MODELLING AND ANALYSIS OF RAILWAY TRANSITION ZONES
Application to the transition between Ballasted and Slab track

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Keywords: Railway infrastructure, stiffness variation, transition, ballasted track, slab track, finite element modelling.

Abstract: The increase in velocity of circulation and frequency of services led to the increment of the dynamic solicitations experienced by the railway, which led to a more accelerated degradation of the track geometric quality. This problem gains increased importance in zones that have abrupt changes in track stiffness, since in these areas the dynamic amplifications reach magnitudes of higher order.

In recent past, several studies were conducted on this subject, but mainly focused on the specific case of the transition to a structure, namely bridges. As such, few studies were made regarding transitions between two different railway typologies, ballasted track and slab track (ex. transition to tunnels or stations), where this problem is also relevant.

With the intent of reducing the effects of these abrupt stiffness changes, the intermediate zones, or transition zones, are being idealized. These zones intend to introduce a gradual stiffness variation. However, regarding the ballasted/slab transition there isn’t sufficient knowledge over the correct solution to achieve the desired structural performance.

Thereby, this dissertation aims to analyse the structural behaviour of the specific transition between the ballasted track and the slab track, with the purpose of studying and evaluating the solution to this problem. This study is developed through the evaluation of the transition presented in the experimental case study Chauconin located in the French high-speed railway line LGV-Est.

To achieve these goals a set of numerical two-dimensional models will be developed and validated with recourse to experimental results provided by SNCF. These tools will provide aid to the process of study and optimization of said transition.

After the models are completed, there will be an analysis of the influence of some aspects in the railway performance, in order to better understand her behaviour and optimize the alternative solutions to conceive for the transition in study.

Lastly a group of alternative solutions to the case study will be designed and modelled and after consideration and analysis, a set of recommendations to reduce the severity of the problem in hands will be redacted.

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1. Introduction

The increase in velocity of circulation experienced in high-speed services is responsible for greater dynamic solicitations in the railway track, which generally implies a greater deterioration of track quality, especially on ballasted track. With this increase of the rate of deterioration, the maintenance actions have to be more frequent and more intense, and therefore more costly. The transition zones, where abrupt track stiffness occurs, are usually subjected to superior dynamic effects and therefore experience a bigger deterioration rate, which may reach four to seven times the requirements for maintenance in standard track. Examples of these zones are the transitions between embankment and structure or between two different railway typologies, like ballast and slab track.

The concerns on this matter led the scientific community to an extensive research on the deterioration processes and dynamic effects in zones that have stiffness variations. Most of the previous work on the matter, focused on the transitions between embankment and structures, namely bridges, since this specific case is by far the most common to occur. To fill the gap of knowledge on the subject of stiffness variations in railway track, the main focus of this thesis is to address the problem of transitions between ballast and slab track and ultimately apply the findings in a real application, the experimental stretch Chauconin, in the French high-speed railway track LGV-Est.

In order to evaluate the dynamic effects of traffic flow in the track, the most efficient viable tool is the two-dimensional finite element model, since a three-dimensional model would imply excessive computation time that in no way would be viable in a dynamic analysis.

2. Vehicle and Railway Numerical Modeling

This section introduces the development of the vehicle and railway model (ballasted track), as also the methodology behind the dynamic analysis conducted. The numerical modeling will be performed using the software ANSYS\textsuperscript{TM} [1].

2.1. Solving the dynamic equilibrium problem

Every dynamic problem can be formulated through the equations of dynamic equilibrium. Depending on the type of problem, the process of solving can differ. The process implemented to the resolution of this specific problem is shown in Figure 2.1.

![Diagram showing the process of solving the dynamic equilibrium problem.]

\[ F_i = F_k \cdot N_i(x_k) \] (1)

2.2. Vehicle model

In order to study the vehicle-railway interaction, through a dynamic analysis, a vehicle model that can properly decode the dynamic behavior of the system is required. In a static analysis the model consists of two forces representing the axle load. For the dynamic analysis two different vehicle models were conceived, a moving force and a sprung mass model.

2.2.1. Moving force model

The moving force model is the simplest model conceived and has been broadly implemented in the study of track vibrations. While this model can, to a certain degree, register the dynamic characteristics of the railway track it doesn’t allow the consideration of the vehicle response, and it’s affection on the track.

The implementation of this model implies the definition of shape functions that allow a good approximation from the applied load into equivalent nodal forces through the load path.

The process of transformation is exposed in the next set of equations and it is represented in Figure 2.2.
\[ N_i(x) = \begin{cases} \frac{x_k - x_{i-1}}{L_{i-1,i}} & x_{i-1} \leq x_k \leq x_i \\ 1 - \frac{x_k - x_i}{L_{i,i+1}} & x_i \leq x_k \leq x_{i+1} \end{cases} \] \hspace{1cm} (2)

Figure 2.2 - Process of transformation of loads into equivalent nodal forces [2]

### 2.2.2. Sprung mass model

The sprung mass model, illustrated in Figure 2.3, allows to consider the influence of the vehicle in the railway structural response, through the vehicle-track interaction.

The model adopted consists in one single mass, that represents the wheels-axle system \((M_V)\) suspended by a spring \((K_h)\), that enacts the hertzian contact between the wheel and rail. The model also includes a gravity force applied in the mass node equal to the sum of the share part of the mass of the bogie and the box, per axle.

\[
\frac{M_biggie}{2} + \frac{M_{box}}{4} \times g
\]

Figure 2.3 – Sprung mass model

### 2.3. Railway track model

In order to have a better grasp of the results obtained, the modelling process of the railway track was divided in two stages: the rail-railpads system and the complete track-soil system.

#### 2.3.1. Rail - railpads system

This first two-dimensional model consists in a beam, representing the rails, discretely supported in equally spaced springs, representing the railpads.

The rail is modelled using a continuous beam element and accordingly to the Euler-Bernoulli theory (BEAM3). This element has two nodes and tensile, compression and bending capabilities. The element has three degrees of freedom: translations in XX, YY and ZZ. The railpad is modelled by a spring-damper element with two nodes (COMBIN14) that has an uniaxial tension-compression behaviour and possesses the same degrees of freedom as the rail element. The mechanical and geometrical properties are represented in Table 2.1.

<table>
<thead>
<tr>
<th>Element</th>
<th>Properties</th>
<th>Element type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rail</td>
<td>( E = 205.8 ) GPa</td>
<td>BEAM3</td>
</tr>
<tr>
<td></td>
<td>( \rho_c = 7850 ) kg/m(^3)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( A = 154 \times 10^{-4} ) m(^2)</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( I_c = 61.1 \times 10^{-6} ) m(^2)</td>
<td></td>
</tr>
<tr>
<td>Railpad</td>
<td>( K_p = 120 \times 10^3 ) KN/m</td>
<td>COMBIN14</td>
</tr>
<tr>
<td></td>
<td>( C_p = 60 ) kN.s/m</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( h_{zz} = 0.2 ) m</td>
<td></td>
</tr>
</tbody>
</table>

The longitudinal domain adopted is 30 m for the static analysis and 60 m for the dynamic. The displacement restrictions are implemented at the end node of the pads, in XX and YY. In order to avoid numerical problems the same restriction is applied in the end nodes of the rail, (see Figure 2.4).

Figure 2.4 – Two-dimensional model of the rail-railpads system.

The aim of this first system is to validate the results obtained from the finite element model with theoretical formulations (Table 2.2), determine the best rail element length to implement and also evaluate the influence of the vehicle model in the track structural behaviour, (Table 2.3).
### Table 2.2 – Rail-railpads system static analysis results

<table>
<thead>
<tr>
<th>Element length (m)</th>
<th>( \delta_{\text{max}} ) (mm)</th>
<th>( \delta_{\text{theory}} ) (mm)</th>
<th>Error (%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6</td>
<td>-0.568</td>
<td>-0.5818</td>
<td>2.45</td>
</tr>
<tr>
<td>0.3</td>
<td>-0.568</td>
<td>-0.5818</td>
<td></td>
</tr>
<tr>
<td>0.1</td>
<td>-0.568</td>
<td>-0.5818</td>
<td></td>
</tr>
</tbody>
</table>

### Table 2.3 – Rail-railpads system dynamic analysis results

<table>
<thead>
<tr>
<th>( \delta_{\text{max}} ) vertical rail (mm)</th>
<th>Sprung Mass</th>
<th>Moving force</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.6</td>
<td>-0.519</td>
<td>-0.576</td>
</tr>
<tr>
<td>0.3</td>
<td>-0.567</td>
<td>-0.577</td>
</tr>
<tr>
<td>0.1</td>
<td>-0.572</td>
<td>-0.578</td>
</tr>
</tbody>
</table>

The results gathered allow to draw the following conclusions:

- There is a good theoretical consistency in the numerical results.
- Since track irregularities, and stiffness variations are not implemented in this model, the maximum vertical displacement varies very little from the static to the dynamic analysis.
- The vertical displacements are inferior when the vehicle model considered in the dynamic analysis is the sprung mass. This aspect results from the extra stiffness imposed by the hertzian spring.
- The rail element size that produces the best results in the dynamic analysis is 0.1 m. An additional iteration with an element size of 0.05 m was conducted, however the results obtained were very similar and not worth the extra computational effort, therefore the adopted element size for future models is 0.1 m.

### 2.3.2. Railway track-platform system (ballasted)

This two-dimensional system contains beyond the rail equipment (rail, rail-railpads and fastening system) the sleepers and the multilayer ballasted system (ballast, sub-ballast) and platform (see figure 2.5). All the relevant mechanical and geometric properties are shown in Table 2.4.

The sleepers and the multilayer system are modelled with a plane element (PLANE42). This element allows to model two-dimensionally solid structures, and consists of four nodes with two degrees of freedom: translations in XX and YY. To simulate the volume effect, this element has a “plane stress with thickness” functionality which permits the model to consider the influence of the load transversely.

In order to calibrate the thickness of the elements, the properties of an existing three-dimensional model were adopted [3].

### Table 2.4 - Railway track-platform model properties

<table>
<thead>
<tr>
<th>Element</th>
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<th>Element type</th>
</tr>
</thead>
</table>
| Sleepers | E = 25 GPa  
\( v = 0.2 \)  
\( \rho_t = 2548,42 \text{ kg/m}^3 \)  
\( hzz = 0.22 \text{ m} \) | PLANE42 |
| Ballast | E = 70 MPa  
\( v = 0.15 \)  
\( \rho_b = 1529 \text{ kg/m}^3 \)  
\( hzz = 0.35 \text{ m} \) | PLANE42 |
| Sub-ballast | E =70 MPa  
\( v = 0.3 \)  
\( \rho_s = 2090 \text{ kg/m}^3 \)  
\( hzz = 0.25 \text{ m} \) | PLANE42 |
| Platform | E = 100 MPa  
\( v = 0.3 \)  
\( \rho_f = 2140,7 \text{ kg/m}^3 \)  
\( hzz = 4 \text{ m} \) | PLANE42 |

Since the railway track has an infinite development there is a need to create fictional lateral boundaries to restrict the domain in study. In order to simulate the effect of continuity and transparency, this border must allow the free crossing of vibration waves to the infinite without creating reflections inside the domain [4]. To achieve this effect a set of spring-damper elements is applied to the lateral borders.

![Two-dimensional model of the railway-platform ballasted system](image)

The longitudinal domain was established in 60 m in order to balance the computational effort.

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<td>PLANE42</td>
</tr>
<tr>
<td></td>
<td>( v = 0.2 )</td>
<td></td>
</tr>
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<td></td>
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<td></td>
</tr>
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</tr>
<tr>
<td></td>
<td>( v = 0.15 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \rho_b = 1529 \text{ kg/m}^3 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( hzz = 0.35 \text{ m} )</td>
<td></td>
</tr>
<tr>
<td>Sub-ballast</td>
<td>E =70 MPa</td>
<td>PLANE42</td>
</tr>
<tr>
<td></td>
<td>( v = 0.3 )</td>
<td></td>
</tr>
<tr>
<td></td>
<td>( \rho_s = 2090 \text{ kg/m}^3 )</td>
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efforts to solve the problem and the necessary extension to stabilize the dynamic results.

The aim of this particular model is to determine the adequate finite element grids (see Figure 2.6), and evaluate the dynamic results produced, in order to validate the model for application to the case study.

<table>
<thead>
<tr>
<th>Element</th>
<th>Grid 1</th>
<th>Grid 2</th>
<th>Grid 3</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rail (m)</td>
<td>0.1</td>
<td>0.1</td>
<td>0.1</td>
</tr>
<tr>
<td>Sleepers (m²)</td>
<td>0.15x0.15</td>
<td>0.075x0.075</td>
<td>0.0375x0.0375</td>
</tr>
<tr>
<td>Layers (m²)</td>
<td>0.15x0.15</td>
<td>0.075x0.075</td>
<td>0.0375x0.0375</td>
</tr>
</tbody>
</table>

Figure 2.6 – Finite element grids considered: 1 (left), 2 (center), 3 (right)

The calibration of the model allowed to conclude the following aspects:

- The two-dimensional model has good correlation with the three-dimensional model after calibration. This consistency only exists in regard to vertical displacements and accelerations. Oppositely, this type of model proves to be a bad tool to predict the development of vertical stresses in the railway-platform system.
- The increase in computational time required to achieve a solution, with grid 2 and 3, does not lead to better and more exact results, therefore the finite element grid 1 is the choice for implementation in future railway models.

To obtain more accurate dynamic results, especially the wheel-rail interaction force, the vehicle model applied was the sprung mass model. The vertical displacement and acceleration of the rail and sleeper are exposed in Figure 2.7 and 2.8. These results allowed to conclude:

- The time-step between iterations applied is 0.0005 s. Observing the continuity of the results this parameter is adequate.
- The passage of both axles is adequately recorded, for there are two sets of maximums in the results.

With regards to the interaction force, (see Figure 2.9), after the initial oscillation of the results they tend to stabilize near 25 m into the domain. When stable, the model proofs to be a good tool to estimate this parameter.
3. Application to LGV-Est Case Study

In order to represent the railway solutions displayed, and to study the dynamic behavior of the Chauconin experimental section (see Figure 3.1), three numerical models were developed. A ballasted track model (VB), a slab track model (VLB) and a model representative of the transition between them (VB/VLB). An experimental campaign was conducted by SNCF and performance measurements were collected with the circulation of real trains (TGV Réseau and EMW). These experimental measurements are crucial for the proper validation of the numerical models created.

3.1. Track and Train Characterization

The ballasted track (VB) is composed by UIC60 rails, elastic rail-railpads (180x148x9mm³), Pandrol Fastclip™ fastening system, Sateba™ M450 monoblock sleepers, a ballast layer (31 cm) and a sub-ballast layer (20 cm), (see Figure 3.2).

The slab track (VLB) applied in this test section is the Stedef™ system developed by SNCF. The system essential features are the two levels of elastic adjustment, easy replacement of sleepers and biphased concrete cast. The sleepers applied are bi-block (Sateba™ M453 IP), surrounded by a shell containing a second elastic level, an under sleeper pad (USP 656x228x12 mm³), as illustrated in Figure 3.3.

The rolling stock in operation on LGV-Est and circulating in the test section are TGV-Réseau (SNCF) and ICE (DB). However the recorded measurements are only those corresponding to the TGV-Réseau circulations. The static axle load of the engine compartment is 164.8 kN, the dynamic properties are exposed in the Table 3.1.

<table>
<thead>
<tr>
<th>Engine component</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wheelset mass $M_w$</td>
<td>1024 kg</td>
</tr>
<tr>
<td>Stiffness of primary suspension (bogie-wheelset) $K_b$</td>
<td>$1226 \times 10^3$ kN/m</td>
</tr>
<tr>
<td>Damping coefficient of primary suspension $C_b$</td>
<td>10 kN.s/m</td>
</tr>
<tr>
<td>Mass of half boggie $M_b$</td>
<td>1191 kg</td>
</tr>
<tr>
<td>Stiffness of secondary suspension (box-bogie) $K_c$</td>
<td>$1226 \times 10^3$ kN/m</td>
</tr>
<tr>
<td>Damping coefficient of secondary suspension $C_c$</td>
<td>20 kN.s/m</td>
</tr>
<tr>
<td>Mass of a quarter box $M_c$</td>
<td>13600 kg</td>
</tr>
</tbody>
</table>

3.2. Numerical Modeling

The numerical modeling of the elements of the railway typologies present in the Chauconin experimental section was performed according to the geometric and mechanical characteristics recorded in the bibliography provided by the Department of Railway Study from the Technical Direction of SNCF, (see Table 3.2), as well as the conclusions regarding the development of a railway numerical model taken from the previous section.
Based on the study developed in the previous section, the longitudinal domain was established to 60 m. In order to account for the influence of the load transversely, the thickness of the plane stress elements was established according to the real dimensions and previous literature on the matter.

### 3.3. Model validation

With the intent of ensuring that the models developed are a good tool for analysis of the railway behavior, they were validated with a set of experimental results obtained through two campaigns conducted by SNCF in the experimental section.

For each type of model (VB and VLB) the parameters of comparison were the vertical displacement on top of the rail, in a quasi-static analysis (EMW wagon) and the vertical displacement and acceleration on top of the sleeper, for the dynamic analysis (TGV-Réseau locomotive). These experimental results were subject to a proper statistical process in order to enhance the confidence in the results, and establish an interval for the parameters. The process of validation of the transition model (VB/VLB) consisted in comparing the results in each of the two distinct railway zones (ballasted and non-ballasted) with the results of the isolated models.

With the conclusion of this process of validation, a few key aspects were gathered:

- Both the isolated models (VB and VLB) proved to fit the experimental results regarding the displacement on top of the rail (EMW wagon) and on top of the sleeper (TGV-Réseau locomotive).
- In spite of the numerical vertical acceleration signal going towards the experimental results, the maximum values are slightly lower than the experimental minor limit. There are three possible reasons that may be the cause of this difference: the modelling process didn’t consider the rail defects and heterogeneity; there is a certain degree of doubt concerning the value of some mechanical properties, like the Young’s modulus of some layers; the Chauconin test zone is implemented on curve.

- The two different railway zones present in the transition model (VB/VLB) behaved similar to the isolated models. The differences in results didn’t exceed the 8%. With the validation process concluded, it’s safe to assume that future results and their specificities are uniquely due to the existence of the transition and the stiffness variation that it implies.

The model proves to be sensible to the stiffness variation induced by the transition between railway types. According to previous studies, the transition from a more flexible railway structure to a more rigid one, results in a superior aggravation of the dynamic effects. In order to identify the most penalizing transition, two different circulations were conducted, from (VB to VLB) and from (VLB to VB).

Contrarily to what was originally expected, the (VLB) proved to be the most flexible railway structure, due to the implementation of a second elastic level, the USP. Therefore the most penalizing transition consists in the transition from VLB to VB, as shown in Figure 3.4.
After validation of the models described above, the required conditions are met in order to evaluate the structural response of the railway transition. In order to develop different solutions to the problem, there is a need to appraise the influence of different aspects in the structural behavior of the transition area.

4.1. Analysis of the Case Study track responses

In order to increase the knowledge of the experimental zone behavior, the influence of a set of aspects in the structural response was evaluated. The parameters established for comparison were: the interaction force between the rail and the wheel and the vertical acceleration of the sleepers and in a lower level, the ballast in VB and the slab in VLB.

- Running Speed

The train speed in the experimental section was limited to 230 km/h. Since the rest of LGV-Est railway line has a circulation speed limit of 320 km/h, a study was conducted to understand the influence of the increase of circulation speed in the transition structural response.

Regarding the interaction forces, illustrated in Figure 4.1, it’s drawn that the increase of the velocity of circulation leads to an increment of the contact forces and of the track length subjected to an amplification of the dynamic solicitations.

From the analysis of the vertical accelerations, exposed in Figure 4.2 and 4.3, it’s concluded that for higher speeds of circulation, the parameter tends to increase in both levels of the track, but especially on the sleeper level. A great variation of both parameters happens on the sleepers just after the transition. Another interesting remark is the drastic difference between the vertical accelerations of the sleepers and the slab in VLB, this is primarily due to the damping effect of the sleeper pad. This effect has the downside of increasing the displacement and acceleration of the rail and sleepers.
- **Type of soil**
  There is a low reliability of the mechanical properties of some materials, namely the granular layers (ballast, sub-ballast, and foundation soil). The foundation soil of all the previous layers is probably the one with the least reliable properties, as such, the understanding of the structural response due to the variation of the soil stiffness is of great interest.

  From the analysis of the results was possible to conclude that the alteration of the soil stiffness has more influence on the acceleration results, than the interaction forces. The use of a softer soil led to an increase of the vertical acceleration results in all components in analysis. On the other hand, the wheel-rail interaction forces suffered very small variations.

- **Type of ballast**
  In recurrence of the previous topic and regarding the major importance that the ballast layer has on the global track performance, the understanding of the influence of the ballast stiffness is important in this study.

  On the follow-up of the previous aspect, the results showed that the variation of the mechanical properties of the multilayer system (ballast, sub-ballast, foundation soil) has much more influence on the track response per say (vertical accelerations) then the vehicle-track interaction (interaction forces).

- **Axle load**
  Since the dynamic solicitations suffered by the railway are mainly due to the vehicle-track interaction, the influence of some vehicle properties (axle load) is relevant to the influence analysis.

  This specific parameter was evaluated in two distinct ways, firstly by changing the axle load, and then by altering the suspended mass, or the non-suspended mass.

  The increase of the axle load led to a growth of the dynamic solicitations, beyond the static increase expected, has shown on Table 4.1.

  Regarding the suspended and non-suspended mass analysis, the results supported the work developed by Prud’Homme in 1970. The increase in non-suspended mass imposed led to higher dynamic amplifications that the same increase in the suspended mass. This fact is due to the frequency of vibration of the wheel-rail system being substantially superior to the frequency of vibration of the suspended masses.

- **Stiffness of the Under-Sleeper Pad (USP)**
  The under-sleeper pad has great influence on the behavior of the elements of the slab track. In order to understand why the slab track exhibits such a flexible behavior and presents such high values for the vertical accelerations of the rail and sleepers, the influence of the stiffness of the USP must be analysed.

  The increase of the USP stiffness led to a decrease of the variation of the wheel-rail interaction forces since the stiffness increase of the USP caused a boost of the global stiffness of the VLB and therefore an approximation of the VB and VLB track stiffness. The wheel displacement on the transition zone (see Figure 4.4) is a good parameter to observe this stiffness approximation.

### Table 4.1 – Wheel-rail interaction forces for different axle loads

<table>
<thead>
<tr>
<th>Wheel-rail interaction forces (kN)</th>
<th>Fm</th>
<th>Fm</th>
<th>ΔF</th>
</tr>
</thead>
<tbody>
<tr>
<td>Load -25%</td>
<td>138.4</td>
<td>122.5</td>
<td>15.8</td>
</tr>
<tr>
<td>Load</td>
<td>177.3</td>
<td>149.8</td>
<td>27.4</td>
</tr>
<tr>
<td>Load +25%</td>
<td>225.5</td>
<td>183.5</td>
<td>42.0</td>
</tr>
</tbody>
</table>

### Figure 4.4 - Wheel displacement for different USP stiffness

The increase of the USP stiffness was also responsible for a major reduction of the vertical accelerations of the sleepers in the VLB and a significant reduction on the acceleration in the VB just after the transition.

### 4.2. Analysis of different track design solutions suggested for the Case Study
Since the ballasted track applied on LGV-Est high speed line is “standard”, the best way to realistically contribute to an alternative solution is through the implementation of a completely different solution, or through the modification of some components of the VLB.

A set of alternative solutions were conceived and evaluated. These solutions can be divided in two groups: two cases were the USP was removed from the track (A and B), and four were an intermediate band was created (A, B, C and D).

From the first set of solutions, the “Removal of the USP – Case A” is characterized by the alteration of the railpad stiffness of the VLB to 60 kN/mm, in order to compensate the suppression of the sleeper pad. In this specific case the the railpad stiffness of the VB (Kpad) was the parameter in scrutiny. A schematic of the solution is illustrated in Figure 4.5.

![Figure 4.5 - Schematic representation of the “Removal of the USP - Case A”](image)

The “Removal of the USP – Case B” is the same type of solution has shown on Figure 4.5 but in this case the Kpad (VB) is 120 kN/mm and the parameter in analysis is the Kpad (VLB).

The second set of solutions regard the introduction of an intermediate band (TI), as shown on Figure 4.6. The “Creation of an Intermediate Band – Case A”, attempts to recreate the solution in existence at the Case Study location. In this case the band has an elastic layer beneath the ballast bed, the Under-Ballast Material.

![Figure 4.6 - Schematic representation of the IB solutions](image)

The intermediate band (TI) of “Case B” has the same typology has VB with exception to the stiffness of the railpad, which was changed to 60 kN/mm and therefore responsible for creating an intermediate stiffness level which results in a smoother stiffness variation.

The “Case C” is technically an upgrade from “Case B”. This solution is defined by two levels of intermediate stiffness, one where the railpad stiffness was altered to 60 kN/mm and other with 80 kN/mm. Besides the introduction of another level of stiffness, the extension of each stiffness band was increased from 4 sleepers (“Case B”) to 10.

The “Case D” alternative is a joint solution. Besides the implementation of an intermediate band with two levels of railpad stiffness (115 and 110 kN/mm), the USP present in VLB stiffness was also increased to 120 kN/mm.

After the analysis of all the previous solutions and parameters, a set of situations produced very interesting results and allowed to draw some conclusions and recommendations to apply on the case study.

- The increase of the USP stiffness led to an approximation of the global track stiffness of VLB to VB. This resulted in a relevant decrease of interaction forces and vertical accelerations, namely in the solution were KUSP = 500 kN/mm.
- The removal of the USP, with the maintenance of the railpad stiffness in 120 kN/mm resolved the problem of the high values of vertical acceleration of the sleeper. The suppression of the USP was also responsible for enclosing both track stiffness values, and therefore reducing the variation of the contact forces.
- The alternative TI “Case C” produced solid results. The introduction of two levels of stiffness resulted in a smoother transition than similar alternatives, and the increase of their extension to 6m (10 sleepers)
allowed the stabilization of the vehicle after the stiffness change, with repercussions on the results.

- The joint solution ("Case D") achieved the best results of all the alternatives. The existence of two levels of stiffness in the TI allowed for a smooth transition just like "Case C", and the increase of the USP stiffness in VLB reduced the vertical accelerations of the rail and sleepers. Concerning only the improvement of the structural response of the railway and the reduction of the repercussions of the stiffness variation, this was the best solution. With this solution it was possible to obtain acceptable values for the parameters in analysis in the transition adjacent area, without the increase of the VLB track stiffness to levels close to the VB, which are very high.

This solution was subjected to a higher vehicle circulation speed (400 km/h) and compared to previous results, (see Figure 4.7) in order to evaluate the structural response and to understand if the exemplar behavior showed at normal speed is maintained at very high speeds.

Figure 4.7 – Wheel-rail contact forces for different speeds in the chosen solution

From the analysis of Figure 4.7 is concluded that even for super high speeds the solution produces better results than the reference case (VLB/VB) for much lower speeds.

5. Conclusions and further developments

This paper presents the study of railway stiffness variations, with special interest in the transition between ballasted and slab track. Beyond this study is also an aim of this paper to analyze the present solution on the test section Chauconin in the French rail line LGV-Est.

Regarding the numerical tools representing the railway types of the case study (VB and VLB) the key aspects concluded are:

- The numerical results obtained acceptable correlations with the experimental data, with exception to the vertical acceleration of the sleeper. This slight difference might be due to the non-account of the rail defects, or the fact that the experimental stretch is set on curve.
- There is some uncertainty about the values of some mechanical properties of the granular materials, namely soil, ballast and sub-ballast. This effect can be responsible for the variance existent between the track stiffness determined through numerical results (95 kN/mm) and the experimental data, which is much superior.
- The VB is very stiff and therefore the dynamic solicitations suffered by the track are much superior resulting in a faster deterioration of the geometric quality of the track. In opposition the VLB is very flexible, due to the presence of the USP. The superior flexibility imposes an increased need of energy for vehicle circulation.

Concerning the influence analysis of some aspects in the transition zone response the key points concluded are:

- The transition from a more flexible (VLB) to a stiffer structure (VB) proved to be a more penalizing transition.
- The alteration of the elastic modulus of the ballast and foundation soil has little influence in the vehicle-track interaction, but is very relevant in the track structural performance.
- The axle load has a vital part in the behavior of the transition zone, a small increase in the static load led to a big increment in the dynamic solicitations, especially if that increase is imposed in the non-suspended mass.
- The USP stiffness proved to be the most relevant aspect in the VLB and transition behavior. The increase of stiffness led to a drastic reduction in the contact forces and the vertical accelerations in the rail and sleeper of the VLB.

Towards the presentation and analysis of different solutions to the case study the most important conclusions drawn are:
• The reduction of the stiffness variation only through the increase of the VLB stiffness obtained by removal of the USP, guaranteed excellent results in the parameters analyzed, but since the model doesn’t account for irregularities on the wheel-rail system, the consideration of the track performance in standard zone is not in place.

• The implementation of an intermediate stretch allowed to introduce a gradual stiffness variation in the transition zone. This intermediate band with a considerable extension and intermediate stiffness levels well defined led to great improvements in the structural behavior of the transition zone.

• In order to improve the global behavior of the experimental area there was a need to develop a joint solution, consisted of both an intermediate band and an alteration of the USP stiffness. With this solution, it was possible to guarantee a reduction of the dynamic solicitations experienced in the surrounding elements of the transition zone and at the same time an improvement of the performance of the VLB with a reduction of the track flexibility to stiffness values near the optimal for high speed railways.

With the advancement of computational power, it is possible to apply more complex analysis for the purpose of better understanding of stiffness variations in railways, the principal guidelines for future research are:

• Development of 3D models for a dynamic analysis, that allow the reduction of the number of simplifying hypothesis to apply in the model and to guarantee a greater reliability in the results.

• The incorporation of numerical mechanisms that allow the consideration of track heterogeneities and irregularities.

• The incorporation of nonlinear material analysis to some elements of the railway structure, like the foundation soil or the elastic elements.

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6. References


