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Study on different solutions to reduce the dynamic impacts in transition zones for high-speed rail

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A B S T R A C T

One of the most important factors influencing the track maintenance is the transitions between parts of the track with different vertical stiffness. The dynamic forces in the super-structure, i.e. from rail to ballast/slab and subgrade, including every layer under ballast/slab until natural ground, are influenced by the type of materials, layer configuration and geometry. One way to mitigate track transition problems is to have a more gradual transition with a reduced stiffness differential. The aim of this research is to reduce vertical transient stresses and displacements under track supports at track transition areas by combining different structural configurations. For this purpose, the train-track dynamic interaction in the transition zones with different vertical stiffness values is analysed using a finite element software. A high-speed train moving on a slab and ballasted track is considered travelling in both directions. The effect of different structural track designs is studied in realistic operation scenarios. The results allow concluding that the sleeper displacements and ballast stresses can be significantly reduced in the transition zones by making small changes in the track structural elements.

1. Introduction

Rail network demands put pressure on the development of innovative solutions for existing and new rail infrastructures in order to increase the traffic speeds and/or the axle loads and to reduce the maintenance costs. One of the major problems occurs in the transition between embankment

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and engineering structure (i.e. bridges, level crossings, etc.), as depicted in Figure 1. Such change between two types of track produces discontinuity in the vertical stiffness of the rail support, increases the effects of the dynamic forces and deteriorates the track geometry due to the track settlement on the embankment side resulting in increased track maintenance costs in the transition zones. A number of authors have devoted their attention to these topics over the last decade[1-14].

The work presented here aims to develop methodologies to reduce the dynamic forces in the transition zones by combining different elements of the infrastructure. The study proposes solutions to create a more gradual transition between different track elements. An overview of the transition-related problems and the most typical solutions is given in the following section. Then, the numerical models developed in this study are described in detail. The case studies and results of the simulations are presented afterwards. Discussion of the results and the main conclusions are given in the final section.

Fig 1: Track section, separation from ballasted track to structure (HSL- Córdoba- Málaga)

2. Description of the problem

2.1 Track transitions

With the growth of train operation speed, dynamic loads increase resulting in track deterioration and shorter time between maintenance procedures and renewal operations. These problems become more severe for high speed operations as the permanent track maintenance and therefore the life cycle costs of the track are significantly increased [15]. Track behaviour can only be understood through knowledge of the different elements that make up a railway system: the vehicle, the track and the interaction between them. Although the correct track configuration is a requirement, it is also important to understand the relationship between the track and the moving vehicles, the wheel-rail contact. There are two types of impacts generated at track transitions: Wheel impact and wheel bounce[16]. It is also relevant to understand that unsprung masses are an important factor in track deterioration.

Train direction is also important in track transition design. Track behaves differently when a train moves from a high stiffness to a low stiffness area [17-19]. In fact, the nature of the track deterioration depends on the direction the train is travelling in. If a train travels from a stiff area to a flexible area (structure-embankment) the wheel falls causing large dynamic loads on the track, so the ballast and sleepers tend to move. If a train travels from a lower stiffness track onto a high-stiffness substructure the wheel is lifted very quickly causing high vertical accelerations in the vehicle components. Also the distance between trains running in opposite directions in double-track high-speed railways affects the vertical displacement in the middle section of bedding[20].

Track stiffness is a global parameter that includes all track deflection usually obtained during rail measurements. It is defined as the ratio between Q (wheel load) and Y (maximum rail deflection in mm). In [21, 22], track stiffness is considered as an indicator of deterioration. Some ways of its measurement and evaluation can be found in[23]. A more detailed geotechnical description can be found in [24].

Problems found in transition areas are generally difficult to solve. There have been many studies researching this subject; examples include work to improve the superstructure and infrastructure elements [18, 25, 26].

2.2 Review of current solutions

Most of the available solutions for transition problems involve relevant changes in track infrastructure and superstructure, depicted in figure 2, to produce a smooth stiffness variation. Some of them are in substructure (under ballast layers) and others are in superstructure (from rail to ballast) and involve geotechnical solutions (soil treatments), changes to the type of material layers (Hot Mixed Asphalt (HMA), geotextiles), modified sleepers, fasteners (rail seat pads, sleeper plate pads) under sleeper pads or ballast mats. Some transition designs focus on controlling the deformation in the approach zone where the track tends to have lower stiffness[27].

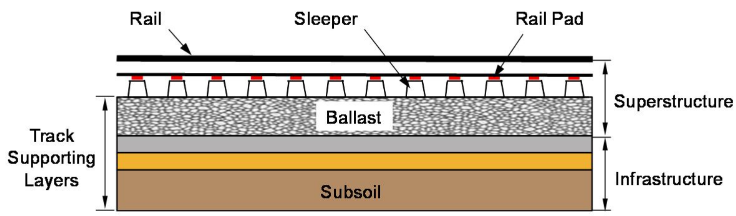


Fig2 .Track infrastructure and superstructure

There are two general ways of improving track transitions: upgrading the infrastructure or the superstructure, and the combination of both[15]. One way to match the stiffness from one part to the other is by improving the subgrade [16]. The track modulus has a great importance in ballasted track. With direct fastenings, the track modulus and deflection are dominated by fastening stiffness. Granular layer thickness, base plate stiffness and sleeper material are secondary influencing parameters. However, for some authors the best way to create a transition without problems is by the careful design of the vertical fasteners in the stiff area [17]. A study about geometric and geotechnical parameters was done on Spanish track transitions (high speed lines) and can be found in[28, 29]. Some recommendations for Spanish network can also be found in ADIF standards[30].

Track faults have to be taken into account first, initial bumps (hanging sleepers) and then a smoother track stiffness change can be made [31]. The most important parameters for track transitions to be studied near bridge abutments are the ballast layers (ballast and sub-ballast). Important variables are the thickness and Young modulus of these layers. Buried structural transitions depend on the thickness of the layer between the sub-ballast and the top of the buried structure. Sometimes track transition could be a problem for track stiffness. This research aims to improve the knowledge about how the proposed solutions influence a transition zone and the relationship between them and differential track stiffness by analyzing innovative layer configurations.

3. Methodology

The methodology consisted on the development of several numerical models (vehicle and track) with the aim of understanding numerically the performance of a track transition. Several solutions to reduce the vertical stress and vertical dynamic displacements have been proposed. Taking as a starting point the state of the art, these solutions were designed and subsequently simulated.

Results are obtained in terms of vertical displacements under sleepers and vertical stresses over ballast. Train body accelerations have been obtained.

The proposed numerical models are described, in subsections 3.1 and 3.2, a description of the case studies is made, in section 4 while the conditions of the numerical simulations are described in section 5. Finally the results and the most important conclusions of the study are presented in section 6 and 7 respectively.

3.1 Numerical models

The dynamic forces acting in the track transition are analysed here numerically. The numerical models are created and analysed using the finite element software DARTS, developed at TU Delft. DARTS program was created for the rapid analysis of the dynamic railway vehicle/track interaction [32]. The software has been successfully used for various railway applications such as the optimisation of a slab track, identification of the dynamic properties of track components [33, 34] and the assessment of various high-speed track structures [35].

The program DARTS was written for the analysis of a rail track on an elastic foundation. Rail tracks can be built up in several ways. The classic track, built up from rail, sleepers, rail pads between sleeper and rail, and the ballast bed and formation (to be called the elastic foundation). However, more recent structures are continuously supported structures, using some fill material between rail and a (rigid) supporting slab.

The stiffness properties of the elastic foundation is modelled either by a Winkler or a Pasternak foundation; the dynamic properties are characterised by damping and inertia properties. To model a train, two ways are possible. The simple way is to model the train by a series of moving loads, travelling on the rail (where no interaction between train and track is considered) and an accurate way is possible by a moving structure, built up from masses, springs and dampers (see figure 3). The interaction between train and track is caused by the roughness of the rail surface (here contact theory is needed).

There are some initial assumptions to make at the beginning:

- The structure is loaded by vertical loads only
- The contributions of the axial forces in the layers are neglected
- In the horizontal plane the structure properties are symmetric with respect to the track axis
- Bending length of deformations stiff layer > thickness of stiff layer
- First we have to model the structure and the loads into equations of motion. Second we have to solve these equations in a numerical way

Despite the fact that these models have not been tested in a real scale (real track) DARTS program has been previously used to study dynamical analysis of High Speed Line slab track design (Settlement Free Plate) in the Netherlands and also in the High Speed Line Ankara-Konya in Turkey.

3.2 Track model

DARTS program allows analyzing a rail track system on an elastic foundation. In order to analyze a track transition, the following two types of tracks with different vertical stiffness have been considered: (a) Slab track (modern railway concrete track structure), (b) Ballast track (classical ballast railway structure).

To make the track models, the track is represented as a combination of stiff and soft layers.

The role of the stiff layers is to redistribute vertical loads by bending and shear deformation of the layer. The stiff layers represent the rail, the slab and the foundation. To model the stiff layer properties the beam elements that take into account the bending stiffness and shear deformation following the Timoshenko beam theory are used. The nodal displacements and nodal forces of a beam element are calculated during the simulations. The inertia properties are modelled by translational inertia and rotational inertia, which results in a mass matrix.

To create the soft layers, the spring and damper combinations are used, taking into account the vertical stresses and displacements. The equations of motion then reads.

$$K \cdot w + c \cdot \dot{\mathrm{w}} + m \cdot \ddot{w} = f \tag{1}$$

where k is spring stiffness, c damping factor and m is density of material. Usually these parameters are given per area unit of the surface. Numerically these parameters are processed in constant shear elements that are

inserted between the bending layers. Two assumptions are made:

the development of the normal forces does not contribute significantly.

the development of the bending moment of the soft layer does not contribute significantly.

Initially, the track model without transition solutions was studied. This was considered as the reference case for comparison with the other solutions proposed here. The train motion over the transition zone in both directions was modelled. In the first case the train travels from the slab track to the ballasted track and in the second case the train moves from ballasted track to the slab track. A schematic representation of the track and vehicle models is shown in figure 3.

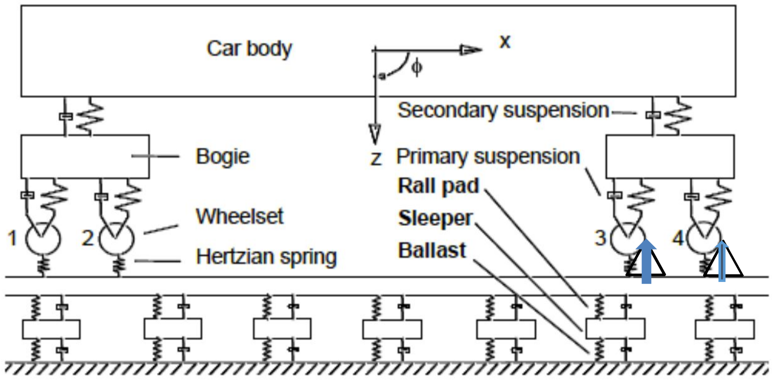


Fig 3. Vehicle and track model used in the study

The length of the finite elements (the rails, sleepers and rail pads) in the models is 0.2 m. In total 1332 elements were used for rail representation, so that the total length of the track is 266.4 m and the transition point is located at 140 m. The details of the elements numbering and the model length in both cases are given in table 1 and table 2. Notice that these track configurations are used in all simulations and that the transition is located at the elements numbered 700 and 701.

Table 1. Element details of transition model from slab to ballasted track.

Rail elements	Track	Length (m)	Length (m)
0-700	slab	700x 0.2	140
701-1332	ballast	632x0.2	126.4
total			266.4

Table 2. Element details of transition model from ballasted to slab track.

Rail elements	Track	Length (m)	Length (m)
0-700	ballast	700x0.2	140
701-1332	slab	632x0.2	126.4
total			266.4

The mechanical properties of the different elements used in the track model of the reference case are shown in table 3, whereas the subgrade properties are given in table 4.

The models can have up to 3 layers, each of them is a beam on an elastic foundation. The stiffness of the elastic foundation is modelled according to the Winkler or the Pasternak foundation models. The dynamic properties are characterised by damping and inertia properties. One of the advantages of the method used here is the fast computing and analysis. Highlights and shortcomings of the method are explained and discussed in[15].

Table 3. Mechanical properties of track models in the reference case.

Rail / Units	Direct fastenings		Pads		Slab		Sleepers		Ballast		Elastic Bed	
	K KN/m	C KNs/m	K KN/m	C KN·s/m	E KN/m	Poisson's ratio	E KN/m	Poisson's ratio	K KN/m	C KNs/m	K KN/m	C KNs/m
UIC 60 (Slab Track)	$3x10^5$	40	-	-	4 $x10^7$	0,2	-	-	-	-	2 $x10^7$	200
UIC 60 (Ballasted Track)	-	-	3 $x10^5$	40	-	-	4 $x10^7$	0.2	2 $x10^5$	80	-	-

Table 4. Subgrade - foundation and formation properties used in the model

Foundation Properties	E (kN/m^2)	Poisson	Density (kN/m3)	Wide (m)	Thickness (m)
	857.000	0.2	25	4	0.2
Formation Properties	**Kf (kN/m)**	**C (kN.s/m)**	**Density (kN/m3)**		
	250.000	200	1	-	-

3.3 Vehicle model

The vehicle is modelled as a mass and spring/damper system, as shown in figure 3, representing a Thalys locomotive, which properties are given in table 5. In general, an operation velocity of 70 m/s (252 km/h) is considered in the numerical simulations performed here. This speed is decreased to 50 m/s (180 km/h) and increased to 90 m/s (324 km/h) in case 9.

Table 5. Vehicle parameters corresponding to a Thalys locomotive

Vehicle			
Bogie spacing (m)- Bogie length	14.00	Body spacing (m) - Vehicle length	22.00
Offset (m)-Distance from reference vehicle position	4.00		
Body properties		Bogie properties	
k1 (kN/m)-Stiffness of secondary suspension	600.00	K (kN/m) - Stiffness of primary suspension	1150.00
c1 (kN.s/m) - Damping properties of secondary suspension	48.00	C (kN·s/m) - Damping properties of primary suspension	15.00
M (Ton) -Vehicle mass	54.15	Mb (Ton) - Bogie mass	2.80
Front (*)	1.00	Mw (Ton) -Wheel mass	1.03
Back (*)	1.00	R (m) - Wheel radius	0.42
(*) Distance from the beginning and the end of a vehicle for placing bogies.		Dw - Distance between axle wheels	3.00

The wheel-rail contact is modelled using a Hertz contact theory[36]. The program calculates Hertz spring parameter through.

$$K_H = \sqrt[3]{\frac{6 \cdot P \cdot E^2 \cdot \sqrt{R_{wheel} \cdot R_{rail}}}{4 \cdot (1-\nu^2)^2}} \tag{2}$$

where E is the Young modulus of steel (2.1·108 kN/m), ν is the Poisson modulus (0.3), R_{wheel} is the radius of the wheel of the vehicle (0.42 m) and R_{rail} is the rail radius (0.3 m), in this case from UIC60. The force P is the wheel load. Basically, P is time dependent, nevertheless DARTS uses the constant static value. All values of masses used in (2) are given in table 5.

To take into account the deformation of a moving vehicle or train it is assumed that the dead weight load deformation of vehicle and track are considered independent from each other; both are defined and solved at the beginning of the integration process. For this analysis DARTS assumes the train to be supported at the wheel axes and ignores the Hertz springs. From this analysis the reaction forces P at the wheel axles (see figure 3 right bogie) are obtained, which will be applied for the computation of the Hertz spring stiffness. The dynamic deformations of the vehicle are taken with respect to this initial deformation shape.

Boundary disturbances are solved in an efficient way by the application of 'cyclic' boundary conditions[32]. Cyclic boundary conditions mean that the deformations at start and end of the analyzed track model are put equal to each other. In fact, a repeating structure with repeating loads is defined which means that a second application of cyclic boundary condition is associated with moving loads and a moving load system may cross the border of the analyzed structure and 'reappear' at the beginning of the structure, allowing to analyze long structures.

4. Case studies

The purpose of the case studies performed here is to analyze the track responses to different transition solutions involving diverse configurations of infrastructure, superstructure, track materials and geometry. The results of these simulations are compared with the track performance of the initial (reference) case where the track has no transition measures applied. Ten cases studies have been considered, as summarized in Table 6. These cases combine superstructure elements in the transition area, namely, additional rails[37], longer sleepers[17], lighter sleepers, etc.

Table 6. List of case studies

Case study	Description
1	No transition
2	Double Rail
3	Sleepers with variable length
4	Mixed solution: Case 2 + Case 3 (Double rail + sleepers' length variation)
5	Lighter sleepers
6	Extra-long sleepers with variable length (6 m long)
7	Case 2 + Case 6 (Double rail + Extra-long sleepers with variable length)
8	Embedded rail (based on case 1)
9	Train speed variation (based on case 1)
10	Influence of rail surface defects (based on case 1)

Case 1: The initial case, or reference case, has a slab track and ballasted track zone. There is an abrupt change in stiffness. Here, there is no transition solution between these two types of track.

These changes in track stiffness can easily be found at stations and on track structures such as bridges, viaducts, tunnels, etc.

Case 2: One of the most commonly used solutions is double rail where the superstructure is strengthened near the track transition on both sides, ballasted and slab track. This solution consists of installing several rails between the running rails and on the field side of the running rails to stiffen the ballasted track panel. In this case, the length considered on ballast track side is 15 m and for the slab area is 5 m.

Case 3: In this case, sleepers with variable length are considered. The sleeper length is variable from the slab width (4 m) to the normal sleeper length considered (2 m). At the starting point next to the slab there are 4 m long sleepers, the length then starts to decrease to 2 m. Table 7 gives information about sleeper's length and thickness used in this case study.

Table 7. Case 3 - Sleeper's length variation in the track transition model.

Type	Number Ballast-Slab	Number Slab-Ballast	Length Cross Section) (m)	Thickness (m)
Slab	700-1332	1-700	4.0	0.400
Sleepers	688-699	701-713	4.0	0.208
	676-687	714-725	3.5	0.208
	664-675	726-737	3.0	0.208
	652-663	738-749	2.5	0.208
	1-651	750-1332	2.0	0.208

In case 4, the double rail and sleeper length variation solutions are considered. The improvement with double rail affects up to 20 m overall, 5 m on slab track and 15 m on ballasted track. Sleeper length variation affects 10.2 m on the ballasted track. If these two solutions are combined the improvement is on a 10.2 m length in the ballasted area, i.e., the intersection zone of both solutions.

Case 5: It consists of using lighter sleepers with a density of about 57 Kg/m^3 (composite materials) to create additional elasticity in the ballasted area and try to reduce stresses. Sixteen lighter sleepers have been placed in the ballasted area. The train travels in both directions from slab track to ballasted track and vice-versa.

Case 6: In this case, sixteen extra-long sleepers have been considered to improve case 3. The sleeper´s length has been varied from 6 m (larger than the 4 m slab width) to the normal sleeper length of about 2 m. This was done for both travel directions and 10.2 m have been improved in the ballasted area. The variable lengths are shown in Table 8.

In case 6, longer sleepers of about 5 and 6 m, could have bending stresses problems. It is not easy to make such long sleepers, however a sleeper manufacturer ensured that pre-stressed concrete sleepers will resist these stresses. In this case the main problem occurs during maintenance works when using tamping machines. Ballast under the sleepers is squeezed and its effect with longer sleepers is unknown. Despite of this, these oversized sleepers are placed close to the slab track and tamping is not allowed in the surroundings of the slab.

Table 8. Extra-long sleepers introduced in case 6.

Type	Number Ballast-Slab	Number Slab-Ballast	Length (Cross Section) (m)	Thickness (m)
Slab	700-1332	1-700	4.0	0.400
Sleepers	688-699	701-713	6.0	0.208
	676-687	714-725	5.0	0.208
	664-675	726-737	4.0	0.208
	652-663	738-749	3.0	0.208
	1-651	750-1332	2.0	0.208

Case 7: It represents a solution with extra-long sleepers and double rail. The overall improved length of 20 m considered in case 2 was 15 m in the ballasted track and 5 m in the slab track. In case 6 the improved length is 10.2 m on the ballasted side only. So, the improved length for case 7 is 10.2 m on the ballasted track where both solutions coexist.

The effects of other important factors on track transition performance, such as embedded rail structures, train velocity and rail surface defects are also studied here in cases studies 8, 9 and 10, respectively. Case 8 results from adding an embedded rail to case 1. It enables studying the effect on vertical stresses and vertical displacement when the rail is continuously supported in the slab track side.

In all case studies, the train is considered to travel at 70 m/s (252 km/h) but in case 9 the speed is decreased to 50 m/s (180 km/h) and increased to 90 m/s (324 km/h). Increasing the speed has negative consequences on track performance[38].

Case 10 consists in adding rail surface irregularities to reference case 1. The irregularities are obtain from a measurement train. The properties of the vertical level geometry profile in various wavebands used in this case are given in Table 9. This methodology was also used in [24].

Table 9. Rail Surface Irregularity Parameters

Wavelength range (m)	Max (mm)	STD (mm)
3-25	4.6058	1.02960
25-70	3.8831	1.38490
70-180	6.4151	2.72110
0-5	2.3161	0.30047
0-10	4.4263	0.70446
0-150	7.4245	2.59740
0-70	7.2074	1.74430
Total profile	10.5941	3.20930

5. Numerical simulations

In order to assess the track transition performance in the case studies considered here, the under sleeper forces, stresses in ballast, vertical displacements and accelerations in sleepers are analyzed and compared. The stresses in the ballasted area are studied in the low frequency band [39]. The rail supports are very important elements in these studies, so special attention is devoted to the rail pads and sleepers in the ballasted area and to the rail pads and fastenings in the slab track. The results calculated at the track supports are given in the time domain.

The forces, stresses and displacements results are filtered. The analyzed frequency range was 0-250 Hz for a vehicle running at 50m/s, 70 m/s and 90 m/s. Explanation for this frequency selection in this track analysis can be found in [34]. The chosen frequency of 250 Hz (medium range) was selected to observe the effects of the dynamic loads on the superstructure, ballast and lower layers. According to[3], this is the most suitable frequency range to study these effects.

The analyses concentrate on the ballasted area where the stresses have to be reduced. The initial analysis is to check the maximum values for each support element near the transition and compare them with the maximum allowed values for the ballasted and slab tracks.

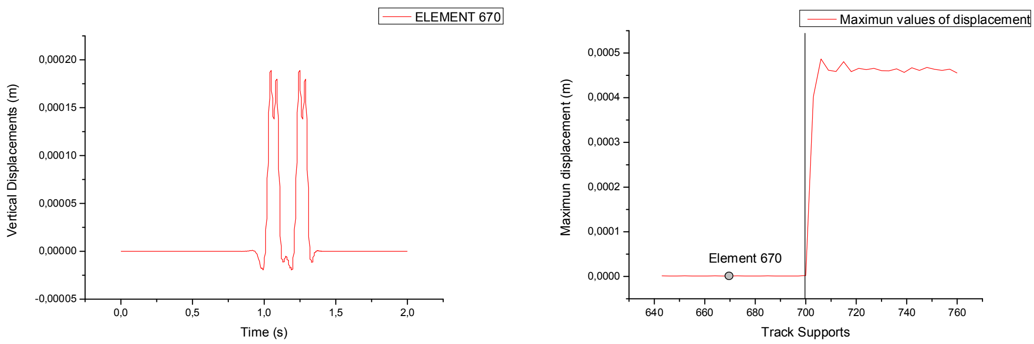


Fig 4. Example of maximum filtered displacements at a sleeper element: load history for one track support (left). Maximum values for the track support (right).

An example of the load history for a sleeper element is shown in figure 4. According to[40], displacement peaks have to be lower than 1 mm and in 200 m values have to be under a standard deviation of about 0.7 mm. On the other hand, ballast stress should be lower than 500 kN/m^2 (no tensile stresses are allowed) [41]. Other authors give the admissible values ≤ 300 kN/m [2 ,42]. The vertical track displacements and total stresses on ballast are obtained for each case study analyzed here and compared with the limit values.

Initial deformation in ballast is not elastic but continuous loads make it elastic in time. In this study ballast is modelled as elastic material (stabilized before first train circulation). Also, the vertical accelerations in the car body have been calculated to analyze the passengers comfort.

The convergence of the solution process is affected by the size of the integration time step dT. Typically, in FEM the step should be small enough in order to obtain accurate results, but not too small, so that the numerical error will prevail and the CPU time become a heavy burden. In DARTS the time step must be smaller than dT*V. In this case, a value of 0.0025 seconds was used (for a speed of 70 m/s). Only in case 9 this was changed to 0.002 seconds to satisfy convergence (because here the analysis was made for both speeds of 70 m/s and 90 m/s).

6. Results and discussion

Vertical displacements under sleepers are depicted in figures 5 and 6 and vertical stresses under sleepers are shown in figures 7 and 8. In these figures, the vertical limit represents the separation between slab and ballast track and vertical settlement is considered positive. Train goes from slab track to ballast and vice-versa in all cases.

Case 7 shows the best performance of all of them and the main improvements are found in this case. Here the goal to smooth the stiffness transition has been achieved that can be seen in the responses (figures).

Red line in all figures are under variables taken from initial case (case 1). Transitions in vertical displacements and vertical stresses under sleepers (ballast track) and under principal slab (slab track) can be seen in the following figures.

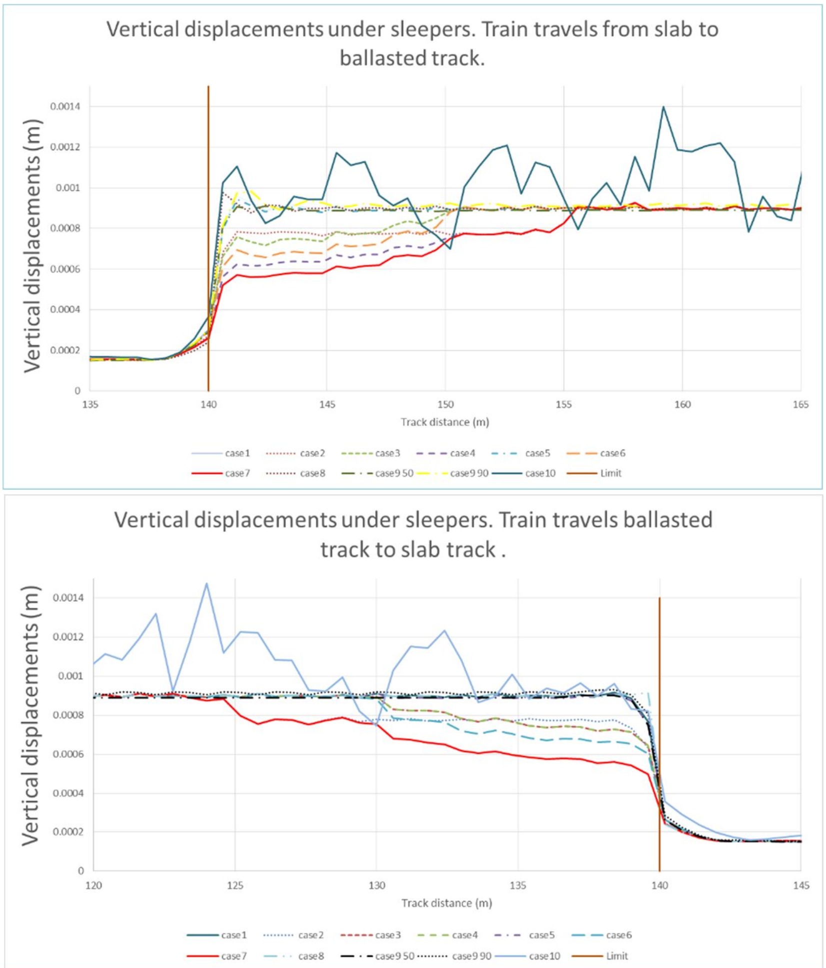


Fig 5. Vertical transient displacements under sleepers (cases 1, 7, 9 and 10). Train goes from slab to ballast track (up) and ballast to slab track (down).

The transition length depends on the parameter that is analyzed. When considering the vertical displacement, the response values vary on the distances from 15.6 m to 17.8 m, depending on the train direction, as shown in table 9. If the vertical stresses are considered, the transition length is from 15.7 m to 15.3 m. In order to have a transition length that contains all scenarios, a value of 18 m is proposed here as an appropriate transition length to cover transient vertical displacements and vertical stresses. The vertical displacements are more restricted in order to give an optimal transition length.

Table 10. Transition length in vertical displacements and in vertical stresses under sleepers

Transition Length	Vertical Displacements	Vertical Stresses
Train Direction Slab to Ballast	15.6 m	15.7 m
Train Direction Ballast to Slab	17.8 m	15.3 m
Optimal length	17.8 m	15.7 m

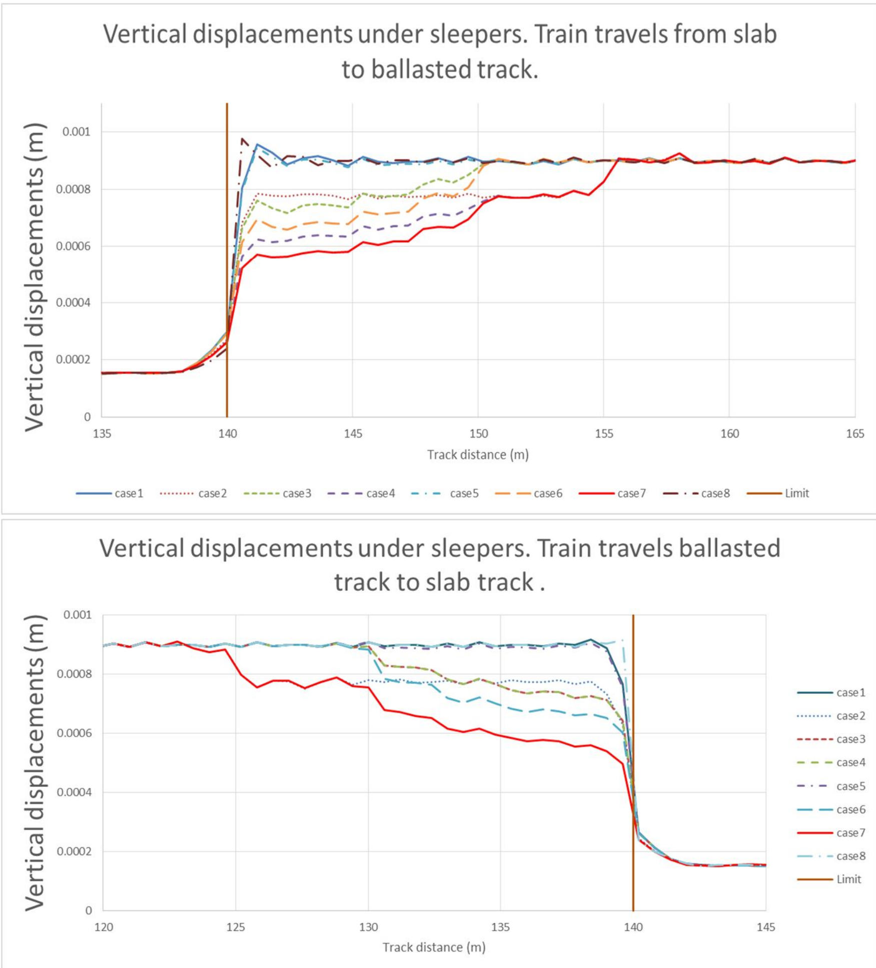


Fig 6. Vertical transient displacements under sleepers (cases 1 to 8). Train goes from slab track to ballast track (up) and from ballast track to slab track (down)

The results from figures 5 and 7 show that when rail surface defects are introduced (case 10) a significant disturbance in vertical displacements appears in the ballast area and vertical stresses in both types of tracks. These rail defects are the source of vertical displacements over 1 mm. It can be seen that in the last support there is an additional settlement when train goes from slab to ballast track. Rail surface defects affect more the vertical stresses and create amplification in the loads. After and before the considered sections, lines tend to be horizontal with constant values in both types of tracks and for both types of variables (vertical displacements and vertical stresses under sleepers).

The results from figure 6 and 8 show that the use of embedded rail (case 8) has no influence in vertical displacements and vertical stresses, although a peak appears just in transition point for both train travel directions.

The results obtained in cases 3 and 4 with longer sleepers when train goes from ballast to slab are generally the same, which means that adding additional rails in this travel direction doesn't improve the vertical displacements or the vertical stresses of the track. However, additional rails with extra-long sleepers (case 7) considerably improve the situation in vertical displacements and vertical stresses. In fact, statistical values for 120 m of track length show minimum vertical displacements average of 0.493 mm and standard deviation of 0.353 mm in case 7. When train goes from ballast to slab track the minimum values are also obtained in case 7.

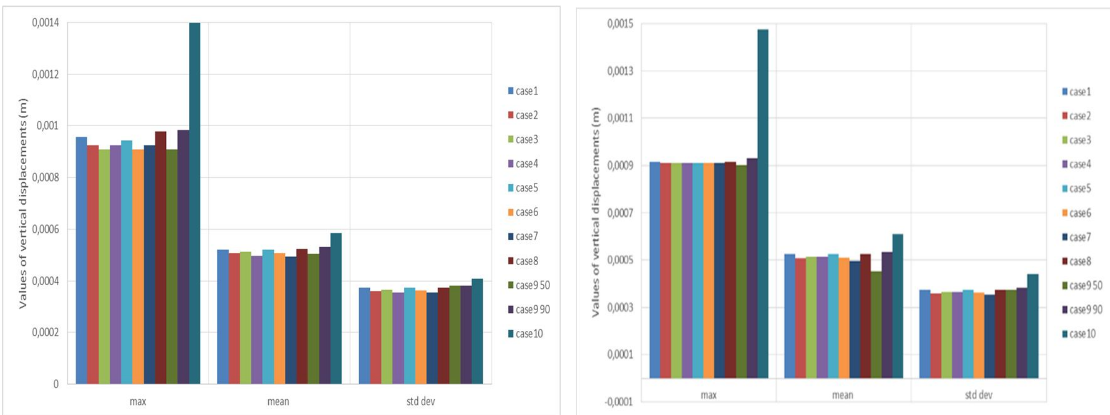


Fig 7. Maximum, average and standard deviation values for vertical displacements along 120 m. Train goes from slab to ballast track (left) and from ballast to slab (right)

From table 10 it can be seen that the largest differences with respect initial case comes from case 9 (50 m/s) for maximum values and case 7 for medium values when train goes from slab to ballast. If train direction changes from ballast to slab best case is case 9 for maximum and medium values.

Also, the statistical values show that increase sleeper size is better than add more rails.

Table 10. Vertical displacements under sleepers within 200 m. Statistical values. Table also shows the improvement for each train direction all cases from case 1.

	Vertical displacemente under sleepers mm in 200 m						% Improvement respect case 1 (Initial)			
Cases	Slab to ballast			ballast to slab			Slab to Ballast		Ballast to Slab	
	max	mean	std dev	max	mean	std dev	max	mean	max	mean
case1	0.958	0.522	0.374	0.916	0.525	0.374	0.000	0.000	0.000	0.000
case2	0.925	0.507	0.361	0.910	0.509	0.360	-3.416	-2.958	-0.679	-3.139
case3	0.910	0.512	0.365	0.909	0.514	0.365	-5.014	-2.011	-0.711	-2.071
case4	0.925	0.497	0.355	0.909	0.514	0.365	-3.437	-4.754	-0.711	-2.071
case5	0.945	0.521	0.374	0.910	0.524	0.373	-1.395	-0.113	-0.707	-0.188
case6	0.910	0.507	0.362	0.909	0.509	0.362	-5.017	-2.903	-0.711	-2.958
case7	0.925	0.493	0.354	0.910	0.495	0.354	-3.429	-5.552	-0.679	-5.698
case8	0.977	0.523	0.374	0.915	0.526	0.374	1.996	0.111	-0.068	0.181
case9 (50)	0.909	0.506	0.383	0.902	0.452	0.374	-5.151	-3.111	-1.496	-13.941
case9 (90)	0.984	0.530	0.381	0.930	0.533	0.381	2.701	1.603	1.556	1.550
case10	1.399	0.586	0.408	1.476	0.610	0.441	45.981	12.272	61.114	16.269

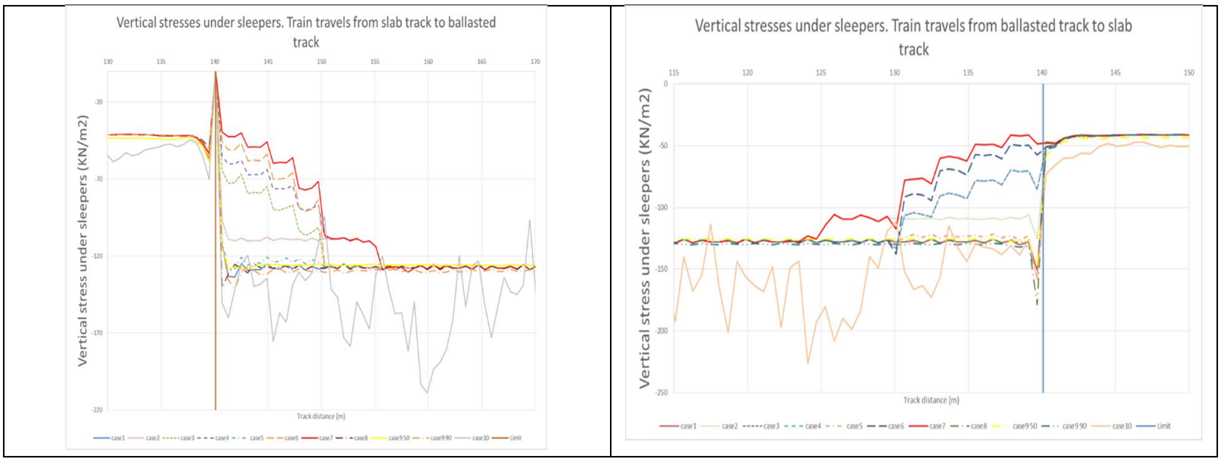


Fig 8. Vertical transient stresses under sleepers (cases 1, 7, 9 and 10). Train goes from slab track to ballast track (left) and ballast track to slab track (right)

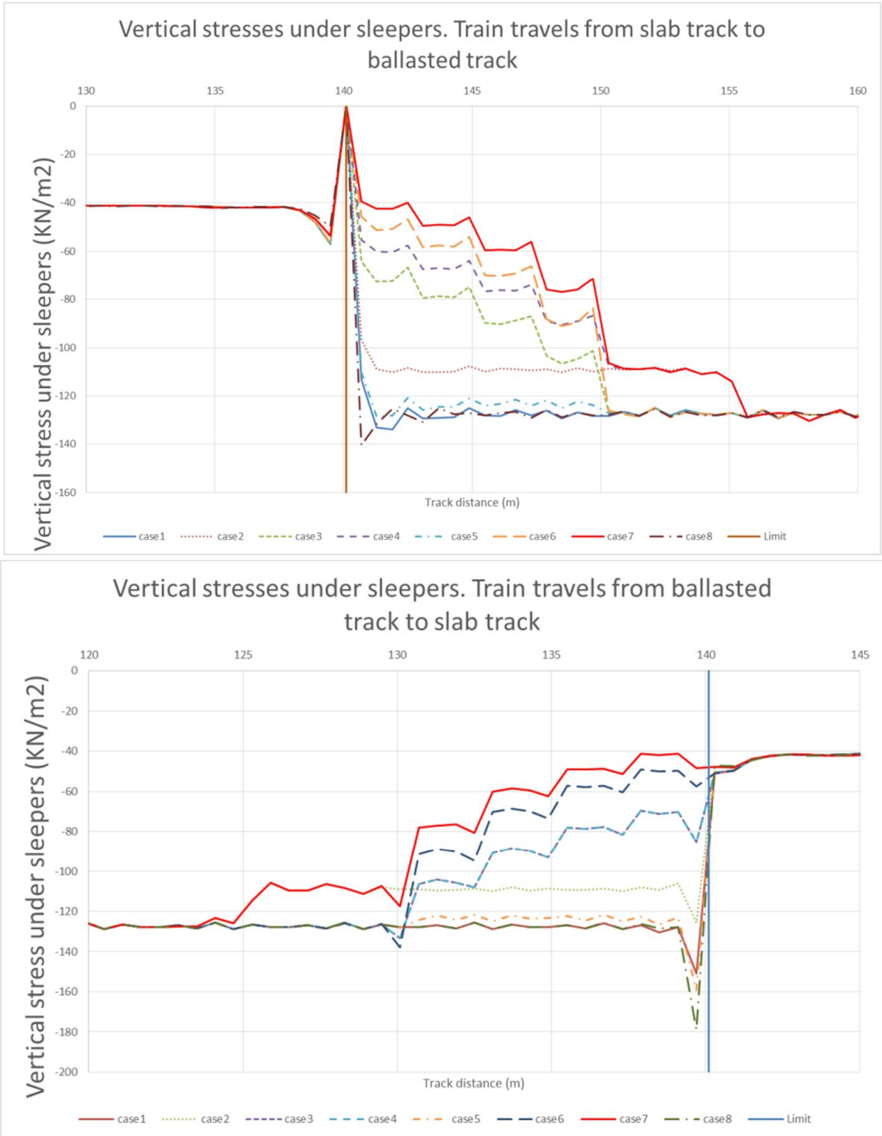


Fig 9. Vertical transient stresses under sleepers (cases 1 to 8). Train goes from slab track to ballasted track (up) and from ballast track to slab track (down)

The results from figure 7 show that the vertical stresses under sleepers in case 7, when train travels from slab to ballast, reach 170 kN/m^2 (compressive). Case 10 produces stresses near to 200 kN/m^2 (compressive). If train goes from ballast to slab, maximum values for stresses are near 250 kN/m^2.

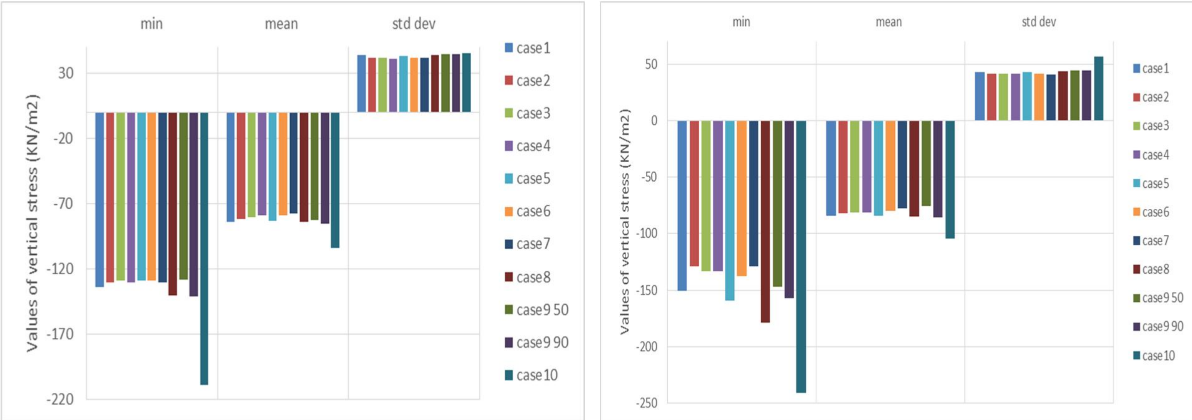


Fig 10. Maximum, average and standard deviation values for vertical stresses along 120 m. Train goes from slab to ballast track (left) and from ballast track to slab track (right)

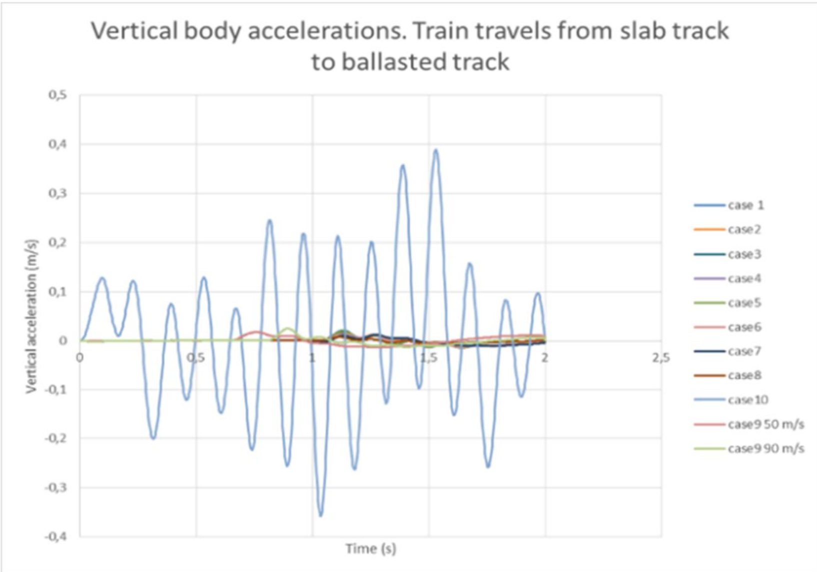
Table 11. Vertical stresses under sleepers. Statistical values of vertical stresses under sleepers. Table also shows the improvement for each train direction all cases from case 1.

	Vertical stresses under sleepers						% Improvement respect case 1 (Initial)			
Cases	Slab to ballast			Ballast to slab			Slab to Ballast		Ballast to Slab	
	max	mean	std dev	max	mean	std dev	max	mean	max	mean
case1	-133.825	-83.824	43.443	-150.975	-84.572	43.297	0.000	0.000	0.000	0.000
case2	-130.385	-81.527	41.532	-128.860	-82.165	41.275	-1.871	-2.740	-14.648	-2.846
case3	-129.080	-80.426	41.665	-133.185	-81.260	41.445	-2.581	-4.054	-11.783	-3.916
case4	-130.350	-78.614	40.916	-133.185	-81.260	41.445	-1.890	-6.216	-11.783	-3.916
case5	-129.185	-83.479	43.106	-159.305	-84.291	43.065	-2.524	-0.411	5.517	-0.332
case6	-129.040	-78.871	41.876	-137.920	-79.725	41.650	-2.603	-5.909	-8.647	-5.731
case7	-130.360	-77.284	41.395	-128.860	-78.027	41.083	-1.885	-7.802	-14.648	-7.739
case8	-140.475	-83.895	43.523	-178.865	-84.721	43.515	3.618	0.085	18.473	0.176
case9 (50)	-128.170	-82.170	44.029	-146.945	-75.629	44.155	-3.076	-1.973	-2.669	-10.574
case9 (90)	-141.275	-85.018	44.543	-157.390	-85.801	44.418	4.053	1.424	4.249	1.453
case10	-208.930	-103.890	45.246	-240.810	-104.766	56.763	40.857	23.939	59.503	23.878

Statistical values for vertical stresses under sleepers in 120 m track length show that the minimum value of vertical stresses average is obtained in case 7 with 77.28 kN/m^2 (compressive) when train goes from slab to ballast and in case 9 (50 m/s) when train goes from ballast to slab 75.63 kN/m^2 (compressive). The maximum value in vertical stresses corresponds to case 10 with 208.93 kN/m^2 (compressive) when train goes from slab to ballast, the minimum standard deviation is obtained in case 4 (40.9 kN/m^2) and case 7 presents a similar value of 41.39 kN/m^2. When train goes from ballast track to slab track, minimum values for vertical stresses are obtained in case 7, with 128.86

kN/m^2 (compressive), which also presents the minimum value for standard deviation, corresponding to 41.08 kN/m^2.

Last four columns in table 11 shows the improvement of each case from case 1. Values are different depending of train running direction. Mean values are good in cases 7 and 9.



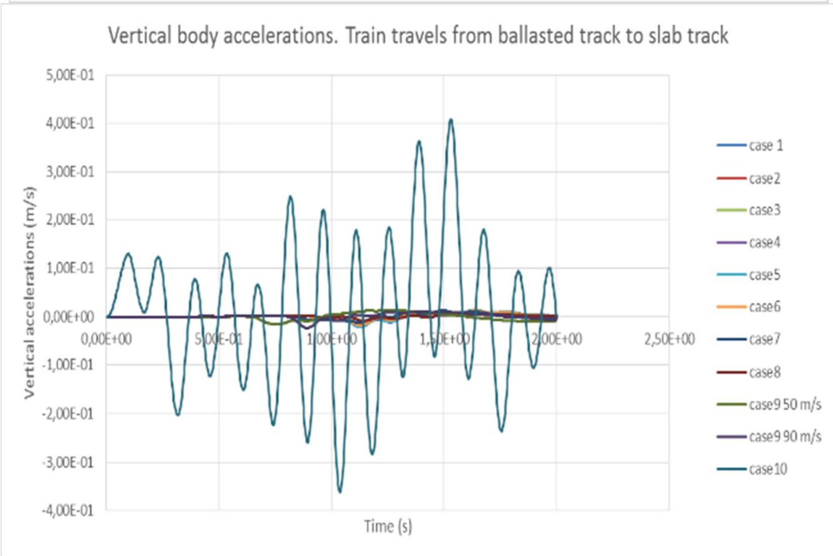


Fig 11. Vertical accelerations in the carbody (cases 1 to 10). Train goes from slab track to ballast track (up) and from ballast track to slab track (down)

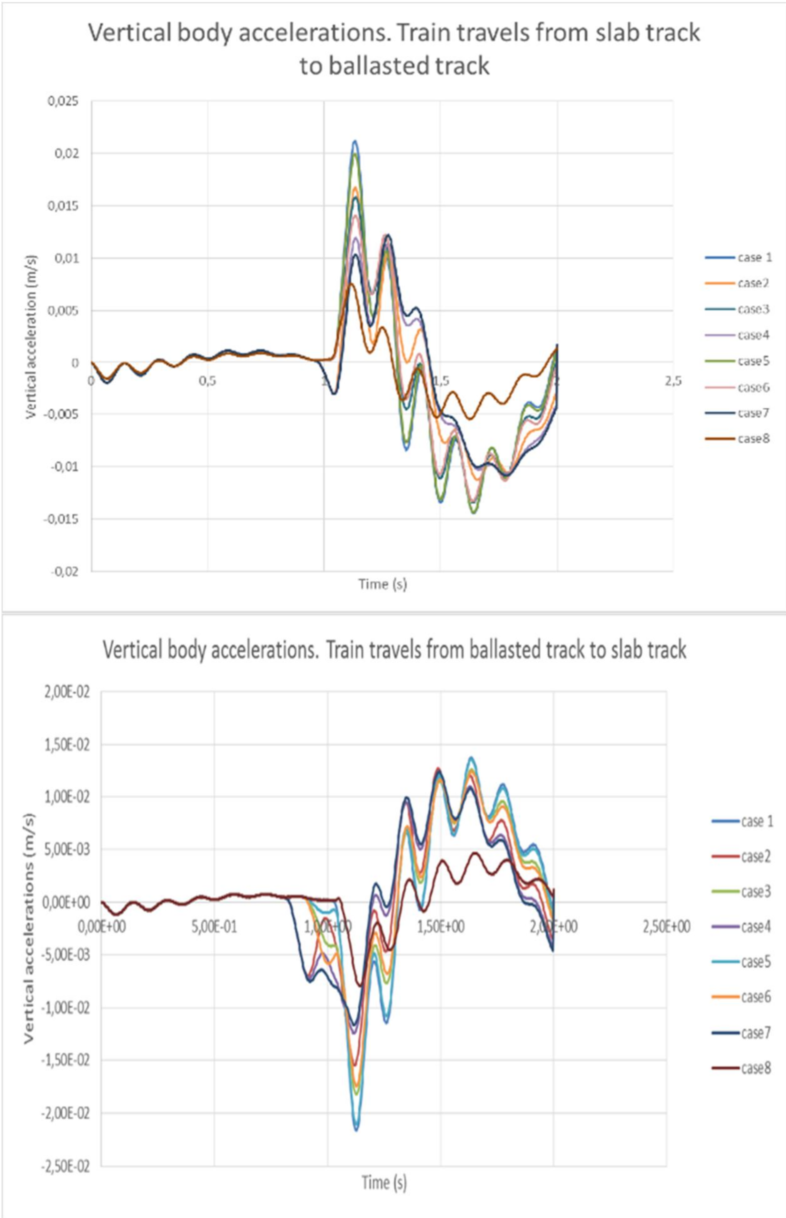


Fig 12. Vertical accelerations in the carbody (cases 1 to 8). Train goes from slab track to ballast track (up) and from ballast to slab track (down)

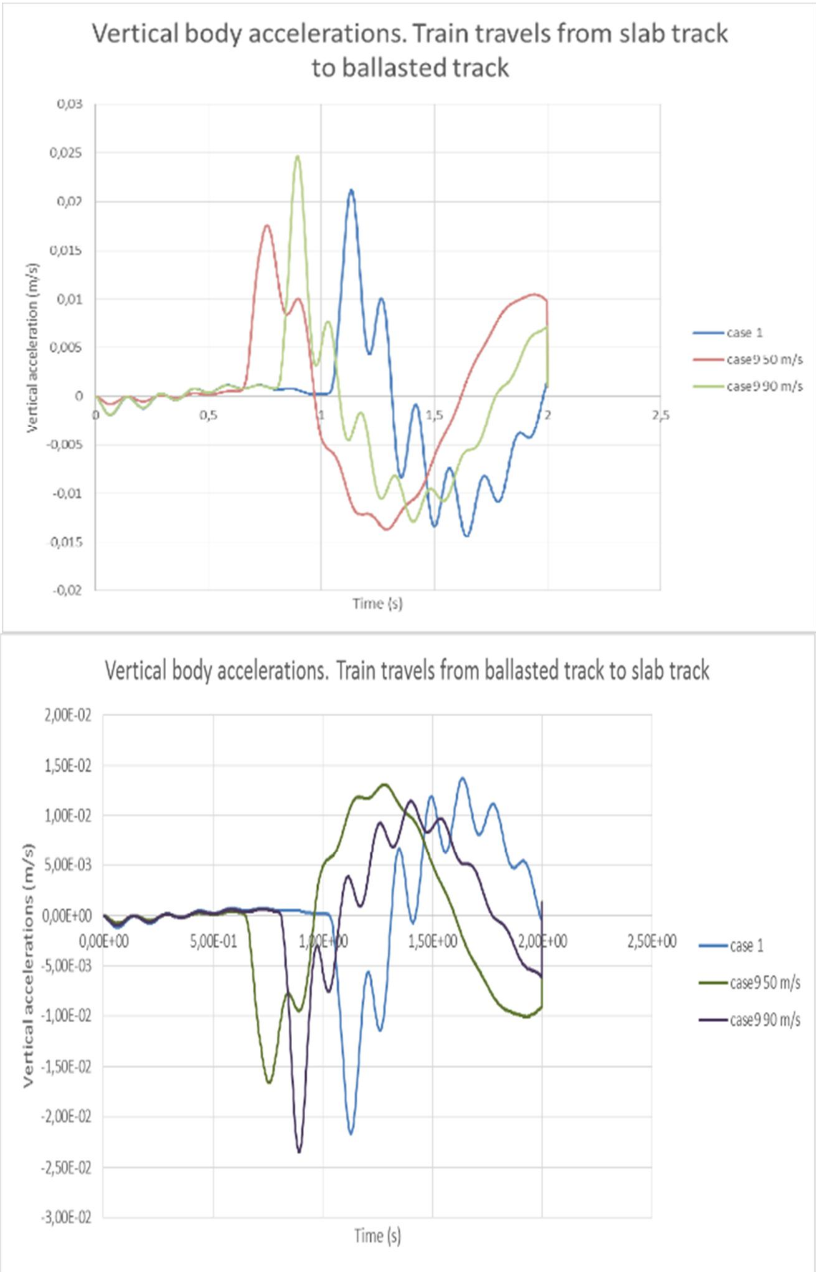
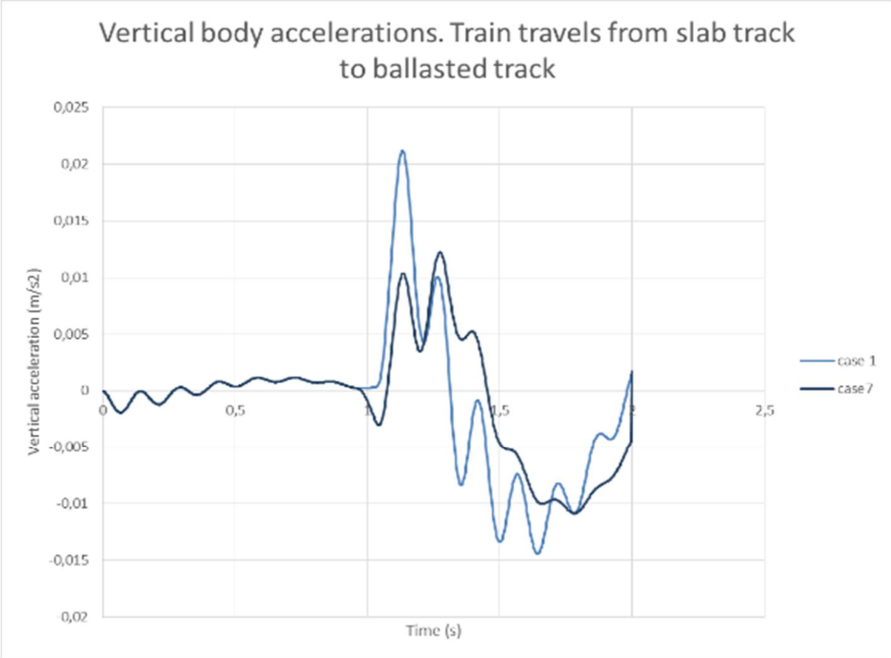


Fig 13 .Vertical accelerations in the carbody (velocity of 50 m/s, 70 m/s and 90 m/s). Train goes from slab track to ballast track (up) and from ballast to slab track (down)



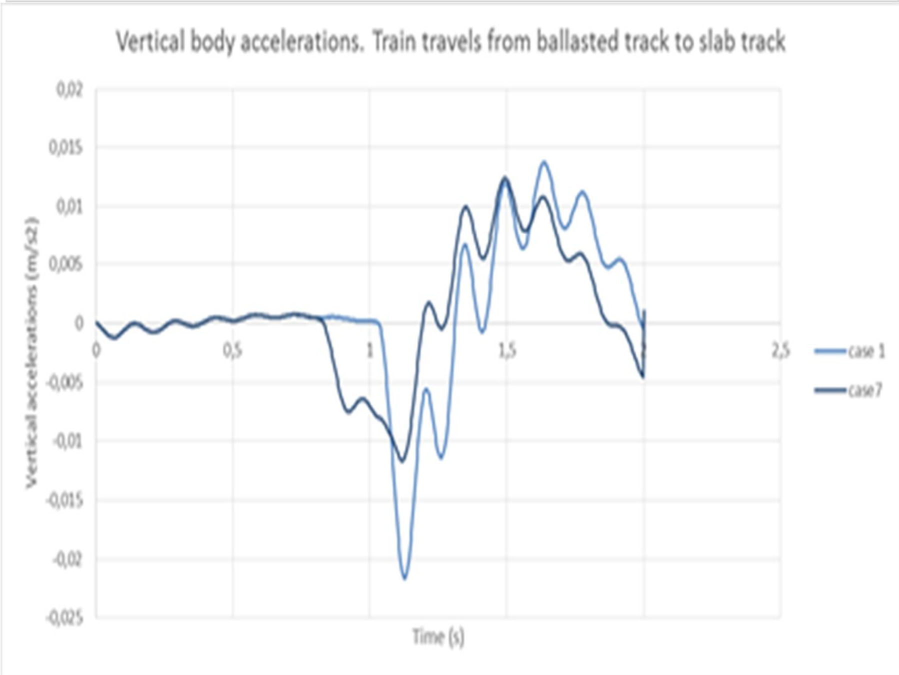


Fig 14. Vertical accelerations in the carbody (case 1 and case 7). Train goes from slab track to ballast track (up) and from ballast to slab track (down)

The time domain car body vertical accelerations, when train reaches the transition between slab and ballast track, are shown in figures 10-13. The change between types of track is between 1 and 1.5 s, depending on the starting point of the train in the track model and of the train.

From slab to ballast track, vertical car body accelerations reach values of 0.025 m/s^2 when train goes from 70 to 90 m/s. If train goes from ballast to slab these values reach -0.0235 m/s^2. Case 8 shows the best performance in vertical accelerations with values under 0.01 m/s^2 for both travel directions.

In case 7 the maximum acceleration values are below 0.0125 m/s^2. The rail surface defects increase the vertical accelerations with values of 0.38 m/s^2. If train changes its direction (from ballast to slab) vertical acceleration of the car body increases to values of 0.4 m/s^2. When comparing case 1 with case 7, acceleration values are reduced from 0.02 m/s^2 to 0.01 m/s^2. Improvements when traveling in the opposite direction are even better as negative peaks of vertical accelerations are attenuated almost three times.

7. Conclusions

This work presents a methodology to study the track transition performance with different track structure solutions. The methodology uses numerical modelling of ballast track, slab track and railway vehicle dynamics in time domain. The behavior of a track transition with high and low vertical stiffness is analyzed using the finite element software DARTS. A high-speed vehicle moving in both directions is considered and the track vertical displacements, stresses and car body accelerations are analyzed.

The results show that the vertical stresses and displacements can be significantly reduced in the ballasted areas by improving the existing solutions and using new technologies. In particular, the best solution achieved here uses extra-long sleepers and double rails (case 7). The rail displacements in this case are less than 1 mm and the stresses in ballast are under the resistance limits of 300 kN/m^2 as well. The vertical accelerations in the car body have also been reduced as compared to the reference case, so that the case 7 is also the best solution in terms of passenger comfort.

This work shows that improvements in track transition stiffness can be achieved by modifications made to 18 m of track in the transition zone, in the worst case scenario, when the train travels in both directions. Other researches have shown the transition lengths to be around 100 m. The solution obtained here using the calculation and simulation process can reduce the length of the track transition zone and, at the same time, avoid the harmful effects caused by the dynamic loads. The numerical models show that small changes in superstructure elements can reduce transient vertical displacements and stresses over ballast layer. Track works have to focus on this part of track structure in the transition areas in order to reduce track deterioration, decrease the maintenance and reduce the life cycle costs of the track.

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