condition of up to 12 per cent on the first trail axle at selected locations along the road. In terms of typical pavement damage, the well known fourth power law can be used to illustrate that such a 12 per cent "overload" is equivalent to 1.57

Equivalent 80 kN axles – or a potential 57 per cent overload on the specific location where this peak load occurs. If – as is apparent from the pavement surface as shown in Figure 1) these peak loads occur at similar locations on the pavement – due to the dynamics of the vehicle and the unevenness caused by the bridge deck joint – then the potential rapid deterioration of the pavement at these locations should be expected.

The effect of differences in the bump height was also investigated. The analysis indicated that the load levels generated increased for higher bumps and decreased for lower bumps, but that the location of minima and maxima remained similar.

Table 2. Summary of percentages of average for Moving Variable Loads calculated for the 123 vehicle and a 10 mm bump.

Percentile	Steer axle	Drive axle 1	Drive axle 2	Trail axle 1	Trail axle 2	Trail axle 3
Minimum [%]	96.92%	97.79%	97.50%	95.53%	95.44%	95.43%
25th percentile [%]	99.84%	99.82%	99.83%	99.87%	99.84%	99.92%
50th percentile [%]	100.00%	100.00%	100.01%	100.00%	100.00%	100.11%
75th percentile [%]	100.16%	100.17%	100.14%	100.11%	100.15%	100.28%
Maximum [%]	108.77%	111.49%	110.21%	**112.14%**	111.43%	111.20%
Average [%]	100.00%	100.00%	100.00%	100.00%	100.00%	100.00%
Coefficient of Variation [%]	0.63%	0.49%	0.55%	0.51%	0.50%	3.64%

In order to investigate the effect of better installation of the bridge deck joint (improved quality control) the simulation was also performed using a bump height of 5 mm. The wavelengths of the Moving Variable Loads remained similar than for the 10 mm bump, but the values of the peak Moving Variable Loads decreased. The summary of the percentages of average of Moving Variable Loads for this simulation is provided in Table 3. It is interesting to note that the potential "overloaded" condition decreased to a maximum of 6.15 per cent. This in turn translates to an equivalent of 1.27 Equivalent 80 kN axles – or a potential 27 per cent overload on the specific location where this peak load occurs. This is 30 per cent less than in the first case – indicating a potential longer life for the pavement at the location where the maximum peak load occurs due to improved installation of the joint.

Table 3. Summary of percentages of average for Moving Variable Loads calculated for the 123 vehicle and a 5 mm bump.

Percentile	Steer axle	Drive axle 1	Drive axle 2	Trail axle 1	Trail axle 2	Trail axle 3
Minimum [%]	98.52%	99.34%	98.91%	98.70%	98.67%	98.89%
25th percentile [%]	99.85%	99.92%	99.89%	99.95%	99.95%	100.05%
50th percentile [%]	100.00%	99.99%	100.00%	100.00%	100.00%	100.10%
75th percentile [%]	100.15%	100.06%	100.09%	100.05%	100.05%	100.17%
Maximum [%]	104.36%	105.78%	105.08%	**106.15%**	105.85%	105.88%
Average [%]	100.00%	100.00%	100.00%	100.00%	100.00%	99.98%
Coefficient of Variation [%]	0.38%	0.27%	0.30%	0.27%	0.25%	3.62%

DISCUSSION AND EFFECTS

The case described is the first case where the author specifically observed the formation of ruts due to dynamic loads generated by unevenness caused by a bridge deck joint, measured the distances between these ruts and modelled a vehicle to determine whether these data sets corresponds.

It clearly illustrates the potential increased damage due to "overloaded" conditions generated at specific locations due to the existence of a bridge deck joint standing proud of the surrounding pavement surface. This is an indication of the practical benefit of a good quality control system and workmanship during installation of such joints.

It was shown that a decrease in height of the joint from 10 mm to 5 mm result in a potential 30 per cent decrease in the "overloaded" condition at the location of maximum peak load. This is potentially a major improvement in the life of the pavement surfacing, mainly caused by improved installation of the bridge deck joint.

A further practical use of this information is that it will indicate to road maintenance authorities that maintenance of the rut developed on the pavement is only solving half the problem. In the case illustrated, the main cause of these types of localised ruts is to be found a distance before the physical ruts themselves, i.e. in this case the unevenness of the bridge deck joint.

In the case study the analysis was done for vehicles traveling in the downhill direction (as the truck left the bridge onto the approach slab) as this is the location at which the specific phenomenon was observed. It is the opinion of the author that similar peak loads are being generated on the bridge deck should a similar discontinuity be present on the joint as the truck goes from the approach to the bridge deck. The resultant bridge deck reactions have, however, not been analysed in this paper.

It is recommended that further analyses of this phenomenon be performed with data from other bridges in South Africa.

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