Dynamic Performance of High-Speed Rail Cable Stayed Bridge with Altered Track Assembly

Jigar P. Variyavwala, Atul K. Desai

Abstract: On fast moving world, the interest is being increased for high-speed trains. It provides a sustainable and effective mode of travelling. Till date India was special among the developing nations for not having a high-speed rail corridor. However, by introducing Mumbai–Ahmedabad high-speed rail corridor, it opens up research directed towards efficient design of the supporting structures. In India, construction and upgradation of railways on its top which attracts the researchers for advance research in the same field. Here an attempt is made to understand the railway track formulation and bridging aspect for high speed rail model as per European standards to assess the influence or impact of rail axle load on bridge behavior. Under this scope, a hypothetical case study of "H-Shaped Cable Stayed Bridge" is taken with Rheda 2000 non-ballasted track, vibrating and non-vibrating ballasted track. The analysis is performed by two approaches; first is the track characterized by its finite element model and second is by introducing track through its equivalent weight. The ballast-less track is already a candid approach for rail lines, showing competitiveness against the conventional ballasted track. The reason for the adaptability of the Ballast-less track is its low maintenance and high accessibility of materials. However, on the basis of vibration analysis "Vibrating Ballasted Track" proved decent in terms of performance.

Index Terms*: *Cable Stayed Bridge, Dynamic Analysis, High Speed Rail Model, simplified Track models.

I. INTRODUCTION

Bridges and rail transportation are the key parameter of any nation's infrastructure and global prestige. Indian Railways is one of the world's leading railway networks, with 115000 kms of tracks and a route of 65,000 kms. Indian Railways carry greater than 25 million passengers every day which is beyond the entire population of Australia. But change is the only constant thing; Indian railway also on the path of revolution. For Mumbai–Ahmedabad high-speed rail corridor Indian government has secured 92 % elevated route, 6 % passing through tunnels and 2 % on ground. So, it leads to intensive research in these area.

The study of the dynamic effects in bridges has been performed by many authors and the need to guarantee the service limit states on the structure, especially in resonance conditions, has pushed forward further research on this area. The beginning of the research on the dynamic effects on railway bridges is attributed to Willis in 1847, through an experimental manner.

In 1849, Stokes had presented one analytical solution for simply supported beam with the circulating moving mass. Afterwards, Timoshenko had done a remarkable research, which is a basis for many subsequent studies which is the case of a simply supported beam subject to harmonic loads, moving at constant speed, which simulated the effects of a multiple axle train circulation. A detailed set of analytic solutions which allowed the computations of accelerations, displacements and bending moments along a beam with an Euler-Bernoulli behavior to diverse loading cases, which could vary randomly on time. Author had also considered different support conditions and span lengths. [7]

More complex formulations appeared with the progress of automatic calculations and finite element methods. These formulations began with the interaction between the vehicle and the deck, allowing a more realistic determination of the dynamic effects, not only on the deck but also on the vehicle in circulation. Now a day, 2D and 3D models are used to project/express this interaction. The 2D models have been defined to represent the train consisting of suspended and non-suspended masses. [6] In the same context, 3D models have been generated in a 3D space. [2, 12, 13] this study is enhanced by the conditions of resonance and cancellation phenomena and defined analytical equations that allowed the understanding of these concepts.

In the beginning of the 1970s decade, the Office of Research and Experiments (ORE) of the International Union of Railways studied the dynamic behavior of bridges. Later on, the Specialists' Committee D214, following the studies made by UIC (Union Internationale des chemins de Fer). That has defined design criteria which allow the adequate dynamic safety of bridges to circulation speeds up to 350km/h with the aim of standardizing all under these concepts. Later on CEN (European Committee for Standardization) had presented documents such as EN 1991-2. [4, 5]

More recently, several studies have been conducted to identify the parameters that most influence the dynamic behavior of the structure, and also define the evaluation criteria to take into account the dynamic effects. [10] The circulation of a high-speed train compliance for very strict criteria of comfort and safety, and the railway plays a very important role in guaranteeing these criteria. A traditional ballasted track was adopted on the first high-speed line, which later revealed a faster deterioration of the track for high-speed trains. Whereas, when designing the first French high-speed line, it was noted that the maximum speed of circulation without the appearance of a significant increase in vertical internal forces. [9] The action must be carried out basically in the reduction of two parameters: first, the non-suspended vehicle mass and secondly, the vertical stiffness of the track.

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The reduction of the first parameter has been achieved with the development of new types of trains. However, the second parameter has not been reduced always due to the need of increasing the load capacity of the infrastructure.[6, 13, 17] In Germany, it was verified that increasing of track stiffness, it impacts negatively & leads to a faster deterioration of the railway track.

To understand the effects of above two parameters on the dynamic behavior of the support structure, the first studies appear in 1867 by Winkler who advocated the modelled track as a supported beam on an elastic ground. Later on, similar model was used which also took into consideration the damping of the structure that supports the rail. [7, 13] The fact that the rails were being discretely supported on the sleepers leads to the development of simplified models. Author modelled the support structure of the track, not in a continuous manner but in a discrete one. [14,17] With the need to solve some of the ballasted track limitations, ballast-less track was used for the first time in 1972 along 700m on the Rheda Station. Nowadays, the Rheda 2000 is a typology largely used in high speed lines. More recently, high speed rail is applied to the cable supported structure. Rail induced vibration on deck is predominant in bridge designing aspects. The vibration analysis is more pronounced to track properties, train modelling approach and vehicle bridge interaction. [1, 16, 18]

II. DESIGN ASPECTS

A. Actions

As all the trains under railway network circulation acquiring different design and carrying different load characteristics, developing different dynamic effects on tracks in each case. For dynamic loads, a series of models named by HSLM are presented by European standards as per Fig. 2, those were developed by taking into account the inter-operability of trains with different characteristics. It also covers the characteristics of the existing trains, their forecasted evolutions and characteristics of the future ultra-modern trains those will be circulated on the high speed lines.

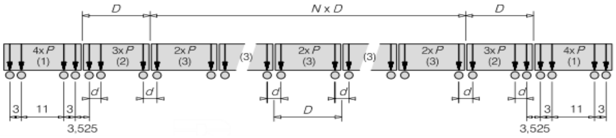

Fig. 1 HSLM-A model, EN 1991-2 [5]

B. Dynamic effects

In the EN 1991-2, various specifications are presented for the efficient railway bridge design. Considering the dynamic effects resulting on the circulation of trains, in a static analysis, the dynamic effects can be considered through

Dynamic amplification coefficient (ϕ):

For carefully maintained track:

$$\phi_2 = \frac{1.44}{\sqrt{L_\phi} - 0.2} + 0.82 \qquad \text{With } 1.00 \le \phi_2 \le 1.67 \qquad (1)$$

For track with standard maintenance:

$$\phi_3 = \frac{2.16}{\sqrt{L_\phi - 0.2}} + 0.73 \qquad \text{With } 1.00 \le \phi_3 \le 2.00 \qquad (2)$$

Where, L_ϕ is a determinant length. In high-speed bridges, the resonance phenomena are not taken into account on a static analysis. To take into account this effect, a dynamic analysis, using the load HSLM models or real trains, should be made. The results are analyzed for a range of circulating speed between 200 km/h to 400 km/h. Track irregularities can be considered using an interaction model train-track-bridge or considering the coefficient presented on the same norm that represents the track irregularities ($\phi''/2$)

C. Design requirements on a dynamic analysis

An excessive deformation of the bridge can compromise the stability of the train circulation. Such deformation changes the track geometry provoking excessive vibrations on the elements which constitute the bridge and in the train itself, resulting on passengers discomfort. In the EN1990-A2, verifications to the limit states of service are prescribed, defining limits to the deck's vertical accelerations, displacements on the supports or deformations on the bridge to assure the safe traffic circulation and the track stability. To guarantee a certain level of passenger comfort, limits regarding the vertical accelerations at the train are also defined.

III. RAILWAY TRACK

The rail track is designed to meet various criteria such as comfort, resistance, construction, velocity and maintenance costs. Railways can be divided into two groups on the basis of track, with ballast and ballast less tracks. Their performance and behavior depends on the properties of the layers that constitute them, as well as their interaction when subjected to different train traffic. Since beginning to till date railway as competent manner of transportation using ballasted track passionately. For railway bridges, the taxonomy is categorized by the superstructure and substructure. Rails, sleepers and fastening systems falls under the Superstructure and the ballast layer is the substructure. The ballast less track was developed in the end of the 1960s and the first solution of this pattern was used along 700m in the Rheda Station in Germany. The Rheda Solution 2000, used nowadays in high speed railway bridges, is made of pre-compressed sleepers embedded on a reinforced concrete layer. The fastening system is made of materials with resilient properties.

A. Ballasted track elements

Although the taxonomy adopted for ballasted railway track accepted broadly, but the demarcation and properties of each element used has some erraticism.

Rails, Fastening Systems, Sleepers and Ballast layer

Rails are the first components which having contact with the vehicle. Its principal tasks are to transmit and disseminate the vertical and horizontal forces and to assist the vehicle's wheels. The adoption of the UIC60 rail is defensible for technical and economical motives. [9] The fastening system ensures a secure connection between the rail and the sleepers limits the dynamic effect by the movement of trains at high-speed. The stiffness of the rail pads ranges from 30 and 500 KN/mm.

Sleepers distribute the stresses coming from the rail to the ballasted layer underneath them. The sleepers have a significant stiffness. The mono-block sleepers are the most common typology used and the distance between two successive sleepers varies in the range of 50 and 70 cm. The ballast layer is most superior part in railway substructure which is designed with the aim to transmit the loads to the supporting structure. As per EN 1991-2 for a sustainable distribution of stresses without damaging a bridge surface or a subsequent ballast mats, the depth of ballast layer must not be lesser then 250 mm. All together, the use of 350 mm depth allows an effective conservation of the track.

B. Ballast less track elements

The Rheda 2000 solution, when applied on high speed railway bridges, is made of a Rheda 2000 slab and by a geo-textile. The main function of the geo-textile is to reduce the effects of the interaction between the supporting structure and the Rheda 2000 slab, preventing the degradation between them, due to the cyclic impacts resulting from the train circulation a value of the vertical stiffness of the geo-textile is 2e10 KN/m. [15]

IV. MODELLING OF THE RAILWAY TRACK

The railway has been modelled by numerically and analytically for the behavioral study and element characterization as well as the vehicle-track-bridge interaction. Facing the wanted objectives, approximated models can be used depending on the requirements and its significance to the results which are pretended. With this intention, the study was made with 2D simplified railway models.

Modelling the ballasted railway track

Based on previous studies two simplified models were used aiming at the understanding of the modelling effects of various elements belonging to the ballasted track.

Non-vibrating Ballast model and Vibrating ballast model

A rail is modeled in such way that it behaves like a beam of Euler-Bernoulli or Timoshenko. [14] The length of the beam is selected such that the structural behavior not affected by the edge restrictions. Sleepers are connected to the beam, like suspended masses. The properties of the pad and ballast are represented by the parallel spring and damper systems at the top and bottom, respectively. For model analysis, the distance between each system is defined by the space between the sleepers. Fig. 3 shows the 2D model of the non-vibrating ballast in which the ballast are modeled as a suspended mass which are connected to the sleepers at the top and to the subsoil/ bridge at the bottom by the parallel system of spring-damper. [15]

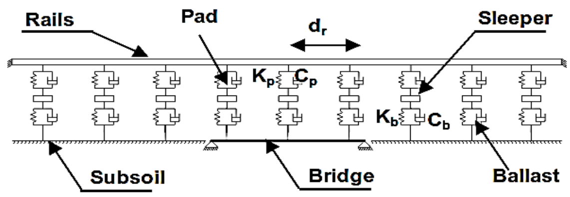

Fig. 2 2D abridged model of a ballasted track without considering the ballast as a vibrating mass [15]

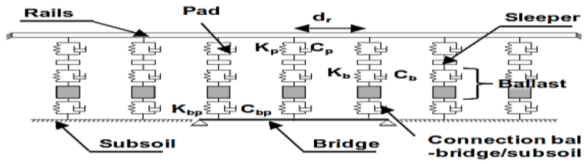

Fig. 3 2D abridged model of a ballasted track with considering the ballast as a vibrating mass [15]

Definition of the properties of the ballast

The calculation of vibrating mass is subjected to the half-sleeper which is associated to the attenuation angle (α) as shown in the Fig. 5. Equation for vibrating mass is given below.

$$M_b = \rho_b h_b [l_e l_b + (l_e l_b) h_b tg\alpha + \frac{4}{3} h_b^2 tg^2 \alpha] \qquad (3)$$

In which ρ_b, h_b, l_e, l_b are the ballast density, depth of ballast, effective supporting length of half sleeper and the sleeper's width at underside respectively. The vertical stiffness of ballasted vibrating mass is calculated by the following equation. [17]

$$K_b = \frac{2(l_e - l_b) tg\alpha}{\ln\left[(l_e l_b).(l_b + 2h_b tg\alpha)/(l_e + 2h_b tg\alpha)\right]} E_b \qquad (4)$$

Where, E_b is the modulus of elasticity of the ballast. Generated Equations are established on hypothesis that there is absence of overlapping over the adjacent cone regions. Values which are incorporate in the equations 3 and 4 are computed as presented in Table 1, values are influenced by the sleeper property. [17]

Table 1 Vibrating mass and vertical stiffness of the ballast corresponds to the influence of a sleeper

Designation	Notation	Value
Effective supporting length of half sleeper	l_e [m]	1.30
Width of sleeper underside	l_b [m]	0.30
Density of ballast	ρ_b [kg/m]	$2.04\,e^3$
Elastic modulus of the ballast	E_b [pa]	$1.10\,e^8$
Ballast stress distribution angle	α [◦]	14.04
Depth of ballast	h_b [m]	0.35
Vertical stiffness of the ballast	K_b [N/m]	$3.30\,e^8$
Vibrating mass of ballast	M_b [kg]	770.94

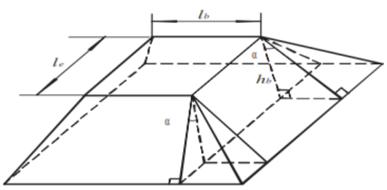

Fig. 4 Ballast model under one rail support point.

Modelling of the ballast-less railway track

The bi-dimensional model represented in Fig. 6 which represents the behavior of the ballast less track for Rheda 2000 typology. In this model, the rails are represented as beam elements with an Euler-Bernoulli behavior. The Rheda 2000 slab is modelled as a segmented beam of 6.5 m length and with a Timoshenko behavior. The connection of this element to the rails and to the bridge's deck is considered discretely through systems of parallel spring damper, with the resilient properties of the pads and of the geo-textile respectively. [15]

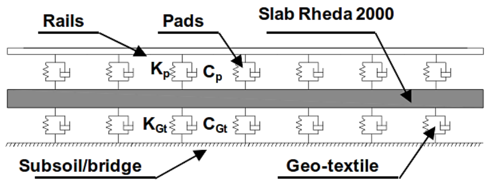

Fig. 5 2D Abridged Model of a ballast-less track [15]

V. GOVERNING EQUATIONS FOR CABLE STYED BRIDGE

CSB can be analyzed by the two approaches which a FE method and stiffness method. For both approaches bridge idealized as a 2D system. Modelling approach for the Deck and towers are an Euler-Bernoulli beam element to carry an axial force. Analysis results from the stiffness method are not satisfactory because of reduction of number of element in global structural assembly and vibration characteristic of cables. [1] therefore FEM approach is selected for bridge analysis. In the same approach cables are modelled by two ways, first is an equivalent modulus and second is cable elements with the original modulus.

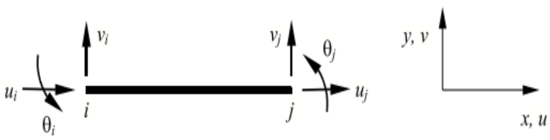

Fig. 6 Beam element with six Degree of Freedom

Let assume the small beam element i-j having L length with six degree freedom system, whose stiffness is incorporate with the summation of stiffness due to bending and prestressing force. The governing equations are given for the stiffness calculation individually.

Here, six end displacements or DOF a cubical variation in displacement is expressed by

$$v = Aa \tag{5}$$

Here, A = (1 x x_2 x_3 x_4 x_5) and a^T = (a_1 a_2 a_3 a_4 a_5 a_6) (Displacement variation within element. There are six DOF corresponding to the displacements u_i, u_j, v_i, v_j and the rotations θ_i, θ_j. Given by,

q = Ca (Nodal displacements) (6)

Where q^T = (v_i u_i θ_i v_j u_j θ_j) and C is the connectivity matrix. From Eq. 5 and 6 we have,

$$V = AC^{-1}q \tag{7}$$

If I is the moment of inertia and E is the Young's modulus then bending moment M for the element is given by

$$M = D\frac{\partial^2 v}{\partial x^2} = DBC^{-1}q \tag{8}$$

Where D = $EI(x)$ and B= $\frac{d^2 a}{dx^2}$

Beam Stiffness as a result of bending

The potential energy UB due to bending is

$$U_B = \frac{1}{2}\int_0^l \frac{d^2 v}{dx^2} M dx \tag{9}$$

And the stiffness is given by

$$kb = \frac{\partial^2 U_B}{\partial d^2} \tag{10}$$

By Eq. 9 and Eq. 10 we get,

$$k\bar{b} = \int_0^l B^T DBdx \quad \text{(Elemental)} \tag{11}$$

$$Kb = (C^{-1})^T k\bar{b} C^{-1} \quad \text{(Assembled)} \tag{12}$$

Beam Stiffness as a result of prestressing force

The potential energy of prestressing force is expressed by,

$$U_Q = \frac{1}{2}\int_0^l Q\left(\frac{\partial v}{\partial x}\right)^2 dx \tag{13}$$

So, the stiffness is presented by,

$$k_Q = \frac{\partial^2 U_Q}{\partial d^2} \tag{14}$$

By putting eq. 13 in 14 we get,

$$\bar{k}_Q = \int_0^l A^T k_Q A dx \tag{15}$$

$$K_Q = (C^{-1})^T k_Q C^{-1} \tag{16}$$

Final complete stiffness is given by

$$K = K_B + K_Q \tag{17}$$

Mass matrix for an element is the equivalent to the nodal mass which represents the actual distributed mass of the element. Those are represented as kinetic energy of the element.

$$T = \frac{1}{2}\int_0^l (\dot{v})^T \rho dV \dot{v} \tag{18}$$

Where, ρ = mass density and $\dot{v}$ = Lateral velocity

$$T = \frac{\rho}{2}(\dot{q})^T (C^{-1})^T \left\{ \int_0^l A^T hxAdx \right\} (C)^{-1} q \tag{19}$$

Then, the mass matrix is given by,

$$m = (C^{-1})^T \overline{m} C^{-1} \tag{20}$$

$$\overline{m} = \rho \int_0^l A^T hxAdx \tag{21}$$

Expression For the free vibration of beam is,

$$[M]\{\ddot{q}\} + [C]\{\dot{q}\} + [K]\{q\} = 0 \tag{22}$$

Expression for forced vibration of beam is,

$$[M]\{\ddot{q}\} + [C]\{\dot{q}\} + [K]\{q\} = \{f\} = [N]^{-1} f_0 \tag{23}$$

For cable stayed bridge, cables are thinner and the effect of pre-stressing force is comparatively high so, the effect of Prestress can be calculated as per above given equations.

Vibration of Cables

Cable vibration is calculated by the Equivalent modulus approach. In the CSB analysis, initially cables are modelled as a single truss element with an equivalent modulus to allow a sag effect as suggested by (F. T. K. Au et al. 2001). The stiffness matrix in local coordinates for a cable element can be written as,

$$k_c = \frac{A_c E_{eq}}{l_c} \begin{bmatrix} 1 & 0 & -1 & 0 \\ 0 & 0 & 0 & 0 \\ -1 & 0 & 1 & 0 \\ 0 & 0 & 0 & 0 \end{bmatrix} \tag{27}$$

The equivalent modulus of elasticity is,

$$E_{eq} = \frac{E_c}{1 + (wH_c)^2 A_c E_c / 12T^3} \tag{28}$$

Where, l_c, H_c, A_c, E_c are the chord length, the horizontal projection length, the cross-sectional area and the effective material modulus of elasticity respectively. W stands for the weight per unit length and T is the updated cable tension of the cable. After getting equivalent modulus, the profile will not affect the final analysis.

Mass matrix for cable

The cable element mass matrix is the same for both the single-element and multiple-element modeling methods. The mass matrix is given as follows,

$$m_c = \frac{m_{cc}}{6} \begin{bmatrix} 2 & 0 & 1 & 0 \\ 0 & 2 & 0 & 1 \\ 1 & 0 & 2 & 0 \\ 0 & 1 & 0 & 2 \end{bmatrix} \tag{29}$$

In which m_{cc} is the total mass of the cable element

Vibration of stay cables

To know local cable vibrations, each cable was analyzed with inclined profile and different support condition. But in reality, the end anchorages are movable. The symmetric in-plane vibration and anti-symmetric in-plane vibration frequencies ω (radians/second) can be expressed,

$$\omega = \frac{\omega^*}{l}\sqrt{\frac{T_\theta}{m}}$$ For symmetric in-plane vibration and,

$$\omega = \frac{2\pi}{l}\sqrt{\frac{T_\theta}{m}}$$ For anti-symmetric in-plane vibration,

Where
$$\tan\left(\frac{\omega^*}{2}\right) = \left(\frac{\omega^*}{2}\right) - \frac{4}{\lambda^2}\left(\frac{\omega^*}{2}\right)^3 \tag{30}$$

Here, l and m represents the chord length, static cable tension and the mass of cable per unit length respectively. [1]

VI. DYNAMIC ANALYSIS OF A CABLE STAYED BRIDGE, RESULTS & DISCUSSIONS.

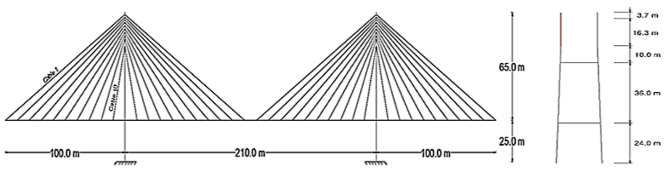

Fig. 7 Simplified model of a cable-stayed bridge

(Dimensions in m)

The streamlined model of the symmetric cable-stayed bridge is considered as shown in fig. 8. The bridge having two anchor piers at the end and two H- shaped pylon, a semi-fan cable arrangement has been provided. Pylon is connected with two cross beam at upper side to resist moment and just beneath the deck. Fixity is assumed at pylon base. Four elastic link is provided, two at the deck- pylon interface and two at the anchor pier respectively. The modulus of elasticity of the deck, cross girder beam and all cables is taken as 200 GPa. The modulus of elasticity for the towers and cross tower beam is taken as 27 GPa. The significant properties of the bridge are given in Table 2.

Table 2 Properties of the cable-stayed bridge

	Bridge component	Area [m^2]	I_{XX} [m^4]	I_{YY} [m^4]	I_{ZZ} [m^4]
1	Cable	0.0052	0.0	0.0	0.0
2	Girder	0.3092	0.0070	0.1577	4.7620
3	pylon	9.2000	19.5100	25.5670	8.1230
4	Girder beam	0.0499	0.0031	0.0447	0.1331
5	Pylon beam	7.2000	15.7900	14.4720	7.9920

A. *Dynamic analysis of the cable stayed bridge without consideration of the railway models*

Considering the track in which a load is distributed uniformly along the deck. For a ballasted track, the EN1991-2 suggests that the analysis should be performed by considering the least density of a clean ballast with a minimum depth and an extreme saturated density / dirty ballast with maximum possible depth of ballast layer.

For the research purpose, the depth of ballast should be taken 350 mm with two different ballast densities $\rho_{ballast} = 17$ kN/m^3 & $\rho_{ballast} = 20$ kN/m^3. For the Ballast-less track Rheda 2000, ballast density should be $\rho = 35$ kN/m^3. Three models have been prepared, model 1 (vibrating ballasted track), model 2 (non-vibrating ballasted track) & model 3 (Ballast less track). The weight of the ballasted track and the ballast-less track corresponds to 30.7% and 12.3% of the total structure's weight. The usage of the ballasted track may carry very high costs on the Verification to the ultimate limit states and it is much heavier.

The maximum accelerations and the vertical displacements obtained for the model 1 due to the circulation of the HSLM-A load models are presented in Fig. 9 and Fig. 10. It is verified that the maximum acceleration peak occurs for the HSLM-A8 load model to a circulation speed V=300 km/h and with the $a_v = 6.3$ m/s^2. This value is exceed the 3.50 m/s^2 limit presented on the EN 1991-2 for a ballasted track. Relatively the maximum vertical displacements obtained, was noticed on the HSLM-A2 load model to a speed of 400 km/h and with a value of δ_v =31 mm.

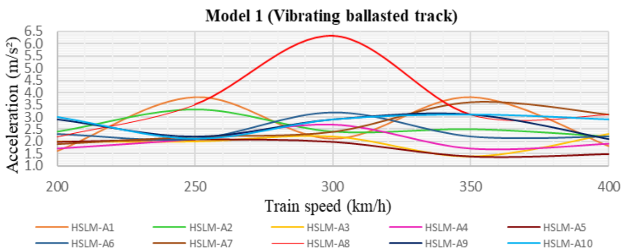

Fig. 8 Maximum accelerations along the load path of model 1, due to the circulation of HSLM-A trains to different circulation speeds

Concern with real trains, the maximum vertical displacement of the structure occurs for the ICE-2, at a speed of 400 km/h, and with a value of $\delta_{lim} = 25$ mm which is lower to the one obtained to the HSLM-A load models. It was thus concluded that the accelerations and the maximum displacements neither occur necessarily due to the same load model nor with the same resonance speed. The obtained results using the HSLM-A load models are an envelope of those obtained for the real trains.

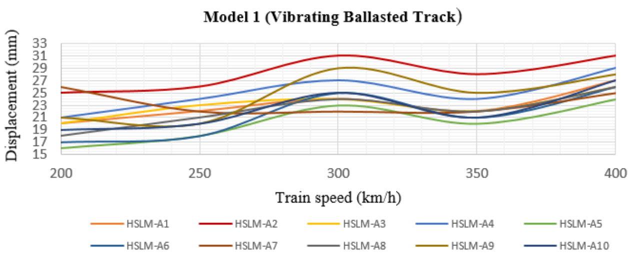

Figure 9 Maximum displacements along the load path of model 1, due to the circulation of HSLM-A trains to different circulation speeds

The accelerations and the maximum vertical displacements obtained in model 2 due to the HSLM-A load models circulation are shown in Fig. 11 and Fig. 12. It was noticed that the results obtained for model 1 by considering a heavier track, corresponding value of maximum vertical acceleration reduced however the maximum vertical displacements remained approximately unchanged.

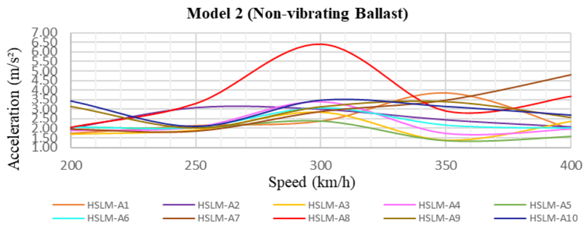

Fig. 10 Maximum accelerations along the load path of model 2, due to the circulation of HSLM-A trains to different circulation speeds

It would be expected that the use of a heavier track would allow a reduction of the maximum accelerations. Nevertheless, as the resonance peaks occur at lower speeds, it is noticed that appearance of a new resonance peak on the HSLM-A8 load model in a speed of V = 300 km/h, with an acceleration value of $a_v = 6.41$ m/s^2, which is higher than the maximum value obtained in model 1. The use of a heavier track is not always the best solution to reduce the maximum accelerations on the bridge.

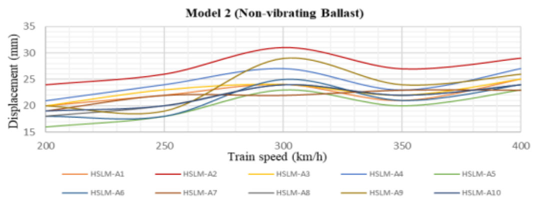

Fig. 11 Maximum displacements along the load path of model 2, due to the circulation of HSLM-A trains to different circulation speeds

The use of a ballast-less track brings some advantages, since a lighter railway leads to an increase of the speed of the resonance peaks. The variation of the resonance speed between model 2 and the ballast-less track corresponds to $\Delta V = 50$ km/h. In the same HSLM-A load model, maximum vertical deck accelerations are increased on the ballasted track model 2 which is reflected in fig. 13. To the same resonance maximum peak, ballast-less track presents an acceleration of $a_v = 5.12$ m/s^2. It is possible to guarantee the 5 m/s^2 limit, presented on the EN1991-2.

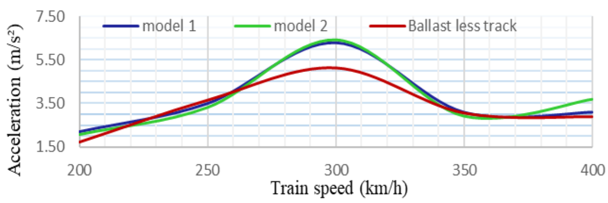

Fig. 12 Comparison of the maximum vertical accelerations obtained by all three models due to the circulation of HSLM-A8 load model

The vertical displacements obtained to the same HSLM-A load model are compared and it is notice that the use of different railways typologies does not influence the maximum computed values, nonetheless their occurrence to different circulation speeds which can be seen in Fig. 14.

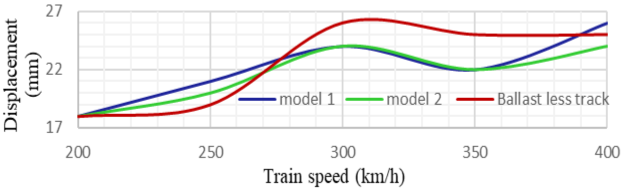

Fig. 13 Comparison of the maximum vertical displacement obtained by the all three models due to the circulation of HSLM-A8 load model

B. Dynamic analysis of the cable stayed bridge considering the railway models.

The parameters considered in the study of the effect of the various belonging elements of the ballasted and ballast less track on the dynamic behavior of the structure are presented in Table 3. Table 4 correspond to the 10 first time period of the structure with vibrating ballast, non-vibrating ballast and non-ballasted track models. Minimum variations on the values of each vibration modes were noticed, variation of first mode shapes is remarkable. Such results indicate that the consideration of more refined railway models do not significantly affect the global structural behavior.

Table 3 Adopted parameters for the ballasted and the ballast less track model [15]

Railway parameters		Notation	values: Non vibrating ballast	values: Vibrating ballast	values: Ballast less track
Rail UIC60	Section Mass Elasticity modulus Poisson's coefficient c/s inertia at XX c/s inertia at YY	S_r [cm²] m_r [kg/m] E_r [GPa] μ_r[-] I_{xx} [cm⁴] I_{yy} [cm⁴]	76.86 60.34 210.00 0.30 3050 515.6	76.86 60.34 210.00 0.30 3050 515.6	76.86 60.34 210.00 0.30 3050 515.60
Pad	Vertical stiffness Vertical damping	Kp (MN/m) Cp (kNs/m)	65 5.50	65 5.50	100 15
Sleeper	Mass Sleeper spacing	Ms [kg] Ls [m]	300 0.60	300 0.60	- 0.65
Ballast	Vertical stiffness Vertical damping Vibrating mass	K_b [N/m] C_b [Ns/m] M_b [kg]	3.30e08 1.20e052 -	3.30e08 1.20e05 770.94	- - -
Connection Ballast/deck	Vertical stiffness Vertical damping	K_{bp} [N/m] C_{bp} [Ns/m]	- -	1000e06 50e03	- -
Slab Rheda 2000	Specific weight Width x height Elasticity modulus Poisson's' coefficient	ρL [kN/m³] Lxh [mxm] EL[GPa] μL[-]	- - - -	- - - -	26.37 2.60 x 0.24 34.00 0.20
Geo-textile	Vertical stiffness Vertical damping	K_{Gt} [MN/m] C_{Gt} [kNs/m]	- -	- -	1.3e10 1.3e07

Table 4 Comparison of the time period of the CSB with different track models

Vibration mode	Time period [sec.]: Vibrating ballast	Time period [sec.]: Non-vibrating ballast	Time period [sec.]: Non-ballasted
1	1.957	1.752	1.408
2	1.382	1.374	1.374
3	0.921	1.282	1.242
4	0.901	1.161	1.143
5	0.842	1.039	1.025
6	0.737	0.841	0.790
7	0.633	0.790	0.717
8	0.597	0.787	0.669
9	0.530	0.759	0.601
10	0.522	0.617	0.595

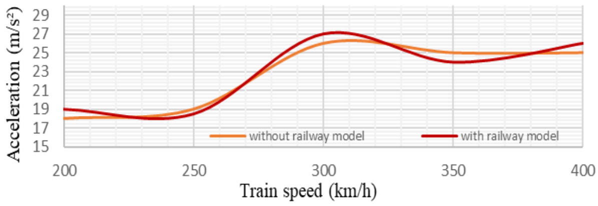

Fig. 14 Comparison of the maximum vertical accelerations with and without railway model for the ballast-less track model due to the circulation of a HSLM-A8 load model

The acceleration values computed using a ballast-less track model present slight variations in the resonance peaks compare to the results obtained on the CSB without considering the track model which is presented in Fig. 15. Such variations are not owed to the track vibration but to the stiffness increment of the global structure due to the increased flexural stiffness resulting from Rheda 2000 slab and rail.

VII. CONCLUSIONS

The safety verification of high speed railway CSB were presented along with considering the effect of various railway models in its dynamic behavior. The key parameters comprised on the structural dynamic behavior correspond, on the one side to the type of train (axles, axles' spacing and loads) and the corresponding circulation speed range as well as conversely to the structural characteristics, namely the vibration frequencies, mode shapes and damping.

By considering various HSLM-A models and compared them with various existing real trains has allowed to conclude that these high speed load models cover the range of effects induced by existing real trains that correctly leading to envelope all relevant effects like deck acceleration and vertical displacement.

The selection of railway taxonomy influences the dynamic behavior of the structure. Considering a heavier track modelled as a load distributed along the deck, the maximum figured of accelerations are lower and the resonance peaks occur at lower speeds. In reality, the first structural resonance phenomena occur at lower speeds. It may generate new resonance peaks within the considerable speed range, possibly leading to other negative effects. The use of a ballasted track, generally leads to lower resonance speeds (and dynamic effects on deck), which does not necessarily imply any advantage. However, the permissible deck accelerations remain lower due to the possibility of the volatility of the ballast layer.

Results represents the use of bi-dimensional railway models does not significantly change the results relevant to bridge deck design and verification, when these are compared to more simplified models without explicit consideration of the track. Concern is only on the cable stayed bridge under analysis (span length of L= 410m). It has also been concluded that the distribution of the load per axis along the sleepers (figured with the use of explicit rail track models) don't contribute in the structural response degradation.

There is no significant differences have been noticed in these ballasted models. That can be clarified through the railway track behavior as a vibrating sub-system. Considering the fact, the first vibration mode shape of the track as a whole corresponds to a frequency is much higher than the limit considered on the dynamic analysis. It implies that railway track vibrates jointly with the bridge and these (local) mode shapes of the track are not excited.

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