ANALYSIS OF THE DYNAMIC BEHAVIOUR OF T-BEAM BRIDGE DECKS DUE TO HEAVYWEIGHT VEHICLES

David A.M. Jawad1 and Anis A.K. Mohamad-Ali2

1Lecturer, Dept. of Civil Engineering, University of Basrah, E-mail: david215jawad@yahoo.com
2Professor, Dept. of Civil Engineering, University of Basrah

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This study investigates the dynamic behaviour of concrete T-beam bridge decks due to heavyweight vehicles. The three-dimensional model of an actual T-beam bridge deck design is implemented within the context of the finite element method, through use of the ANSYS 5.4 computer code. The deck is modeled with 20-node brick elements. Axle loads and configurations which correspond to the “permit vehicle” loading model are adopted for the vehicle model. The case study is considered for static, free vibration, and forced vibration analysis. The dynamic loading for forced vibration analysis is a harmonically (sinusoidal) varying load with magnitude equal to 10% of the axle load and a forcing frequency equal to the first (fundamental) frequency of the bridge deck, thus simulating a case of resonance. Dynamic amplification factors are evaluated at certain locations on the bridge deck for vertical displacement, normal stress in the longitudinal direction, and shear stress. Numerical results show a general trend for higher values than those specified by the AASHTO design code. It is also concluded that the values of dynamic amplification factors are response dependent, which suggests the use of three different types of dynamic amplification factors for the analysis of bridge decks.

1. INTRODUCTION

T-beam bridge decks are one of the principal types of cast-in-place concrete decks. T-beam bridge decks consist of a concrete slab supported on, and integral with girders. They are especially economical for spans in the 15–25 m range. The girders are commonly spaced at 2.10–2.80 m and the stem thickness varies between 350–550 mm, set by the required horizontal spacing for the positive moment reinforcement. The depth-to-span ratio for this type of decks is generally 0.07 for simple spans and 0.065 for continuous spans.

Bridge structures have traditionally been designed with the primary objective of avoiding failure under static loads. The response of bridge structures to static loading can be determined quite satisfactorily by any of a number of classical analysis techniques. The response to dynamic loading due to moving vehicles, however, is not as easy to predict. Under current design practice, dynamic effects are taken into account by increasing the static live load by an impact fraction, I, which is a function only of span length. This results in attractive bridge designs that may satisfy safety and strength requirements but may have undesirable dynamic response in the form of large displacements and, consequently, increased stresses, which affect the long-term performance of the bridge. Stresses developed with heavy vehicles moving over a bridge deck may exceed those obtained by incrementing static live loads by the factors specified in bridge design codes. Accordingly, there is a need for a more
elaborate investigation into the dynamic behaviour of bridges due to moving vehicles. Despite its long history, the finite element method remains to be a predominant strategy for the analysis of engineering systems. The ANSYS finite element computer code provides a unique program interface for the analysis of engineering systems. The wide range of modeling and analysis capabilities available to the program makes it highly suitable for the static and dynamic (free and forced vibration) analysis of concrete bridges within the context of the finite element method.

The objective of this study is to analyse the dynamic behaviour of concrete T-beam bridge decks caused by heavy-weight vehicles. Dynamic amplification factors are evaluated for different types of response and compared with the values given by AASHTO specifications for highway bridges.

2. LITERATURE REVIEW

The dynamic behaviour of bridges due to vehicle loading has been the subject of considerable experimental and analytical research.

The first important report on the matter was published in 1931 by a special committee of the ASCE[1]. The recommendations of this committee, which were based on data obtained from a series of field tests, constitute the basis of American design specifications. In particular, it was recommended that the live load due to vehicles should be increased by an Impact Fraction (I) given by

\[ I = \frac{0.30}{125 + L} \]

in which L is the loaded length in feet, and the maximum value of I is 0.25. The American Association of State Highway Officials conceived and sponsored a major experimental investigation in 1962 to determine the dynamic effects of moving vehicles on short span highway bridges[2]. The standard specifications for highway bridges of AASHO[3] cite a variation on the impact fraction given by equation(1), whereby the maximum value of I should not exceed 0.30.

In 1981, the ASCE committee on loads and forces[4] confirmed current AASHTO practice concerning live load impact, except that the term impact be replaced wherever appropriate in current design specifications by the more descriptive term dynamic allowance for traffic loading.

The dynamic factor or dynamic amplification factor is generally defined as the ratio of maximum dynamic response to maximum static response. O’Connor and Pritchard[5] conducted dynamic field tests on a short span, steel and concrete highway bridge. The authors reported a wide scatter in the values of impact fractions I, with a maximum value of 1.32. Chan and O’Connor[6] describe further field studies on the bridge referred to above and reported values for the impact fraction I, consistent with the values obtained previously. In a companion paper, the same authors Chan and O’Connor[7] present a vehicle model in which each axle load includes a dynamic load component that varies sinusoidally at the first natural frequency of the bridge. The field data of the above paper was used to calibrate the magnitude of the dynamic component at 10% of the static axle load. Hwang and Nowak[8] presented a procedure to calculate statistical parameters for the dynamic loading of bridges. The authors used a Monte Carlo method to represent the random nature of traffic and studied the effects of the static parameters on dynamic loading.

Wang et al.[9] studied the dynamic behaviour of highway prestressed I-beam concrete bridges due to moving vehicles. The bridges were modeled using one-dimensional, two-node elements with six degrees of freedom at each node. The results indicated that the dynamic amplification factors for bending moment and deflection are higher than those for end shear. Wang and Huang[10] studied the dynamic and impact characteristics of continuous steel beam bridge decks and slant-legged rigid frame bridges. The former type was modeled as a grillage beam system, whereas the latter was modeled as a space bar system. The authors concluded that the dynamic amplification factors over interior supports for certain span ranges of steel beam bridge decks might exceed the maximum value given by AASHTO. They also reported a difference in values of the dynamic amplification factors for bending moment and axial force at most sections of the slant-legged rigid frame bridge. Fafard et al.[11] investigated the effect of dynamic loads on the dynamic amplification factors for an existing continuous concrete bridge using three-dimensional vehicle and finite element models. The bridge was modeled using a combination of 8-node shell elements and 3-node beam elements. Experimental testing of the bridge was also conducted. Comparison of the results showed good agreement at some sections and a large discrepancy at other sections; which the authors attributed to the variation in road roughness. The authors reported values of dynamic amplification factors, which are higher than the AASHTO value. They recommend a dynamic amplification value of 1.55 for the ultimate limit state. Barefoot et al.[12] outlined a procedure for developing finite element models to predict the static and dynamic response of steel beam bridges. The procedure uses ANSYS 5.0 and is validated through comparison with field data of a typical bridge. Challal and Shahawy[13] provided a state of the art review on dynamic testing procedures for bridges, with a special emphasis on experimental evaluation of dynamic amplification factors. Broquet et al.[14] described a parametric study, based on the simulation of bridge-vehicle interaction, to investigate the distribution of dynamic amplification factors throughout bridge deck slabs. The bridge structure was represented by a three-dimensional finite element model using quadrilateral shell and beam elements.
Results indicated that dynamic amplification factors do not vary significantly for different response effects or for different parts of a deck slab. Martin et al. developed a procedure for representing moving loads on a bridge model within the context of ANSYS. The relative influence of various design and load parameters was investigated using a finite element model of a simply supported beam with the equivalent properties of a longitudinal section for an actual bridge. The authors concluded that the most important factors affecting dynamic response are the basic flexibility of the structure and, more specifically, the relationship between the natural frequency of the structure and exciting frequency of the vehicle. Wang and Liu carried out a static and dynamic analysis for typical span ranges of steel beam bridge decks and prestressed I-beam bridge decks. The authors synthesized truck traffic data collected through weigh-in-motion measurements. The bridge decks were modeled as grillage beam systems, and the dynamic amplification factors for moment at midspan and end shear were analysed.

From the foregoing review of available literature, it can be seen that the impact formula specified by AASHTO is based on traffic loads in operation more than 40 years ago, and since then, the volume of traffic has increased considerably and the configuration of heavy vehicles has changed. Therefore, there is a need for a more rational approach. Several researchers report values of dynamic amplification factors, which exceed the AASHTO value. It is also noted that the dynamic amplification factors are affected by the type of response.

3. MATHEMATICAL MODELS

i- Bridge Model

The bridge deck behaviour in static and dynamic (free and forced vibration) analysis is governed by the following equations:

i- Static Analysis

\[ K u = f \] (2)

ii- Free Vibration Analysis

\[ (K - \omega^2 M) u = 0 \] (3)

iii- Forced Vibration Analysis

\[ M \ddot{u} + C \dot{u} + Ku = f(t) \] (4)

In equations (2) K is the stiffness matrix of the bridge deck, u is the displacement vector, and f is the load vector. The equations are solved for known forces f to give equilibrium displacements u. The method of solution employed is the frontal solution technique. In equations (3) M is the mass matrix of the bridge deck, the equations are solved to yield the mode shapes u and natural frequencies \( \omega \) of the bridge deck. The solution method is subspace iteration. In equations (4) C is the damping matrix of the bridge deck, \( \ddot{u}, \dot{u}, \) and u are the acceleration, velocity, and displacement vectors, and f(t) is the load vector due to vehicle loading. The equations are solved by transformation to the frequency domain using the complex response method.

ii- Vehicle Model

The vehicle model developed for analysis in this study is based on the permit vehicle loading which is a major model for the structural analysis of bridges traveled by heavyweight vehicles. The term permit vehicle is related to the requirement of heavyweight vehicles to obtain a permit to travel on highway bridges, since they have the capacity to damage or possibly collapse the bridges. The main classes of permit vehicle loadings are designated as P7, P9, P11, and P13.

It is well known that a moving load on a bridge produces a greater deflection and greater stresses than does the same load acting statically. Using the principle of virtual work, it has been established that the deflection of a simply supported beam, of span L, due to the passage of a single concentrated load P at a constant velocity v, figure(1), is given by the following expression:

\[
y = -\frac{2PL^3}{m\pi^2} \sum_{i=1}^{\infty} \frac{\sin(i\pi x/L)}{i^2 (i^2 \pi^2 a^2 - v^2 L^2)} \sin \frac{i\pi vt}{L} \\
+ \frac{2Pv L}{m\pi^2 a} \sum_{i=1}^{\infty} \frac{\sin(i\pi x/L)}{i^2 (i^2 \pi^2 a^2 - v^2 L^2)} \sin \frac{i^2 \pi^2 at}{L^2} \] (5)

where m, EI are the mass and flexural rigidity of the beam and so = EI/m. The foregoing expression is the same as that for a simply supported beam acted upon by a harmonically (sinusoidal) force with a forcing frequency of \( \frac{L}{L^2} \) applied at a distance x=vt. The first series in the solution represents forced vibrations, and the second series pertains to free vibrations of the beam. As the velocity v increases, a condition is reached where the value of one of the denominators in equation (5) becomes equal to zero. If it is assumed that:

\[ v^2 L^2 = a^2 \pi^2 \] (6)

denominators in the first terms of both series in equation (5) become equal to zero; and the sum of these two terms will be

\[
y = -\frac{2PL^3}{m\pi^2} \frac{\sin \frac{\pi x}{L}}{\sin(\pi vt / L) - (Lv / 2a) \sin(\pi^2 at / L^2)} \\
\] (7)

The expression has the indeterminate form 0/0 and is evaluated by applying L’Hopital’s rule

\[ \lim_{v \to 2\pi / L} \frac{PL}{m\pi^2} \frac{\sin \frac{\pi vt}{L} \sin \frac{\pi x}{L}}{\cos \frac{\pi vt}{L} - L\pi} \]

\[ = -\frac{PL}{m\pi^2} \frac{\sin \frac{\pi vt}{L} \sin \frac{\pi x}{L}}{\cos \frac{\pi vt}{L} - L\pi} \] (8)
Equation (8) has its maximum value when \( t = L/v \) and is then equal to

\[
\begin{align*}
    y_{\text{max}} &= -\frac{PL}{m\pi^2v^2} \left( \sin \frac{\pi vt}{L} - \frac{\pi vt}{L} \cos \frac{\pi vt}{L} \right)_{-L/4} \sin \frac{\pi x}{L} \\
    &= -\frac{PL}{EI\pi^2} \sin \frac{\pi x}{L}
\end{align*}
\]

(9)

The system is linear, hence for multiple axles superposition applies, and the value of response at any point along the bridge deck is equal to the summation of the responses at the same point caused by the loads taken separately.

iii- Dynamic Amplification Factors

The Dynamic Amplification Factor (DAF) (8,10,11,13) is given by

\[
    \text{DAF} = 1 + \text{DA}
\]

(10)

Where \( \text{DA} \) (Dynamic Amplification) is the ratio of maximum dynamic response to maximum static response, and both response values are of identical signs. The Standard Specifications for Highway Bridges of AASHTO\[18\] have made provision for the dynamic effects of vehicle loads on bridges by the inclusion of an impact fraction (I) defined by the equation (1), the live load response is multiplied by (I+1) to obtain the total response due to vehicle load. The maximum value of the impact fraction (I) suggested by AASHTO is 0.3, this corresponds to a simple span of 41.7 ft (12.71 m) and all shorter spans use the value of 0.3 for impact.

4. DESCRIPTION OF THE BRIDGE

The structure considered in this case study is a 25 m two-span, overhang T-beam bridge deck, and is designed by the service load design method according to the procedure outlined in Heins and Lawrie\[18\]. The main span has a length of 20.00 m and the cantilever span has a length of 5.00 m. The bridge deck width is 5.70 m. Figure(2) shows a plan view of the bridge deck. The slab thickness is 200 mm and is reinforced in the transverse direction with distribution steel added longitudinally. The slab is integral with three girders spaced at 2.15 m c/c. The girders dimensions are 400x1300 mm. The bridge deck is monolithic with the substructure at both supports. Figure(3) shows the cross-section of the bridge deck. The design stresses are: concrete strength \( f_c = 0.4f'_c = 9.0 \text{ MPa} \) and Grade 60 reinforcing bars \( f_s = 166 \text{ MPa} \).
5. FINITE ELEMENT MODEL

The T-beam bridge deck is modeled using the SOLID 95 three-dimensional brick element. The element has twenty nodes, and each node possesses three translational degrees of freedom corresponding to the x-, y-, and z- directions. The overall model comprises 572 elements. The total number of nodes included in the model is 3934 resulting in a system with 11082 degrees of freedom. The model is illustrated in Figure (4). The origin of axes is shown in figure, where the z- and x- axes are located in the plane of the bottom surface of the slab along the longitudinal and transverse directions, respectively. The y-axis is perpendicular to the deck and positive upwards. The three-dimensional discretisation of the model is achieved by the extrusion facility available in the ANSYS software, in which the cross-section of the bridge deck is meshed using two-dimensional elements, and the meshed area then used as a pattern and extruded along the length of the bridge model. The bridge deck model support conditions are specified to be constrained in translation along x-, y-, and z-directions at both supports. This is attributed to the monolithic construction of the bridge deck with the substructure. The support conditions are also shown in Figure 4 Finite Element Model of T-Beam Bridge Deck.

A convergence study to validate the adequacy of the adopted discretisation, is carried out by comparing the results obtained with twice the number of elements. The difference in results is insignificant, excluding the need for further refinement. The material properties used in the analysis are:

- Density of Concrete = 2380 kg/m³.
- Modulus of Elasticity of Concrete = 22.39x10³ MPa (Based on the ACI formula \( E_c = 4730 \sqrt{f_c} \) for normal-weight concrete).
- Poisson’s Ratio = 0.20
- Damping Ratio = 0.02. This value is based on the lower-bound value for damping in concrete bridges cited in (13) from a series of tests on 225 bridges in Europe. The value was adopted with the aim of maximizing the dynamic response of the bridge.

6. LOADING

The applied loading considered for the static analysis of this case consists of a series of concentrated loads which correspond in magnitude and spacing to the permit load P9, Figure(5). The concentrated loads are applied to the nodal points along the centreline of the deck, as illustrated in the finite element model of the bridge deck. The loading considered for the forced vibration analysis consists of concentrated loads which vary harmonically (sinusoidally) at a magnitude equal to 10% of the static load, within a forcing frequency range that includes the first natural (fundamental) frequency of the bridge deck. This is an upper-bound value and was calibrated by field studies where wheel loads were obtained from measured strains on a highway bridge. The concentrated loads for the forced vibration analysis have the same spacing and location Figure (6) as for the static analysis cited above.

![Figure 5 Schematic diagram for load magnitudes (kN) along centreline of T-beam bridge deck-static analysis](image-url)
7. RESULTS

i- Static Analysis

Results of the static analysis are presented in Figures (7) to (10).

Figure (7) shows the deflected shape pattern of the T-beam bridge deck as a result of the loads applied to the model, and Figure (8) shows contours of distribution of the vertical displacement throughout the bridge deck. The maximum vertical displacement is attained at the midspan region of the main span. The displacement at the midpoint of the main span (on the soffit of the central girder) is evaluated as 2.60 mm. Figure (9) shows contours of distribution of the normal stress in the (longitudinal) z-direction $\sigma_z$ throughout the T-beam deck. The normal stress at the midpoint of the main span (on the soffit of the central girder) is $+1.53 \text{ MPa}$. The normal stress at the top surface of the interior support is $+2.40 \text{ MPa}$.

Figure (10) shows the contours of distribution of the shear stresses in the xy-directions $\tau_{xy}$ throughout the T-beam deck. The shear stresses are distributed in an anti-symmetrical pattern with respect to the centreline of the deck, resulting in near-zero values at the centerline and a maximum value of $-1.00 \text{ MPa}$ at the outer face of the edge girder.
Figure 10 Distribution of shear stress throughout T-beam deck-static analysis.

**Free Vibration Analysis**

The natural frequencies and mode shapes are determined from the model. Table (1) gives the natural frequencies of the first six modes of vibration of the T-beam bridge deck. Figures (11) to (16) illustrate the corresponding mode shapes. The first, fourth, and sixth modes are identified as longitudinal bending modes. The second and fifth modes are torsional modes. The third mode is a transverse bending mode.

Table 1. Natural Frequencies of T-Beam Bridge Deck.

<table>
<thead>
<tr>
<th>Mode</th>
<th>Frequency (Hz)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 (fund.)</td>
<td>10.127</td>
</tr>
<tr>
<td>2</td>
<td>10.831</td>
</tr>
<tr>
<td>3</td>
<td>19.059</td>
</tr>
<tr>
<td>4</td>
<td>24.788</td>
</tr>
<tr>
<td>5</td>
<td>25.647</td>
</tr>
<tr>
<td>6</td>
<td>26.372</td>
</tr>
</tbody>
</table>

Figure 11 First mode; frequency(10.127 Hz).

Figure 12 Second mode; frequency(10.831 Hz).

Figure 13 Third mode; frequency(19.059 Hz).

Figure 14 Fourth mode; frequency(24.788 Hz).
iii- Forced Vibration Analysis

In this analysis a system of loads which vary harmonically (sinusoidally) at a magnitude equal to 10% of the static loads, is applied to the T-beam bridge deck model, within a definite range of frequencies that includes the first natural (fundamental) frequency of the bridge deck 10.13 Hz. The bridge deck’s response (displacement) is calculated for the specified frequency range. Peak response is then identified and stresses are reviewed at the corresponding peak frequency. The frequency range considered for this case is (7.0–13.0 Hz) and the solution is calculated at intervals of 0.2 Hz. Damping is included as a constant damping ratio and assigned the value of 0.02.

Figures(17) and (18) show plots of the vertical displacement at the midpoint of the main span (on the soffit of the central girder), within the given range of frequencies. In figure(17) the results are illustrated as amplitudes, whereas figure(18) depicts the real part of the solution; since the results are essentially complex numbers due to the inclusion of damping. From both figures it is evident that the maximum response occurs at a forcing frequency close to the first natural frequency of the T-beam bridge deck 10.13 Hz. This is characterized as a case of resonance and the value of the forcing frequency at which this occurs (the resonant frequency) is to be seen as 9.8 Hz. The slight discrepancy between the values of the forcing frequency and the natural frequency of the bridge deck is representative of the phase lag in the peak structure displacement behind the load, due to damping, wherein the maximum response occurs at a frequency ratio (forcing/natural) that is slightly less than unity.

In figures(19) to (24), displacements and stresses are reviewed along the bridge deck at the same locations considered in the static analysis, for the adopted forcing frequency 9.8 Hz. However, the maximum value for shear stresses $\tau_{xy}$ was obtained at a forcing frequency of 10.4 Hz which is closer to the second (torsional) mode of vibration of the bridge deck(10.83 Hz). Figures(19)and(20) depict plots of the variation of vertical displacement along the centreline; on the soffit of the central girder of the T-beam deck,
for the respective cases of static analysis, and forced vibration analysis. Figure (21) shows the deflected shape pattern of the T-beam bridge deck and figure (22) shows the contours of distribution of the vertical displacement throughout the bridge deck. The vertical displacement at the midpoint of the main span (on the soffit of the central girder) is 2.3 mm. Figure (23) shows the contours of distribution of the normal stress in the (longitudinal) z-direction $\sigma_z$ throughout the T-beam deck. The normal stress at the midpoint of the main span (on the soffit of the central girder) is $+1.70$ MPa. The normal stress at the top surface of the interior support is $+1.03$ MPa. Figure (24) shows the contours of distribution of the shear stress in the xy-directions $\tau_{xy}$ throughout the T-beam deck. The shear stress distribution is anti-symmetrical with respect to the centerline and attains a maximum value of $-0.16$ MPa at the exterior face of the edge girder.

Figure 19 Variation of vertical displacement (m) along centreline of T-beam deck-static analysis.

Figure 20 Variation of vertical displacement (m) along centreline of T-beam deck-forced vibration.

Figure 21 Deflected shape pattern of T-beam deck-forced vibration analysis; forcing frequency(9.8Hz).

Figure 22 Distribution of vertical displacement (m) throughout T-beam deck-forced vibration analysis; forcing frequency(9.8Hz).

Figure 23 Distribution of normal stress throughout T-beam deck-forced vibration analysis; forcing frequency(9.8Hz).
iv- Dynamic Amplification Factors
In this case study, the dynamic amplification factors for the T-beam bridge deck are calculated for the maximum response at resonance, of vertical displacement, normal stress in the (longitudinal) z-direction $\sigma_z$, and shear stress in the $xy$-directions $\tau_{xy}$. The dynamic amplification factor for vertical displacement, at resonance, is evaluated at the midpoint of the main span of the bridge deck. The dynamic amplification factors for normal stress in the (longitudinal) z-direction $\sigma_z$, at resonance, are evaluated at the midpoint of the main span and at the interior support of the bridge deck. The dynamic amplification factor for shear stress in the $xy$-directions $\tau_{xy}$, at a forcing frequency of 10.4Hz, is evaluated at the outer face of the edge girder. The values of the dynamic amplification factors for maximum response at the locations referred to above are given in Table(2).

An examination of these values reveals that the maximum value of the dynamic amplification factor for this case study 2.11 is notably higher than the value given by the AASHTO standard specifications, and is attained for normal stress at the midpoint of the main span of the T-beam bridge deck. The value of the dynamic amplification factor for vertical displacement at the midpoint of the main span 1.87 is also considerably high. The value of the dynamic amplification factors are evaluated at 1.43 for normal stress at the interior support and 1.16 for shear stress at the outer face of the edge girder.

<table>
<thead>
<tr>
<th>Response Type and Location</th>
<th>Maximum Static</th>
<th>Maximum Dynamic</th>
<th>DA</th>
<th>DF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Vertical Displacement of Midpoint of Main Span (mm)</td>
<td>-2.60</td>
<td>-2.27</td>
<td>0.87</td>
<td>1.87</td>
</tr>
<tr>
<td>Normal Stress $\sigma_z$ at Midpoint of Main Span (MPa)</td>
<td>+1.53</td>
<td>+1.70</td>
<td>1.11</td>
<td>2.11</td>
</tr>
<tr>
<td>Normal Stress $\sigma_z$ at Interior Support (MPa)</td>
<td>+2.40</td>
<td>+1.03</td>
<td>0.43</td>
<td>1.43</td>
</tr>
<tr>
<td>Shear Stress $\tau_{xy}$ at Outer Face of Edge Girder (MPa)</td>
<td>-1.00</td>
<td>-0.16</td>
<td>0.16</td>
<td>1.16</td>
</tr>
</tbody>
</table>

8. CONCLUSION AND RECOMMENDATION
The following conclusions are drawn upon:

1. The type of response influences the value of the dynamic amplification factor. Results obtained from this study indicate that different values of dynamic amplification factors are calculated for the responses of vertical displacement, normal stress in the longitudinal direction, and shear stress. It is therefore recommended that three types of dynamic amplification factors be included in the design/review of concrete bridges. These factors are based on the three types of response given above and correspond to the limit states of serviceability, collapse for moment, and shear, respectively.

2. The dynamic amplification factor for the same response type has different values with respect to location throughout the bridge deck.

3. Several of the values obtained for the dynamic amplification factors exceed the maximum value specified by AASHTO. In AASHTO specifications, the impact factor is a function of span length only. However, dynamic response depends to a great extent on vehicle properties and bridge characteristics. Thus bridge designs that comply with current codes may satisfy safety and strength requirements, but may also have undesirable dynamic response in the form of large displacements and consequently, increased stresses, which may affect the long-term performance of the bridge. Therefore there is a need to incorporate the bridge dynamic characteristics as an important parameter for the design of bridges.

Generally, the cutting rates decreased with carbon content of the steel. At the same oxygen pressure,
cutting rates have decreased up till about 65 pct with increase in carbon content of about 0.17wt%, the reason for the observed trend being the retardation to the formation of FeO by decarburization reactions between iron oxide (FeO) and carbon (C), iron oxide and carbon monoxide (CO), carbon monoxide and carbon.

REFERENCES