iJRASET
International Journal For Research in
Applied Science and Engineering Technology

INTERNATIONAL JOURNAL FOR RESEARCH

IN APPLIED SCIENCE & ENGINEERING TECHNOLOGY

Volume: 9 **Issue:** VIII **Month of publication:** August 2021

DOI: https://doi.org/10.22214/ijraset.2021.37746

International Journal for Research in Applied Science & Engineering Technology (IJRASET)
ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor: 7.429
Volume 9 Issue VIII Aug 2021- Available at www.ijraset.com

Vehicle Bridge Interaction in High-Speed Rail Corridors

Jitendra Rastogi[1], Mirza Amir Beg[2]
[1]M. TECH (Structural & Foundation Engineering), Department of Civil Engineering AL-FALAH UNIVERSITY, Faridabad INDIA
[2](Assistant Professor), Department of Civil Engineering AL-FALAH UNIVERSITY, Faridabad INDIA

Abstract: In this paper, our aim is to establish that Dynamic impact factor (DIF) is not only dependent on the span and type of the bridge but also dependent upon speed of the train and distance between axles of the train as well. Our current code i.e. Indian Railway Standards specify that DIF or Coefficient of Dynamic Augment (CDA) is dependent on span length and type of bridge but it is applicable for design speed up to 160 kmph. For any speed greater than that CDA shall need to be computed as per the dynamic analysis as per available international codes. As mentioned earlier that there is imminent need of high-speed rail network in India due to increase in economic activity, increase in travel choices, improvement in mobility, reduction in congestion and to boost productivity.

Our objective of this project is to study dynamic response of a various types of bridges under high speed trains currently being used in India for high speed rail projects like RRTS (Delhi to Meerut and other corridors) and High speed rail project from Mumbai to Ahmedabad to accurately assess the DIF in bridges under the effect of different governing factors (vehicle speed, vehicle load, bridge superstructure type, etc). This study could be beneficial in upcoming projects of high-speed rail as it is our future need.

This study is based on the current semi-high-speed rail network i.e. Delhi Meerut Rapid Rail Transit System (RRTS) being constructed and other corridors are to be implemented. Design speed of this project is 180 kmph hence existing IRS codal provision for DIF cannot be used, therefore, dynamic analysis is needed to establish the DIF. Dynamic analysis has been carried out with two types of boggie length i.e. 21.34m and 22.34m.

In this project, we have started with the understanding of dynamic analysis by mentioning various codal provisions and parameters influencing the DIF. Subsequently, procedures for computation of dynamic analysis for given superstructure, loading, train type, span, etc have been explained including the modelling part. Last part of this study covers the dynamic analysis of various types of superstructure for given data.

I. INTRODUCTION

A. Background

It has been known for decades that vehicles and bridge interact dynamically and the dynamic response of the bridge during vehicular movement is more than the static response. The vehicle and bridge are two elastic systems which interact dynamically during vehicular movement through contact forces. The dynamic properties of vehicle and bridge such as modal frequency, damping and stiffness as well as velocity of the vehicle significantly affect the structural response of bridge during vehicular movement. The conventional approach to bridge design for dynamic effects is based on amplifying the static effects on bridge by a factor called Dynamic Impact Factor (DIF). Most of the design codes of practice specify the value of DIF based on bridge length or first natural frequency of flexural mode of vibration of the bridge. In this approach, various parameters that affect the dynamic response of the bridge such as:

1) The Train speed across the bridge.
2) The span length of the bridge and its structural configuration.
3) The mass of the bridge structure.
4) The natural frequency of the entire structure.
5) The number of train axles, their loads and distribution.
6) The damping of the structure.
7) The suspension characteristics of the vehicle.
8) The vertical irregularities of the track.
9) The wheel defects.

B. *Need and Importance of the Study*

The transport infrastructure is developing at a rapid pace throughout the world. In developing countries, new bridges are being constructed while in developed countries, repair and rehabilitation of existing bridges is being undertaken.

Currently, only developed countries like America, European countries, Japan, China and Far east countries are having high speed rail corridors. Whereas in current situation, high speed rail corridors are very much required in developing countries like India due to below benefits:

1) Increase in economic activity
2) Reduces Congestion and Boosts Productivity
3) Reduces the Nation's Dependence on Foreign Oil
4) Expands Travel Choices and Improves Mobility

Due to above benefits, cost optimised structures are the need of the hour to meet the demand of ever-growing population. Therefore, it is required to develop complex bridge modelling to accurately predict dynamic response of bridges leading to cost optimisation of new bridges. Dynamic Impact Factors (DIF) specified in various design codes may not be accurately able to forecast the maximum dynamic response of the bridge especially different type of bridges with various types of trains. Hence, it is imperative to conduct a detailed structural analysis on such bridges to precisely understand its response to moving loads. The aim of this project is to study dynamic response of a various types of bridges under high speed trains currently being used in India for high speed rail projects like RRTS (Delhi to Meerut and other corridors) and High speed rail project from Mumbai to Ahmedabad to accurately assess the VBI effects in bridges under the effect of different governing factors (vehicle speed, vehicle load, bridge superstructure type, etc). This study could be beneficial in upcoming projects of Semi high-peed/ high-speed rail as it is our future need.

C. *Scope and Objectives*

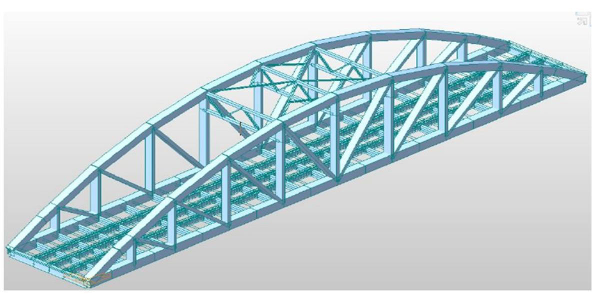
The objectives of present study are:

1) To develop a procedure for study of the Dynamic response of bridge under action of moving vehicular traffic.
2) To analyse the various type of superstructure type under the effect of governing parameters like vehicle speed, bridge superstructure type, etc to obtain its dynamic response.
3) To obtain the DIF using Vehicle bridge interaction and to compare it vis-a-vis results from codal provisions.

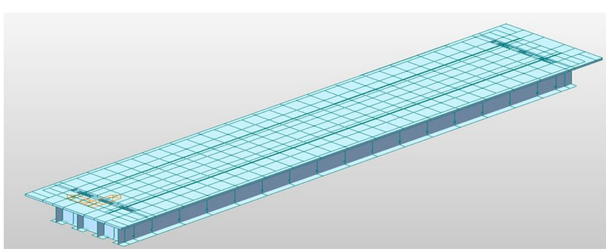
II. METHODOLOGY

Dynamic analysis is performed using MIDAS software for below mentioned type of structures for two train boggies length 21.34m and 22.34m and for design speed of 180 kmph.

A. *73m Span Steel Truss*



B. 50m Span Steel Plate Girder.



International Journal for Research in Applied Science & Engineering Technology (IJRASET)
ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor: 7.429
Volume 9 Issue VIII Aug 2021- Available at www.ijraset.com

C. 34m PSC Concrete box Girder.

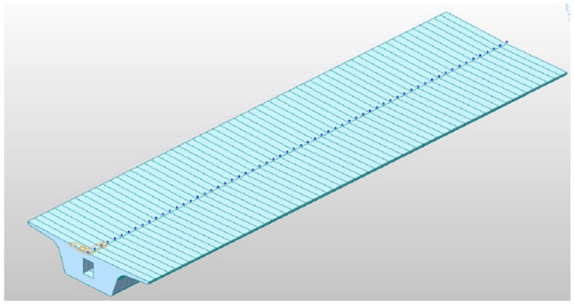


Fig. 7 Model showing 3D view

Various steps and results of dynamic analysis for mentioned structures type is explained in the subsequent sections.

1) Codal Provisions for Dynamic Analysis

a) Codes for Dynamic Analysis

- BS EN 1990:2002 - Basis of structure design
- BS EN 1991-2:2003 - PART-2 traffic load on the bridges
- NA to BS EN 1991-2:2003
- UIC 776-2 - Design Requirement of rail-bridges based on interaction phenomena between train, track and bridge

b) Codal Provisions

- *Speed*
  - Speed > 200km/h
  - In Dynamic analysis speed to be considered = 1.2 x maximum line speed at site.
  - Individual project may vary the definition of maximum line speed and it can be increased for future rolling stock.
  - For analysis Speed to be considered from 40m/sec (144km/h) to 1.2 times of maximum speed in the step of 5km/h to10km/h.
  - Spanish Code (IAPF 2007) indicates that calculations should be made from 20 km/h up to 1.2 times of maximum Line Speed with speed steps of 10 km/h.

- *Acceleration:* In the first high speed line built in France (Paris & lyon) track deformation appeared due to high vertical acceleration. Usually, ballast starts to lose its integrity in ballasted track when acceleration is greater than 0.7g to 0.8g (Report ERRI-214, experiment done by SNCF and confirmed by UIC). Also, loss of wheel rail contact in ballast less track occurs when acceleration is greater than 1.0g.

Codal provisions for acceleration is provided below:

- Limit as per Euro Code
  For ballasted track – 3.5m/s2
  For direct fastened track – 5.0m/s2
- Limit as per UIC (Clause. 3.2.3 of UIC-776)
  For ballasted track – 0.35g m/s2 (3.43 m/s2)
  For direct fastened track – 0.5g m/s2 (4.91 m/s2)
- Maximum frequency considered = maximum (30Hz, 1.5 x frequency of first mode, frequency of third mode) (Clause A2.4.4.2.1 of BS EN 1990:2002)
- Acceleration to be checked at the serviceability limit state to prevent track instability.

- *Number of Track Considered in Dynamic Analysis:* When verifying the acceleration or dynamic factor of a deck with dual track in both running direction, only one track is considered. (clause 6.4.6.1.2).

Table 6.5 - Summary of additional load cases depending upon number of tracks on bridge

Number of tracks on a bridge	Loaded track	Loading for dynamic analysis
1	one	Each Real Train and Load Model HSLM (if required) travelling in the permitted direction(s) of travel.
2 (Trains normally travelling in opposite directions) [a]	either track	Each Real Train and Load Model HSLM (if required) travelling in the permitted direction(s) of travel.
	other track	None.

[a] For bridges carrying 2 tracks with trains normally travelling in the same directions or carrying 3 or more tracks with a Maximum Line Speed at the Site exceeding 200km/h the loading should be agreed with the relevant authority specified in the National Annex.

- *Frequency*

Frequency needs to be checked:

- First natural Bending frequency of permanent load (n0)
- First Natural frequency of lateral vibration > 1.2 Hz
- First natural Torsional frequency of permanent load (nT)
- First natural frequency

NOTE 8 For a simply supported bridge subjected to bending only, the natural frequency may be estimated using the formula :

$$n_0\,[\mathrm{Hz}] = \frac{17{,}75}{\sqrt{\delta_0}} \qquad (6.3)$$

where:

δ_0 is the deflection at mid span due to permanent actions [mm] and is calculated, using a short term modulus for concrete bridges, in accordance with a loading period appropriate to the natural frequency of the bridge.

First natural Bending frequency of permanent load (n_0)

The upper limit of n_0 is governed by dynamic enhancements due to track irregularities and is given by :

$$n_0 = 94{,}76L^{-0{,}748} \qquad (6.1)$$

The lower limit of n_0 is governed by dynamic impact criteria and is given by :

$n_0 = 80/L$ for $4\mathrm{m} \le L \le 20\mathrm{m}$

$n_0 = 23{,}58L^{-0{,}592}$ for $20\mathrm{m} < L \le 100\mathrm{m}$ (6.2)

where:

n_0 is the first natural frequency of the bridge taking account of mass due to permanent actions,

L is the span length for simply supported bridges or L_Φ for other bridge types.

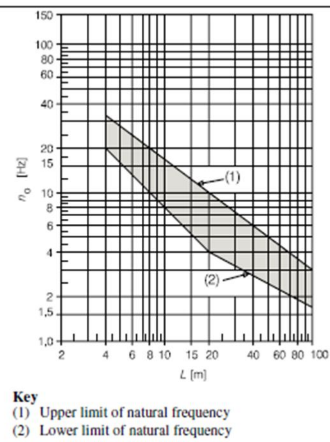


Figure 6.10 - Limits of bridge natural frequency n_0 [Hz] as a function of L [m]

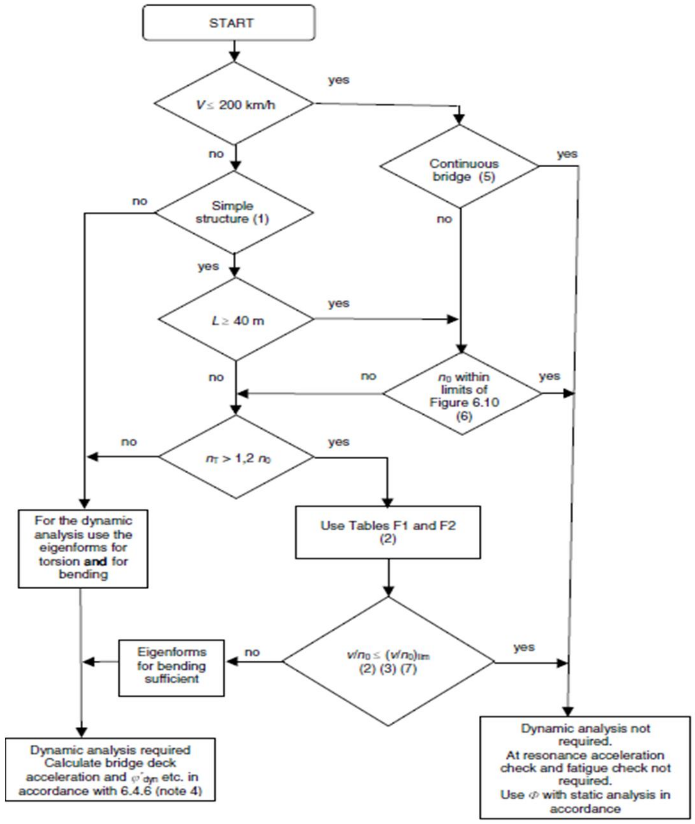
- *Critical Speed*

Critical speeds for simple spans:

V = f d / i
f = deck vibration period
d = distance between bogies
i = 1, 2, 3, …

International Journal for Research in Applied Science & Engineering Technology (IJRASET)
ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor: 7.429
Volume 9 Issue VIII Aug 2021- Available at www.ijraset.com

- *Flowchart for Dynamic analysis*



- *Bridge Parameters*
- Bridge Damping

Table 6.6 - Values of damping to be assumed for design purposes

Bridge Type	ζ Lower limit of percentage of critical damping [%]	
	Span $L < 20$m	Span $L \geq 20$m
Steel and composite	$\zeta = 0{,}5 + 0{,}125\,(20 - L)$	$\zeta = 0{,}5$
Prestressed concrete	$\zeta = 1{,}0 + 0{,}07\,(20 - L)$	$\zeta = 1{,}0$
Filler beam and reinforced concrete	$\zeta = 1{,}5 + 0{,}07\,(20 - L)$	$\zeta = 1{,}5$

- Additional damping span < 30.0m

$$\zeta_{TOTAL} = \zeta + \Delta\zeta \qquad (6.12)$$

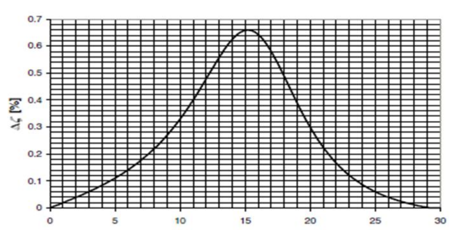


Figure 6.15 - Additional damping $\Delta\zeta$ [%] as a function of span length L [m]

where:

$$\Delta\zeta = \frac{0{,}0187L - 0{,}00064L^2}{1 - 0{,}0441L - 0{,}0044L^2 + 0{,}000255L^3}[\%] \qquad (6.13)$$

International Journal for Research in Applied Science & Engineering Technology (IJRASET)
ISSN: 2321-9653; IC Value: 45.98; SJ Impact Factor: 7.429
Volume 9 Issue VIII Aug 2021- Available at www.ijraset.com

- Bridge Stiffness & Mass
  - Any overestimation of bridge stiffness will overestimate the natural frequency of the structure and speed at which resonance occurs.
  - A lower limit of the mass of the deck to obtain maximum acceleration
  - An upper limit of the mass of the structure to obtain the lowest speed at which effects of resonance occur
- Dynamic Impact Coefficient

$$\varphi'_{dyn} = \max\left[\gamma_{dyn} / \gamma_{stat}\right] - 1$$

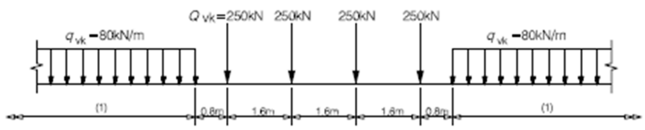
where γ_{dyn} represents the dynamic deflection of the deck under the high-speed load diagram or real trains and γ_{stat} represents the static deflection of the deck.

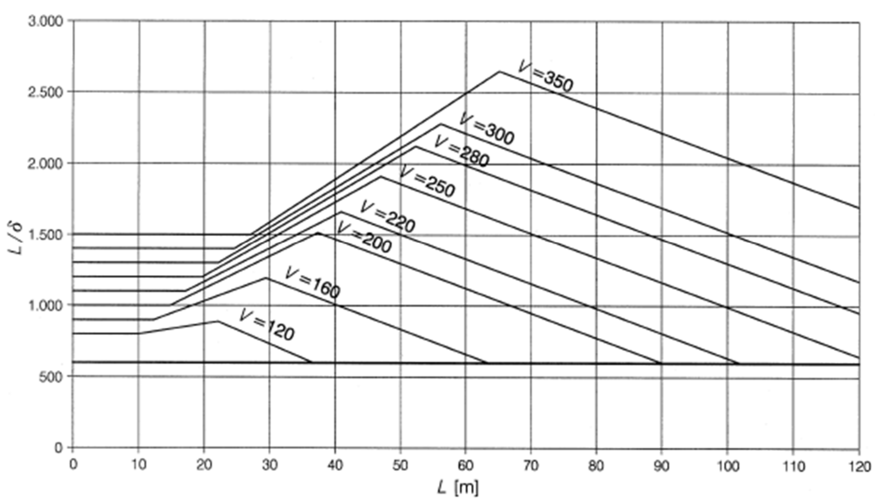
  - Passenger comfort depends on vertical acceleration inside the coach.
  - Limiting values for maximum vertical deflection for passenger comfort inside the coach.

Table A2.9 - Recommended levels of comfort

Level of comfort	Vertical acceleration b_v (m/s^2)
Very good	1,0
Good	1,3
Acceptable	2,0

  - Comfort is generally satisfied by applying the deflection criteria under LM71 (which in eurocode 0 depends on the span, the speed, the statical scheme and number of successive spans)





- Necessary to Perform Comfort Criteria
  - For classical railway, when the allowable deflection criteria equivalent to comfort checking are not applicable (Clause A2.4.4.3 of EN1990)
  - span > 120m, or continuous bridges with very different spans
  - large inertia variation within a span.

2) *Steps And Data For The Dynamic Analysis For The Various Superstructure Type For Given Loading*

a) *Introduction:* All developed and developing countries are having high/ semi high-speed train network available to minimise the travelling time of commuters to the increase business activities for improvement of economy. In India neither we have high speed / semi speed railway network nor the codal provisions for high speed trains. Considering the above, Indian government has started constructing high speed rail corridor (Mumbai-Ahmedabad) and semi high-speed rail corridors (Delhi-Meerut RRTS). Therefore, it is imperative to study the impact on various type of bridge superstructures for the high-speed trains as it may assist in choosing the appropriate type of superstructure. In this thesis, dynamic analysis for various type of superstructure has been carried out for Delhi-Meerut RRTS project which is having design speed of 180 kmph.

b) *Various steps involved for the Dynamic analysis*

- Data collection as per project requirement (structural type, speed, track type etc.).
- Modeling of structure (Geometric property, Material property, Damping etc.).
- Permanent Load Application (DL, SIDL converted to model masses).
- Eigenvalues analysis check first natural bending frequency and compare with codal first natural frequency.
- Check upper and lower bound values of first natural frequency.
- Check critical speed.
- Create Time History Function for Live Load.
- Dynamic Nodal Load Generation (time step, damping etc.).
- Moving Load Analysis.
- Results extract (Acceleration v/s Time & displacement v/s time graph.
- Calculate dynamic impact factor from dynamic and static displacement and compare with code.
- Checking of passenger comfort criteria

c) *Data considered for modelling of Structures*

- *Design Basis*

The Dynamic Analysis is carried out in accordance with the following documents:

- Design Basis Report-Viaduct – (Document: DM12DDC-GEN-GEN-VDC-REP-000005) which summarizes the general assumptions for the whole project.
- BS EN 1991-2 (Eurocode 1 : Actions on structures – Part 2 : Traffic Loads on Bridges)
- BS EN 1990-2002+A1(Basis of structural Design)

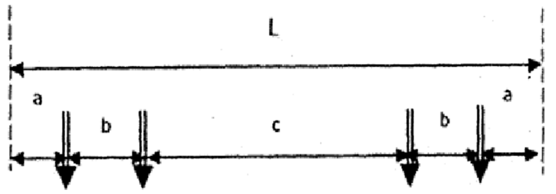
- *Loadings*

- Dead Load (DL): Self-weight of the structure is automatically considered by the software.
- Super Imposed Dead Load (SIDL): Details of SIDL for two tracks:

S.No.	Element	Unfactored Load (t/m)
1	Railing 1m high – 3 Nos (50Kg each - both side & Central)	0.150
2	Both Track structure	6.240
3	Rail Guards between Rails (both tracks)	0.606
4	Rail + Pads (both tracks)	0.300
5	Cables	0.070
6	Cable trays/Hanger	0.100
7	Parapet (Excluding Integral Part of Box)	0.574
8	Walk way cum Cable Trough + Pedestal for Central Railing + Pedestal for OHE Mast	0.920
9	Miscellaneous (OHE Mast, Signaling etc.)	0.400
		9.36 t/m

say	**9.40 t/m**
Fixed	**1.65 t/m** (railing/walkway/Parapet)
Variable	**7.75 t/m**

- *Live Load (LL):* Each component of the structure shall be designed/checked for all possible combinations of these loads and forces. They shall resist the effect of the worst combination:



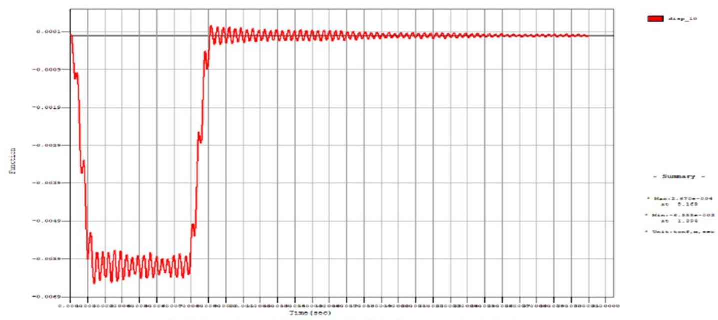


Fig. 26 Summary of the Highest Peak Displacements for Bottom Chord

All axle loads = 17 tons
Maximum number of successive cars = 12 Where,
L = 21.340m / 22.340m (Length of a car)
a = 1.920m (overhang)
b = 2.500m (Wheel base in a bogie)
c = 12.500m / 13.500m (Distance between Axle-2 and Axle-3 in the car)

3) *Dynamic Analysis For The Various Superstructure Types*

a) *Dynamic Analysis of 73m span Steel Truss:* The Standard Span of 73m Steel Truss has been analyzed for the dynamic application of RRTS Railway Vehicular Load and deflections to identify the range of applicability of the steel truss for speed upto 180 kmph. The analysis is aimed at deriving the deck accelerations for RRTS vehicular speed. Calculation has been carried out for RRTS train load model for the maximum speed ($Vmax$ = 216 kmph). The analysis is aimed at deriving the deflection for RRTS vehicular speed. Calculation has been carried out for RRTS train load model for the maximum speed ($Vmax$ = 180 kmph). The obtained maximum acceleration of 2.299m/s² and maximum deflection of 6.552mm (as shown below) are acceptable when compared to the limit values of 5.0m/s² and 101 mm respectively. It can be concluded that the choice of the 73m Span steel truss with the provided geometrical arrangements is acceptable and the selected configuration allows meeting the dynamic design criteria for RRTS structures. Also, the dynamic factor obtained from Dynamic Analysis as 1.082 is less than the dynamic factor calculated as per Annex-C, BS EN 1991-2 i.e. 1.133 and as per Cl. 2.4.1.1 of IRS Bridge Rules i.e. 1.254. So, Dynamic Factor as per IRS Bridge Rule be considered in design on conservative side.

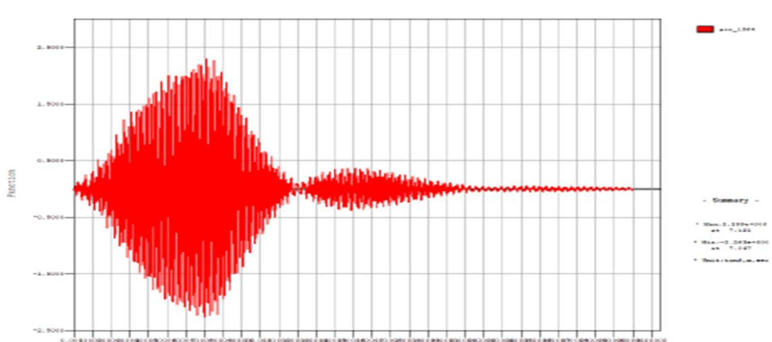
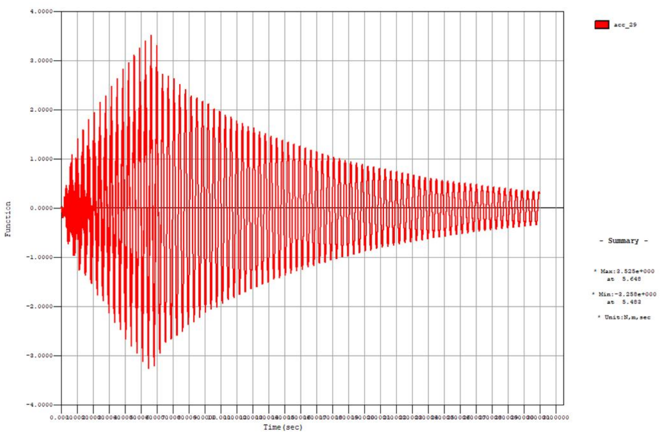
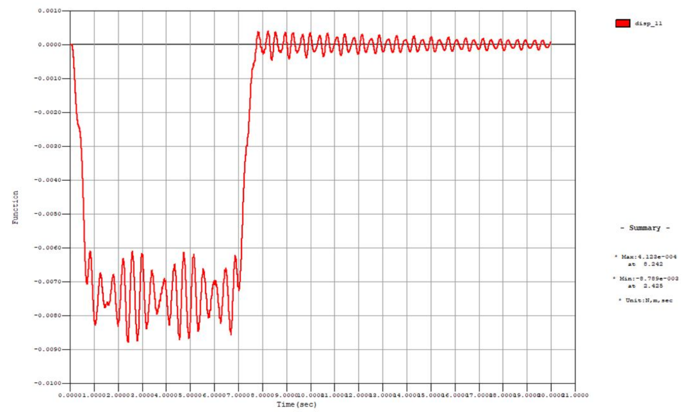


Fig. 13 Summary of the Highest Peak Accelerations for Velocity 200kmph Velocity (for bogie length 22.34m)

b) *Dynamic Analysis of 50m span Steel Plate Girder:* The Standard Span of 50m Steel Plate Girder has been analyzed for the dynamic application of RRTS Railway Vehicular Load and deflections to identify the range of applicability of the steel plate girder for speed upto 180 kmph. The analysis is aimed at deriving the deck accelerations for RRTS vehicular speed. Calculation has been carried out for RRTS train load model for the maximum speed ($V_{max} = 216\ kmph$). The analysis is aimed at deriving the deflection for RRTS vehicular speed. Calculation has been carried out for RRTS train load model for the maximum speed ($V_{max} = 180\ kmph$). The obtained maximum acceleration of 3.525m/s² and maximum deflection of 8.816mm (as shown below) are acceptable when compared to the limit values of 5.0m/s² and 40 mm respectively.





It can be concluded that the choice of the 50m Span steel plate girder with the provided geometrical arrangements is acceptable and the selected configuration allows meeting the dynamic design criteria for RRTS structures.
Also, the dynamic factor obtained from Dynamic Analysis as 1.146 is less than the dynamic factor calculated as per Annex-C, BS EN 1991-2 i.e. 1.229 and as per Cl. 2.4.1.1 of IRS Bridge Rules i.e. 1.298. So, Dynamic Factor as per IRS Bridge Rule be considered in design on conservative side.

c) *Dynamic Analysis of 34m PSC Box Girder:* The Standard Span of 34m PSC Box Girder has been analyzed for the dynamic application of RRTS Railway Vehicular Load to identify the range of applicability of the Box Girder for speed upto 216 kmph. The analysis is aimed at deriving the deck accelerations for RRTS vehicular speed. Calculation has been carried out for RRTS train load model for the maximum speed ($Vmax$ = 216 kmph). The analysis is aimed at deriving the deflection for RRTS vehicular speed. Calculation has been carried out for RRTS train load model for the maximum speed ($Vmax$ = 180 kmph). The obtained maximum acceleration of 1.612m/s² and maximum deflection of 4.041 mm (as shown below) are acceptable when compared to the limit values of 5.0m/s² and 23.75mm respectively.

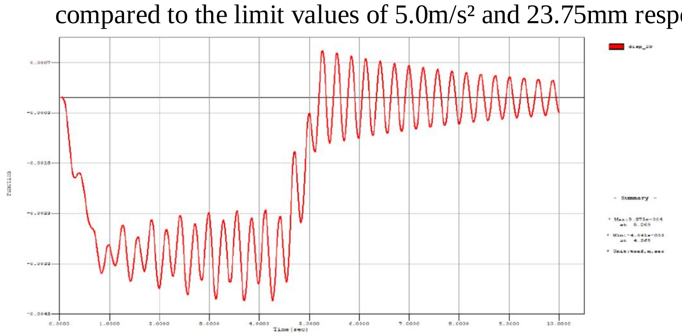


Fig. 37 Summary of the Highest Peak Displacements

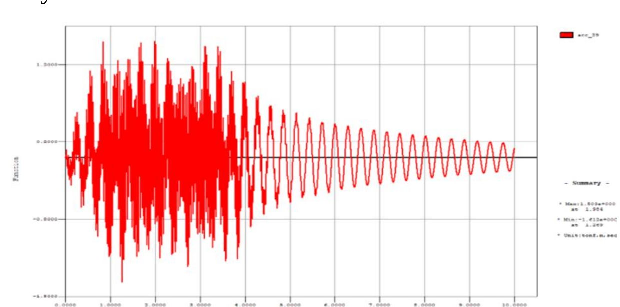


Fig. 12 Summary of the Highest Peak Accelerations for Velocity 210kmph Velocity (for bogie length 21.34m)

It can be concluded that the choice of the Standard Span as 34.0m with the provided geometrical arrangements is acceptable, and the selected concrete outline allows meeting the dynamic design criteria for RRTS structures.
Also, the dynamic factor obtained from Dynamic Analysis as 1.247 is less than the dynamic factor calculated as per Annex-C, BS EN 1991-2 i.e. 1.292 and as per Cl. 2.4.1.1 of IRS Bridge Rules i.e. 1.359. So, Dynamic Factor as per IRS Bridge Rules may be considered in design on conservative

III. CONCLUSIONS

A. It has been observed that dynamic deflection and vertical acceleration is dependent upon the distance between the axles and the span length for given speed, superstructure type. For example, vertical acceleration and dynamic deflection for 34m PSC box girder is as below:

PSC box girder 34m span	144 kmph (21.34 m Boggie)	144 kmph (22.34 m Boggie)	180 kmph (21.34 m Boggie)	180 kmph (22.34 m Boggie)
Vertical acceleration (m/s2)	0.528	0.994	1.051	1.079
Dynamic deflection (mm)	3.916	4.041	3.736	3.731

B. Dynamic impact factor for 180 kmph design speed with 17t axle load is majority of the times lesser than that mentioned in the IRS bridge rules as seen is all superstructure type considered in this study.

C. It has also been studied that if span length is in multiples of axles spacing, then chances of resonance or increased vertical deflection is possible. Therefore, spans in multiple of axles spacing should be avoided in the initial exercise.

D. Above conclusion is drawn for project having similar speed that of RRTS i.e. 180 kmph. However detailed study for speed higher than that of RRTS is to be done as future study.

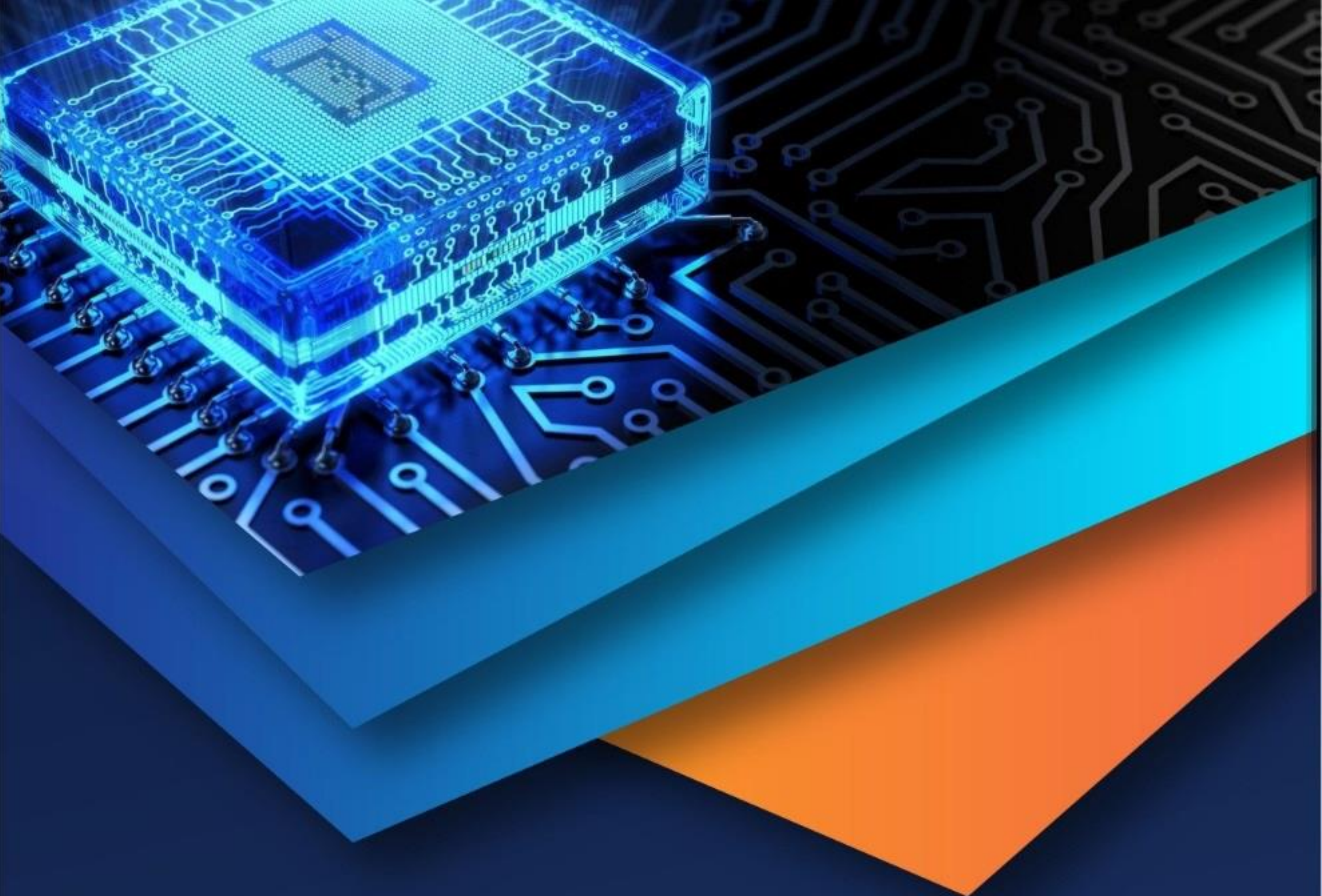
REFERENCES

[1] BS EN 1990:2002 - Basis of structure design
[2] BS EN 1991-2:2003 - PART-2 traffic load on the bridges
[3] NA to BS EN 1991-2:2003
[4] UIC 776-2 - Design Requirement of rail-bridges based on interaction phenomena between train, track and bridge.
[5] Design Basis Report-Viaduct – (Document: DM12DDC-GEN-GEN-VDC-REP-000005)
[6] Azimi, H. (2011), "Development of VBI models with vehicle acceleration for bridge vehicle dynamic response." PhD Thesis, Concordia University, Canada.
[7] Chang, D.; Lee, H. (1994) "Impact factors for simple-span highway girder bridges.", Journal of Structural Engineering, 3(704), 704-715.
[8] Chatterjee, P. K.; Datta, T.K.; Surana, C.S. (1994) "Vibration of continuous bridges under moving vehicles.", Journal of Sound and Vibration, 169(5), 619-632.
[9] Deng, L.; Cai, C.S. (2010) "Development of dynamic impact factor for performance evaluation of existing multi-girder concrete bridges."; Journal of Structural Engineering, 32(1), 21-31.
[10] Huang D.; Wang T.L.; Shahawy M. (1992) "Impact analysis of continuous multigirder bridges due to moving vehicles.", Journal of Structural Engineering, 18:12(3427), 3427-3443.
[11] MATLAB, The Math Works, Incorporation; Natick; Massachussets, 7.10.0 (R2010a).
[12] Mehmood, A.; Khan, A.A.; Mehdi, H. (2014) "Vibration analysis of beam subjected to moving load using finite element method.", IOSR Journal of Engineering, 4(5), 7-17.
[13] Paraskeva, T.S.; Dimitrakopoulos, E.G.; Zeng, Q. (2017) "Dynamic vehicle-bridge interaction under simultaneous earthquake excitation.", Bulletin of Earthquake Engineering, 15, 71-95.
[14] SAP2000, Computers and Structures Incorporation, Version 15.1.0.
[15] Shepherd, R.; Aves, R.J. (1973) "Impact factors for simple concrete bridge.", Proceeding of the Institute of Civil Engineering, 55(1), 191-210.
[16] Timoshenko S.P. (1922) "On the forced vibration of bridges.", Philosophical Magazine Series 6, 43(257), 1018-1019.
[17] Yang, Y.B.; Lin B.H. (1995) "Vehicle-bridge interaction analysis by dynamic condensation method.", Journal of Structural Engineering, 11(1636), 1636-1643.
[18] Yang, Y.B.; Wu, Y.-S. (2001) "A versatile element for analysing vehicle-bridge interaction response.", Engineering Structures, 23, 452-469.
[19] Yang, Y.B.; Yau, J.-D. (1997) "Vehicle-bridge interaction element for dynamic analysis.", Journal of Structural Engineering, 123(11), 1512-1518.

BIOGRAPY

JITENDRA RASTOGI
Student of M.Tech-Structure and foundation (Alfalah University).

Mr. MIRZA AMIR BEG,
Asst. Professor (Alfalah University)



10.22214/IJRASET

45.98

IMPACT FACTOR: 7.129

IMPACT FACTOR: 7.429