Structure/Embankment Transitions in Railway Infra-structures

Behaviour and National and International Practices

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Abstract
Portugal is planning the construction of a totally new hi-speed railway line. This investment will challenge rail engineering. In that matter, Portugal has the advantage of looking to the practices and solutions used in countries with more experience and tradition in hi-speed railways. In this article, a specific part of this railway system will be studied: the transition between engineering structures and embankments. The references found in the available bibliography showed that these zones are one of the main problems in railway and road infra-structures.

A review in literature was done and the main problems that are usually found, as well as some mitigation solutions were identified. Track-bridge interaction is one the problems showed at the ERRI D 230-1/RP3, State of the Art Report (1999). In the available bibliography this problem was identified but never discussed. An introduction to it is found in this paper with an objective to show how the bridge effects might be adverse to the transition and how the structure can condition the actions in the transition zone.

Portuguese REFER EP (Public entity responsible for construction and maintenance of Portuguese railway lines) practices for these zones are described and related to the French SNCF/RFF transition practices.

Finally, thanks to REFER EP, it was possible to accompany the earthworks and control quality methods in a embankment-structure transition near a viaduct in the South Portuguese railway line.

Key words: Embankment, transition, railway, Bloc Technique, bridge approach.

1. Introduction

Transitions zones are sensible points of transport infrastructures. They are hidden to the eyes of the passenger but, many times, the passenger knows when he has just passed by one. In this environment stresses are measured in MPa and settlements must be measured in mm. So, the challenge of this zone consists in maintaining low deformability levels, allied to very high stresses and good passenger’s comfort levels. Insa (2008) describes railway lines has a complex system made of many sub-systems like infra-structure, power supply, security systems, superstructure. Transition zones embrace different sub-systems and have the challenge to connect different zones in the best possible way, like an embankment or a bridge. For them to be to study it is necessary to know each part of this sub-system: the adjacent structure, the upper superstructure and geotechnics.

2. General Aspects about transitions zones

In railways a transition zone occur each time there are two different stiffness zones. Fig. 2.1 shows this problem and relates it to another, the differential settlement between this two zones. According to Insa (2008), the key to reduce maintenance costs in railways consists in defining an optimum level of the vertical stiffness of the track and maintain it. This is surely a problem in transitions between embankment and bridges or tunnels. One of the objectives of the transition should be transforming the stiffness variation described in Fig. 2.1 to the showed in Fig. 2.2.

The ERRI (1999) indicates that the factors influencing the behaviour of the track in transitions zones are mostly external to the track (axle loads, weather conditions, speed and vibrations), of geo-technical
nature (sub-grade and soil conditions), structural (static system, bending stiffness, lateral movements and interaction between track and bridge) and of track design and layout (stiffness, location of track dilation devices or presence of CWR). This chapter will pass mainly in, structural, track design and layout factors.

### 2.1 Track design and Layout

Track dilation devices (TDD) allow longitudinal relative displacements between adjacent rails maintaining the track gauge and direction. These devices are mostly used in abutments of long railway bridges in order to compensate the different temperature dilation behaviours of rail, bridge deck and abutment (Hess, 2007). In some cases, these stresses may be too high. Rails have to be cut and TDD has to be implemented (Schmitt, 2007). The specifications for the maximum admissible values of stresses and displacements can be found in the EN1991-2. Usually associated with these devices, are the retaining ballast ones. These allow a separation between the ballast over the bridge deck and the abutment. This way, in the SNCF practices, for concrete bridges with a dilatable length over 100m, retaining ballast joints are used (Schmitt, 2007 and Ramondenc, 2008). TDD have high maintenance cost (Gil, 2006), and LWR must be as long as possible in order to minor maintenance; therefore bridge engineers have to take this recommendation into account (Schmitt, 2007) and for that some solutions for long railway bridges do not need TDD like described in Fig. 2.3:

![Fig. 2.3 – Long bridge solutions without TDD (Adapted from Schmitt 2007)](image)

The use of TDD or retaining ballast devices reduces the normal stresses of the rail in the top of the embankment in the transition zone. According to Reis et. al.(2007), when a CWR crosses a support discontinuity, such as embankment-bridge transition, horizontal forces are transmitted to the support, producing stresses concentration in the rails. The opposite is also valid, because continuous rails restrain the free movement of the bridge deck, so its deformations of the bridge deck (e.g. due to thermal variations, vertical loading, creep and shrinkage) will produce longitudinal forces in the rails and in the fixed bridge bearings (Reis et. al. 2007). These horizontal forces in the rails will produce shear stresses at the top of the transition zone. The EN 1991-2:2003 specifies the actions to take into account for the calculation of these rails: traction and braking forces, for double track bridges, should be considered the braking forces in one track and the traction forces in the other; thermal effects in combined structure and track system. According to EN1991-1-5 an uniform temperature variation in the bridge should be taken as ±Δ35°C; vertical traffic loads (including SW/0 and SW/2 where required) without the associated dynamic effects (these may be neglected); and other actions such as creep, shrinkage, temperature gradient etc, shall be taken into account for the determination of rotation and associated longitudinal displacement of the end sections of the decks were relevant. Yet, in the SNCF experience, according to Schmitt (2007), these long term effects may be neglected, specially creep and shrinkage due to track maintenance and long term rearrangement of ballast.

### 2.2 Structural influence in the transition zone

In railway bridges, track-bridge interaction will take place and influence the transition zone, where shear efforts could be stronger. The next lines will show these effects for, uniform thermal effects, traction and braking forces, and vertical loads.

For thermal effects, Schmitt (2007) shows, for a simple supported bridge case, the variation of the normal effort in the rail (F_{rail}) for the expanding of the bridge deck (Fig. 2.4a). In this case, F_{rail} will be higher near movable supports due to the different longitudinal displacements (u_2) > (u_1). Since the deck will be expanding, the rail will be compressed at the abutments, therefore, at the top of transition zone, will appear shear stresses according to the action: bridge contraction
(Traction in the rail) or bridge expanding (compression on the rail). For braking forces (Fb), Schmitt (2007) considers that the deck will not deform (ε = 0) so the displacement (u) will be the same in fixed or movable support. In the case described in Fig. 2.4b, the rail over the transitions will be compressed ahead of the train and in traction behind it. Finally, for vertical loads, the deck will be deformed (Fig 2.4c). At the top of the bridge decks, the longitudinal displacements will be different depending of the abutments type. Since the fixed support, will have only rotation, and the movable support will have both rotation an longitudinal translation, the normal effort in the transition zone near the fixed abutment will be higher than the effort in the transition near the movable support.

This way, track-bridge interaction will be responsible for shear stresses on the top of transitions zones. Even when there are any train passing, due to track bridge interaction, the top part of the transition zone will be supporting shear stresses due to thermal effects, creep and shrinkage. During train passage, the deck will deform and the CWR will adapt to this deformation. The sleepers above the transition zone will tend to float (Fig. 2.5a) and the shear stresses in the top of the transition will be increased. When the train is in both sides (bridge ant transition) the sleepers will be pushed to the ballast and the vertical loads will increase due to this dynamic effects (Fig. 2.5b). Finally, only the transition will be load by the train (Fig. 2.5c).

Besides track-bridge interaction, White (2005) described, for road bridges, the influence of lateral movements of bridges at the transitions zones near integral abutment. In his study, in the USA, many transition zones had problems related to horizontal movements of the bridge. In the cases of integral abutment, seasonal temperature fluctuations and concrete thermal strain characteristics. The bridge superstructure will expand or contract and the integral abutment will move together. As temperature increases, the abutment moves toward the adjacent soil (Fig. 3.4a). With the decrease of temperature, superstructure and abutment tend to move away from the soil, creating a void between soil and abutment (Fig. 3.4b). This void may lead to soil erosion which will increase the void size near the abutment and below the approach slab visible on Fig. 3.4c (White, 2005).
The expansion and contraction movements of the bridge can be calculated with the above equation [1]. White (2005) refers to the studies of Wahls (1990) and Holts (1982), that in addition to the temperature change effects, indicate embankment settlement as a primary factor leading to formation differential settlements.

### 2.3 Embankment settlement

Embankment settlement can be caused by settlement of foundation soil, poor compaction of fill material, poor drainage, and loss of fill material by erosion (White 2005). In his studies, White (2005) mentions Briau et al. showing a synthesis, visible in Fig. 2.5 of the problems identified in road bridges by this author.

#### Fig. 2.5 Problems leading to differential settlements in transitions
(Adapted from Briau et al.)

#### Fig. 2.6 Track modulus for different sleeper/ties
(Adapted from Read and Li, 2006)

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### 2.4 Mitigation

In the available literature many solutions are used to provide a gradual stiffness variation. A study by Read an Li (2006) showed that a good solution to surpass it acts in parallel between reducing structure stiffness and increasing the embankment stiffness. Read and Li (2006) made a comparison of the track modulus for three different ballasted deck bridge sleepers types. Fig. 2.6 shows that in their studies, both composite sleepers and concrete sleepers with rubber pads, were a successful solution for reducing the modulus in the bridge. The solution with composite ties decreased the track modulus over the bridge to an almost equal value of the embankment modulus. The rubber pads solution decreased this modulus, in a factor of 2.8, to a minor value of the embankment modulus (Read and Li 2006). According to Riessenberg (2006) rubber pads allow an improvement between the contact of ballast and sleepers from a contact percentage of 3 to 4% (using sleepers without pads) to 30% (using pads). The rubber pads used in the studies of Read and Li (2006) were placed between sleepers and ballast. They are designed to achieve a specific stiffness rate and their use indicates a reduction in the ballast wear (Read and Li 2006).

Another solution, also described in Read an Li (2006) studies, acts at the geotechnical level by means of invert soil wedges. These wedges tend to increase vertical stiffness of the track with the bridge/structure approximation. A review on the UIC CODE 719 R (2006) shows that this is the typical European solution. The next chapter will focus on the French and Portuguese Solutions, both using the treated and non treated crushed aggregates (cement treated near abutment and non treated in the wedge near the normal embankment).

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### 3 National and International experience

In the UIC 719R, many solutions are showed for transitions zones. Most of them use soil wedges as a solution for the different stiffness of the embankment and structure zone. In this chapter an presentation of the French and Portuguese solution will be done. The Portuguese solution here presented is very similar to the Spanish solutions present in the UIC 719R. The French SNCF/RFF transitions solutions, consulted in the “Remblais contigues aux Maconneries” (SNCF/RFF 2002), shows different solutions according to the structure type (inferior passage, abutment characteristics, hydraulic passages, etc), embankment height - H (H > 10m, 4m ≤ H ≤ 10m, 3m ≤ H ≤ 4m e H < 3m ), and construction timing.
(embankment constructed before or after the structure). The Portuguese solution adopted in the South Portuguese line has two main types of transitions: one for bridges and under crossing "box-culvert" type, with two different soil wedges) and another for smaller underground inferior passages, such as hydraulic passages with only one soil wedge. The wedge near the abutment is cement treated (Grave Traitéé with 3% of cement). This wedge has the particularity of maintaining a constant dimension with 1m beside the abutment and with a thickness of 3m (except for H=4, in this case a thickness of 4m should be used). The non treated wedge has a minimum longitudinal length at the top of 4m. In both Portuguese and French solutions, in the interface between regular embankment and non treated soil wedges, a decline of 3/2 is adopted; in the cement treated a decline of 1/1 is used. So the main differences between the SNCF solutions and the REFER EP solutions are: the higher longitudinal lengths of the Portuguese soil wedges, (total Portuguese length of 20m); the SNCF definition of a fixed cement treated wedge height at the “sous couche” level for a minimum length of 10m.; the changing applied in the ballast thickness with a transition height from 0.35m to 0.45m and also , a higher compaction level (100%OPM) at sous couche. This is visible in Fig. 3.1.

The introduction of a treated top layer might be a good solution for the track-bridge interaction problems, and the dynamic loads ones.

### 4 Portuguese Study Case

#### 4.1 General Presentation

The case study was obtained in co-operation with REFER E.P. (Portuguese responsible for maintenance and construction of railways infra-structures). The studied transition zone, was inserted in a track stretch of the Portuguese South railway line that links Lisbon to Algarve. This track has a design speed of 220km/h. The circulation will have either passengers and freight. The maximum axle load allowed is 25T with an axle spacing of 1.6m. Fixation between rail and sleeper will be vox. The railway structure used in embankment is described in table 4.1. It was possible to accompany the construction of some earth/structure transitions. This paper focus particularly the design and construction of transitions near the Água Cova viaduct and the results of track modulus tests (portancemètre) made at the form layer level in two hydraulic inferior transitions. All charts, figures and tables showed in this chapter were done from REFER data.

#### 4.1 Água Cova Viaduct

Água Cova viaduct has a pre-stressed concrete deck with 12,50m that allows a double ballasted double track solution. The structural solution adopted was studied to allow continuous welded rail without track dilation devices and is similar to the showed in fig. 2.3a. The North abutment is integral and south abutment has movable bearings. The maximum dilatable length of the deck was limited to a maximum length of 90 m. Track-bridge interaction forces were obtained by finite element calculation in SAP2000. The additional normal effort in the rail obtained and the displacements where less then the maximum imposed in UIC 774-3, and EN 1991-2:2003. The maximum values obtained in the south abutment were:

\[ N_{\text{max}} = 483 \text{ kN/rail} \]
\[ N_{\text{min}} = 478 \text{ kN/rail} \]
4.2 Materials and construction descriptions of the transition studied

The solution adopted for the transition zone was already described. Four different soils in the two distinct wedges were used. Near the abutment it was used a cement treated crushed aggregate corresponding to the soils 629/08 and 776/08. Between these soils and the embankment, crushed aggregates in two soils were used: 483/08 and 680/08. Fig. 4.1 sketches the solution adopted:

Each one of this soils (cement treated and non treated) fulfilled the conditions established by REFER EP, plus the granulometric fuse described in table 4.2:

- % of retained material in 20 mm ASTM sieve minor or equal to 30%;
- Liquid limit minor than 25%;
- Plasticity index minor or equal to 6%;
- Sand equivalent major or equal to 40%;
- Value of methylene blue minor or equal to 2;
- Value of Los Angeles minor or equal to 45%
- Sulphate content minor or equal to 0.5%
- Index of shape minor or equal to 35%

4.3 Construction of the South Transition zone of Água Cova viaduct

At the date of the visits, earthworks had already finished in the Norte transition. Yet, were just starting in the south abutment. So, it was possible to monitor the earthworks in the south embankment transition. In the background, all the materials and compaction methods had already been properly approved. The aggregates were deposited near the central mixing located at the construction work. This ensures that time between central mixing of the crushed aggregates, and compaction works will not be longer than two hours. In the central, water contents of the soil is made in a way that at compaction time, the soil water contents will be near it’s optimum value. To ensure this, after a first mixing all aggregates pass in a weighing machine and cement and water are added in the proper percentages for the soil. The soil is re-mixed, transported, bead in situ, uniformly spreader, so that the final thickness of this layer maintains a maximum of 30 cm after compaction, and finally compacted. Compaction levels were reached using two different rollers: a vibrator tandem and a vibrator compaction roller (19T). The smaller roller (tandem), was used near the structure and the bigger vibrator (simple) compaction roller was used in the other zones of this layer. In each layer, the rate of compaction, dry density and water contents were checked with a troxler equipment. So, after some iterations, the minimum of 98%OPM was to be guaranteed in this layer or, the layer should be replaced.

<table>
<thead>
<tr>
<th>Sieves EN 932-2 [mm]</th>
<th>Passed material [%]</th>
</tr>
</thead>
<tbody>
<tr>
<td>40</td>
<td>100</td>
</tr>
<tr>
<td>31,5</td>
<td>75 - 100</td>
</tr>
<tr>
<td>20</td>
<td>56 - 86</td>
</tr>
<tr>
<td>14</td>
<td>48 - 78</td>
</tr>
<tr>
<td>10</td>
<td>41 - 71</td>
</tr>
<tr>
<td>6,3</td>
<td>32 - 58</td>
</tr>
<tr>
<td>5</td>
<td>28 - 54</td>
</tr>
<tr>
<td>4</td>
<td>26 - 51</td>
</tr>
<tr>
<td>2</td>
<td>22 - 46</td>
</tr>
<tr>
<td>0,42</td>
<td>12 - 30</td>
</tr>
<tr>
<td>0,125</td>
<td>5 - 15</td>
</tr>
<tr>
<td>0,063</td>
<td>1 - 8</td>
</tr>
</tbody>
</table>

Table 4.2 – Granulometric fuse
During the earthworks, some problems were identified. In some zones, it’s impossible to fulfil the needs of the compaction rate. Fig. 4.2 shows that under the wing-wall’s the alteration of geometry doesn’t allow the use of compaction equipment.

4.3 Quality control

For each soil, a sample was studied in laboratory to see if it as fulfilled the characteristics described in table 4.2. It was calculated the proctor (modified) curve for each soil sample. Since these soils present a good percentage of aggregates with a diameter above the #3/4” the results obtained were corrected for the real soil grading. The in situ quality control was based in troxler tests at each layer. The results of the troxler equipment were previously checked with sand bottle tests with good results. Also, before the spreading of the cement treated layers, some sampler soil was obtained and cylinder test pieces were diametral fractured at 7, 28 and 90 days to compare this values with the specifications minimum requirements: Average Diametral Resistance > 0.25 MPa at 7 days (CE REFER E.P.). At form layer level and sub-ballast the track modulus is controlled in continuous with an portancemètre.

4.4.1 Non Treated layers wedge

In the tout-venant (non treated) layers two soils were used corresponding to the laboratory tests 483/08 and 680/08. The first one was applied on the two bottom layers of this wedge, and the second filled the rest of the soil wedge. The construction was phased so that layers of treated and non treated soil could be linked. The modified proctor results tests are visible in figures 4.3a e 4.3b. In the 483/08 soil, water contents results show a dispersion between wet and dry side between ±20% Wopt (Fig. 4.4a). This dispersion did not restrain that the compaction grade results could archived values of 101%OPM and 100%OPM, corresponding to 33% and 67% of the troxler tests in this soil (Fig. 4.3b). In the 680/08 soil the deviation of water contents was still large and it did not show any tendency either to results in the dry or wet side. In this case, a satisfying compaction rate was still obtained in all the troxler tests with 33% of tests showing compaction grades of 98% and 100% (Fig. 4.4a and 4.4b).
4.4.2 Cement Treated wedge

As shown in Fig. 4.1, in this wedge two soils, described with samplers 629/08 and 776/08, were used. Both soils were studied at laboratory and the modified proctor results are visible in Fig. 4.6a and 4.7a. The in situ tests, lead, in the case of 629/08 soil, to water contents results mainly in the dry side. In this wedge, some trowel tests revealed compaction rates below the minimum specified of 98% OPM. These layers were replaced. The results obtained show 37% and 34% of tests with 100% OPM and 99% OPM (Fig. 4.5b).

In the soil represented by sampler 776/08, the majority of the water contents were in the wet side. This might be the reason why 37% of compaction rate the tests showed a 101% OPM; as Fig. 4.6b show, all the tests showed compaction grades above the minimum 98% OPM (Fig. 4.6b).

4.4.3 Diametral Resistance Tests

In the top layers of the cement treated wedge diametral resistance tests were done. The whole process was made under the specifications present at BS 1924 - test 5, and the test pieces were fractured at 7, 28 and 90 days. The REFER specifications indicate that the percentage of cement in these soils should be studied so that at 7 days the average diametral resistance should be bigger than 0.25. Yet, instead of a really study, the solution adopted was a simple check that for 5% of cement this minimum requirements were guaranteed. So, after checking this, the solu-
tion adopted was an addition of 5.5% of cement. This is almost the double of the SNCF/RFF solution (3%). As expected, the results of diametral resistance of the tests pieces in this transition, visible in Fig. 4.7, showed higher values than the minimum required. The average of the tests showed a crescent resistance in time with a tendency to stable. Some tests pieces showed more resistance at 60 days than at 90 days, in an abnormal behaviour. The average results were 0.567 MPa (7 days), 0.723 MPa (28 days) and 0.758 MPa (90 days). These results are more than two times bigger than the minimum of 0.25 MPa (7 days).

4.5 Deformation test with portancemètre

The portancemètre allows to continuously check the track modulus. These tests were done at form layer level and sub-ballast layer in order to guarantee that the minimum values were respected and, more important, in order to see how uniform was the behaviour of this modulus in the track. Fig. ASD is described a test done in 425m at form layer level. The two hi jumps correspond to a passage above a hydraulic passage (HP 13.1 and HP 14.1) visible in Fig. 4.8. The bottom of the form layer was at the same level of the top of both the HP, the solution for the transition is described in Fig. 4.9. The minimum EV2 required for the form layer is of 80MPa. The majority of the results obtained, showed values between two to four times the minimum value requirements.

5 Conclusions

Some of the highly critical aspect of railways were discussed thought this paper: transitions zones and track-bridge interaction. The structure/bridge has an influence in the transition zone; bridge contractions and expanding generate a void development near integral abutments (White 2005), specially in transitions with slab, where a void beneath the transition slab tends to occur. Also, in railways, the effects of track-bridge-interaction are transmitted to the transition zone by the rail. The top of the transition zone is in constant shear stresses. When a train is over a bridge, the deformation of the bridge will develop a release in sleepers weight, above the transition zone, in an effect that might lead to floating sleepers. So, when the train passes in both bridge and transition, these released sleepers will now push the ballast, producing higher stresses in this zone. This work was centrally in the connection between structure and embankment, so dynamic aspects due to wheel/rail interaction weren’t discussed but should be taken in account in transitions zones. Studies of Li and David (2006) indicate a good solution for the differential stiffness problems is to act simultaneously in structure and embankment. In their studies the solutions of composite sleepers and rubber pads under concrete sleepers proved to work, at the stiffness level, indicating that a smooth vertical stiffness transition is possible.

The French transitions described, were empirical dimensioned and are being used, according to SNCF, without the typical maintenance problems. These solutions have a longitudinal development much smaller than the Portuguese’s. Both treated and non treated wedges are smaller in the SNCF solution. Also the cement percentage is almost half of the used in REFER case study (with cement of the same class 32,5 N). Another difference is the application of a treated zone above the transition at the sous de couche level with a higher proctor and 3% cement added. This treated structure might be a good solution for the track-bridge interaction problems discussed, since a major compaction level and cement addition should raise loading capacity to shear and vertical stresses. SNCF solution applies a higher ballast thickness over
the structure (35 cm over embankment and 45 cm over structure), this way this solution acts on both embankment and structure with a reduction of the stiffness level over the structure. Finally, the SNCF has a total of 44 solutions depending of the type of structure, the embankment height, the recurrent thickness and the working process.

The Portuguese construction practices showed, in this study case, good compaction grades and high levels of track modulus at the portancemètre test. The main difficulties passed through guaranty that, at compaction time, the water contents levels were near the optimum values. Also, it’s necessary to define an optimum cement percentage that might allow cost reductions.

Transitions instrumentation, monitoring, are urgent to allow a well calibrated finite element calculation. This models should also include the adjacent structures and could answer to how important are the influences of the embankment heights, the presence of track dilation devices and/or retaining ballast joints, the abutment type (integral, with movable bearings), the wedges geometry, and the recurrent thickness.

6 References


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