

# EFFECT OF INCREASING SPEED ON DYNAMIC IMPACT – AN ANALYTICAL STUDY ON STANDARD STEEL GIRDER BRIDGES

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## 1.0 INTRODUCTION

Dynamics of railway bridges involves the response of bridges to the movement of vehicles and to the influence of a number of parameters which increase dynamic strains or stresses. The most important parameters influencing the dynamic stresses in railway bridges are the frequency characteristics of bridge structures (i.e., the length, mass, and rigidity of individual members), the frequency characteristics of vehicles (i.e., the sprung and unsprung masses, the stiffness of springs), the damping in bridges and in vehicles, the velocity of vehicle movement, the track irregularities, and so on. The vehicles affect the bridges not only by vertical forces, but also by movements which generate longitudinal and transverse horizontal forces. The coefficient of dynamic augment which is used in IRS Bridge Rules is a function of loaded length only and does not represent the actual dynamic impact of moving train loads at high speeds. The paper presents the results of an analytical study undertaken on standard railway bridges which suggested that dynamic analysis of bridges is required to be done for design of bridges for high speeds.

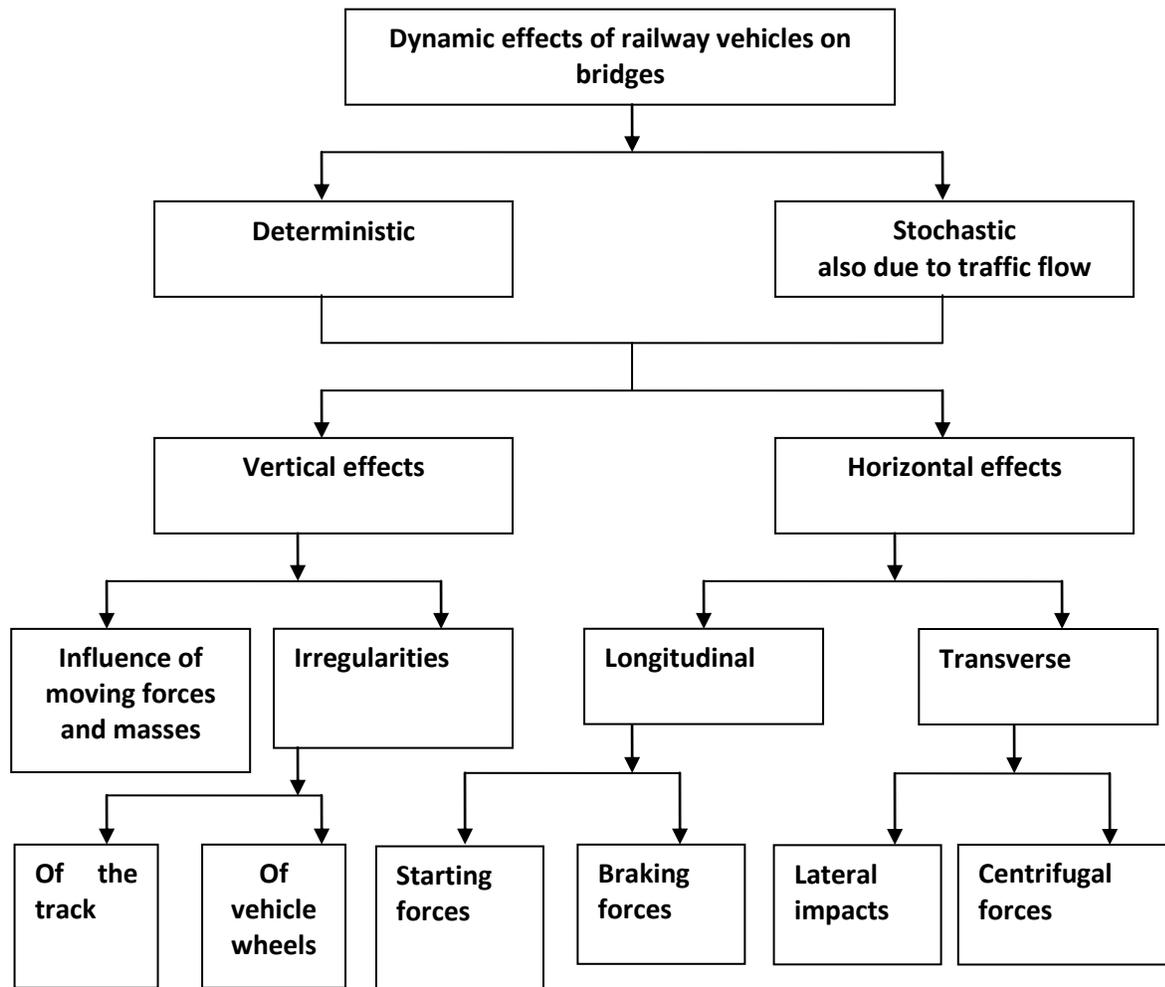
## 2.0 IMPORTANCE OF DYNAMIC EFFECTS

- 2.1 With more and more high-speed railways being built in the world, more and more emphasis has been given on the subject of dynamic interaction of vehicles and bridges. On the one hand, the train running with high speed induces dynamic impact on the bridge structures, influencing their working state and service life. On the other hand, the vibration of the bridge in turn affects the running stability and safety of the train, and thus becomes an important factor for evaluating the dynamic parameters in bridge design. Therefore, in many countries, the dynamic behaviors of bridges have been systematically studied in the development of high speed railway.
- 2.2 The railway bridges subjected to high speed trains provide intensive vibration similar to the resonance phenomenon. The resonance occurs if the frequency of an input force coincides with one of the natural frequencies of the system. The resonant vibration of railway bridges results in the deterioration of passenger comfort, reduction of traffic safety (a possibility of derailment of vehicles), the destabilization of ballast (higher maintenance costs) and increased damage in the bridge system from fatigue considerations.

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2.3 The dynamic effects of vehicles on railway bridges as shown in Fig. 1, [2] highlight the cause and effect relationships that exist. It may be appreciated that the problems to be addressed are multi-disciplinary in nature and require a reasonable degree of interaction between the executing agencies and the research people. The codal provisions of other countries can not be directly adopted without quantifying the parameters in indian context and verifying the relationship for Indian condition.



**Fig. 1 Dynamic effects of railway vehicles on bridges**

2.4 In practice the design of railway bridges includes the dynamic effect of the moving load by increasing the live load by an impact factor. However recent codes of practice address the problem by giving the full recognition to the fact that resonance in bridges may occur depending upon the characteristics of the train, its passage and the bridge characteristics. Considerable research has

been done elsewhere on the subject of dynamic interaction of vehicles and bridges. However, significant work has not been done in India on existing bridges to confirm their adequacy for high speed trains.

- 2.5 Under current design practice, the impact factor is considered a function of length only. However, dynamic response depends on a number of factors, including vehicle properties, bridge characteristics, and pavement roughness [9, 10]. Although designs that comply with current codes may satisfy safety and strength requirements, they may also greatly underestimate actual bridge response in many cases [11, 12, 13, 14]. Consequently, some bridge structures may suffer distress as a result of unexpected dynamic response. For example, stresses generated by heavy vehicles moving at high speeds over a rough bridge deck may greatly exceed those predicted by incrementing static live loads by a dynamic impact factor prescribed in bridge codes. Existing analysis and design procedures do not always predict these unexpected and undesirable results.

### **3.0 CODAL PROVISIONS FOR DYNAMIC EFFECTS OF TRAIN LOADS**

#### **3.1 Steel Bridge Code and Bridge Rules in India**

- 3.1.1 The present design of railway bridges is done in accordance with guidelines and provisions existing in Steel Bridge Code [22] and Bridge Rules [23]. In design practice, dynamic effect of the moving load is taken care of by increasing the live load by impact factor or dynamic augmentation factor or dynamic coefficient. This factor depends on many parameters like the type of loading, speed, type of structure, material of structure, loaded length etc. But for simplicity an impact factor is specified by the Bridge Rules in India involving only one parameter, i.e., the loaded length. All the other parameters are considered as constants in the expression for impact factor. For Broad Gauge and Meter Gauge steel railway bridges carrying a single track, the Coefficient of Dynamic Augment (CDA) is given by the following expression-

$$CDA = 0.15 + \frac{8}{6 + L}, \text{ subject to a maximum of 1.0}$$

Where L is defined as given below:

- a) L is loaded length of the span in meters for the position of the train giving the maximum stress in the member under consideration. For the design of chord members, it will be the whole span of the truss and for the web members only part of the span is to be loaded.
- b) L is taken as 1.5 times the cross-girder spacing for finding stresses in the stringers (rail-bearers).
- c) L is taken as 2.5 times the cross-girder spacing for finding moments in the cross-girders (floor-beams).

3.1.2 It is important to note that the expression for Co-efficient of Dynamic Augment (CDA) was proposed in 1981 based on actual field observations made on existing bridges. The formula is applicable for speeds upto 160 kmph on Broad Gauge and 100 kmph on Meter Gauge for passenger trains. It is apparent that the effect of higher speeds is not reflected in this expression.

### 3.2 **BS 5400-2: 1978**

In this standard [24], dynamic effects are considered in clause 8.2.3. Here equivalent static loadings (RU and RL loading) are multiplied by appropriate dynamic factors to allow for impact, oscillation and other dynamic effects including those caused by track and wheel irregularities.

The dynamic factor for RU loading applies to all types of track and is given in Table 2.1 (Table no. is with reference to BS:5400 – 2)

**Table 2.1**  
**Dynamic factor for type RU loading**

Dimension L (m)	Dynamic factor for evaluating	
	Bending Moment	Shear
upto 3.6	2.00	1.67
from 3.6 to 67	$0.73 + \frac{2.16}{\sqrt{(L-0.2)}}$	$0.82 + \frac{1.44}{\sqrt{(L-0.2)}}$
over 67	1.00	1.00

In deriving the dynamic factor, L is taken as the length (in m) of the influence line for deflection of the element under consideration. For unsymmetrical influence lines, L is twice the distance between the point at which the greatest ordinate occurs and the nearest end point of the influence line. In the case of floor members, 3 m should be added to the length of the influence line as an allowance for load distribution through track.

The dynamic factor for RL loading, for evaluation of moments and shears, shall be taken as 1.20, except for unballasted tracks where, for rail bearers and single track cross girders, the dynamic factor shall be increased to 1.40.

### 3.3 **EN 1991-2: 2003(E)**

3.3.1 In European standard [25] dynamic effects (including resonance) are considered in clause 6.4. In this code dynamic effects are taken care of in a better way. A

static analysis shall be carried out with the load models (Load Model 71 and where required Load Models SW/0 and SW/2). The results shall be multiplied by the dynamic factor,  $\Phi$  to consider the dynamic effects. Generally the dynamic factor  $\Phi$  is taken as either  $\Phi_2$  or  $\Phi_3$  according to the quality of track maintenance as follows:

(a) For carefully maintained track:

$$\phi_2 = \frac{1.44}{\sqrt{L_\phi} - 0.2} + 0.82, \text{ with: } 1.00 \leq \phi_2 \leq 1.67$$

(b) For track with standard maintenance:

$$\phi_3 = \frac{2.16}{\sqrt{L_\phi} - 0.2} + 0.73, \text{ with: } 1.00 \leq \phi_3 \leq 2.00$$

Where  $L_\phi$  is “Determinant” length (in m.).

The dynamic factor  $\Phi$  shall not be used with:

- the loading due to Real Trains.
- the loading due to Fatigue Trains.
- the load model HSLM.
- the load model “unloaded train”.

3.3.2 The requirements for determining whether a dynamic analysis is required are shown in Fig. 6.10 (Figure no. is with reference to Euro Code)

Where:

V is the Maximum Line Speed at the site (km/h)

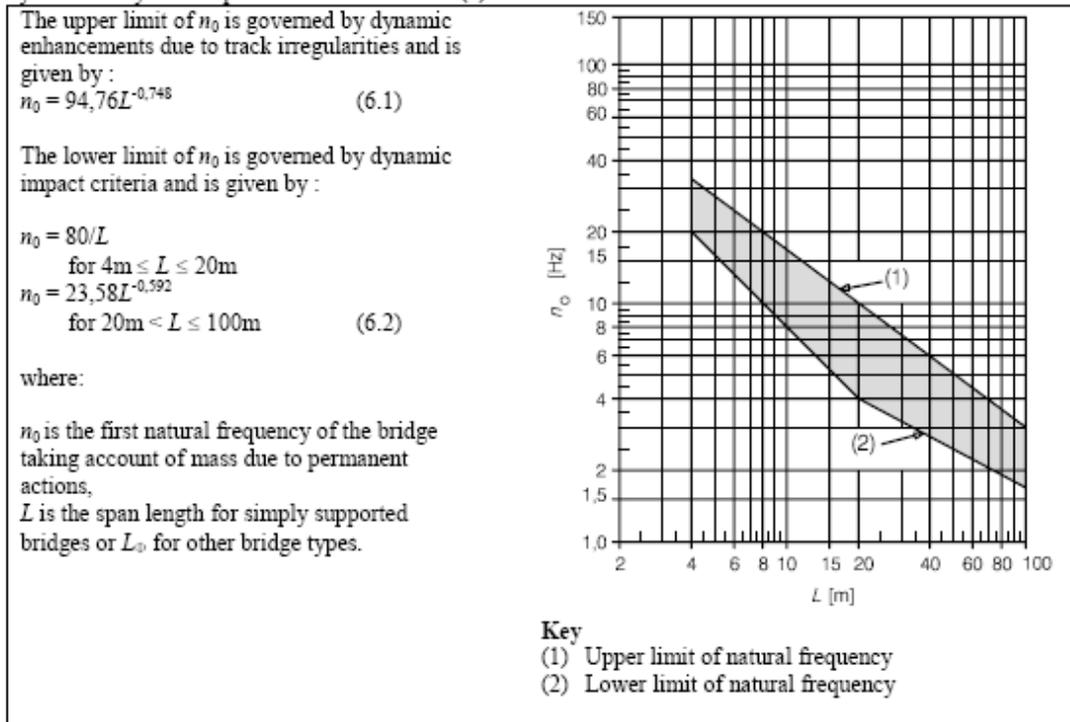
L is the span length

$\eta_0$  is the first natural bending frequency of the bridge loaded by permanent actions (Hz)

$\eta_T$  is the first natural torsional frequency of the bridge loaded by permanent actions (Hz)

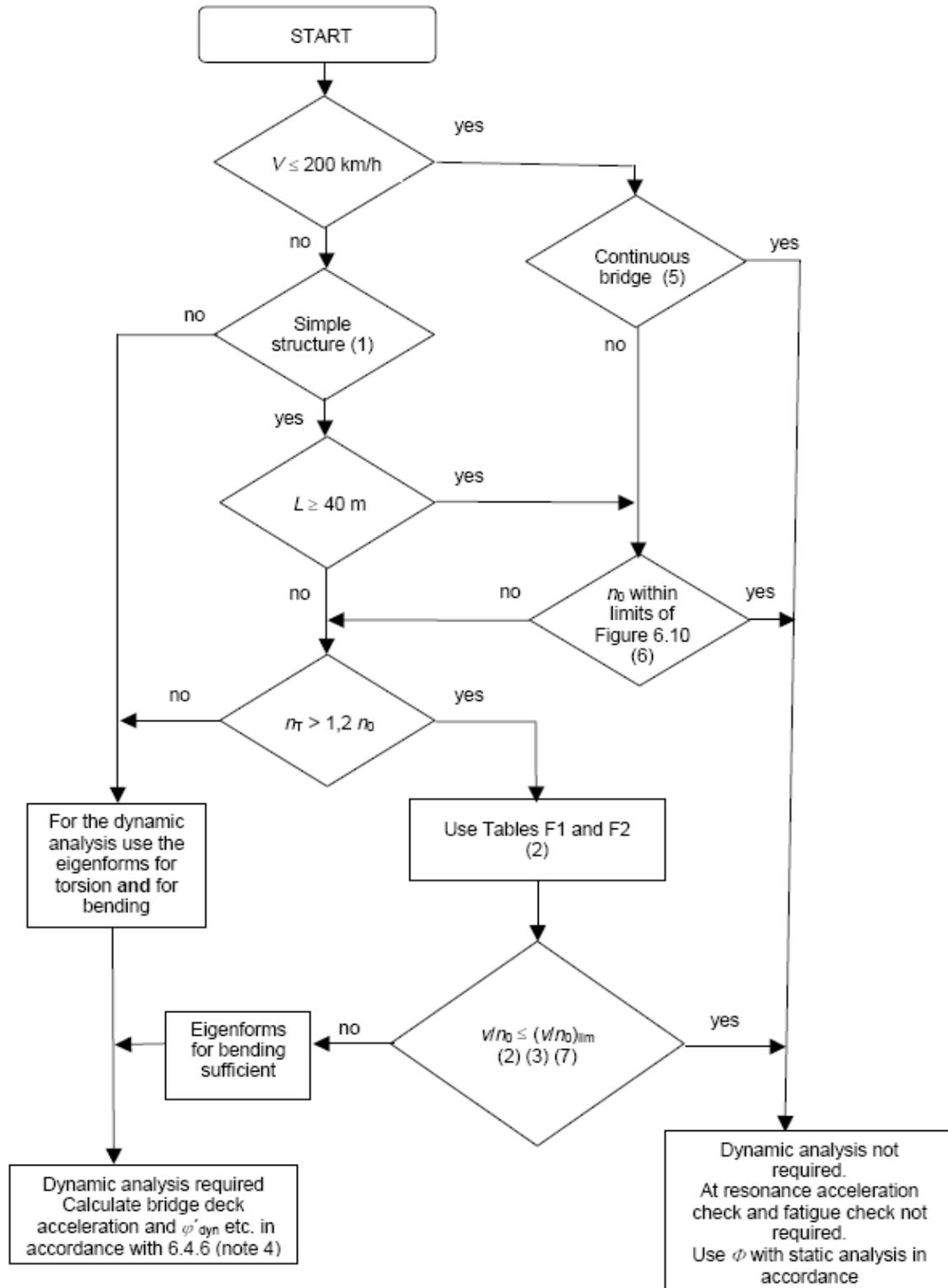
v is the Maximum Nominal Speed (m/s)

$(v/\eta_0)_{lim}$  is given in annex F (EN 1991–2: 2003(E))



**Fig. 6.10 – Limits of bridge natural frequency  $n_0$  [Hz] as a function of  $L$ (m)**

- 3.3.3 Quasi static methods which use static load effects multiplied by the dynamic factor  $\Phi$  are unable to predict resonance effects from high speed trains. Dynamic analysis techniques, which take into account the time dependant nature of the loading from the High Speed Load Model (HSLM) and Real Trains (e.g., by solving equations of motion) are required for predicting dynamic effects at resonance.
- 3.3.4 The standard [25] gives a flow chart (Figure 2) for determining whether dynamic analysis is required. This chart cannot be used directly for bridges on Indian Railways as the procedure given involves parameters which are meant for train given in European Load Models. As per this flow chart for continuous girder dynamic analysis is not considered necessary if design speed is less than 200 kmph.



**Fig. 2 Flow chart for determining whether a dynamic analysis is required (§ EN 1991–2: 2003(E), page no 77)**

#### 4.0 MATHEMATIC MODELING OF MOVING LOAD

4.1 Mondal [28] has obtained the response of a load train by assembling the responses of the different point loads  $P_k$  (Fig. 3). The differential equation corresponding to mode 'n' is given as under:

$$M_n \ddot{q}_n(t) + 2\xi_n \omega_n M_n \dot{q}_n(t) + \omega_n^2 M_n q_n(t) = \sum_{k=0}^k P_k \langle \phi_n(vt - d_k) \rangle$$

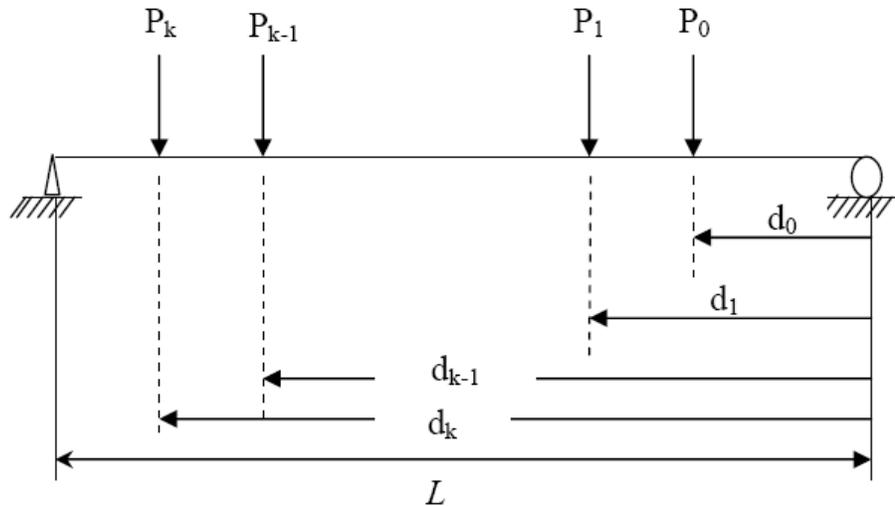


Fig. 3 Train load travels across the beam

4.2 Software package SAP2000 (Ver. 10.0.5) has been used to validate the results of mid span dynamic displacement with respect to time for a simply supported beam which is subjected to a single point wheel load traversing from one end to other end. The results are shown in Fig. 4.

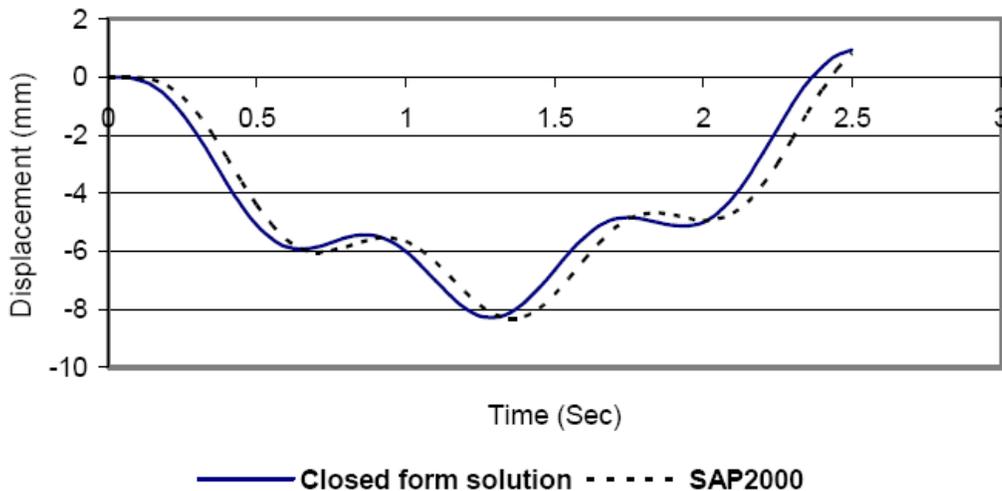


Fig. 4 Comparison between moving load analysis and SAP2000 results

### 4.3 Moving load analysis of standard spans

Two standard plate girder bridges and two standard welded truss bridges have been considered as per details given below:

#### a) 12.2 m Span Plate Girder Bridge:

It consists of two girders each having 2 Flange plates 550 x 25, 1 Web plate 1250 x 10

Clear Span = 12.2 m

Effective Span = 13.1 m

#### b) 24.4 m Span Plate Girder Bridge:

It consists of two girders each having 2 Flange plates 620 x 45, 1 Web plate 1980 x 14

Clear Span = 24.4 m

Effective Span = 25.6 m

#### c) 30.5 m Span Welded Truss Bridge:

Clear Span = 30.5 m

Effective Span = 31.926 m

#### d) 61 m Span Welded Truss Bridge:

Clear Span = 61 m

Effective Span = 63 m

### 4.4 Train definitions

For analyzing all the above mentioned bridges the moving load analysis has been carried out by the train descriptions. Trains are classified on the basis of their usage like passenger and goods train. For the present study 10 Modified Broad Gauge trains (which are defined as MBGT1 to MBGT10) and 12 Modified Broad Gauge trains for HM routes (which are defined as HMT1 to HMT12) have been considered as per Bridge Rules. Trains are modeled as a series of concentrated axle loads moving across the bridge.

### 4.5 Moving load analysis and dynamic analysis

After completing the modeling of all type of bridges, for all type of trains, MBGT1 to 10 and HMT1 to 12 moving load analysis and dynamic analysis are performed in SAP2000 v 10.0.5.

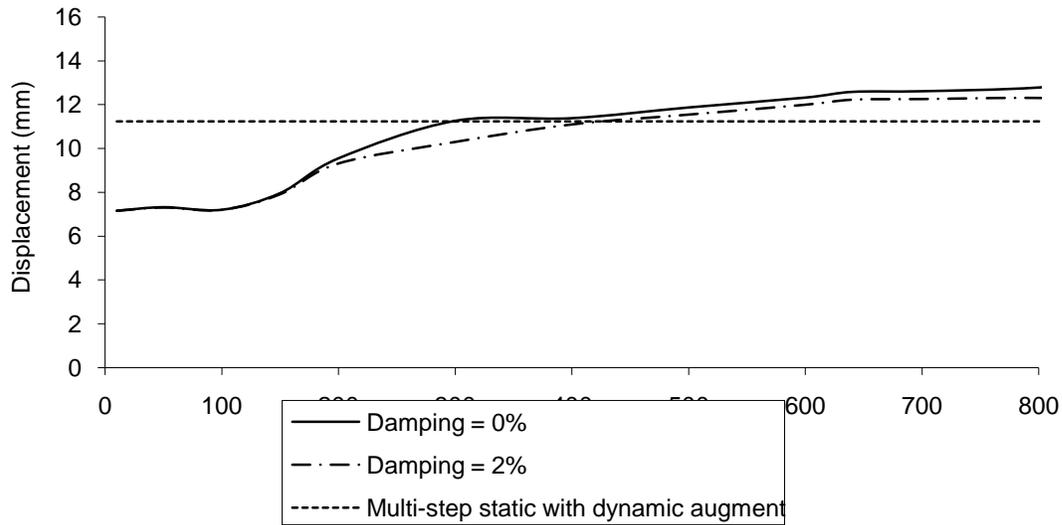
- 4.5.1 In moving load analysis *multi-step static analysis* is performed to get the bending stress histories at midspan of girder bridges and combined stress histories (axial and bending) of different members of truss bridges considering coefficient of dynamic augment as per Bridge Rules [23].
- 4.5.2 The parameters which are involved in moving load analysis mainly are (a) duration of loading i.e. the time required of a train to pass over the bridge (b) time step for discretization of the load. For *multi-step static* analysis speed of the train ( $v$ ) and time step ( $dt$ ) are considered as 10 m/sec and 0.05 sec respectively for finding the stress histories. The minimum spacing between two wheels ( $l_w$ ), considering all the trains is 1.7 m. For getting all the cycles in the stress history time step should be less than ( $l_w/v$ ).
- 4.5.3 Dynamic analysis is performed by *linear direct-integration time-history analysis* for different velocities of train considering zero percentage (0 %) and two percentage (2 %) damping ratio. From *modal analysis* time periods of two vertical modes are taken for defining mass and stiffness proportional damping.
- 4.5.4 The main important parameters for dynamic analysis are (a) velocity (b) time step

#### **4.6 Results and discussions**

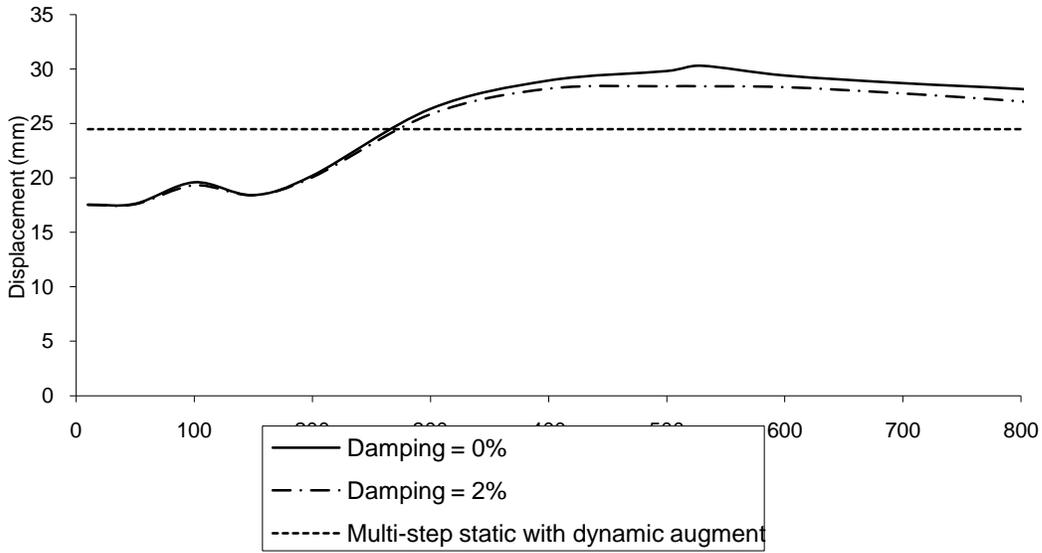
- 4.6.1 The midspan displacement responses of *plate girder bridges* for 0.0 % damping and 2.0 % damping due to passage of *MBGT5* with respect to the change of velocities are shown in Figs. 5 and 6, considering time step as  $l_w/v$ .
- 4.6.2 The midspan displacement responses of *welded truss bridges* for 0.0 % damping and 2.0 % damping due to passage of *MBGT5 and HMT8* with respect to the change of velocities are shown in Figs. 7, 8, 9 and 10 considering time step as  $l_w/v$ .
- 4.6.3 Also the dynamic response is compared with the response of moving load analysis i.e. multi-step static with coefficient of dynamic augment. When the response of moving load analysis crosses the response of dynamic analysis then the corresponding velocity is named as cut-off velocity. Table 1 shows the cut-off velocities of all the bridges due to passage of *MBGT5* and *HMT8*

**Table 1 Cut-off velocity of Girder and Truss Bridges due to passage of MBGT5 and MBGT8**

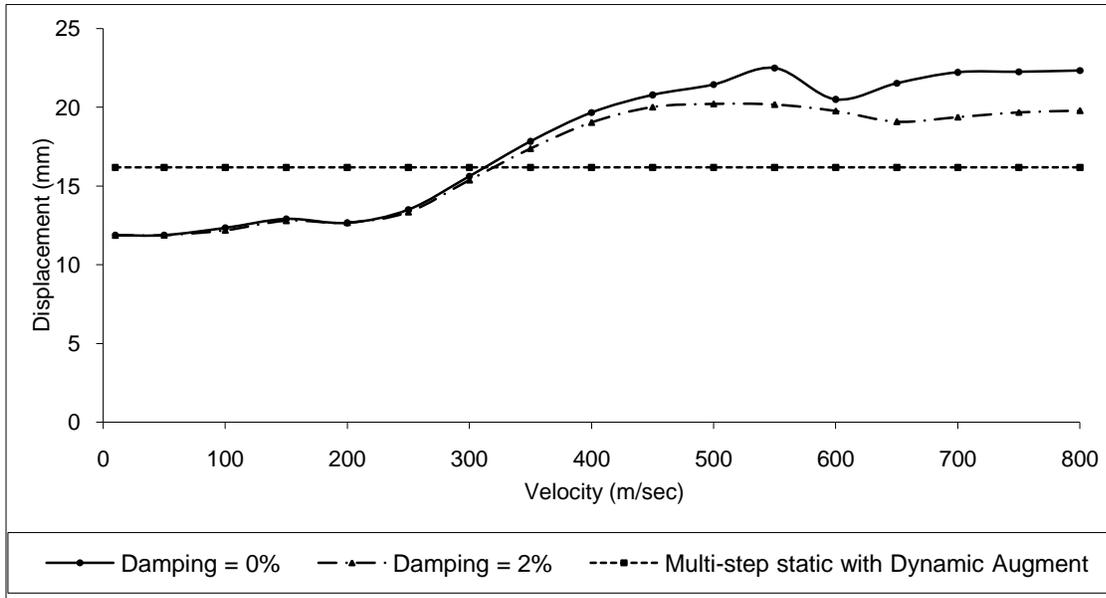
Type of Bridge	Type of Train	Effective Span (m)	Cut-off Velocity (m/sec)	
			Damping ratio = 0.0%	Damping ratio = 2.0%
Truss	MBGT5	31.926	312.67	320
		63	233.65	240.76
	HMT8	31.926	85.89	557.22
		63	286.26	296.49
Girder	MBGT5	13.1	298.25	428.89
		25.6	269.87	276.47



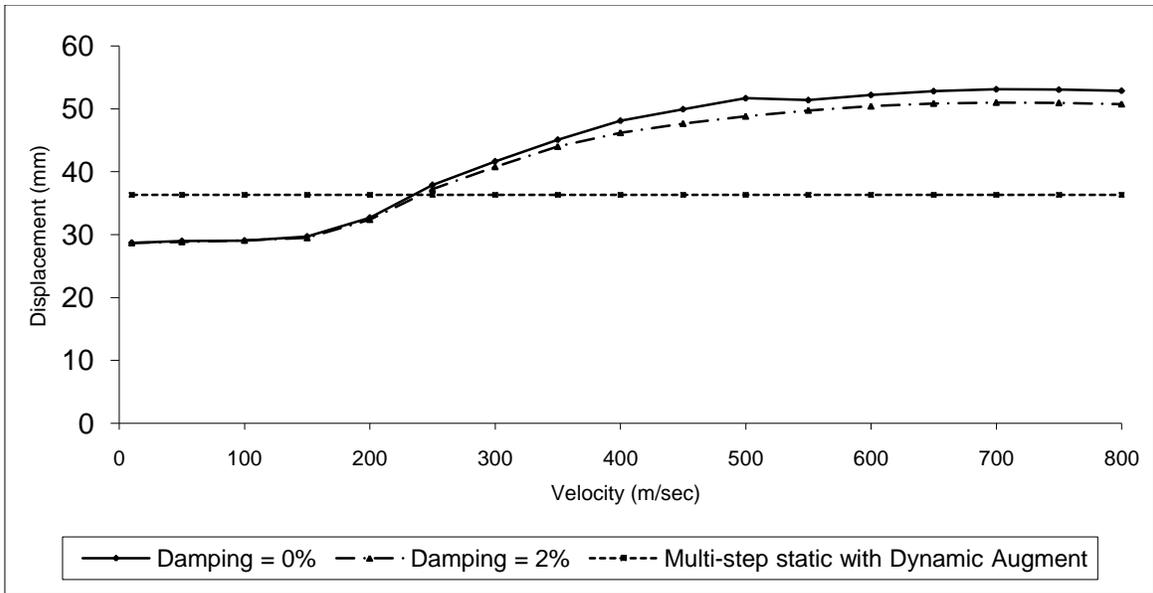
**Fig. 5 Variation of midspan displacement due to passage of MBGT5 on 12.2 m Plate Girder Bridge with respect to velocity**



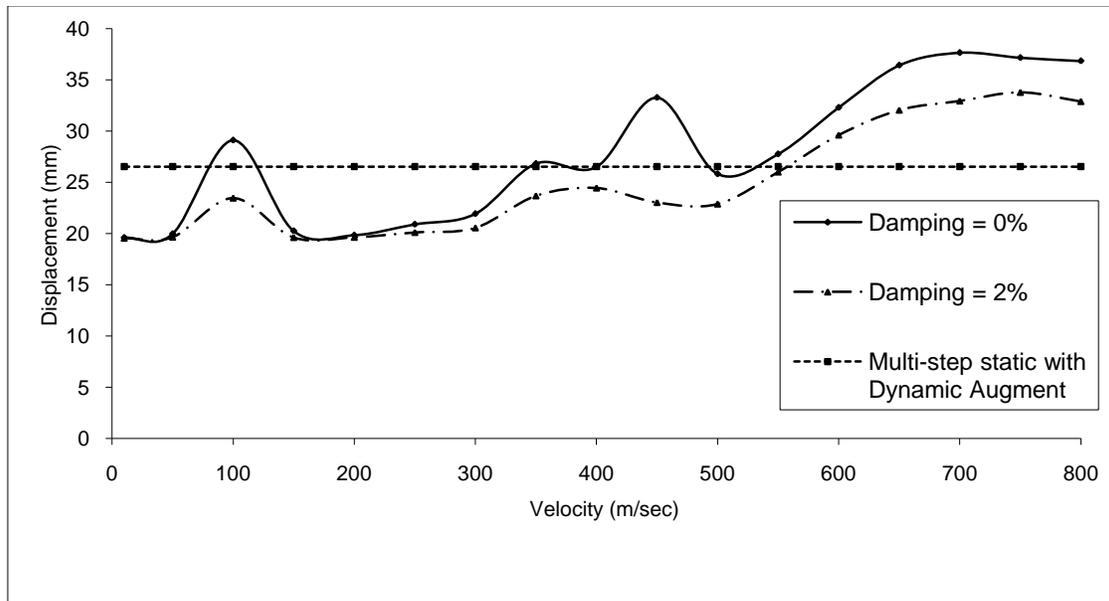
**Fig. 6 Variation of midspan displacement due to passage of MBGT5 on 24.4 m Plate Girder Bridge with respect to velocity**



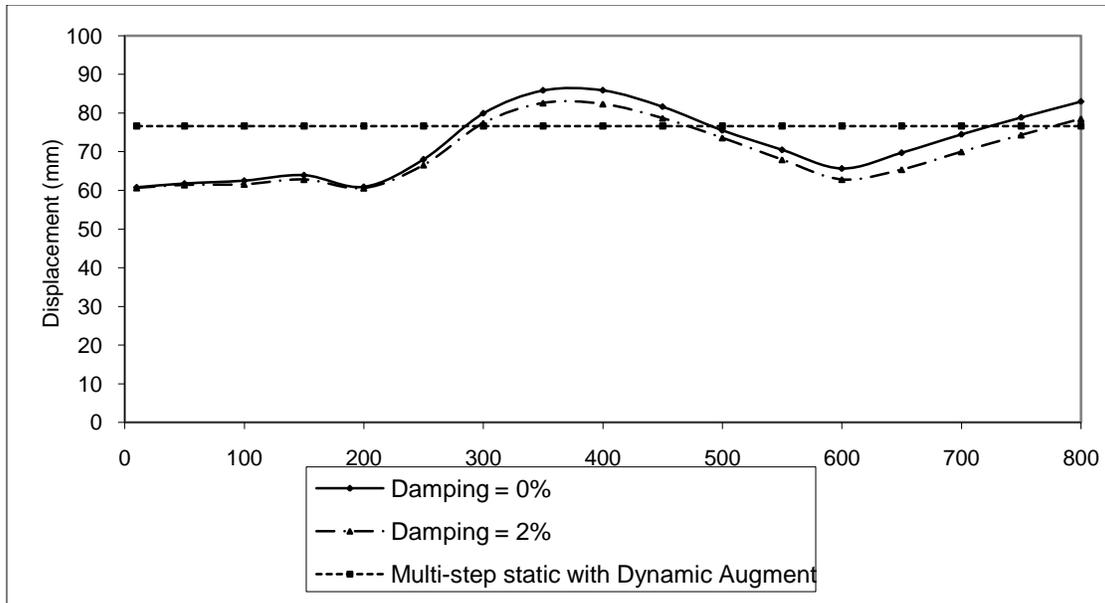
**Fig. 7 Variation of midspan displacement due to passage of MBGT5 on 30.5 m Welded Truss Bridge with respect to velocity**



**Fig. 8** Variation of midspan displacement due to passage of MBGT5 on 61.0 m Welded Truss Bridge with respect to velocity



**Fig. 9** Variation of midspan displacement due to passage of HMT8 on 30.5 m Welded Truss Bridge with respect to velocity



**Fig. 10 Variation of midspan displacement due to passage of HMT8 on 61.0 m Welded Truss Bridge with respect to velocity**

## 5.0 CONCLUSION

1. Mid span displacement on standard spans as per dynamic analysis are varying with speed.
2. The static analysis is adequate upto a cut off velocity which is generally more than 160 kmph. Dynamic analysis is required for different types of spans for permitting higher speeds.
3. The cut off velocity is different for different type of bridges and length of spans for a standard type train.
4. The cut off velocity is different for different types of trains in a load model.
5. Cut off velocity can be increased by providing external damping devices on the bridge.
6. For present speeds (i.e. upto 160 kmph) CDA based static analysis is safe.

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