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ESCUELA TÉCNICA SUPERIOR DE INGENIEROS DE CAMINOS, CANALES Y PUERTOS

ANALYSIS OF THE PROBLEMS ASSOCIATED WITH DYNAMIC INTERACTION BETWEEN TRAIN, TRACK AND STRUCTURE AT TRANSITION ZONES

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2015
Para la mayoría de nosotros, la ciencia es un conjunto de conocimientos que explican cómo es el mundo. Pero sorprendentemente, para los que se dedican a ella no es así. Los investigadores definen esa cosa llamada ciencia más como un método de conocimiento que como un punto de llegada. Es decir, que lo importante no es lo que se descubre, sino cómo se descubre. Lo importante es la pregunta y no la respuesta, porque esta va variando a lo largo de la historia: nuevos hallazgos destronan anteriores teorías. Lo que define la ciencia, por tanto, es el método científico, la forma de llegar a esas respuestas. Y el método científico se ha ido forjando con las sucesivas preguntas que se han hecho los hombres y mujeres que han escrito la ciencia. LO IMPORTANTE ES LA PREGUNTA...

(Párrafo recuperado de la ciencia, para la ciencia, por mi padre Ángel Miguel Fragoso)
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**Resumen**

El gran desarrollo experimentado por la alta velocidad en los principales países de la Unión Europea, en los últimos 30 años, hace que este campo haya sido y aún sea uno de los principales referentes en lo que a investigación se refiere.

Por otra parte, la aparición del concepto *super – alta velocidad* hace que la investigación en el campo de la ingeniería ferroviaria siga adquiriendo importancia en los principales centros de investigación de los países en los que se desea implantar este modo de transporte, o en los que habiendo sido ya implantado, se pretenda mejorar.

Las premisas de eficacia, eficiencia, seguridad y confort, que este medio de transporte tiene como razón de ser pueden verse comprometidas por diversos factores. Las zonas de transición, definidas en la ingeniería ferroviaria como aquellas secciones en las que se produce un cambio en las condiciones de soporte de la vía, pueden afectar al normal comportamiento para el que fue diseñada la infraestructura, comprometiendo seriamente los estándares de eficiencia en el tiempo de viaje, confort de los pasajeros y aumentando considerablemente los costes de mantenimiento de la vía, si no se toman las medidas oportunas.

En esta tesis se realiza un estudio detallado de la zonas de transición, concretamente de aquellas en las que existe una cambio en la rigidez vertical de la vía debido a la presencia de un marco hidráulico. Para realizar dicho estudio se lleva a cabo un análisis numérico de interacción entre el vehículo y la estructura, con un modelo bidimensional de elemento finitos, calibrado experimentalmente, en estado de tensión plana. En este análisis se tiene en cuenta el efecto de las irregularidades de la vía y el comportamiento mecánico de la interfaz suelo-estructura, con el objetivo de reproducir de la forma más real posible el efecto de interacción entre el vehículo, la vía y la estructura.

Otros efectos como la influencia de la velocidad del tren y los asientos diferenciales, debidos a deformaciones por consolidación de los terraplenes a ambos lados el marco hidráulico, son también analizados en este trabajo.

En esta tesis, los cálculos de interacción se han llevado a cabo en dos fases diferentes. En la primera, se ha considerado una interacción sencilla debida al paso de un bogie de un tren Eurostar. Los cálculos derivados de esta fase se han denominado cálculos a corto plazo. En la segunda, se ha realizado un análisis considerando múltiples pasos de bogie del tren Eurostar, conformando un análisis de degradación en el que se tiene en cuenta, en cada ciclo, la deformación de la capa de balasto. Los cálculos derivados de esta fase, son denominados en el texto como cálculos a largo plazo.

Los resultados analizados muestran que la utilización de los denominados elementos de contacto es fundamental cuando se desea estudiar la influencia de asientos diferenciales, especialmente en transiciones terraplén-estructura en las que la cuña de cimentación no llega hasta la base de cimentación de la estructura. Por otra parte,
tener en cuenta los asientos del terraplén, es sumamente importante, cuando se desea realizar un análisis de degradación de la vía ya que su influencia en la interacción entre el vehículo y la vía es muy elevada, especialmente para valores altos de velocidad del tren.

En cuanto a la influencia de las irregularidades de la vía, en los cálculos efectuados, se revela que su importancia es muy notable, siendo su influencia muy destacada cuanto mayor sea la velocidad del tren. En este punto cabe destacar la diferencia de resultados derivada de la consideración de perfiles de irregularidades de distinta naturaleza. Los resultados provenientes de considerar perfiles artificiales son en general muy elevados, siendo estos más apropiados para realizar estudios de otra índole, como por ejemplo de seguridad al descarrilamiento. Los resultados provenientes de perfiles reales, dados por diferentes Administradores ferroviarios, presentan resultados menos elevados y más propios del problema analizar. Su influencia en la interacción dinámica entre el vehículo y la vía es muy importante, especialmente para velocidades elevadas del tren. Además el fenómeno de degradación conocido como danza de traviesas, asociado a zonas de transición, es muy susceptible a la consideración de irregularidades de la vía, tal y como se desprende de los cálculos efectuados a largo plazo.
Abstract

The major development experienced by high speed in the main countries of the European Union, in the last 30 years, makes railway research one of the main references in the research field.

It should also be mentioned that the emergence of the concept superhigh-speed makes research in the field of Railway Engineering continues to gain importance in major research centers in the countries in which this mode of transportation is already implemented or planned to be implemented.

The characteristics that this transport has as rationale such as: effectiveness, efficiency, safety and comfort, may be compromised by several factors. The transition zones are defined in railway engineering as a region in which there is an abrupt change of track stiffness. This stiffness variation can affect the normal behavior for which the infrastructure has been designed, seriously compromising efficiency standards in the travel time, passenger comfort and significantly increasing the costs of track maintenance, if appropriate measures are not taken.

In this thesis a detailed study of the transition zones has been performed, particularly of those in which there is a change in vertical stiffness of the track due to the presence of a reinforced concrete culvert. To perform such a study a numerical interaction analysis between the vehicle, the track and the structure has been developed. With this purpose a two-dimensional finite element model, experimentally calibrated, in a state of plane stress, has been used. The implemented numerical models have considered the effects of track irregularities and mechanical behavior of soil-structure interface, with the objective of reproducing as accurately as possible the dynamic interaction between the vehicle the track and the structure.

Other effects such as the influence of train speed and differential settlement, due to secondary consolidation of the embankments on both sides of culvert, have also been analyzed.

In this work, the interaction analysis has been carried out in two different phases. In the first part a simple interaction due to the passage of a bogie of a Eurostar train has been considered. Calculations derived from this phase have been named short-term analysis. In the second part, a multi-load assessment considering an Eurostar train bogie moving along the transition zone, has been performed. The objective here is to simulate a degradation process in which vertical deformation of the ballast layer was considered. Calculations derived from this phase have been named long-term analysis.

The analyzed results show that the use of so-called contact elements is essential when one wants to analyze the influence of differential settlements, especially in embankment-structure transitions in which the wedge-shaped backfill does not reach the foundation base of the structure. Moreover, considering embankment settlement
is extremely important when it is desired to perform an analysis of track degradation. In these cases the influence on the interaction behaviour between the vehicle and the track is very high, especially for higher values of speed train.

Regarding the influence of the track irregularities, this study has proven that the track’s dynamic response is heavily influenced by the irregularity profile and that this influence is more important for higher train velocities. It should also be noted that the difference in results derived from consideration of irregularities profiles of different nature. The results coming from artificial profiles are generally very high, these might be more appropriate in order to study other effects, such as derailment safety. Results from real profiles, given by the monitoring works of different rail Managers, are softer and they fit better to the context of this thesis. The influence of irregularity profiles on the dynamic interaction between the train and the track is very important, especially for high-speeds of the train. Furthermore, the degradation phenomenon known as hanging sleepers, associated with transition zones, is very susceptible to the consideration of track irregularities, as it can be concluded from the long-term analysis.
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1 Introduction

1.1 Motivation

High Speed lines (HSLs) network has undergone considerable development in the past 30 years in Europe.

Several European countries decided to develop a new and fast mode of transport with the highest standards of efficacy, efficiency, safety and comfort.

Italy was the first European country to inaugurate an HSL (Florence-Rome) in 1977. France introduced the first high-speed train (Paris-Lyon) in 1981 German High Speed began ten years after the French High Speed (Hamburg- Altona) in 1991. Spain was the next country which introduced the Alta Velocidad Española AVE (Madrid-Seville) in 1992.

The reduced traveling times, the increment of passenger comfort and the low environmental impact enable HSLs to compete with another complement modes of transport. Notwithstanding the railway track suffers degradation over its life cycle. Degradation phenomenon is particularly evident at locations with abrupt changes in the track stiffness, such as transition zones to bridges, culverts, ends of tunnels, rail crossings or passages from slab tracks to ballasted tracks. This has negative consequences and
could adversely affect the aforementioned standards of efficacy, efficiency, safety and comfort if no preventive actions are implemented. The maintenance incidence at transition zones may be three to eight times higher than that in normal plain track. The research works carried out by Paixão (2014) and Ribeiro (2012), highlight the importance the need to take into consideration the problem of track degradation at transition zones in ballasted tracks. These studies have been carried out by conjugating both the numerical and experimental analysis with an Alfa Pendular train at a maximum speed of 220 km/h. Main conclusions to be drawn from the previous works, were the importance of developing a more comprehensive analysis bearing in mind a new type of train, traveling at different speeds, and considering also the effect of differential settlements and track irregularities.

On the other hand, research of transition zones is particularly important in the context of the main European Railway Research Projects, namely: SUPERTRACK, INNOTRACK and CAPACITY4RAIL.

Despite the importance of transition zones in the railway lines, the fundamental causes of their poor performance are not fully understood.

This thesis focuses on the influence of certain parameters as: frictional behaviour of the soil/structure interface; longitudinal level track irregularities and consolidation settlement in embankments, in the dynamic interaction behaviour between the train and the track at a particular transition zone of the Portuguese North Railway Line. The choice of this transition zone, consisting of a culvert and the corresponding wedge-shaped backfills, is justified by the existence of available information to characterize it and the existence of an experimentally calibrated bidimensional finite element model developed by Ribeiro (2012). This numerical model has been modified and adapted in order to take into account the effect of frictional behaviour of the soil/structure interface, track irregularities and differential settlements caused by consolidation of embankments.

1.2 Aim of the research

The main objective of this thesis is to improve the knowledge of the degradation process of a ballasted track at a transition zone, when certain parameters as: track irregularities (longitudinal level) and frictional behaviour of the soil/structure interface are considered. In order to get that far, several goals have been reached and described in this work:

- Summarize and present the main issues associated with transition zones in a ballasted track, developing a state-of-the-art that provides a solid background of the problem.
- Undertake a detailed review of the numerical models used to assess the dynamic behaviour of the railway track and transition zones, scope of this study.
• Perform an assessment on the frictional behaviour in different contact interfaces, evaluating the main features of the contact elements used to simulate the interface between different elements of the transition zone analyzed.

• In order to take into account track irregularities in this work, it is necessary to analyze the geometrical characteristics for both the artificial profiles, generated according to the requirements of different Railway Administrators, and the experimental profiles that come from the monitoring operations of REFER and ADIF Railway Administrators. In this analysis, geometrical quality of the longitudinal level irregularity profiles is evaluated according to specifications provided by TSI (2008) and EN13848-5 (2010).

• Adapt the existing bidimensional numerical model of a transition zone by including the effects of longitudinal level track irregularities and frictional behaviour of the soil/structure interface in the dynamic interaction behaviour between the train and the track.

• Analyze the effect of consolidation settlements in the embankments, that cause differential settlements in the vicinity of the culvert, in the dynamic interaction behaviour between train and track systems.

• Evaluate the influence of differential settlements, frictional behaviour in the soil/structure interface, speed of the train and longitudinal level track irregularities in a short-term dynamic analysis: when the bogie of the train moves over the transition zone.

• Evaluate the influence of the frictional behaviour in the soil/structure interface, track irregularities (longitudinal level) and speed of the train in the degradation process of the track at the transition zone. In this degradation process, several number of loading cycles are considered with the aim of performing a long-term analysis in which, a common track degradation phenomenon at transition zones as hanging sleepers, could be properly detected and analyzed.

1.3 Outline of the thesis

This thesis is composed of seven Chapters plus an additional bibliography section that complete the dissertation document. A brief description of the thesis is given below.

The first, and current, Chapter provides a short introduction on the background and motivation of the work presented herein. The main objectives of the work are also stated in this Chapter.

Chapter 2, contains a literature a review on transition zones. In this Chapter, a special attention is given to the vertical track stiffness, settlements of the track and adopted solutions in the ballasted track. Normative limits that define the track quality and other specifications are also presented in this Chapter.
Chapter 3, contains a review of the numerical models used to analyze dynamic behaviour of the track and transition zones. In this Chapter not only models for the track and transition zone are included, but also the main models of the vehicle used to analyzed track-train dynamic interaction. Description of the tool used to perform the long-term simulation at transition zones is presented at the end of this Chapter.

Chapter 4 presents the methodology that enables to have into account certain variables as: the frictional behaviour in the soil/structure interface, longitudinal level track irregularities and ballast settlement in the numerical model with which dynamic interaction results both in the short-term and long-term are obtained.

In Chapter 5, short-term results of the dynamic interaction analysis are evaluated, for different speeds of the train and different scenarios. These results justify the analysis carried out in the degradation assessment developed in the following Chapter.

Chapter 6 presents the simulation of the degradation process that occurs at the transition zone, that is the subject of this study. In this Chapter, a special attention is given to the hanging sleepers phenomena, one of the main causes for excessive degradation in ballasted track at transition zones.

Finally, Chapter 7 presents the conclusions of the research and gives suggestions for future work.
2 Railway transition zones in a ballasted track

2.1 Introduction

Transition zones are commonly defined as a region of the track in which there is an abrupt change of track stiffness (mainly in the vertical direction). Such a change of stiffness may cause an increment of interaction forces between the train and the track and consequently an acceleration of the degradation process, when a train is moving on it.

Examples of this kind of elements are:

- Bridge-embankment transition.
- Regular track - culvert transition.
- Plain track - tunnel transition.
- Ballasted track to slab track transition.

Because of it, a special attention has to be paid on these sections, building it properly to mitigate several problems associated with these critical zones.

It is also important to mention that these zones behave as a discontinuity at which the track geometry degrades at an higher rate than on the regular normal free track. Some causes of such a degradation are referred by several authors: Briaud & James (1997), Li & Davis (2005), Li et al. (2006), Lundqvist & Dahlberg (2005), Coelho et al. (2011) and Varandas (2013).

The reasons of track degradation, at transition zones, can be summarized as follows:

1. If the stiffness of the track changes abruptly, the interaction forces between train and track increase, leading to localized differential settlement of the track.

2. In transition zones with embankments, sections above it, tend to have a greater settlement than sections above the stiff structure, leading again to the problem of differential settlements.

Some examples of transitions zones in a railway line are depicted in Figure 2.1.
2. Railway transition zones in a ballasted track

The differential settlements at these zones are commonly denoted as bump (protuberance on the surface), see Figure 2.2.

Figure 2.2: Typical bump development at transition zones, adapted from ERRI-D 214/RP9 (1999)

The aforementioned interaction effects in transition zones, induce a fast deterioration of the track with the consequently increment in the maintained costs, reliability of the circulation on it, and affecting the specified requirements of security and comfort.

Also, the special geometry requirements for a high speed railway lines imply a special attention at these critical points to try to design and keep in good conditions the existent lines, in the frame of efficacy and efficiency.

The main purpose of this Chapter is to provide an overview of the problem, in terms of state of the service, of this critical sections of the track as well as analyzing the main features and components of the track that lead us to have a better understanding of the problem and its possible solutions.

2.2 Track components

Ballasted track is composed of two subsystems: the superstructure and the substructure. Superstructure is formed of rails, sleepers, ballast and sub-ballast. Substructure is composed of a formation layer and the sub-ground. Some authors consider ballast
and sub-ballast layers as a part of the substructure, may be because in on the top of these layers where the sleepers are embedded, Selig & Waters (1994).

![Figure 2.3: Track with its different components, adapted from Dahlberg (2003)](image)

Below, a brief description of each component of the track will be made.

**Rail:**

It is the first resistant element of the track, it receives the load from the wheel and transmit it to the sleepers. The rail has other important function, it works as a guide element of the train in the direction of the track. Charles Vignoles improved the original I-shape profile of rails until the commonly used rail profiles nowadays, UIC-60, where 60 refers to the mass of rails in kg per meter. The section of the rail has to provide stiffness, through a flexural strength, it has to carry and transmit vertical and horizontal forces that act on the track.

![Figure 2.4: Cross section of the UIC-60 rail](image)

**Fastening system and railpad**

These are the elements that transmit the load from the rail to the sleeper, see Figure 2.5 (a). Fastening system connects the rail and the sleeper and has the following functions:
2. Railway transition zones in a ballasted track

- Keep the track width and inclination of the rails within reasonable limits.
- Keep the connection between rail and sleeper without loss of contact.
- Prevent longitudinal displacements of the rail.
- Provide an appropriate elasticity to the rail-sleeper support.
- Provide electrical insulation to the track.
- Absorbing vibrations and impact loads of the train.

![Image](image1.png)  ![Image](image2.png)

Figure 2.5: Fastening system and railpad: (a) Detail of a fastening system; (b) Detail of the railpad

Railpad is an important element, especially in high speed lines, it is placed between the rail and the sleeper, see Figure 2.5 (b). It protects sleepers from impact damage, provides electrical insulation and makes the load transition from rails to sleepers less severe.

Railpad has a great influence in the overall track stiffness. Furthermore, from a dynamic point of view, it improves the behaviour of the track, by isolating sleepers from high frequency vibrations.

**Sleepers**

Sleepers are the support of rails, they keep the gauge, level and alignment of the track. Therefore sleepers transmit forces (vertical, horizontal and longitudinal) from rails to the ballast layer. As the previous elements, sleepers provide electrical insulation to the track. Earlier sleepers were made primary of wood, but nowadays, in high speed lines almost every sleepers are made of reinforced concrete.

Concrete sleepers may be divided into two main groups, according to the shape and mechanical characteristics: mono-block and bi-block sleepers. Mono-block sleepers are the most used into the European high speed lines net, see Figure 2.6 (a).
2.2 Track components

![Concrete sleeper and Under sleeper pad images](image)

Figure 2.6: Concrete sleepers and Under sleeper pad: (a) Detail of a typical concrete sleeper of a HSL; (b) Detail of the under sleeper pad between the sleeper and the ballast layer

**Under sleeper pad (USP)**

The primary objective of this element is to modify the global track stiffness. They are specially used in transition zones and some railway managers have introduced USP as a standard solution for special cases of applications within their networks, see Figure 2.6 (b). Some positive effects of USP are:

- Improvement of the initial track geometry quality and of the deterioration rate.
- Reduction of structure-borne noise and vibration.
- Reduction of long pitch corrugation in tight radius curves.
- Reduction of maintenance efforts; possible stretching of leveling lining and tamping.

Additional benefits from USP:

- Compensation of locally heterogeneous conditions.
- Reduction of high-frequency vibration and structure-borne noise.

**Ballast**

It is in this layer where the sleepers are embedded. Ballast layer is made up by coarse stones and receives the load from the sleepers. The materials from which ballast layers are usually constructed are granite, basalt, limestone, slag and gravel. The typical ballast material is gravel-size with most particles between 6 and 64 mm diameter. The main functions of the ballast layer are:

- Receive the load from the sleepers and distribute it softly to the substructure, by limiting the magnitude of permanent settlements of the track.
- Limit displacements of sleepers due to vertical, horizontal and longitudinal forces transmit by the rails.
- Drain the water, facilitating the evacuation of the rainwater.
The depth of the ballast layer is approximately 0.30 m, depending on the railway administration. It is packed to 0.5 m around the sleepers end to ensure lateral stability. According to (Dahlberg, 2003), traditionally angular, crushed, uniformly graded hard stones and rocks have been considered good ballast materials. A lot of authors have studied its constitutive laws. These laws nowadays are under development.

Understanding of dynamic behavior of ballast need to do specific vibration and tamping tests as well as numerical simulations works.

**Sub-ballast**

It is used as a transition layer between the ballast layer and the subgrade. Sub-ballast materials consist of broadly graded sand-gravel mixtures or broadly graded crushed natural aggregates or slags, as long as they fit necessary filtering requirements given by each Railway Administrator. Some functions of this layer are presented:

- Sub-ballast layer prevents migration of particles from the subgrade to the ballast layer.
- It improves drain capacity of the track.
- It transmits, softly, the load from the sub-ballast to the subgrade, improving resistance characteristics of the track, from a deterioration point of view.

**Subgrade**

Also called formation layer, the subgrade is the layer where the whole track rests. Its importance is based on the fact that if this layer fails, in contrast with ballast and sub-ballast layer, little or nothing can be done to alter it and recover original characteristics. Therefore a special attention needs to be paid to this layer by increasing its resistance and stability if necessary by tamping it or adding materials to achieve a good quality of the subgrade.

This layer can either consist of the existing natural soil, which is most likely to be fine grained soil with silt and clay components, or placed fill.

**2.3 Dynamic behaviour of the track**

Dynamic behaviour of the track is of great interest in this work. As in many other structures, track and its components behaviour is highly influenced by the characteristics of the load acting on it, e.g. Man (2002) and Popp et al. (1999).

When the track is excited by a dynamic load it becomes deformed and it vibrates for a certain period of time.

If a resonant effect is attained in this process, the track and its components may experiment a high value of the stress level and consequently, suffer significant damages and accelerate degradation.
2.3 Dynamic behaviour of the track

Ballasted track response may present several frequencies contained, when a dynamic load is acting on it.

Commonly, these frequencies may be divided into three different main groups. Limits of this classification may vary between different publications, see Table 2.1.

Table 2.1: Damage in vehicle and track for different frequencies, adapted from Man (2002)

<table>
<thead>
<tr>
<th>Frequency range</th>
<th>Low</th>
<th>Mid</th>
<th>High</th>
</tr>
</thead>
<tbody>
<tr>
<td>Interval</td>
<td>0 – 40 Hz</td>
<td>40 – 400 Hz</td>
<td>400 – 1500 Hz</td>
</tr>
<tr>
<td>Structural damage type</td>
<td>type A</td>
<td>type C</td>
<td>type E</td>
</tr>
<tr>
<td>Track</td>
<td>type B</td>
<td>type D</td>
<td>type D</td>
</tr>
<tr>
<td>Vehicle</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Damage types in Table 2.1 are defined as:

- type A: Damage of substructure and engineering structures.
- type B: Damage of carriage bogies, axles and wheels.
- type C: Damage of structure.
- type D: Damage of wheels.
- type E: Damage of rails.

Track response in a low frequency interval is closely related with mechanical properties of the foundation soil. For medium and high frequencies, response of the track is mainly influenced by properties of track components as: rails, railpads, sleepers and ballast layer.

One of the most common methods to analyze track dynamic, consists in applying an impulsive force through an impulse hammer, obtaining the receptance function, that as seen later on, has been used by several authors to analyze dynamic behaviour of the track.

Excited frequency intervals will depend on the characteristics of the impulse hammer. Analysis of track response is usually made in frequency domain using transfer functions. Transfer functions are mathematical representations of the relation between an input factor and the corresponding output factor in frequency domain.

Depending on the type of the response analyzed, transfer functions may be of different types, see Table 2.2:

Table 2.2: Different types of transfer functions

<table>
<thead>
<tr>
<th>Response parameter</th>
<th>Displacement</th>
<th>Velocity</th>
<th>Acceleration</th>
</tr>
</thead>
<tbody>
<tr>
<td>Function</td>
<td>Receptance</td>
<td>Impedance</td>
<td>Inertance</td>
</tr>
<tr>
<td>Relation</td>
<td>[ H = \frac{\text{Displacement}(f)}{\text{Force}(f)} ]</td>
<td>[ Y = \frac{\text{Velocity}(f)}{\text{Force}(f)} ]</td>
<td>[ A = \frac{\text{Acceleration}(f)}{\text{Force}(f)} ]</td>
</tr>
</tbody>
</table>
From the list of functions presented in Table 2.2, receptance function is the most used to analyzed dynamic behaviour of the track. It is also known as dynamic flexibility function. To have this function it is necessary to obtain the response of the track, in terms of dynamic displacement, which is usually done by double integration of the measured accelerations.

Characteristics of the track may be observed in the receptance function. Figure 2.7 shows a typical receptance function of a ballasted track.

![Receptance Function](image)

**Figure 2.7:** Receptance function obtained when a load is applied right on the sleeper (black) and on the middle of the span between two consecutive sleepers (red). Numerical results from Ribeiro (2012)

From Figure 2.7, one can observe that receptance function depends on the frequency content of the track response, in terms of displacements.

Also, main resonance frequencies of the track can be easily identified:

- A: full track resonant frequency.
- B: Anti-resonant frequency of the sleepers.
- C: Vibration frequency of the rail on the railpads.
- D: Pin-pin frequency.

Vibration mode shapes, corresponding to the main resonance frequencies of the track, are presented in the paragraphs below. In this case, the track is considered as a mass-spring system. In the next figure, vibration mode shapes, corresponding to the main resonance frequencies of the track, are presented. In this case, the track is considered as a mass-spring system.

Figure 2.8 represents the full track vibration mode: Its main feature is that all suspended mass of the track structure moves in vertical direction relative to infinitely stiff boundary of the structure. In this case, value of frequency A differs from one author to others: 40 – 140 Hz Man (2002), 30 – 250 Hz Popp et al. (1999), 100Hz Dahlberg (2003). This frequency is strongly influenced by ballast layer and subground
properties as denoted by Knothe & Wu (1998) and Popp et al. (1999)). Sometimes, 
when the track is built on a soft ground, one resonance may appear in the frequency 

Figure 2.8: Mode shape belonging to the full track resonant frequency A, adapted from Man 
(2002)

Figure 2.9 represents vibration mode corresponding to frequency B, anti-resonance 
frequency of the sleepers. According to Man (2002), between every pair of resonant 
frequencies, an anti-resonant frequency can be expected. At this anti-resonant fre-
quency, the rail does not present any movement, whereas the sleeper or the block 
seem to experience a resonant frequency. According to Man (2002) a typical range 
for this frequency is between 80 – 300 Hz.

Figure 2.9: Mode shape belonging to the sleeper anti-resonant frequency B, adapted from Man 
(2002)

Figure 2.10 represents the vibration mode corresponding to frequency C. In this 
mode, there is a vertical movement of the rail on the sleepers. At this frequency there 
is no movement of sleepers neither the ballast layer. Frequencies for this vibration 
mode may vary from 250 – 1500 Hz Man (2002).

Figure 2.10: Mode shape belonging to the rail resonant frequency C, adapted from Man (2002) 
Ribeiro (2012)

Finally, Figure 2.11 represents the vibration mode corresponding to frequency D. 
This last is called pin-pin frequency and it is characterized by presenting a wavelength, 
of the bending shape, that is double than sleepers distance. Pin-pin frequency may 
be in the range of 400 – 1200 Hz Man (2002) and 1000 Hz Dahlberg (2003).
2. Railway transition zones in a ballasted track

Figure 2.11: Pin-pin mode shape belonging to the first order pin-pin resonant frequency $D$, adapted from Man (2002)

Figure 2.12 shows both, experimental and numerical receptance curves, obtained in the dynamic calibration process of a numerical model developed by Ribeiro (2012). As it can be seen, the experimental receptance curve is composed of two different curves coming from the excitation of the track by two different hammers. In this case it is necessary the use of a big hammer to excite the higher frequencies of the track and a small hammer to do the same with the lower frequencies.

![Figure 2.12: Typical receptance curves, numerical (cyan colour) and experimental (black and red colours), adapted from Ribeiro (2012)](image)

2.4 Dynamic behaviour of transition zones

The experience shows that transition zones are the origin of several problems, mainly due to the development of differential settlements. These settlements are usually linked to changes in the track stiffness, leading to the emergence of different problems associated with safety, maintenance costs and passenger comfort, as well as the development of track defects that contribute to track degradation.

PrudHomme (1970) contributed to the study of the dynamic overloads in railway lines:

$$\Delta Q_{NS} = \frac{0.45}{100} b \frac{V}{mK}$$  \hspace{1cm} (2.1)
2.4 Dynamic behaviour of transition zones

where:

\((\Delta Q_{NS})\): Standard deviation of the dynamic overload due to the unsprung mass.

\(V\): Speed of the train.

\(b\): Variable related to the track defects and wheel defects.

\(m\): Unsprung mass of the vehicle.

\(K\): Vertical track stiffness.

\(\phi\): Damping of the track.

From equation 2.1, it can be deduced the importance of vertical track stiffness to control the value of the dynamic overload. The lower the values of the vertical stiffness and the unsprung masses of the train, the less effect of the dynamic overloads. Also variation of stiffness value from one sleeper to another is important to determine the value of the dynamic overload.

Several authors have studied the effect of variation of stiffness value of two consecutive sleepers. The most important are: Amielin (1974), Hettler (1986), Hunt (1997), Esveld (2001) and Teixeira (2003).

In these previous works, it has been proven that when the difference between the stiffness values of two consecutive sleepers increases, the reaction on the sleepers increases too, leading to the development of the effect called hanging sleepers. The phenomena of hanging sleepers is characterized by the existence of unsupported sleepers on the ballast layer, so that they are hanging from the rail.

If no preventive measures are taken, hanging sleepers will cause an irregular distribution of the load, increasing stresses in the transition zone and speeding up the degradation process, causing other degradation phenomena such as broken stones in the ballast layer and broken sleepers.

Lei & Mao (2004) and Nicks (2009) analyzed the effects due to a variation of a track geometry similar to the one presented in Figure 2.13:

![Figure 2.13: Geometry of the transition zone, from Lei & Mao (2004)](image-url)

The authors concluded that the higher the value of \(\alpha\) (angle of inclination), the higher the value of train-track interaction forces and accelerations on the vehicle. As well, the above mentioned effects are increased when higher values of train speed are considered.
Schooleman (1996) developed a study in which the main goal was to establish some limits for the vertical variation of geometry in a transition zone.

Schooleman concluded that in the design of transition zones it will be more important to establish restrictions for admissible differential settlement value than for stiffness variations, when a dynamic load is moving along it.

Ribeiro (2012) studied the influence of stiffness variation in a track. Several scenarios of a sudden stiffness variation between two adjacent soils have been studied. Table 2.3 shows different scenarios and deformability characteristics, given by deformability modulus E, for each case.

Table 2.3: Different scenarios of a sudden change in the subground stiffness, adapted from Ribeiro (2012)

<table>
<thead>
<tr>
<th>Scenario</th>
<th>Designation</th>
<th>Soil 1 (MPa)</th>
<th>Soil 2 (MPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>j</td>
<td>Sj1</td>
<td>320</td>
<td>1600</td>
</tr>
<tr>
<td></td>
<td>Sj2</td>
<td>160</td>
<td>1600</td>
</tr>
<tr>
<td></td>
<td>Sj3</td>
<td>80</td>
<td>1600</td>
</tr>
<tr>
<td></td>
<td>Sj4</td>
<td>40</td>
<td>1600</td>
</tr>
<tr>
<td>k</td>
<td>Sk1</td>
<td>80</td>
<td>1600</td>
</tr>
<tr>
<td></td>
<td>Sk2</td>
<td>80</td>
<td>8000</td>
</tr>
<tr>
<td></td>
<td>Sk3</td>
<td>80</td>
<td>8000</td>
</tr>
<tr>
<td></td>
<td>Sk4</td>
<td>80</td>
<td>8000</td>
</tr>
</tbody>
</table>

From scenarios Sj1 to Sj4, deformability modulus of Soil 2 remains constant and equal to 1600 MPa. Deformability modulus of Soil 1 presents decreasing values of 320 MPa, 160 MPa, 80 MPa and 40 MPa. From scenarios Sk1 to Sk2 deformability modulus of Soil 1 remains constant and equal to 80 MPa, presenting increasing values for Soil 2. To evaluate dynamic effects in this zone of stiffness variation, the author developed two numerical models, one in 3D and the other in 2D. An interaction analysis has been carried out considering an Eurostar train moving at 350 km/h. In Figure 2.14, results of wheel/rail interaction forces are presented.

For all scenarios, a perturbation of the interaction forces, between the wheel and the rail, arises when the train approaches to the stiffness variation point (Position 0). It can be clearly seen how this perturbation, in the interaction force is greater, the higher the variation of adjacent stiffness value.

### 2.5 Problem description of transition zones

Existence of many different structures along a railway line (bridges, tunnels, culverts...) generate differential settlements that may affect the behaviour of the track
2.5 Problem description of transition zones

Figure 2.14: Wheel-Rail interaction forces for different scenarios of a sudden change in the sub-ground stiffness: (a) sj1-sj4; (b) sk1-sk4, numerical results adapted from Ribeiro (2012)

and its components. These differential settlements will have a great influence in the dynamic of interaction forces between train and track, which causes a fast degradation of the track in the surroundings of the existent structures. Such degradation can be measured either on the track or through measurements carried out in the vehicle. In Figure 2.15 it is shown a comparison of the results of tests on average track settlement on four ballast deck railroad bridges and their corresponding approaches Li et al. (2006).

Figure 2.15: Comparison of track settlements over one maintenance interval, from Li et al. (2006)

As it can be seen, the approaches present the highest track geometry degradation. In the work developed by López Pita (2006) the exceedance density parameter is defined as the vertical acceleration exceedance (to an acceleration limit) measured on axle boxes. In this case, the acceleration limit to compare is $30 \, m/s^2$. This limit is established to indicate the need of making maintenance operations in the track.
Table 2.4 shows the values of vertical acceleration exceedance, in different sections of the track, measured in a high speed train of the Spanish line Madrid - Seville.

Table 2.4: Survey of vertical acceleration exceedance measured on axle boxes in the period from April 1992 to March 2003. From López Pita (2006)

<table>
<thead>
<tr>
<th>Section considered</th>
<th>Exceedance density</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Track 1</td>
</tr>
<tr>
<td>Viaduct</td>
<td></td>
</tr>
<tr>
<td>Tunnel</td>
<td>0.031</td>
</tr>
<tr>
<td>Start of transition</td>
<td>0.237</td>
</tr>
<tr>
<td>Transition center</td>
<td>0.254</td>
</tr>
<tr>
<td>Abutment</td>
<td>0.687</td>
</tr>
<tr>
<td>Viaduct</td>
<td>0.064</td>
</tr>
<tr>
<td>Underpass</td>
<td>0.393</td>
</tr>
<tr>
<td>Pontoon bridge</td>
<td>0.086</td>
</tr>
<tr>
<td>Frame</td>
<td>0.026</td>
</tr>
<tr>
<td>Culvert</td>
<td>0.303</td>
</tr>
<tr>
<td>Pipe or siphon</td>
<td>0.115</td>
</tr>
<tr>
<td>Line with no crossovers, expansion equipment or masonry structures</td>
<td>0.075</td>
</tr>
</tbody>
</table>

In Table 2.4, it can be seen that in those sections in which an abrupt change in the track stiffness is involved (viaduct, underpass and culvert), a considerable increase occurs in the exceedance density parameter.

It has been proven that exceedance density parameter depends not only on the section the track but also on other parameters as amount of traffic and speed of the train, López Pita (2006). See Table 2.5

Table 2.5: Vertical acceleration exceedance density measured on axle boxes, adapted from López Pita (2006)

<table>
<thead>
<tr>
<th>Speed (km/h)</th>
<th>Exceedance density</th>
</tr>
</thead>
<tbody>
<tr>
<td>250 &lt; v 270</td>
<td>0.135</td>
</tr>
<tr>
<td>220 &lt; v 250</td>
<td>0.038</td>
</tr>
</tbody>
</table>

Table 2.5 demonstrates how speed of the train affects significantly density parameter, especially for speeds values higher than 250 km/h.

A great amount of research projects, in HSL, have as an objective to develop an improved understanding of ballasted track behaviour, reducing the maintenance costs while maintaining the safety limits.
2.5 Problem description of transition zones

One of these projects is EUROBALT-II (2000). Conclusions of this project were that relevant track parameters influencing the track behaviour are:

- Track stiffness.
- Settlement of different layer of the structure.

In the following, a special attention will be paid to this parameters, due to the important role that both of them play at transition zones.

2.5.1 Settlement of different layers of the structure

Railway track will settle due to permanent deformation in different layers of the track, as ballast and subgrade soil.

The problem arises when different sections of the track settle unevenly. In such a situation this phenomena is called differential settlement and it could seriously affect the value of train-track interaction forces.

Settlements of the track come mainly from three different sources:

- Settlements of the embankments and the subgrade soils due to a consolidation process, including the secondary compression phenomena.
- Settlements due to traffic loading.
- Settlements due to the structural interaction.

Settlements of ballasted track occurs mainly in two phases Dahlberg (2001):

In a short-term, after tamping, the settlement is typically fast until the gaps between the ballast particles have been reduced and the ballast is consolidated.

In a long-term, there is a slower phase with an almost linear relationship between settlement and time.

This phase is caused by several basic mechanisms of ballast and subgrade behaviour. These mechanisms can be divided in two main groups, depending on the mechanical behaviour of the layers.

The first one, due to a densification of both the ballast and the subgrade soil and the second one due to an inelastic behaviour of the two mentioned layers: ballast and subgrade soil.

2.5.1.1 Settlement of the embankments and the subgrade soils

Most of the deflection in the track is caused by vertical deformation of embankments and foundation soils. The analysis of this type of settlements is very important and it has a certain level of complexity.

Special attention should be paid to settlements caused by static actions. This problem is properly referenced in Melis (2006) and Soriano (1989).
Melis (2006) states that settlements that derive from the static actions are negligible after 10 – 15 years, but within that period of time, settlements could be very significant, having a great influence in the degradation behaviour of the track. Total settlement is divided into two different components: settlement of embankment and settlement of the foundation soil.

In the same work it states that in Holland a 5 m height embankment may settle until 2.5 m only by consolidation effects. In Spain settlements of embankments do not reach that magnitude, but they might have a significant influence if no preventive measures are implemented.

Tables 2.6 and 2.7 show post constructive settlements, in some of the embankments belonging to the Madrid-Sevilla and Madrid-Barcelona high speed railway lines.

Analyzing the results of the previous tables, next conclusion are reached:

- On one hand, settlement of embankments reach significant values that may affect the future behaviour of the track degradation.
- It does not exist a well-defined relationship between the height of the embankment and the post construction settlement.

2.5.1.2 Settlement caused by moving trains

Vertical deformation of ballast and sub-ballast layers is a phenomenon that presents a certain degree of complexity, mainly due the characteristics of the material. Is in these two layers were vertical deformations, caused by train loads, are more significant.

Its behaviour, when a moving train load is acting on it, is nonlinear and the stress level is not very high.

Dilatation\(^1\) of the ballast, increases when confinement stresses are low.

However, breakage of sides and corners of the particles occurs when the confinement increases, and this is the main reason of differential settlements.

This phenomenon was properly evaluated by several authors as: Schultze & Coesfeld (1961), Marsal (1967), Leps (1970), Raymond & Davies (1978) and Aubry (1999).

To have an accurate prediction of permanent settlement in these layers, three different solutions can be adopted:

The first one is based on the use of deformation laws according to laboratory testing. The second one consists in developing scaled down models to assess the evolution of settlements. Finally, another way to analyze the value of permanent settlement is to carry out an experimental campaigned to measure settlements in the track itself.

Regarding the transition zones Bruni et al. (2002) analyzed permanent settlement growth of the track in the vicinity of a viaduct – bridge transition, see Figure 2.16.

\(^1\)phenomenon that consists in the increase of the volume of ballast when shear forces act on it, due to sliding between ballast particulates
Table 2.6: Post constructive settlements, in ten years, of some embankments of the Madrid-Seville high speed line, adapted from Melis (2006)

<table>
<thead>
<tr>
<th>Location (KP)</th>
<th>Height (m)</th>
<th>Length (m)</th>
<th>Settlement (mm)</th>
<th>Maximum speed allowed (km/h)</th>
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<tbody>
<tr>
<td>56.60</td>
<td>10</td>
<td>220</td>
<td>48</td>
<td>270</td>
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</tr>
<tr>
<td>233.90</td>
<td>10</td>
<td>425</td>
<td>129</td>
<td>250</td>
</tr>
<tr>
<td>274.90</td>
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<td>784</td>
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<td>250</td>
</tr>
<tr>
<td>256.00</td>
<td>20</td>
<td>962</td>
<td>170</td>
<td>250</td>
</tr>
<tr>
<td>262.50</td>
<td>17</td>
<td>754</td>
<td>149</td>
<td>250</td>
</tr>
<tr>
<td>269.00</td>
<td>15</td>
<td>1070</td>
<td>34</td>
<td>250</td>
</tr>
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<td>273.50</td>
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<td>1215</td>
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<td>275.80</td>
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<td>745</td>
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<td>215</td>
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<td>250</td>
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<td>324.60</td>
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<td>685</td>
<td>33</td>
<td>250</td>
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<td>337.60</td>
<td>16</td>
<td>220</td>
<td>80</td>
<td>250</td>
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</table>
Table 2.7: Post constructive settlements, in three-five years of some embankments of the Madrid-Barcelona high speed line, adapted from Melis (2006)

<table>
<thead>
<tr>
<th>Location (KP)</th>
<th>Height (m)</th>
<th>Settlement (mm)</th>
</tr>
</thead>
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<td>235.00</td>
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<td>230</td>
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<td>490</td>
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<td>232.46</td>
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<td>233.60</td>
<td>35</td>
<td>84</td>
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<tr>
<td>244.39</td>
<td>32</td>
<td>178</td>
</tr>
<tr>
<td>157.16</td>
<td>30</td>
<td>131</td>
</tr>
<tr>
<td>238.55</td>
<td>25</td>
<td>112</td>
</tr>
<tr>
<td>245.42</td>
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<td>226.36</td>
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<td>243.19</td>
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<td>246.86</td>
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<tr>
<td>248.58</td>
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<td>300.47</td>
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<td>83</td>
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<tr>
<td>306.44</td>
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<td>171.98</td>
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<td>81</td>
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<td>180.66</td>
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<td>188.04</td>
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<td>195.81</td>
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<td>173.18</td>
<td>16</td>
<td>232</td>
</tr>
<tr>
<td>183.88</td>
<td>15</td>
<td>309</td>
</tr>
</tbody>
</table>
2.5 Problem description of transition zones

Figure 2.16: Pattern of settlement growth in the vicinity of viaduct-bridge transition, from Bruni et al. (2002)

Figure 2.16 shows settlements evolution vs number of transits. Settlements present a more significant increment right according with the location of transition zone (Position 0).

Differential settlements in this case, causes the arising of track irregularities. These induce train excitation and consequently variations on the dynamic load. It is important to note that in some cases these variations may be higher than those causes by variations of stiffness.

Mauer (1995) analyzed the evolution of the permanent settlement in a track where a train moves along a transition zone. In this case the train travels from a zone with a certain stiffness, $K_{th}$, to a zone in which stiffness is supposed to be half of the initial stiffness, $k_{lo}$. Distance along the track, in which such a change of stiffness occurs, is equal to 8 m. See Figure 2.17.

Figure 2.17: Evolution of track settlement in the transition zone, from Mauer (1995)

In this case, deformation law adopted by Mauer is considered only in the ballast layer and it depends solely on the reaction force of the sleepers.
Due to the fact of having higher values of reaction force in the stiffer zone, permanent deformation is higher in this one, as it can be observed.

Hunt (1996) analyzed the effect of vertical stiffness variation on the total settlement evolution. The author considered a transition zone in which stiffness increased three times. This change of stiffness was not developed abruptly but in a soft variable way within a length of 2 m.

Hunt concluded that settlement ratio of the track on the flexible zone was higher than on the rigid zone.

Bodin (2001), studied the evolution of settlement on a track where there is an abrupt stiffness variation, as Mauer (1995) did. Furthermore, Bodin considered an initial defect on the transition zone as it can be seen in Figure 2.18.

These results invalidate the main conclusions from Mauer (1995). In fact, settlement evolution studied by Bodin (2001), fitted pretty well to observations made in in transition zones of real tracks.

Dahlberg (2010), pointed out that variations of track stiffness will induce variations in the wheel-rail contact, increasing dynamic effects and speeding up track settlements. Moreover in this study, the authors have concluded that regarding the dynamic amplification effects, going from soft to stiff zones is worse than going from stiff zones to soft zones.

Ribeiro (2012) evaluated the wheel-rail interaction force when a vehicle travels from a soft zone to a stiff zone and vice versa. The author validated the conclusions pointed out by Dahlberg (2010), as illustrated in Figure 2.19.
2.5 Problem description of transition zones

Figure 2.19: Wheel-rail interaction force when the train moves from a flexible zone to a rigid zone and vice versa. Numerical results from Ribeiro (2012)

In the same work, the author developed a numerical example to simulate the evolution of degradation in a transition zone of a railway track. This study takes on account the worst case scenario, that is when train moves from soft to stiff zone. Results are taken from the points placed at the top of the ballast layer, right under the sleepers.

Figure 2.20: Evolution of settlement, depending on the number of loading cycles, along a transition zone on the top of the ballast layer. Numerical results from Ribeiro (2012)

Some conclusions of the previous figure can be drawn:

- It exists a higher permanent settlement, right in the transition zone, when the number of loading cycles increases.
- In the normal track, either in the soft or in the stiff zone, the evolution ratio of permanent deformation reduces as the number of cycles increases. However,
this trend, is only verified only for last cycles right in the boundary of transition zone.

- Permanent deformation is higher in the stiff section than in the soft section.

### 2.5.1.3 Settlements caused by the structure

Depending on the type of foundation, or the type of bridge, in transition zones, dynamic behaviour and degradation evolution may be highly influenced.

Some aspects that take part in the interaction are: characteristics of the bridge, type of the track and construction process of the transition zone.

Other features that can have a significant impact in the interaction process are: thermal effects, traction and braking forces and vertical loads.

According to the ERRI-D-230.1/RP1 (1999) main problems associated with transition zones in railway bridges are:

- Movements of the bridge.
- Expansion joints positioning.
- Foundation and geometry of the abutment.
- Badly drained in the abutment.

Geometry of the track near a transition zone is highly influenced by movements experimented in the bridge. Due to: low resistance of abutments to horizontal movements, cyclic rotations of the bridge deck and temperature effects, horizontal displacements are caused, inducing also track degradation.

According to Dahlberg (2003) in cases of thermal variations with integral abutments, seasonal temperature changes may induce that the abutment and the superstructure tend to move away from the soil, creating a void between the soil and the abutment.

As temperature decreases, abutment and superstructure move away from the soil, creating a void between the soil and the abutment, see Figure 2.21.

A bad drainage of the transition zone, may cause erosion problems with a localized flow of water from the backside of the abutment, from the deck or from the expansion joint. In Figure 2.22(a) a layout of the drainage problem is depicted. Figure 2.21(b) shows a real picture of damages caused by a bad drainage of the backfill zone.

If a special attention is not paid to the previous effects, great deformations of the foundation soil will arise, speeding up the process of degradation, when cyclic loads are acting on the track.

### 2.5.2 Importance of the vertical track stiffness

Other track parameter highlighted in EUROBAL II (2000) as one of the most relevant track parameters that have a significant influence in the track behaviour is track
stiffness. Importance of this parameter is very high, not only in the behaviour of the ballast track but also in the particular case of transition zones, above all when variations of track stiffness take place. A brief definition of track stiffness will be given in this section, highlighting the importance of this parameter pointed out by some relevant authors.

Firstly a differentiation between track stiffness and track modulus has to be made. In this work, vertical stiffness of the track will be used as an important parameter that takes the of indicator of the dynamic behaviour between train and track.

This indicator becomes more important when a special evaluation of track deterioration is made.

Vertical stiffness of the track, $k_t$, can be defined as the ratio between the vertical punctual load applied on the rail and the vertical settlement of the track set at the same point, López Pita (2001).

$$k_t = \frac{Q}{y}$$

(2.2)

Track modulus, $u_m$, is defined as the applied force per unit length of rail per unit deflection:

$$u_m = \frac{q}{\delta}$$

(2.3)

Where $q$ is the vertical foundation supporting force per unit length. The relationship between track stiffness and track modulus is given by the next expression:

$$u_m = \frac{k_t^4}{(64EI)^{\frac{3}{2}}}$$

(2.4)
The difference between $u_m$ and $k_t$ is that $k_t$ includes the rail bending stiffness $EI$, whereas $u$ is related only to the remainder of the superstructure and the substructure.

Some authors highlighted the track stiffness as an important factor for track behaviour and maintenance costs. Methods of track stiffness measurements (UIC, 2009).

Ebersohn & Selig (1994): “The continuous measurement of track deflection or stiffness and the correct interpretation of the results will be a tool for the track maintenance engineer to correctly direct the maintenance activities which will result in optimal use of the maintenance budget”.

Esveld (2001): “Track stiffness has be found to be very useful for the purpose of determining the cause of certain substructure problems. Unfortunately, in most of the cases railways do not possess the right equipment for this type of measurement and thus, do not utilize the insight these measurements could have provided them with”.

Frohling (1997): “Spatial variation of the track stiffness contributes significantly to track deterioration, both in terms of differential track settlement and increased dynamic vehicle loading. It is thus recommended that track maintenance procedures should be used to reduce the variation of the special track stiffness”.

Selig & Li (1994): “The factor affecting the track modulus most is the character of the subgrade layers. The influence of subgrade condition on track modulus is further enhanced by the fact that the subgrade resilient modulus is the most variable quantity among all the track parameters, subject to change of soil type, environmental conditions, and stress state. Therefore, a change of track modulus in the field is primarily an indication of a change of subgrade condition. Since the subgrade condition is subjected to weather, extremes of temperature and moisture, the track modulus may vary with seasonal changes”.

Figure 2.22: An example of a bad drainage in transition zones: (a) layout; (b) a real picture from Pacheco (2005)
Sussmann (2007): “Track stiffness test provides a potentially useful technique for system wide evaluation of track safety and performance. The data can be used to provide an additional indicator of track conditions to inspectors and to guide maintenance planning and execution”.

### 2.6 Stiffness variation in transition zones

The track stiffness varies along the track because of different causes. These variations may occur within limited spaces or in more gradually way. It may have its origin in the substructure: track foundation, subgrade soil or in the superstructure: rails, railpads, sleepers, ballast... As previously mentioned, in some cases the stiffness variations can be very significant within short distances, causing significant variations in the wheel-rail contact.

Some examples of stiffness variations along the track can be: hanging sleepers, embankments, bridges, tunnels, transition areas from an embankment to a bridge.

According to Dahlberg (2010) the rate of degradation of track components and the rate of track settlement will depend on the severity of the stiffness variation. As soon as the track geometry starts to deteriorate, the variations of the train-track interaction forces increase and this accelerate the track degradation rate.

In Figure 2.23, it is shown a diagram of train-track interaction forces along a piece of track in which there is a transition zone from a flexible zone (normal track) to a stiff zone (culvert). It is extracted from the work of Ribeiro (2012). The objective is to assess the behaviour of interaction forces when an Eurostar train travels at a speed of 350 km/h. Two different scenarios will be analyzed:

- A transition soil is used to ease the stiffness variation.
- No transition soil is considered.

![Figure 2.23: Influence of the transition wedge in the wheel-rail interaction force, along the transition zone, numerical results adapted from Ribeiro (2012)](image-url)
The red line, represents interaction forces along a transition zone when no soil-cement transition wedge is taken into account. The black line represents the same variable but considering the transition wedge. Not considering a smooth transition causes an increment in the amplitude of interaction forces, especially in the vicinity of the culvert section.

In the same work Ribeiro (2012), the effect of the influence of variations of track stiffness in the degradation profile of the track, is assessed. The above mentioned analysis is carried out by simulating the effect of a total number of 5 million of cycles of a bogie of a train moving along the transition zone.

![Graph](image)

Figure 2.24: Evolution of ballast settlement depending on the number of cycles along the transition zone, numerical results adapted from Ribeiro (2012)

Figure 2.24 illustrates how degradation profile is directly associated with variations of stiffness along the track. Furthermore the change of settlement in different parts of the track increases and becomes more accentuated with the number of cycles.

### 2.7 Adopted solutions in transition zones

In the light of the above information, it is very important to minimize the effect that variation of stiffness cause in the dynamic behaviour of the train-track interaction.

Traditionally the more complex type of transition, occurs in bridges due to construction system, drainage system and selection of appropriate backfill materials. Compaction should be done with thin layers of soil and abutments will be designed to resist compaction forces.

Drainage system is a critical issue in which a special attention has to be paid. A good drainage system prevents material erosion and hydrostatic pressures that leads to a decrease in loading capacity of the structure. Moreover, the presence of water inside the transition zone may induce undesirables variations of volume if it is frozen at extremely low temperatures.
2.7 Adopted solutions in transition zones

Overall, the most usual constructive solutions in transition zones for railway tracks can be found in Gallego et al. (1999):

- Technical block: backfill behind the abutment either with granular material treated with cement or materials of a high compression level.
- Transition slab: A concrete slab directly connected to the abutment of the bridge.
- Use of geosynthetics to achieve an abutment reinforced backfill.
- Introduction of horizontal layers on a track formation of different materials.
- Treatment of the track bed and sub-ballast with cement.

Paixão & Fortunato (2009) suggest interesting options to adopt in transition zones in order to mitigate the undesirable effects associated with these critical points of the railway track. Amongst other, solutions analyzed by the authors are:

- Use of internal rails fixed to the sleepers.
- Use of longer and heavier sleepers placed closer together, in transition zones.
- Use of more flexible and tighter track elements.

The aim of using internal rails in transition zones is basically to increase the global stiffness of the transition.

Regarding to the use of longer sleepers with a shorter distance, this allows a better distribution of the loads transmitted by the train, mitigating the effect of hanging sleepers.

Adoption of more flexible elements, allows to absorb dynamic excitations, reduces the propagation of deterioration phenomenon along the transition area and consequently reduces the maintenance costs. Nowadays this type of elements are common in track construction and they can be found under the rails, under the sleepers or on the base of sub-ballast layer.

Figure 2.25: Different solution to mitigate the effect of stiffness variation: (a) Use of internal rails; (b) Use of longer sleepers
The growing number of HSL, either already built or planned, implies a constant evolution of the transition zones morphology. This evolution follows a general trend in the most of European countries in which HSL exist. Some of the main construction features of the aforementioned trend are:

- Minimum length for the transition area of 20 m.
- Use of granular materials treated with cement in the backwalls of the abutments.
- Provision of drainage layers as a part of the backfill materials in the abutments.
- Treatment of different formation layers of the track with cement.

Regarding to complementary solutions for transition zones it is interesting to point out the state-of-the-art made by Ribeiro (2012), in which an interesting and a brief summary of the main complementary solutions that are used today in transition zones, is presented.

In the following, some of these solutions will be briefly explained.

### 2.7.1 Geosynthetics reinforcement grid

This type of elements improve the granular material stability and the drainage of the water located in the backwall.

This technique is commonly used in both, rehabilitated and new construction tracks, particularly in transition zones.

Cyclic loading studies show that geosynthetic reinforcement provides the soil a higher resistance and consequently a better behaviour to the long-term degradation.

### 2.7.2 Incorporation of the resilient elements

This type of elements is very important due to their influence in the track stiffness. With their use, adverse effects due to the variation of stiffness, can be better controlled.

Among these type of elements are: railpads, under sleeper pads (USP) and elastic blankets located on the top of the ballast layer. Pita & Teixeira (2003) concluded that railpads help to reduce not only global stiffness values, but variation of stiffness as well.

Larsson & Dahlberg (2006) evaluated the effect of the utilization of USP in transition zones. The authors have proven that this type of element allows to reduce dynamic effects between the train and the track, as well as reducing deformation of the ballast layer.
2.7 Adopted solutions in transition zones

2.7.3 Changes made in sleepers

Modification of sleeper geometry is other different way of control the track stiffness in transition zones.

Namura & Suzuki (2007) studied the effect of taking into account four different types of sleepers in a track degradation assessment. In this study, different ratios of ballast settlement were deduced, regarding to each type of sleeper.

The fact of increasing the size of the sleepers, as it is suggests in TTCI (2006), leads to a higher contact area and consequently a better distribution of the load, avoiding a great stress concentration that contributes to reduce the value of track settlements.

Changing the distance between the sleepers could be another different way to modify and improve the track stiffness at transition zones.

2.7.4 Use of bituminous material

This is not a typical solution in transition zones. Its use is more focused on the rehabilitation operations. It is commonly designed as HMA (Hot Mixed Asphalt).

Its application is located under the ballast layer and the aim of using this material is to reduce, due to its flexibility, the level of stress concentration in track layers. Moreover it behaves as a barrier to avoid particle migration from layers on the bottom to the ballast.

2.7.5 Transition zones configuration in European countries with HSL

In the following, a brief description of solutions adopted by different European countries is presented. These solutions correspond to the design of transition zones in four different countries with HSL as Spain, Germany, France and Italy.

Spain (ADIF)

![Figure 2.26: Solution adopted by ADIF for transition zones](image)

45
Configuration adopted by the Spanish Administrator ADIF is illustrated in Figure 2.26. In this case, the treated soil wedge presents an inclination 1:1. The top of this wedge is aligned with the bottom part of the sub-ballast layer and it has a length of 3 m. Between the treated soil wedge and the concrete structure, a vertical drainage system is placed.

The wedge of soil that does not have any treated material, its slope is 3:2 and it extends 20 m from the abutment location. The top of this layer is aligned with the bottom part of the formation layer.

The height of the technical blocks and embankments should be defined according to the characteristics of the foundation soils.

In the case of inadequate foundation soils, these should be removed and the base of the wedge should be properly treated.

**Germany (DB)**

![Figure 2.27: Solution adopted by DB for transition zones](image)

Figure 2.27 shows the solution adopted by the German Administrator. In this case, the solution is very similar to those provided by ADIF. The top of the treated soil wedge is placed approximately at 1 m from the down of the ballast layer and it has a length of at least 1.5 m.

The treated soil should have between 2.5% and 3% of cement content.

In this solution, the height of technical blocks is not directly specified but the foundation where they are placed on, should have deformability characteristics equal or greater than 45 MPa in the second phase of the plate load test.
France (SNCF)

Solution adopted by the French Administrator is depicted in Figure 2.28. This configuration corresponds to the case in which there is an open abutment solution and a height of the technical block higher than 10 m. Similar solutions are adopted in those cases in which there is a close abutment configuration or a culvert.

In this case, the treated soil wedge exhibits a height of 3 m, whereas the height of the technical block in the transition zone.

The width of this wedge will be of approximately 1 m, on the top, and it has an inclination of 1:1. Furthermore, the cement content in it is 3%.

The height of the wedge that does not have any treated soil and it is similar than the height of the technical block.

The length of this wedge, on the top part, is 5 m and it has an inclination 3:2 as it can be seen in Figure 2.28. As in the previous cases, the height of the technical block is not directly obtained, and it depends on the conditions of the foundation soil. If the above mentioned height is higher than 10 m, a foundation layer of select and well compacted material should be adopted.
Italy (FS)

Finally, Figure 2.29 shows the typical solution adopted in the Italian HSL. As in the French case, the height of the treated soil wedge of the technical block is 3 m for heights of the abutment higher than 4 m.

If the height of the abutment is less than 4 m, the height of the treated soil wedge will be similar than the height of the abutment.

Geometry of the wedge is similar than that presented by SNCF. The height of non-treated material wedge is similar than the height of the treated material wedge and it has a slope of 2:1, as in the German solution.

The width of the technical block is similar than the height of the abutment but always exceeding 8 m, as depicted in Figure 2.29.

2.8 Normative limits

Considering transition zones as a critical point within a railway track, in terms of degradation, safety and comfort and due to a lack of an appropriate normative to design transition zones in railway lines, different normative, especially related to bridges and geotechnical science, have to be used as a guide to design transition zones.

Several documents are used to define geometrical quality of the track, attending to critical parameters as: safety, maintenance costs and passengers comfort. The importance of establish a control of the track geometry quality, arose in 20th century. At that time, European railway networks developed their own track recording vehicles, allowing a continuous measurement of the track geometry.

Geometric quality of the track can be defined using three different indicators, EN13848-5 (2010):

---

Figure 2.29: Solution adopted by FS for transition zones
2.8 Normative limits

- Extreme values of isolated defects.
- Standard deviation over a defined length (typically over 200 m).
- Mean value.

Some of the documents in which specifications of geometrical quality of the track can be found are: TSI (2008), EN13848-5 (2010) and UIC (2005b). Specifications of the previous documents are based on three different main levels that have to be considered. Immediate action limit, intervention limit and alert limit. Such levels are well defined in EN13848-5 (2010).

**Immediate Action Limit** (IAL): ‘Refers to the value which, if exceeded, requires taking measures to reduce the risk of derailment to an acceptable level. This can be done either by closing the line, reducing speed or by correction of track geometry’.

**Intervention Limit** (IL): ‘Refers to the value which, if exceeded, requires corrective maintenance in order that the immediate action limit shall not be reached before the next inspection’.

**Alert Limit** (AL): ‘Refers to the value which, if exceeded, requires that the track geometry condition is analyzed and considered in the regularly planned maintenance operations’.

A brief summary of the quality limits provided by each of the above mentioned documents, is listed in the following paragraphs.

**TSI (2005b):** Suggests limit values for three different quality levels: $QN_1$, $QN_2$ and $QN_3$. These values have to be considered for irregularities of the track with a wavelength between 3 m and 25 m (Group D1).

**TSI (2008):** This document provides, for HSL, limit values of: twist, gauge variation, cant, level and alignment. The limits are defined for three different levels of: safety, intervention and alert.

A layout of the gauge, cant, level and alignment is illustrated in Figure 2.30.

Figure 2.30: Layout of the different track dimensions: gauge, level, alignment, cant and twist

**TSI (2008):** provided Tables 2.8 and 2.9. In the first, the standard deviation limit in longitudinal level, over 200 m of track, is provided. This limit depends on both the quality level of the track and the speed of the vehicle. In Table 2.9, the maximum
2. Railway transition zones in a ballasted track

table value of peak, relative to the average value of the longitudinal level, is given. As in the previous case, this limit depends on the value of the quality level of the track and also on the speed of the vehicle.

Table 2.8: Standard deviation limit in longitudinal level (mm) over 200 m of track. TSI (2008)

<table>
<thead>
<tr>
<th>Speed (km/h)</th>
<th>Quality level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>QN1</td>
</tr>
<tr>
<td>v ≤ 80</td>
<td>2.3</td>
</tr>
<tr>
<td>80 &lt; v ≤ 120</td>
<td>1.8</td>
</tr>
<tr>
<td>120 &lt; v ≤ 160</td>
<td>1.4</td>
</tr>
<tr>
<td>160 &lt; v ≤ 200</td>
<td>1.2</td>
</tr>
<tr>
<td>200 &lt; v ≤ 300</td>
<td>1.0</td>
</tr>
</tbody>
</table>

Table 2.9: Maximum value of peak, relative to the average value of the longitudinal level (mm). TSI (2008)

<table>
<thead>
<tr>
<th>Speed (km/h)</th>
<th>Quality level</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>QN1</td>
</tr>
<tr>
<td>v ≤ 80</td>
<td>12</td>
</tr>
<tr>
<td>80 &lt; v ≤ 120</td>
<td>8</td>
</tr>
<tr>
<td>120 &lt; v ≤ 160</td>
<td>6</td>
</tr>
<tr>
<td>160 &lt; v ≤ 200</td>
<td>5</td>
</tr>
<tr>
<td>200 &lt; v ≤ 300</td>
<td>4</td>
</tr>
</tbody>
</table>

EN13848-5 (2010): This document provides limit values to follow when distributed or isolates irregularities of the track exist. The previous limit values are given for alignment and longitudinal level irregularity with wavelengths between 3 m and 25 m (group D1) and between 25 m and 70 m (group D2).

Table 2.10: Longitudinal level-isolated defects-mean to peak value, EN13848-5 (2010)

<table>
<thead>
<tr>
<th>Speed (km/h)</th>
<th>AL</th>
<th>IL</th>
<th>IAL</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Wavelength rate</td>
<td>Wavelength rate</td>
<td>Wavelength rate</td>
</tr>
<tr>
<td></td>
<td>D1</td>
<td>D2</td>
<td>D1</td>
</tr>
<tr>
<td>v ≤ 80</td>
<td>12-18</td>
<td>-</td>
<td>17-21</td>
</tr>
<tr>
<td>80 &lt; v ≤ 120</td>
<td>10-16</td>
<td>-</td>
<td>13-19</td>
</tr>
<tr>
<td>120 &lt; v ≤ 160</td>
<td>8-5</td>
<td>-</td>
<td>10-17</td>
</tr>
<tr>
<td>160 &lt; v ≤ 200</td>
<td>7-12</td>
<td>14-20</td>
<td>9-14</td>
</tr>
<tr>
<td>200 &lt; v ≤ 300</td>
<td>6-10</td>
<td>12-18</td>
<td>8-12</td>
</tr>
</tbody>
</table>
2.8 Normative limits

The mean, in the table above, is calculated over a length of at least twice the higher wavelength in the D1 or D2 range. In practice the mean will be close to zero and therefore zero to peak values may be used.\(^2\)

<table>
<thead>
<tr>
<th>Speed (km/h)</th>
<th>D1 (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>v ≤ 80</td>
<td>2.3-3.0</td>
</tr>
<tr>
<td>80 &lt; v ≤ 120</td>
<td>1.8-2.7</td>
</tr>
<tr>
<td>120 &lt; v ≤ 160</td>
<td>1.4-2.4</td>
</tr>
<tr>
<td>160 &lt; v ≤ 200</td>
<td>1.2-1.4</td>
</tr>
<tr>
<td>200 &lt; v ≤ 300</td>
<td>1.0-1.5</td>
</tr>
</tbody>
</table>

If it is desired to take into account passengers comfort, the document Eurocode (2005) recommends some interesting values that may be analyzed in transition zones. These values are recommended for certain levels of comfort and they have to be measured inside the coach during the travel on the approach to, passage over and departure from the bridge. See Table 2.12.

<table>
<thead>
<tr>
<th>Level of comfort</th>
<th>Vertical acceleration (m/s(^2))</th>
</tr>
</thead>
<tbody>
<tr>
<td>Very good</td>
<td>1.0</td>
</tr>
<tr>
<td>Good</td>
<td>1.3</td>
</tr>
<tr>
<td>Acceptable</td>
<td>2.0</td>
</tr>
</tbody>
</table>

López Pita (2006) provides also different values of accelerations that can be measured on different points of the vehicle.

For our particular case, it can be interesting to show limit values of acceleration measured on the axle box.

Table 2.13 outlines the control values corresponding to the different accelerations measured and recommend actions in each case.

\(^2\)For speeds less than or equal to 40 km/h, the ALs and ILs can be relaxed.
Table 2.13: Vertical acceleration levels, on the axle box, in a dynamic inspection of the AVE Madrid-Seville Line, adapted from López Pita (2006)

<table>
<thead>
<tr>
<th>Acceleration intervals ( (m/s^2) )</th>
<th>Recommended action</th>
</tr>
</thead>
<tbody>
<tr>
<td>0-30</td>
<td>Normal control level</td>
</tr>
<tr>
<td>30-50</td>
<td>Internal control level</td>
</tr>
<tr>
<td>50-70</td>
<td>Schedules checking and correction</td>
</tr>
<tr>
<td>&gt;70</td>
<td>Immediate checking and correction</td>
</tr>
</tbody>
</table>

2.9 Transition zones in European research projects

Research in transition zones has become of great interest in the last years. This is evidenced by the fact that some of the most relevant European projects of railway engineering bear in mind these critical zones. In this section, a brief description of the above mentioned projects is made, highlighting the importance given in these works to the transition zones in railway lines.

June 2004 - RAVE & FEUP

This document is a Partnership and Cooperation agreement between the Portuguese institutions RAVE and FEUP. The main topic of this document is to define the sensors to monitoring the structures, among others, of high speed lines.

Regarding transition zones, the work will be focused in embankment-structure transition.

In this type of transitions, and due to variations of stiffness, dynamic forces experiment important variations that lead to track alterations, causing it to deteriorate.

Some critical aspects in which monitoring may contribute to a better understanding are:

- Permanent vertical deformation of the track: Relation between the length \( L \) and the rise of transition \( h \) is very important to guarantee the stability in the wheel-rail contact and in the passengers comfort.

- Vertical accelerations in in the axle box of the train: According to Eurocode (2005), with the aim of guarantee a good level of passengers comfort, accelerations inside the coach should be less than \( 1 \, m/s^2 \).

Esveld (2001), carried out a parametric study in which a defined relation between \( L \) and \( h \) were established, for those cases in which a TGV train travels along the transition zone at a speed of 300 km/h, to guarantee vertical accelerations in the axle box of the train less than \( 1 \, m/s^2 \).

December 2005 - SUPERTRACK
2.9 Transition zones in European research projects

The project was motivated by the need in the railway industry to improve the performance of railway lines for making them a more competitive means of transportation. For the networks to expand, it is vital for the railway administrators to be able to design more durable new tracks and reduce the maintenance costs of existing lines.

Measurements, testing and numerical simulations in this study have revealed that an important factor contributing to track deterioration is the heterogeneity along the track. The non-homogeneity, which can be measured through the variation of track stiffness, can arise due to a rapid variation of track structure, for example at the junction of a bridge or transition from a shallow to a deep embankment, or as a result of insufficient track compaction leading to loose of hanging sleepers.

Results and achievements from SUPERTRACK (2005):

- The cyclic tri-axial tests on large scale samples at NGI and medium scale sample at CEDEX represents major developments in material testing. Very few tests of comparable size, quality and detail have been performed in the world. The results of these will be used in the future to calibrate the constitutive models for granular material.
- The construction of the track box facility, which is one of the biggest of its kind in the world, is a major contribution of this project to the future research on railway track.
- The numerical models for simulation of nonlinearity and non-homogeneities in the track and their effects on the long-term response of the track represents state-of-the-art researches in this field.
- The measurements at the tests sites were made with state-of-the-art sensors and instrumentation set-ups.
- The innovate grouting technique implemented successfully at a site in Spain is a valuable contribution of this project to the state of practice.

2010-INNOTRACK

Within the general highlights of the INNOTRACK (2009) project are:

**Track Stiffness:**

It is an important feature in the interaction between train and track. Track stiffness govern the track’s impact on the vehicle. This is especially important for high-speed and heavy freight operations. It should here be noted that it is normally not the specific stiffness that is of most importance, but rather the variation of stiffness. Further, the track stiffness has a natural variation due to climate.

The results clearly show the significant potential for reducing dynamic component of the forces. The measurements carried out in this project give a tool for monitoring and maintaining proper stiffness distribution in transition zones.

**The concept of track stiffness:**

Historically, most attention has been paid to inspection techniques targeted at the superstructure. Several such techniques are standard measurements used worldwide. Inspection of the substructure has been given much less consideration, especially the sub-ballast and subsoil components, even though it has a major influence on the cost of track maintenance.
Most of the substructure investigation techniques are not standard measurements and are not performed regularly.

The term global stiffness is used if the whole track structure is considered. It is often measured as applied force to rail divided by rail displacement. Global track stiffness varies both with frequency, dynamic amplitude, applied preload and position along the track. Global track stiffness is an important interaction parameter in the wheel/rail contact, and variations of track stiffness as well as extreme values will affect the degradation of the track.

**Variation of track stiffness in transition zones:**

Three *in situ* campaigns have been carried out to verify the behaviour of a transition zone.

Wheel loads and rail deflections induced by the trains operating the line at 200 km/h to 250 km/h in 2007 and at 250 km/h to 300 km/h in 2008 have been measured at five cross sections over the transition zone (one of them at the interface concrete-ground of the bridge abutment) and at one cross section in the plain track. The behaviour of each cross section zone has been assessed by determining the rail deflections induced in five consecutive sleepers by trains coming out from the bridge and trains entering the bridge using sensors external to the track mounted on the rails.

Besides the construction features, train speed and traveling direction seem to be the most influential factors affecting the behaviour of the transition zone analyzed in this work.

For bogie trains leaving the bridge at 200 km/h to 250 km/h, wheel loads 16 % less than the nominal static values have been recorded at both the transition zone and the plain track. At the interface concrete-soil I the edge of the abutment, and for bogie trains traveling at 200 km/h to 300 km/h, higher wheel loads and rail deflections have been recorded from trains entering the bridge than for trains leaving it.

At his interface variations in the rail deflections between the stiff side and the soft side of the track ranging between 1:2 and 1:3 depending on train speed, have been found. Although for this particular case it is believed that the most cost effective way to improve the behaviour of the transition zone must rely mainly on the modification of the mechanical behaviour of the track superstructure components, it may be not so for other cases. Whether to act on the superstructure or infrastructure components of the track in a given transition zones case will depend on the nature and magnitude of the problem found.
3 Dynamic models adopted in a ballasted track. Application to transition zones

3.1 Introduction

Before presenting the content related to the models adopted in this work, it would be useful to define the concept ‘model’.

“A model can be defined as a simplified description, especially a mathematical one, of a system or process, to assist calculations and predictions”

Hawking & Mlodinow (2011) in their work ‘The Grand Design’ take a step forward and they define the necessary characteristics to a model be considered appropriate. According to them a model is good if:

- Is elegant.
- Contains few arbitrary or adjustable elements.
- Agrees with and explains all existing observations.
- Makes detailed predictions about future observations that can disprove or falsify the model if they are not borne out.

Regarding the analysis methodologies it is useful to distinguish between analytic methods and numerical methods.

In analytic methods it is possible to predict how the system will behave for any given excitation. Analytic methods tend to work properly only for simple models and they cannot be used in complex problems nonlinear behaviour is to be expected.

On the other hand, numerical methods are techniques that allow to obtain an approximate solution of the problem that can be represented by a mathematical model. Numerical methods provide an approach to the exact solution of the problem. When a numerical method is used, other important aspects must be taken into consideration as: The rate of convergence, the accuracy of the answer and the completeness of the response.

Within this group of methodologies of analysis are, amongst other, Finite Element Method (FEM), Discrete Element Method (DEM), Boundary Element Method (BEM)
3. Dynamic models adopted in a ballasted track. Application to transition zones

and a combination of the BEM and FEM methodologies. A brief description of each methodology is made in this section.

It is important to note that this work will be developed in the frame of the Finite Elements method, due to the advantages of this methodology on some issues involved in the current research as the consideration of general loads, different materials, Variable elements size, dynamic analysis and non-linear problems.

**Finite Element Method FEM**

Finite element method is a numerical technique for solving problems which are described by partial differential equations or can be formulated as functional minimization. A domain of interest is represented as an assembly of finite elements. Approximating functions in finite elements are determined in terms of nodal values of physical field which is sought and values inside the finite elements can be recovered using nodal values.

In the finite element method, precision of the solution depends on the number of elements in which the model is discretized.

This methodology is widely used in the HSL research, obtaining very accurate results which allows to have a better understanding of the different problems.

**Discrete Element Method DEM**

The main area of application of DEM is focused in granular materials modeling. The particle properties and interaction laws are inserted into DEM, which is also referred to as a molecular dynamics MD, and lead to the collective behaviour of the dissipative many particle system.

They have an extended use in the modeling and simulation of the ballast bed Saussine et al. (2006) and Lu & McDowell (2007). In the beginning and due to the computational resources, particles were considered with a spherical shape. Recently, modeling techniques of ballast particles with angular shapes has been also incorporated.

According to Luding (2004), these microscopic simulations of a certain sample, representative of the ballast layer, can be used to derive macroscopic constitutive relations needed to describe the material within the framework of a macroscopic continuum theory.

**Boundary Element Method**

Reflections generated from edges of numerical grids have always presented a difficulty in applying discrete methods to simulate physical phenomena. Formulation presented by Lysmer, J; Kuhlemeyer (1969) was the first to deal with this problem. Until now and even with other new techniques developed, the formulation presented in 1969 is the most used from a general point of view, due to three main reasons: Compatibility of the formulation with no regular geometries; possibility to apply this method either
to harmonic or transient excitations; simplicity of mathematical formulations and facility of implementation in the finite element method.

**Combination of Boundary element method MEC and Finite element method FEM**

BEM is considered as one of the most powerful rigorous methods for modeling infinite domains and furthermore it lends itself easily for coupling with other methods.

Regarding to ballast track modeling, both BEM and FEM methods can be implemented together.

Applications of this methodology can be found in the works of Dominguez (1993), Firuziaan & von Estorff (2003), Savidis & Hirschauer (2003), Galvín & Domínguez (2007) and dos Santos *et al.* (2010).

In this methodology, a part of the track soil can be model using finite elements joint to the boundary elements which represent the rest of the foundation soil as a semi-infinite medium.

In the current chapter, methodologies used to analyze ballasted track problems will be presented, taking special attention to the evolution and improvement of the main methods and explaining the principal characteristics of the FEM methodology used in this thesis. A description of the particular models, used to do assessments in transition zones, will be made as well.

Finally, the last part of the Chapter, will be focused on the explanation of the methodology developed by Ribeiro (2012) to analyzed degradation experimented by the track, in the surroundings of transition zones. This methodology has been used to perform a degradation analysis of the transition zone in Chapter 6.

### 3.2 Evolution of ballasted track models

Track modeling is important to have a better understanding of the dynamic behavior of its different components. Interaction of the track with the vehicle implies a certain grade of complexity, in the modeling process, but also of accuracy that allows to have a better knowledge of the problem.

The past years have seen significant improvements on the field of ballasted track models. Timoshenko (1926) tried to model dynamic behavior of the track. In this work the author has modeled rails as an Euler infinite beam supported uniformly over a continue elastic foundation with damping characteristics.

Other authors developed similar models composed of a beam representing the rail, laid on as elastic or visco-elastic foundation as: Kenney (1954), Mathews (1958), Choros & Adams (1979), Jezequel (1980), Vesnitskii (1993) and Zhai & Cai (1997). See Figure 3.1.
3. Dynamic models adopted in a ballasted track. Application to transition zones

The advances in computing power seen in the past years meant that more complex models could be developed keeping the running times within acceptable limits.

Complexity level of the models is strongly correlated with the characteristics to be analyzed. For example, a model that may be good to analyze the long-term degradation of a transition zone, may be not good enough to analyze a fatigue crack propagation problem.

After Timoshenko, models of the track evolved. Rail was considered as a Timoshenko beam, including shear deformations. This model is valid only for static loading of a track on a soft support. Due to the fact that model does not contain any mass, no dynamic effects can be analyzed. For example pinned-pinned frequency of the track, that was explained in Chapter 2, can not be estimated analyzing the previous model.

To have into account pinned – pinned frequency in the track model, a discrete support of the rail has to be considered. The support may consist of a discrete spring-damper system or spring-mass-spring systems. Railpads, sleepers and ballast layer are modeled too, as it is shown in Figure 3.2:

Grassie, S.L. and Cox (1984) studied the behaviour of the track support, and concluded that the large sleeper strains were associated with poorly damped sleeper resonances.

They decided to introduce in their model the effect of railpad and it was found that a softer railpad provides an isolation of the sleepers, reducing strains therein.
3.2 Evolution of ballasted track models

With the aim of capture a resonance frequency at low frequencies (20-40Hz), Oscarsson (2001) incorporated more masses into the model. See Figure 3.3 in which: (1) Rail, (2) Railpad stiffness and damping, (3) Rigid sleepers, (4) Ballast stiffness and damping, (5) Mass of the ballast and subgrade and (6) Subgrade stiffness and damping.

![Figure 3.3: Rail on discrete supports with an increase number of masses, from Dahlberg (2006)](image)

By making the ballast and subgrade mass larger than the mass of sleeper and rails, and fitting the subgrade stiffness, a resonance at low frequency can be achieved.

Nowadays more complex 3D models are available mainly due to the advances in computer sciences that made possible to perform this analysis within reasonable runtime. 3D models allow to properly simulate different components of the track leading to obtain more refined results than the previous models.

To use this 3D models, finite element method is the most used numerical method in the last years. An illustration of a typical 3D model is shown in Figure 3.4.

This type of models are the more realistic, in which rails and sleepers are modeled as beams or three-dimensional bodies; railpads are modeled as elastic elements located between the rails and the sleepers; sleepers are embedded on a continuous layer, the ballast, that is modeled by three-dimensional finite elements.

![Figure 3.4: Rail on sleepers. Sleepers are embedded in a continuous ballast medium, from Dahlberg (2006)](image)
3. Dynamic models adopted in a ballasted track. Application to transition zones

3.3 Numerical models

As aforementioned, the main source to obtain dynamic results in this work, are the numerical models. The significant number of different elements in the track and the accuracy of the response justify this type of models instead of those based in analytical solutions.

On the other hand, numerical models allow to consider any type of non-linear behavior of the track and their uses are more versatile to model complex geometries given in certain parts of the track as transition zones.

Within the numerical models, finite element method has been chosen to developed analysis due to characteristics commented in previous sections.

While this work has been developed with a 2D numerical model, it is appropriate to do a brief introduction not only on the 2D numerical models, but also on the more complex 3D numerical models. The last have been the basis, in terms of numerical calibration, of the 2D model with which results have been obtained.

3.3.1 Nonlinear and dynamic analysis

The numerical analyses performed in this work, have considered two features of the problem: On one hand, nonlinear behaviour of the system analyzed, mainly due to the modification of the stiffness matrix in each step of analysis. On the other hand dynamic analysis, due to the modification of the system response over time. Below, a brief description of each of the previous methodologies will be made.

**Nonlinear analysis**  This methodology should be taken into account when the behaviour of the system that we want to analyze is not linear.

Nonlinearity arises because of different reasons that can be place into three different categories:

- Geometric nonlinearity.
- Material nonlinearity.
- Contact nonlinearity.

In geometric nonlinearity a variation in geometry, as the structure deforms, is included in setting up the strain-displacement and equilibrium equations.

In material nonlinearity, material behaviour depends on the history of the deformation and on current deformation state as well.

In contact nonlinearity (known also as changing status analysis), surfaces involved may or may not be in contact, this fact is unknown until the analysis is made. Moreover, some contact problems need to a take into account friction which implies the use of nonlinear laws to define different status.
3.3 Numerical models

**Dynamic analysis** This is a methodology that is used to determine dynamic response of a structural system under the action of any general time-dependent loads.

The main forces acting on the general case of a multi-degree of freedom system are:

- The forces of inertia, \( f_I \).
- The forces of resistance or damping, \( f_D \).
- The elastic forces, \( f_E \).

The inertial forces may be expressed by a set of influence coefficients called mass coefficients, Clough & Penzien (1975). These coefficients represent the relationship between the accelerations, of the degrees of freedom, and the resulting inertial forces.

Symbolically, previous relationship may be expressed by equation 3.1.

\[
  f_I = m\ddot{u} \tag{3.1}
\]

in which \( m \) represents the mass matrix of the system and \( \ddot{u} \) is the acceleration vector.

The damping forces may be expressed similarly by a set of damping influence coefficients that represents the relationship between the velocities of the different degrees of freedom and the resulting damping forces. Symbolically this relationship is expressed by equation 3.2.

\[
  f_D = c\dot{u} \tag{3.2}
\]

where \( c \) represents the damping matrix of the system and \( \dot{u} \) is the velocity vector.

The elastic forces are expressed by a set of stiffness influence coefficients. These coefficients represent the relationship between the displacements of the degrees of freedom and the resulting elastic forces.

As in the previous cases, this relationship may be expressed by equation 3.3.

\[
  f_E = ku \tag{3.3}
\]

where \( k \) represents the stiffness matrix of the system and \( u \) is the displacement vector.

For equilibrium, the sum of the above-mentioned forces should be equal to the external forces acting along the coordinates of the system. If these external forces are denoted by \( F \), the equation of motion becomes:

\[
  f_I + f_D + f_S = F \tag{3.4}
\]

or

\[
  [M] \dddot{u}(t) + [C] \dot{u}(t) + [K] u(t) = F \tag{3.5}
\]
3. Dynamic models adopted in a ballasted track. Application to transition zones

When the forces in the system act at different locations than those defined by the coordinates, the former must be replaced by equivalent forces acting at the coordinates.

In Humar (2001), an appropriate classification regarding the formulation of the differential equation of motion 3.5 can be found.

### 3.3.2 3D model

This model is actually a background of this dissertation that has been used as a reference. It is not directly used in this work but according with this 3D model was possible to calibrate the 2D model which has been used to carry out the dynamic analysis of the current work.

A brief explanation of this 3D model will be made below, describing its main features.

The 3D reference model was developed by Ribeiro (2012) in the finite element software LS-DYNA that is based on explicit integration methods. The fact of considering only longitudinal effects is enough to develop an accurate degradation analysis. For this reason, only one half of the track is modeled.

In the 3D model, finite elements of eight nodes are used. A rectangular cross section was taken into account to model the rail. Mechanical and geometrical characteristics of an UIC – 60 profile are taken into account.

Sleepers have a trapezoidal cross section and the mass is the half of one single sleeper, 315/2 kg. Railpads are modeled through solid elements. Mechanical features of this elements can be found in section 5.5.

The rest of the elements that make up the numerical model are also modeled with solid elements of 8 nodes.

Some of the advantages of this explicit algorithm can be found in Clough & Penzien (1975) and Wriggers & Laursen (2006). This methods are conditionally stable: may require use of extremely short time steps to avoid instability, in the higher mode responses of the numerical process.

### 3.3.3 2D model

In this model, 2D elements in plane stress are considered. This model has been developed in ANSYS, that is based on an implicit integration method. This is tool in which dynamic interaction analysis and degradation analysis have been carried out.

The rail is here modeled with truss elements, taking into account mechanical and physical properties corresponding to an UIC-60 profile. Railpads are modeled using the spring-damper element available in ANSYS.
3.3 Numerical models

The rest of the components involved, including the sleepers, are modeled with 2D finite elements of 4 nodes in a plane stress state.

A layout of the 2D numerical model is shown in Figure 3.5.

![Figure 3.5: Details of different elements in the 2D model: (a) culvert; (b) rail, railpad, sleeper and ballast](image)

**Implicit integration method**

As in the explicit algorithm case, the objective is to solve the differential equation of motion 3.5.

The aforementioned software ANSYS uses an implicit time integration algorithm. In this algorithm it is needed to invert $[K]$ matrix corresponding to equation of motion 3.5. The fact of invert $[K]$ matrix, makes the process computationally expensive.

The implicit algorithm is given by equation:

$$
[K] \{x\}_{n+1} = \{E\}_t
$$

with

$$
[K] = \frac{1}{2\Delta t^2} [M] + \frac{1}{2\Delta t} [C]
$$

and

$$
\{E\}_t = \{F(t)\}_t - \left( [K(x)] - \frac{2}{\Delta t^2} [M] \right) \{x\}_t - \left( \frac{1}{\Delta t} [M] - \frac{1}{\Delta t} [C] \right) \{x\}_{n-1}
$$

Since previous mass and damping matrices are not diagonal matrices, equation 3.8 represents a set of coupled equations.

As it was said before, this methodology may turn computationally very expensive.
Between different implicit methodologies, Newmark method has been used in ANSYS to carry out the numerical analysis of the current dissertation.

Some of the advantages of this implicit algorithm can be found in Clough & Penzien (1975) and Wriggers & Laursen (2006). This method is unconditionally stable: the errors are not amplified from one step to the next no matter how long a time step is chosen and hence can be applied with a far bigger time step than explicit schemes.

**Absorbing boundaries in 2D model**

The fact of trying to model an infinite space with a finite geometric region in combination with a certain constrains conditions in its edges, leads to spurious results because of the reflected waves phenomenon. This is a numerical problem that has been studied by many authors as: Lysmer, J; Kuhlemeyer (1969), Engquist & Majda (1977), R.Kosloff (1986), Trefethen & Halpern (1986), Kausel (1988), Pinsky & Abboud (1991) and Pinsky (1992).

Therefore, it is necessary to apply a proper methodology to remove the above mentioned problem of waves reflection. This methodology is known as absorbing boundaries and its application is described below.

In the frame of a dynamic analysis in time domain with a finite element model, an effective technique is suggested by Lysmer, J; Kuhlemeyer (1969), by the application of viscous dampers in the proper edges of the finite element model.

The proposal from Lysmer was the first one to be presented but remains a very efficient methodology that is still used in a great amount of numerical modeling problems. Advantages of this method have already been presented in Section 3.1.

In 2D model of ANSYS, Absorbing boundaries were normally applied. Definition of damping parameters has been made according to the work developed by White *et al.* (1977).

The damping matrix proposed by the authors does not depend neither the frequency of the waves nor the direction of waves propagation.

For a plane stress state, the above mentioned formulations is:

\[
C = \frac{8G}{15} \begin{pmatrix} 18s + 2 - 20s^3 & 0 \\ 0 & 2s + 3 \end{pmatrix}
\]

where:

- $G$: shear modulus.
- $V_s$: propagation velocity of S waves in the soil.
- $s$: is a parameter given by the next expression: $s = \frac{1-\nu}{2(1-\nu)}$

where $\nu$ is the Poisson coefficient of the soil.

Non-reflection condition is simulated through the application of viscous dampers, for both normal and tangential directions at the edges of the model.
3.3 Numerical models

In Figure 3.6 it is shown the effect of taking into account absorbing boundaries in the 2D numerical model when a vertical load is applied on the rail.

![Graph showing displacement over time with and without absorbing boundary conditions](image)

**Figure 3.6: Influence of the absorbing boundary conditions in the dynamic displacement of the rail**

**Calibration of the 2D model**

Calibration of the 2D numerical model is divided into two main phases. First a static calibration, where the thickness of the finite elements is calculated according to the position, in terms of depth, of the elements in the model. Then a dynamic calibration is carried out. In this dynamic calibration an adjust of the damping parameters of the model is made.

Both static and dynamic calibration have been performed taking the 3D model, explained in Section 3.3.2, as reference.

The static calibration methodology consists on the application of a vertical static load of 100 kN on the rail. This static load has been applied in both, 3D and 2D models. Static calibration initiates by making an iterative analysis in which the width of the elements of the 2D model changes. This iterative process stops when the difference in vertical deformations for both 3D and 2D models is smaller than a given tolerance. It has to be noted that the above mentioned vertical deformation is obtained in the vertical alignment of the load.

In Figure 3.7, a complete layout of the iteration process, is depicted.

In the previous scheme \( n \) is the number of finite element layers of the model (depends on the vertical discretization adopted).

For a given iteration of the process \((i + 1)\), the width of each finite element layer, \((b_n)_{i+1}\), depends on the strain of the layer in the 2D model, obtained in the previous iteration, \((\Delta h_p)_i\), and also on the strain of the equivalent layer of the 3D model\(^1\).

This procedure begins with \( i = 1 \), in this step, vertical strains of all the \( n \) finite element layers of the 2D model are calculated. Width for all layers is taking into

\(^1\)It is important to adopt the same vertical discretization for both: 2D and 3D models
3. Dynamic models adopted in a ballasted track. Application to transition zones

account, in this step, and it is equal to 1 m. At the end of each iteration, the error \( \varepsilon_n \), define as a difference between vertical strains in the 2D model and the 3D model, is evaluated. When the error is equal or less than 1%, the iteration process is canceled and the width of each layer is obtained. Even if the error is exceeded only in one layer, the iteration process goes on until the error is less than 1% for every layers.

This iteration procedure has been implemented in MATLAB and it allows a connection between MATLAB, ANSYS and LS-DYNA software, to carry out the iteration analysis.

The dynamic calibration methodology is a complement to the static calibration. 3D model allows to simulate properly radiation damping that actually occurs. This is not possible in the 2D numerical model because of limitations of bidimensional models, in a plane stress state.

It is necessary to perform an adequate calibration of the parameters that are contained in the Rayleigh damping matrix considered in ANSYS.

\[
C = M + \sum_{i=1}^{N_{mat}} \alpha_i K_i
\]

(3.10)

where \( \alpha \) and \( \beta \) are parameters that are multiplied by the mass and stiffness matrices,
respectively. Unlike parameter $\alpha$, in damping Rayleigh matrix is defined for each material $i$ that constitutes the structural system.

and parameters are obtained by fitting receptance curves of the rail, from the 2D and 3D numerical models. Receptance curves of both 2D and 3D models, are obtained through the excitation of the rail right above a sleeper. Calibration is done by using the adjust function of MATLAB with 'lsqcurvefit'. Through this calibration, it was possible to find the right damping parameters that better fit receptance curves from 2D to 3D numerical models.

3.4 Specific models for transition zones in a ballasted track

In this section, a brief summary of the main models developed to study the influence of transition zones in the dynamic behaviour of the track, is presented. Soriano (1989) developed a 2D numerical model with the purpose of analyzing the behaviour of transition soil wedges. This study allowed to develop a design criteria to define the transition soil wedges of the high speed railway line Madrid-Seville.

In the cases of buried structures it was observed the great influence of the $R/a$ relation, where $R$ is the height of the soil above the structure and $a$ is the height of the buried structure.

Hunt (1996) developed two analytical models based on the Winkler theory. The models were adapted in order to modeling track stiffness variations. Also in 1996, Hunt developed more complex finite element models that allow to evaluate situations in which permanent settlement of the track was taking into account.

In order to have a good ratio approach / time consumption, as in the general modeling of a ballasted track, several 2D models were developed. In this models the rail is supported by a set of springs and dampers.

Some authors that developed the above mentioned models were: Hunt (1996), Lei & Mao (2004), Namura & Suzuki (2007) and Varandas (2013).

On the other hand, this type of models do not work properly when complex geometries are considered.

In order to solve the problem of complex geometries, 3D models present a better behaviour because of their versatility, with the well known disadvantage of extra computational effort and additional runtime.

Lundqvist et al. (2006) developed a 3D numerical model in which variations of foundation stiffness were evaluated.

Nicks (2009) worked with a 3D model in which a complete embankment-bridge transition zone was assessed.
As in the general modeling of the regular track, in transition zones complexity of modeling should be in agreement with the characteristics that we want to analyze. There are models that are more focused in the dynamic analysis of the effects in the train-track system when an abrupt change of stiffness occurs: Hunt (1996) and Lundqvist et al. (2006). On the other hand there are more appropriate models to bear in mind effects due to changes in the longitudinal profile of the track Lei & Mao (2004), and Nicks (2009) or able to simulate hanging sleepers phenomena: Ishida et al. (1999), Lundqvist & Dahlberg (2005), Ribeiro (2012) and Varandas (2013).

Arroyo, JC. Benito, A. Pastor (2004) developed an elastic 2D numerical model in which geometric and geomechanical features of different layers of the infrastructure have been taking into account. This model was also appropriate to consider actions from different velocities of the trains, as well as the behaviour of the transition wedge in the train-track-subground group. The above mentioned 2D model has been calibrated with a more complex 3D model, developed for this purpose.

The presence of culverts or other different elements, that may cause an heterogeneity of the railway infrastructure, has been studied by González, P; Cuadrado, M; Romo (2001) using both the 2D and 3D numerical models. Through these models, the authors obtained global variation stiffness curves of the rail and also values of stresses on the subgrade soil. Moreover, the same authors presented a train-track dynamic interaction study, in which the effect of a train moving along a transition wedge, has been analyzed. This study was assessed with a 2D finite element model that was calibrated with a more complex 3D model. In this study effects as: the variation of the stiffness in the transition zone, the inertial behaviour of unsprung masses and its relation with the degradation phenomena of the track, are evaluated.

Mellat et al. (2014) carried out a dynamic behaviour analysis of a short span soil-steel composite bridge for high-speed railways. Other suggestions included in this work, was to improve the FE-model, assigning frictional behaviour between the backfill and the culvert with a non-linear behaviour in the interface.

With the aim of simulating the soil/structure interaction, it is important to highlight some works from different research fields: Goodman et al. (1968) were the first to develop interface elements to account for the relative displacements between rock joints. The above mentioned element consists of two lines, each with two nodal points. This element allows to simulate both tangential slippage across the interface and debonding of the soil and the culvert.

Chan & Tuba (1971) and Katona et al. (1976) used constraint equations to represent interface behaviour. In the case of Katona et al. (1976), the authors used constraint equations in the formulation of interface elements in culverts. The interface element is composed of two nodes, the first one belonging to the structure and the second one to the soil. Each of the previous nodes allow displacements in the horizontal and vertical directions. In this case, a third node is assigned inside of the defined set of nodes, with the purpose of providing equation numbers for the normal and tangential interface forces.
3.5 Models of the vehicle

Kim & Yoo (2005) investigated the effect of the soil-structure interface between the exterior box culvert wall and the filling soil. The authors developed two different methodologies to analyze the problem: a shear interface model and a spring interface model.

Shamsabadi (2007) used a 3D model to study the seismic soil-abutment-foundation-structure interaction. For this goal, interface elements have been used to model the abutment backwall-backfill interaction. In this case a bilinear model was used to describe the behaviour of interfaces for modeling the transition between the backfill and the abutment. The Coulomb’s Law was used to distinguish between a elastic domain (sticking) and plastic behaviour (sliding).

Finally, in order to simulate the long-term behaviour of the ballasted track in a transition zone, Lundqvist & Dahlberg (2004) considered an elastoplastic constitutive law for the subground materials.

In the same field Hunt (1996), Mauer (1995) and Bruni et al. (2002) developed bidimensional models in which the rail was modeled as a beam supported on a group of dampers and springs. The aim of these models was to analyze the long-term behaviour of the track in a transition zone by incorporating certain settlement laws for the ballasted layer.

Ribeiro (2012) developed a 2D numerical model in a plane stress state that is the background of the present dissertation. This model was calibrated using a more realistic tridimensional version that is better described in Section 3.3.2. A complete explanation of the referred bidimensional model is presented in Chapter 5.

3.5 Models of the vehicle

In the European network rail, there are three different types of passenger trains: conventional trains, regular trains and articulated trains.

In conventional trains, each carriage has two bogies, each bogie has a couple of axles. Vehicles of this type are: ICE2, ETR-Y, VIRGIN, AVE S-103.

In regular trains the carriages are articulated, but the hinge does not support in a bogie but only in an axle that is shared by the two adjacent carriages. Vehicles of this type are: TALGO, AVE S-102.

In articulated trains, carriages have in each edge a bogie with two axles. Bogies in this case are shared between two adjacent carriages. Trains of this type are: THALYS, EUROSTAR, TGV, AVE S-101.

In Vale (2010) a layout of bidimensional vehicle models can be found, see Figure 3.8. The way of modeling the vehicle depends on the efficiency of the procedure. Modeling can be more complex if the purpose is to analyze a wide range of frequencies, or
3. Dynamic models adopted in a ballasted track. Application to transition zones

![Diagram of vehicle models](image)

Figure 3.8: 2D Models to represent a part of the vehicle: (a) Half of the bogie; (b) Half of the vehicle, from Vale (2010)

less complex if what is required to obtain are only vertical components of the main parameters of the dynamic analysis: forces, velocities, accelerations...

Generally speaking, if it is desired to take into account passengers comfort, a 3D complete model should be taken into account.

According to Popp et al. (1999) a vehicle can be modeled as a multi-body rigid system or as a multi-body elastic system. Models of multi-body rigid system are the more appropriate to analyze problems with a range of frequencies $0 - 50$ Hz. On the other hand multi-body elastic systems are indicated to analyze a wide range of frequencies $50 - 20000$ Hz. These last models have been developed to analyze, above all, acoustic problems.

A different classification of the vehicle models can be found in Nguyen (2013).

### 3.6 'LongTermSim'

#### 3.6.1 Introduction

This program was developed at FEUP in the framework carried out by Ribeiro (2012), to analyze the long-term behaviour of the ballasted track.

Settlement of the track grows in time. Several studies show the importance of cyclic loading in the general permanent settlements of the track, as Abdelkrim et al. (2003), Ribeiro (2012), Varandas (2013) and Nguyen (2013).

As referred in Abdelkrim et al. (2003), the global settlement of the track is composed by two different terms. The first term is a plastic or residual component, this represents the permanent settlement that has been accumulated over the historic loading process of the track.
The second term, \( e_t \) is an elastic component and refers to a settlement that is restored as the load is removed.

A layout of the global settlement process, is shown in Figure 3.9.

![Diagram of global settlement process](image)

Figure 3.9: Evolution of the global settlement of a platform subject to repeated traffic loading, from Abdelkrim et al. (2003)

The increment of the permanent settlement between two adjacent cycles \((N, N + 1)\) is very low. For this reason and in order to improve the efficiency of the computational process, Abdelkrim suggests to consider loads increment as the sum of \( N^2 \) cycles.

### 3.6.2 Simulation of the permanent deformation

Methodology that has been used to simulate permanent settlement of the track, consists on an iterative process that has been implemented by articulating ANSYS and MATLAB.

Numerical modeling of the vehicle and the track, pre-processing and post-processing of data are made in ANSYS. On the other hand, reading of the dynamic results as well as the application of permanent deformation laws and determination of permanent deformation values, are performed in MATLAB.

In Figure 3.10 a complete layout of the simulation of permanent deformation process is made.

Summarizing, to have a good predictive tool it is necessary:

- An extensive knowledge of the long-term behaviour of the different track materials.
- Consideration of adequate models that can, as closely as possible, reproduce the dynamic behaviour of the track.

\(^2\text{N could be a quantity of thousands of cycles}\)
3. Dynamic models adopted in a ballasted track. Application to transition zones

The simulation process starts assuming that every layers of the track are modeled with finite elements, whereby provide information of the stress state that arises when a vehicle is moving along the track, is provided.

A moving train induces a multidirectional stress field in the layers below. This multidirectional stress field has both normal and tangential components that have a greatest weighting when permanent settlement of the track is obtained.

After performing the dynamic interaction analysis, values of stresses (vertical, horizontal and shear) of certain finite elements\(^3\), are taken.

Variation of stresses over the time, for each layer of the track is stored in a matrix. The size of this matrix depends on the number of time steps as well as the number of the finite elements contained in each layer.

Once the value of stresses has been taken, evolution of the principal stresses, over the time, is obtained in each finite element. Maximum values of the principal stresses are stored as a vector that is crucial to obtain permanent settlements, using deformation laws of the materials.

Permanent deformation of the track due to the load of a moving train induces a very low deformation. In order to do the process more efficient, a set of \(\Delta N\) cycles are

\(^3\)Finite elements that belong to the layers which have an influence on the permanent settlement of the track
taken into account in each simulation, assuming that during this interval of cycles \( \Delta N \), values of strain and stress for all materials remains constant.

Permanent deformation values that is applied to the finite elements has been determined in MATLAB, according to the deformation laws of the track materials. In MATLAB are also written the files used to apply permanent settlements in each finite element of the ANSYS model, updating the track geometry in a closed-loop process.

The following considerations and simplifications are made to develop the above mentioned methodology:

- For every dynamic analysis, track/train interaction is considered.
- For a certain number of cycles \( \Delta N \) of the iterative process, values of stresses in the finite elements, remains constant.
- The resolution of permanent deformation deformation is done element by element.
- Only permanent deformation in the vertical direction is considered.

### 3.6.3 Settlement laws

Settlement of the track is the result of permanent deformation in different layers of the track infrastructure: ballast, sub-ballast and railway platform.

According to Selig & Waters (1994) and Guérim (1996), ballast settlement represents 50% - 70% of the total track settlement. See Figure 3.11.

Figure 3.11: Contribution of the different layers to the total track settlement, in a new construction track, adapted from Selig & Waters (1994)

In Figure 3.11, contribution of each layer to the total track settlement is shown for a new construction track. In this diagram, ballast contribution is approximately 50% of the track settlement.
It should be noted that the phase represented in the previous figure corresponds to the lowest contribution of the ballast layer (new construction track).

In tracks with more advanced serviced phase, contribution of the rest of the layers is minimum. On the other hand, this is the phase in which the influence of the ballast settlement in the total degradation process of the track is the highest.

Permanent deformation of the ballast layer consists on the compaction of this granular layer. Such compaction is due to the relocation and the erosion of its particles. Variation of the height of this layer as well as the presence of tiny particles show signs of wear.

Holtzendorff & Gerstberger pointed out the next parameters as the main variables to take into account in the ballast settlement phenomenon:

1. Deviatoric stress: For a certain level of deviatoric stress, the friction resistance of the ballast particles is exceeded. Consequently the particles start to move and rotate and horizontal spreading of the ballast layer can be observed.

2. Vibrations: The effect of passing trains produces vibrations in the ballast stones that may start to flow in lateral directions.

3. Particle abrasion and breakage that are caused by particle slip, tamping actions and other different environmental sources.

4. Subgrade stiffness: The more the subgrade deforms elastically, the more the ballast particles are able to move into different directions.

According with the study developed by ORE (1970), growth rate of the permanent deformation of the ballast layer is reduced as the number of cycles of the degradation process increases. Authors proved that in first loading cycle, vertical deformation of the ballast layer is very high. After that vertical deformation of the ballast layers evolves following a logarithmic law:

\[
N = 1 (1 + C \log(N))
\]

where: \( N \) represents permanent deformation of the ballast layer after \( N \) cycles, \( C \) is a constant parameter, that is consider by the authors as 0.2 and \( 1 \) is permanent deformation of the ballast layer due to the first loading cycle. This value is given by equation 3.12:

\[
1 = 0.082 \left(100n_p - 38.2\right) \left( \sigma_1 - \sigma_3 \right)^2
\]

where: \( \left( \sigma_1 - \sigma_3 \right) \) is the deviatoric stress value for the specific material, \( n_p \) is the ballast layer porosity parameter. It varies from 0.4-0.5, see ORE (1970) and Selig & Waters (1994).

According to Ionescu (2004), permanent settlement of the ballast layer, that comes from the degradation law suggests by ORE, fits very well to the reality, specially from the first 100000 cycles of the loading process. See Figure 3.12.
Permanent deformation of the ballast layer may be divided into two different phases. The first one is characterized by a rapid increase of the permanent deformation that is associated to an individual stabilization of the ballast, while the second phase is characterized mainly by the evolution of the permanent deformation due to the trains loading.

In this work, only the second phase of the ballast deformation is considered. The evolution of the permanent deformation in the ballast layer is:

\[
N = 1 \cdot C \log \left( \frac{N + N_i}{N_i} \right)
\]  

(3.13)

where \(N_i\) is the number of loading cycles in the first phase, \(n_1\) given by equation 3.12, is permanent deformation in the first loading cycle, \(C\) is a constant parameter, equal to 0.20, \(n_p\) is a porosity parameter. If ballast layer is well compacted \(n_p = 0.4\), otherwise \(n_p = 0.5\).

According to Ionescu (2004) the first phase of the permanent deformation of the ballast layer may be considered as 100000 loading cycles.

Gidel (2001), proposes a permanent deformation law that allow to simulate evolution of settlement for granular materials.

This law depends on the loading cycles as well as the stress values existing in the granular material. Gidel suggests to obtain permanent deformation of materials \(p\) by multiplying two functions, as shown in equation 3.14.

\[
p = f(N)g(p_{\text{max}}, q_{\text{max}})
\]  

(3.14)
3. Dynamic models adopted in a ballasted track. Application to transition zones

Function $f$ depends on the number of loading cycles $N$, see equation 3.15:

$$ f(N) = A \left(1 - \frac{N}{100}\right)^{-B} $$

(3.15)

where: $A$ and $B$ are parameters experimentally determined.

Function $g$ depends on the maximum value of the applied stresses:

$$ g(p_{\text{max}}, q_{\text{max}}) = 1 - \frac{\frac{l_{\text{max}}}{p_a}}{n} \frac{1}{m + \frac{s}{q_{\text{max}}} - \frac{q_{\text{max}}}{p_{\text{max}}}} $$

(3.16)

where $q_{\text{max}}$ is the maximum value of the deviatoric stress, given by equation 3.17.

$$ q = 1 - 3 $$

(3.17)

and $p_{\text{max}}$ is the value of the maximum stress, given by equation 3.18.

$$ p = \frac{1 - \frac{2}{3}}{3} $$

(3.18)

where $\sigma_1$ and $\sigma_3$ are the values of principal stresses, maximum and minimum respectively, $p_a$ is the constant values of the atmospheric pressure and $l_{\text{max}}$ is given by

$$ l_{\text{max}} = p_{\text{max}}^2 + q_{\text{max}}^2. $$

Parameters: $s$, $m$, $n$ and $1$ are experimentally determined and their value depends on the type of material.

In Fortunato (2005), it is possible to find suggestions for the the above mentioned parameters. Furthermore, Soreze and Poulmarch defined also these parameter values for granular materials.

### 3.6.4 Input data in the 'LongTermSim' program

As aforementioned, this program was developed in the LESE (Laboratorio de Engenharia Sismica e Estrutural) of the Faculty of Engineering of the University of Porto.

The program is implemented in MATLAB, in articulation with ANSYS where the dynamic interaction analysis is carried out.

---

*For each layer the maximum value of deviatoric stress, in the horizontal discretization, is the average of the maximum values of deviatoric stresses in the finite elements aligned in the vertical direction*
Entry of data  The program LongTermSim is well structured, presenting an easy interface to work with. In the first part it is necessary to select the track layers where permanent deformation is going to be considered, see Table 3.1.

### Table 3.1: Input options for track layers in LongTermSim

<table>
<thead>
<tr>
<th>Input option</th>
<th>Sub-options</th>
</tr>
</thead>
<tbody>
<tr>
<td>Permanent deformation in ballast layer</td>
<td>Yes</td>
</tr>
<tr>
<td>Permanent deformation in sub-ballast layer</td>
<td>Yes</td>
</tr>
<tr>
<td>Permanent deformation in form layer</td>
<td>Yes</td>
</tr>
<tr>
<td>Permanent deformation in subgrade</td>
<td>Yes</td>
</tr>
<tr>
<td>Consider any extra layer</td>
<td>Yes</td>
</tr>
<tr>
<td>Number of extra layers</td>
<td>Number</td>
</tr>
</tbody>
</table>

After defining the layers in which permanent settlement is going to be considered, it is necessary to give information about each of the layers considered. With this information, some important values of the response are obtained and saved. Some of the values referred are:

- Stresses in different elements of the track.
- Wheel/rail interaction forces.
- Vertical accelerations of the axles of the train.
- Vertical displacements of the wheels.
- Ballast/Sleeper interaction forces.
- Vertical displacements of the bottom of sleepers and top of the ballast layer.
- Acceleration of sleepers.

Below, information relative to each particular layer is entered, see Table 3.2.

### Table 3.2: Input options for a particular layer in LongTermSim

<table>
<thead>
<tr>
<th>Layers information</th>
<th>Inputs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Material Property number</td>
<td>value</td>
</tr>
<tr>
<td>Thickness</td>
<td>value</td>
</tr>
<tr>
<td>Vertical discretization</td>
<td>value</td>
</tr>
<tr>
<td>y-coordinate of the top of the layer</td>
<td>value</td>
</tr>
<tr>
<td>x-coordinate of the first node of the layer</td>
<td>value</td>
</tr>
<tr>
<td>x-coordinate of the last node of the layer</td>
<td>value</td>
</tr>
<tr>
<td>Longitudinal discretization</td>
<td>value</td>
</tr>
</tbody>
</table>
After entering information relative to the layers, it is necessary to introduce some information regarding to the sleepers, see Table 3.3.

Table 3.3: Input information for the sleepers in LongTermSim

<table>
<thead>
<tr>
<th>Sleepers information</th>
<th>Inputs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sleeper material property number</td>
<td>value</td>
</tr>
<tr>
<td>Number of the sleepers in the model</td>
<td>value</td>
</tr>
<tr>
<td>Sleepers spacing</td>
<td>value</td>
</tr>
<tr>
<td>Sleeper base width</td>
<td>value</td>
</tr>
<tr>
<td>Longitudinal discretization of the sleeper base</td>
<td>value</td>
</tr>
<tr>
<td>Sleeper height</td>
<td>value</td>
</tr>
<tr>
<td>Vertical discretization of the sleeper</td>
<td>value</td>
</tr>
</tbody>
</table>

When information of different layers and elements of the track has been entered, wheel/rail contact data can be written, see Table 3.4.

Table 3.4: Input information for the wheel/rail contact

<table>
<thead>
<tr>
<th>Wheel/rail information</th>
<th>Inputs</th>
</tr>
</thead>
<tbody>
<tr>
<td>Number of wheels considered</td>
<td>1 – 2</td>
</tr>
<tr>
<td>Wheel mass element type number</td>
<td>value</td>
</tr>
<tr>
<td>Wheel/rail contact spring Element type number</td>
<td>value</td>
</tr>
<tr>
<td>Wheel-rail contact spring-height</td>
<td>value</td>
</tr>
</tbody>
</table>

After defining information about the layers and the wheel/rail contact elements, it is possible to define options to carry out the long-term degradation analysis. With this purpose information will be divided into two different phases:

- A first part, in which the number of iterations of the simulation process and the number of cycles to consider in each iteration are defined.
- A second part, relative to the long-term behavior of the different track layers. Here it is important to distinguish two different types of layers: the ballast and the rest of the constitutive layers.

For ballast layer, degradation law given in ORE (1970) has been used. Furthermore, for this layer both: initial number of cycles to discount (to take into account ballast stabilization) and ballast porosity can be chosen.

On the other hand, degradation laws for the rest of the layers may be considered as well.
3.6 'LongTermSim'

Finally it is necessary to enter information related to dynamic analysis options. With this information it is possible to read the script file (APDL), in which data about dynamic analysis and numerical model are written.
3. Dynamic models adopted in a ballasted track. Application to transition zones
4 Methodology in the dynamic interaction analysis

4.1 Introduction

This chapter describes and explains the methodology, applied in this thesis, to obtain results from both the short-term and long-term analyses.

In the current chapter, a special assessment of two different variables that have a significant influence in the dynamic behaviour of the train-track interaction, is carried out.

- Contact elements in the different interfaces.
- Track irregularities.

Regarding to contact elements in the different interfaces, it is important to be mentioned that the different interfaces that has been considered in this work are: Soil/structure interface, Ballast/ballast interface and sub-ballast/sub-ballast interface.

Track irregularities phenomenon has had a great development in high-speed railway research. Track irregularity research had a contribution of numerous studies, especially during the early. Subsequent section present a brief description of this subject. In this work, consideration of track irregularities has been performed by changing longitudinal level (vertical direction) geometry of the rail profile.

Justification for the choice of both the track irregularities and contact element in different interfaces, has been already provided by Ribeiro (2012) and the most relevant conclusions are listed below.

- «It was verified that during evolution of permanent deformation of the track, deformations in the zone with a higher variation of stiffness have also a greater value». This fact justifies the use of contact elements in the different interfaces, specially in the soil/structure interface, in order to simulate the frictional behaviour at these points. Behaviour of this type of elements is analyzed in Section 4.7 of the current chapter.

- «It is concluded that considering dynamic component of the interaction force is very important to estimate evolution of the permanent deformation of the track in a transition zone. Instead of presenting a low influence at the beginning, its contribution is higher and higher as the deformation process goes on. It was
verified that for a high value of the loading cycles, accelerations in the axles of the train as well as wheel/rail interaction forces reached immediate intervention level. This fact justifies the use of track irregularities, particularly longitudinal level irregularity profiles, to assess its influence in the global degradation process of the transition zone.

Since both the track irregularities and interface influence phenomena, actually exists in a real track problem, it is important to take into account these factors in order to have an approach closer to the reality.

### 4.2 Contact elements theory

In this work, contact elements have been used with three different purposes.

The first one is to study the contact phenomenon between the wheel and the rail. This fact leads to the development of a dynamic interaction analysis in which two systems: the train and the track are interconnected. In this case, not only the effect of a moving load but also inertial effects of the train moving over the track have been considered.

The use of contact elements between the wheel and the rail allows to simulate the loss of contact between the train and the track, as well as the accelerations transmitted from the track to the vehicle and vice versa.

Another purpose of using contact elements in this work is to model sleepers/ballast interface. This use allows to simulate the hanging sleepers phenomenon, commonly associated with degradation states of the track, especially in transition zones.

Finally, contact elements have also been used in this work as a essential elements to simulate the behaviour of soil/structure interface. This fact is very important particularly in this work in which an embankment/structure transition is assessed.

The use of contact elements has been proven crucial to simulate the effect of differential settlements between the soil and the structure sections, in numerical models with finite elements.

Below, a general description of the contact algorithms that have been used in this thesis, is provided.

#### 4.2.1 Contact problem

Contact problem allows to determine the movement of deformable solids that experiment a mechanical interaction. The objective is to solve the set of configurations that solids have in each interval of time.

In this section, a mathematical resolution of the contact problem is briefly presented.
4.2 Contact elements theory

![Figure 4.1: Layout of contact between two bodies](image)

Figure 4.1, represents two bodies $C^1$ and $C^2$ that can be in contact after experiment a certain displacement $d^1$ and $d^2$. In Figure 4.1 $X^1$ and $X^2$ represent generalized coordinates of the contact points of the bodies $C^1$ and $C^2$ and vector $\vec{n}$ represents the direction in which the bodies separate from each other.

Two different situations may arise in a contact problem: stick contact and slip contact. In both cases, contact mechanic between the two bodies is a nonlinear global optimization problem with some restrictions. Below, the general expression for potential energy between two bodies in contact is shown:

$$ (u) = \sum_{\alpha=1}^{2} \int_{C_{\alpha}} \frac{1}{2} \alpha^{\alpha} dv - \int_{C_{\alpha}} b^{\alpha} dv - \int_{a} t^{\alpha} ds + c $$ \hspace{1cm} (4.1)

where: $b$ is volume forces vector, $t$ surface vector applied on body, $\sigma$ are stress vector on a certain point, $\varepsilon$ deformation vector on a certain point, $\varepsilon$ is an additional function that takes into account the contact phenomenon in the potential energy of the system, Bathe (2006).

In a stick contact problem the total energy in the contact points of the bodies $(u)$ has to be minimized. Limitations in this case are that separation between the contact points is $g_N \geq 0$.

This condition allows loss of contact or contact without penetration.

Tangential stress in the contact point is lower than the maximum value of tangential stress, given by multiplying the normal stress in the contact point $N$ and the friction coefficient $\mu$. See equation 4.2

$$ \tau = \mu |N| $$ \hspace{1cm} (4.2)

In a slip contact problem the total energy in the contact points of the bodies has to be minimized as well, but in this case the second condition is given by equation 4.3:
4. Methodology in the dynamic interaction analysis

\[ \tau = \mu |N| \]  \hspace{1cm} (4.3)

In the previous equations, function \( g_N \) : separation between the contact points is given by equation 4.4, according to Figure 4.1.

\[ g_N = x^2 - x^1 \overline{n} \]  \hspace{1cm} (4.4)

Solution of the problem for both the stick and slip contact, consists on the control of no penetration condition between the contact surfaces. There are some methodologies to solve this type of problem. In this work, two of these methodologies have been used depending on the particular conditions of the contact problem: Penalty methodology and Augmented Lagrange methodology.

Below, a brief description of each methodology is presented regarding the particular implementation that these methods have in ANSYS code. For both methodologies and in order to simplify the explanation, only frictionless cases are presented.

### 4.2.2 Penalty method

In this method it is assumed that contact force depends on two parameters: the contact stiffness given by parameter \( e_N \) (in ANSYS called FKN), and the existing distance between two points of the two separated bodies \( g_N \) (gap).

This method includes penetration restrictions directly in the formulation. In such a way, formulation problem has no restrictions.

For the general frictionless case, potential energy is given by:

\[ \epsilon = \frac{1}{2} \int_{c} e_N \bar{g}_N^2 \, ds \]  \hspace{1cm} (4.5)

with \( e_N > 0 \)

In equation 4.5, \( e_N \) is the penalty parameter and \( \bar{g}_N \) is the penetration function.

One of the problems of the penalty method is to satisfy the no penetration condition. Theoretically this condition would be satisfied if the penalty parameter is infinite, but from a physical point of view, this fact occurs only in certain situations. On the other hand in ANSYS, very high values of \( e_N \) may reduce the value of penetration at the expense of the analysis convergence.

Therefore, it can be concluded that penalty method presents a good behaviour, in terms of computational analysis, because of the simplicity in its formulation. On the other hand, if values for \( e_N \) parameter are too high, problems of convergence may occur.
4.2.3 Lagrange multiplier method

This methodology is characterized by adding the contact force as an additional degree of freedom in the formulation.

Advantage of the formulation is that no penetration condition can be satisfied without using contact stiffness or imposed penetrations.

For a frictionless case, formulation is given by equation 4.6:

\[ e = \int e ( N g_N) ds \]  

where \( N \) is the Lagrange multiplier and \( g_N \) is the separation between points of the contact \( C^1 \) and \( C^2 \).

\( N \) can be identified as the normal component of the stress vector \( p_N \) and it represents the force applied to avoid penetration between the two bodies in contact.

One of the main disadvantages of this method is the convergence problem that may occur because of a sudden transition between an open status through a close status. This problem can be solved by changing the values of TOLN (maximum penetration value allowed) and FTOL (maximum force value allowed), implemented in ANSYS.

Other problem, associated with this method is the additional runtime when compared with other methodologies such as the above mentioned Penalty method.

4.3 Contact elements in ANSYS

4.3.1 Resolution of the contact process

Contact analysis is considered into the group of nonlinear problems. Contact problem is included into particular case to the category of changing-status nonlinearities.

Dealing with contact problems may lead to two difficult situations: First, contact areas are unknown until the problem is solved, surfaces can be in contact or can experiment loss of contact depending on the type of material, load value, boundary conditions... Secondly, in many contact problems it is necessary to take into account the friction between the contact surfaces. There are many different laws that take into account frictional behaviour and all of them exhibit a nonlinear behaviour.

The first thing is create the solid model entities that represent the geometry of the contact solids. After that, it is necessary to identify contact pairs where contact might occur during the deformation of the multi-body model.

Once potential contact surfaces have been identified, the next step is define them via contact elements. These are define in ANSYS through the implemented elements: TARGE and CONTA.
4. Methodology in the dynamic interaction analysis

Contact situations involved in this work, are considered as a Node-Surface case. For this purpose, particular node-surface contact elements: CONTA 175 and TARGE 169 are used.

4.3.2 Description of a node-surface contact with CONTA175 and TARGE169

In Figure 4.2, a layout of the node-surface control used in ANSYS, is shown.

![Figure 4.2: Layout of the node-surface contact used in ANSYS](image)

Target surface TARGE 169, can be rigid or deformable. In a bidimensional model, target surface can be described as a set of straight lines, arches and parabolic curves.

For a rigid-flexible contact, target elements are always attributed to the stiff part. If contact is a flexible-flexible type, attribution of target and contact elements is not simple and there are some basic rules that may help to do the best choice, ANSYS (2009).

Contact element CONTA 175, is defined by one node. This element is located on the surfaces of the solids. Contact occurs when this element penetrates one of the target segment elements on a specified target surface. CONTA 175 supports both isotropic and orthotropic Coulomb friction. This element also allows separation of bonded contact to simulate interface delamination.

4.4 Wheel/rail interaction model

Hertz theory (1882) suggests that contact area between two bodies, with certain conditions, is elliptic and normal stresses distribution is a semi-ellipsoid, where stress value is zero at the borders of the ellipse and it has a maximum value in the center. See Figure 4.3.
4.4 Wheel/rail interaction model

Hertz theory is commonly used to analyze contact analysis in the field of HSL. This theory provides good levels of accuracy with little computational demand as referred by Antolín (2012).

In Hertz theory, some hypothesis are made, to take into account the effect of the contact Johnson (1985):

- Surfaces of the solids are continuous and non conforming.
- Each solid can be considered as an elastic half-space.
- The surfaces are frictionless.
- The strains are small.

![Hertzian contact, general case](image)

Figure 4.4: Hertzian contact, general case, from Ayasse, J. Chollet (2006)
Instead of using formulation for two generic solids, it is more efficient and simple to consider that both solids are solids of revolution. In this case, overestimation of deformations in the contact are only 5% with an overestimation of the maximum value of the contact pressure \( p_0 \) equal to 8%.

For the particular case of two solids of revolution:

\[
A_1 = B_1 = R_1 \tag{4.7}
\]

\[
A_2 = B_2 = R_2 \tag{4.8}
\]

In this case, contact are is a circle with radius \( a \) given by equation 4.9

\[
a = \frac{3PR}{4E} \tag{4.9}
\]

where \( P \) is the total compression load between the two bodies and \( R \) is the equivalent bend radius given by equation 4.10, and \( E \) is given by equation 4.11.

\[
R = \frac{R_1R_2}{\bar{R}} \tag{4.10}
\]

\[
\frac{1}{E} = \frac{1 - \nu_1^2}{E_1} + \frac{1 - \nu_2^2}{E_2} \tag{4.11}
\]

In equation 4.11 \( E_i \) is deformability modulus corresponding to body \( i \) and \( \nu_i \) is the Poisson modulus corresponding to body \( i \), where \( (i = 1, 2) \).

The gap between two points of the solids is obtained in equation 4.12.

\[
\delta = \frac{a^2}{R} = \frac{3}{16} \frac{9p_0^2}{RE^2} \tag{4.12}
\]

where \( p_0 \) is the maximum value of contact pressure:

\[
p_0 = \frac{3P}{2a^2} = \frac{3}{16} \frac{6PE^2}{R^2} \tag{4.13}
\]

From equation 4.12 it can be easily deduced that:

\[
P = \frac{16RE^2}{9} \frac{3}{7} \tag{4.14}
\]

The root term in equation 4.14 can be named as \( C_H \).
4.4 Wheel/rail interaction model

\[ C_H = \frac{16RE^2}{9} \]  

(4.15)

where \( C_H \) is designed as Hertzian constant. This constant depends on the bend radius as well as the mechanical properties of the contact bodies.

Contact stiffness is obtained by derivation \( P \) over \( \delta \).

\[ K_H = \frac{dP}{d\delta} \]  

(4.16)

equation 4.16 leads to:

\[ K_H = \frac{3}{2} C_H \frac{3}{2} P^{\frac{3}{2}} \]  

(4.17)

According to equation 4.11 and substituting 4.15 in 4.17, next value of contact stiffness is obtained:

\[ K_H = \frac{3 \sqrt{3RPPE^2}}{2 (1 - \nu^2)^2} \]  

(4.18)

As it is shown in equation 4.18 relationship between \( P \) and \( K_H \) is nonlinear.

In this work, a linearization of \( K_H \) is made, according to the limitations proposed by Vale (2010) and Wu & Thompson (2000).

Wu & Thompson (2000): The non-linear wheel/rail dynamic interaction model can be well approximated using an equivalent linear model when the roughness level between the rail and the wheel is not extremely severe and a moderate static preload is applied to keep the wheel and the rail in contact.

Vale (2010) pointed out:

- In cases in which track has no irregularities, linear and nonlinear contact stiffness present similar results.
- In cases of track with distributed irregularities with a standard deviation equal or lower than 1.25 mm it can be adopted an interaction model with a linear stiffness in the contact.

The fact of taking into account a multi-body system instead of a moving load system, can be critical in certain situations. As an example, Figure 4.5 shows the differences between displacements and accelerations at middle span of a simply supported bridge, Rigueiro (2007).

In order to analyze the dynamic interaction behaviour between the train and the track, a simple 2D model is created in ANSYS. With this model, several features of the
dynamic interaction analysis can be analyzed such as: Penalty/Lagrange multipliers differences, frequency content in interaction forces, influence of sleepers distance in the interaction forces magnitude...

Below, a description of the 2D model is provided. The available elements in ANSYS as well as the mechanical characteristics of the different materials that make up the model are also here described.

The bidimensional model has a total length of 20 m and represents a section of the track. To carry out this analysis only rail, railpad and supported condition given by sleepers are taken into account. The track is subjected to a vehicle in motion, that is model as a sprung mass system, moving along the the model at a speed of 300 km/h. Longitudinal discretization of the rail is done with beam elements of 0.05 m length . The time step used to carry out the dynamic analysis is $dt = 0.0005 \text{s}$. A layout of the above mentioned model can be found in Figure 4.6:

![Layout of the bidimensional model](image)

Figure 4.6: Layout of the bidimensional model used to carry out the dynamic interaction analysis

Mechanical characteristics for the rail, railpad and sprung mass system are defined in Table 4.1.
Table 4.1: Mechanical characteristics of the elements that make up the bidimensional model

<table>
<thead>
<tr>
<th>Sprung mass</th>
<th>$M, (kg)$</th>
<th>1000</th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_h, (\frac{N}{m})$</td>
<td>1.43E9</td>
<td></td>
</tr>
<tr>
<td>$C_p, (\frac{N\cdot s}{m})$</td>
<td>30E3</td>
<td></td>
</tr>
<tr>
<td>$K_p, (\frac{N}{m})$</td>
<td>200E6</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Railpad</th>
<th>$A, (m^2)$</th>
<th>7.7E-3</th>
</tr>
</thead>
<tbody>
<tr>
<td>$\rho , (\frac{kg}{m^3})$</td>
<td>7850</td>
<td></td>
</tr>
<tr>
<td>$I, (m^4)$</td>
<td>30.55E-6</td>
<td></td>
</tr>
<tr>
<td>$\nu, (-)$</td>
<td>0.35</td>
<td></td>
</tr>
<tr>
<td>$E, (\frac{N}{m^2})$</td>
<td>200E9</td>
<td></td>
</tr>
</tbody>
</table>

A simple illustration of the elements adopted in ANSYS to build the model is shown in Figure 4.7. In Table 4.2 a correlation between the real elements of the track and the particular elements adopted in ANSYS, to model both physical and geometrical characteristics of the problem, is established.

![Figure 4.7: Layout of the particular elements available in ANSYS to carry out the interaction analysis](image)

Dynamic interaction results are obtained, from two different contact algorithms: Penalty method and Lagrange Multipliers method. Interaction forces between the track and the sprung mass system, are depicted in Figure 4.8 for both the time and frequency domains.

In Figure 4.8 it can clearly be seen the similarity of interaction forces response using either Penalty method or Lagrange Multipliers method. Analyzing the frequency contain of contact forces, two main frequencies can be identified. The first one, with a value of 83.190 Hz, that corresponds to the value of the global vibration frequency of the system. Results for this frequency, obtained in ANSYS is 83.323 Hz as it can be seen in Figure 4.10(a). The second one with a value of 139.8 Hz, that corresponds to the parametric excitation frequency due to the distance between the sleepers. This frequency is given by the ratio between the distance between sleepers and the speed of the train. For this particular case, distance between the sleepers is $d = 0.60\, m$. 

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4. Methodology in the dynamic interaction analysis

Table 4.2: Relation between the structural systems and the elements adopted in ANSYS

<table>
<thead>
<tr>
<th>System</th>
<th>ANSYS elements</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sprung mass</td>
<td>structural mass, MASS21</td>
</tr>
<tr>
<td></td>
<td>spring-damper, COMBIN 14</td>
</tr>
<tr>
<td>Track</td>
<td>rail, BEAM 3</td>
</tr>
<tr>
<td></td>
<td>railpad, COMBIN 14</td>
</tr>
<tr>
<td>Contact</td>
<td>contact element, CONTA 175</td>
</tr>
<tr>
<td></td>
<td>target surface, TARGE 169</td>
</tr>
</tbody>
</table>

Figure 4.8: Comparison between Penalty method and Lagrange Multipliers method: (a) interaction forces; (b) frequencies contain for a distance between sleepers d=0.60 m

and velocity of the sprung mass is \( v = 300 \text{ km/h} \). Therefore parametric frequency is

\[
f = \frac{1}{T} = \frac{v}{d} = \frac{300}{(0.60 \times 3.6)} = 183.33 \text{ Hz}.
\]

If distance between the sleepers decreases from 0.60 m to 0.50 m, results for both cases are depicted in Figure 4.9.

In this case, when the distance between the sleepers is 0.5 m, the system becomes more rigid and consequently the fundamental frequency of vibration increases to 86.52 Hz, very close to the value obtained in ANSYS for the fundamental frequency of the system: 87.82 Hz. See Figure 4.10(b). On the other hand the parametric frequency given by the sleepers distance increases to 166.40 Hz, which is also very similar to the value obtained dividing the speed of the system and the sleeper distance:

\[
f = \frac{1}{T} = \frac{v}{d} = \frac{300}{(0.50 \times 3.6)} = 166.67 \text{ Hz}.
\]

In this case, it has been shown how results obtained either through Penalty method or Lagrange Multipliers are very similar using both methods. However, there are some cases in which results may have differences, mainly due to the penetration value allowed in the Penalty methodology, see Figure 4.11.
4.5 Contact elements in ballast/sleeper interface.

The use of this type of element in ballast/sleeper interface is fundamental in cases in which a rigorous track degradation analysis is made.

Hanging sleepers phenomenon, at transition zones, consists on the presence of some voided sleepers, in the vicinity of the sections where variations of the track stiffness occurs. These sleepers are badly supported and in many cases do not have a contact with the ballast layer.

The voided sleepers are not able to support the train load and the do not transfer any force to the ballast layer. Instead the load is distributed on the adjacent sleepers around the voided one.

Hanging sleepers phenomenon causes a decrease in the vertical track stiffness. This change of track stiffness causes high interaction forces between the wheel and the rail which in turn increases the track settlement rate.

According to Bezin et al. (2009) in a general hanging sleepers case, the voided sleeper is not able to support the vehicle axle load and consequently, it does not transfer any force to the lower element, in this case the ballast layer. Instead, the force is distributed on the adjacent sleepers around the voided one. Figure 4.12 shows a pattern of a voided sleeper in the case in which the axle load is above the sleeper \(i\). The general pattern emerging is that of the nominal static axle force, named \(F_n\), increasing slightly on sleepers \(i + 2\) and \(i - 2\) and increasing more significantly on

---

Figure 4.9: Comparison of interaction forces using different values of distance between the sleepers

\(d\): (a) interaction forces; (b) frequencies contain.

Differences between both methodologies have been evaluated again in subsequent sections.
Figure 4.10: Full track resonant frequency: (a) distance between the sleepers \(d=0.60\,\text{m}\); (b) distance between the sleepers \(d=0.5\,\text{m}\)

sleepers \(i+1\) and \(i-1\). Sleepers further away do not seem to be affected by any force increase.

### 4.6 Contact elements in the soil/structure, sub-ballast/sub-ballast and ballast/ballast interfaces

In this work, contact elements have been used with great success to model two different types of contact interfaces at the transition zone analyzed.

The first one (contact elements type 1) is the Soil/structure interface. Inclusion of contact elements here, is important to model the real frictional behaviour existing between two different materials. In this work, these contact elements have been used to model the frictional behaviour between the concrete of the wedge-shape backfill.

The second one (contact element type 2) is: Sub-ballast/sub-ballast and ballast/ballast interfaces. Inclusion of this elements allows to simulate the shear strength of both the ballast and sub-ballast layers. These elements are a vertical extension of the contact elements that simulate the soil/structure interface. All of them, allow to introduce differential settlements between sections A and B, Figure 4.13.

Figures 4.14 and 4.15 show the effect of taking into account contact elements, in the different interfaces, when a static load is applied in the system. In Figure 4.14 no contact elements have been considered in the soil-structure interface, as a consequence, the culvert is not isolated and shows the same vertical displacement field than the surrounding soil.
4.7 Mechanical behaviour of the contact elements used in this work.

If contact elements are considered, they exert great influence on the final the vertical displacement field of the culvert. In this case, the structure can be considered as an isolated body that experiment a vertical displacement field different than the rest of the surrounding soil. In Figure 4.15, contact elements of type 1 and 2 (see Figure 4.13) were used to model the different interfaces. The use of this type of contact elements is fundamental when it is desired to assess situations with differential settlements in the vicinity of the soil/structure interface.

**4.7 Mechanical behaviour of the contact elements used in this work.**

In order to validate the mechanical behaviour of the contact elements used in ANSYS, a comprehensive study of different solids in contact, has been developed and is presented in the current section.

The analysis was primary focus on the frictional performance of the contact interface between the solids.
As aforementioned, the use of contact elements in this work is very common, such as in wheel/rail contact and several interfaces modeling.

The use of contact elements in ballast/sleeper interface is important to model, in the subsequent sections, the hanging sleepers phenomenon. In the case of soil/structure interface as well as in ballast/ballast and sub-ballast/sub-ballast interfaces, when using contact elements it is important to account for differential settlements between two sections of the track at transition zone.

Finally, contact elements have also been used to model the wheel/rail interface but in this case, the purpose is to take into account the dynamic interaction between the train and the track. In this case, frictionless behaviour between the wheel and the rail was considered.

The use of this contact elements in the numerical model, involve to carry out a changing status non-linear analysis.

The main objective of this section was to provide some basic examples in which the mechanical behaviour of the solids is analyzed when friction is taking into account in the contact interfaces.

Since the numerical model used in this work is bidimensional, composed of bidimensional elements in a plane stress state, this basic examples have also been generated with 2D finite elements in a plane stress state.

### 4.7.1 Friction between bodies. Explanation of the mechanical phenomena

In cases in which contact between bodies exists, if tangential stress in the interface is non-zero, a frictional law to model the real mechanic behaviour of the interface, should be considered.
4.7 Mechanical behaviour of the contact elements used in this work.

Figure 4.14: Vertical displacements field when no contact elements are taken into account in the interfaces

Due to the inherent complexity of this phenomenon, a behavioral law should be used to consider the principal characteristics of the frictional interface behaviour.

Frictional force is defined as the reaction that exists between contact areas that slide relative to each other. This reaction tends to prevent the relative movement between solids in contact. Frictional force has a microscopic origin in the microscopic existing imperfections.

Leonardo da Vinci (XV century) and the french engineer Amontons (1699) were the first scientists that analyze friction.

Frictional coefficient is a feature of the contact surfaces of the solids and its value is independent of the size of the solids.

Frictional force is parallel to the contact surface of the solids and its direction is the opposite to the direction of the relative movement. The value of frictional force is in direct proportion to the normal force acting on the contact surfaces.

For a couple of contact forces, the friction coefficient is higher in a static state than when the surfaces experiment a relative movement. Therefore, friction force can be classified, depending on the dynamic state of the contact solids in:

- A static friction force, when solids do not have any relative displacement. This force can be considered as a resistance that has to be exceeded to reach a relative displacement between the solids.

- A dynamic friction force or kinetic force, when solids have a relative displacement once the initial static resistance has been exceeded, as it was above mentioned. This kinetic force is not dependent on the relative velocity of the solids in contact.
Figure 4.15: Vertical displacements field when contact elements, type 1 and 2 are entered in the interfaces

A diagram of these two forces is shown in Figure 4.16.

Figure 4.16: Diagram of static and dynamic states in a frictional problem

In this work, it is considered that that friction between solids in contact is static. The adopted mechanic expression to define mechanical behaviour of the solids in contact is Coulomb’s friction law.

At the beginning , if solids in contact do not have any relative displacement, it is assumed that tangential velocity is zero. This state is called sticking and in it, surfaces in contact are coincident.

Once the tangential force in the contact exceeds a certain level, surfaces are no longer stick to each other. The surfaces, in this state, experiment relative displacements.
4.7 Mechanical behaviour of the contact elements used in this work.

This relative tangential movement is called sliding and it is classically described by the Coulomb’s friction law, see Wriggers & Laursen (2006).

\[ t_T = -\mu |p_N| \frac{\dot{\gamma}_T}{\dot{\gamma}_T} \]  \hspace{1cm} (4.19)

If \( t > \mu |p_N| \), where \( \mu \) is the sliding friction coefficient, that is a constant coefficient in the classical Coulomb law; \( \dot{\gamma}_T \) is the relative sliding velocity between the contacting bodies and \( |p_N| \) is the contact normal pressure in the contact surface. In this case, for zero sliding velocity, the friction coefficient is equal to the static value \( \mu_S \).

Table 4.3 contains some reference values for the static friction coefficient in different interfaces.

Table 4.3: Friction coefficients for different interfaces, from Wriggers & Laursen (2006)

<table>
<thead>
<tr>
<th>Surfaces</th>
<th>Friction coefficient ( \mu_S )</th>
</tr>
</thead>
<tbody>
<tr>
<td>concrete-concrete</td>
<td>0.5 – 1</td>
</tr>
<tr>
<td>concrete-sand</td>
<td>0.35 – 0.6</td>
</tr>
<tr>
<td>concrete-steel</td>
<td>0.2 – 0.4</td>
</tr>
<tr>
<td>metal-wood</td>
<td>0.3 – 0.65</td>
</tr>
<tr>
<td>rubber-steel</td>
<td>0.15 – 0.65</td>
</tr>
<tr>
<td>steel-steel</td>
<td>0.2 – 0.8</td>
</tr>
<tr>
<td>steel-teflon</td>
<td>0.04 – 0.06</td>
</tr>
<tr>
<td>steel-concrete</td>
<td>0.2 – 0.4</td>
</tr>
<tr>
<td>steel-ice</td>
<td>0.015 – 0.03</td>
</tr>
<tr>
<td>wood-steel</td>
<td>0.5 – 1.2</td>
</tr>
<tr>
<td>wood-wood</td>
<td>0.4 – 1.0</td>
</tr>
</tbody>
</table>

In ANSYS it is possible to apply Coulomb’s law to simulate the friction resistance between bodies in contact.

A diagram of both states: Sticking and Sliding is shown for different values of normal force and tangential force is shown in Figure 4.17.

In the previous figure, \( p \) is the total normal force applied on the contact surface, \( \tau \) is the tangential force on the contact surface and \( b \) is a parameter that refers to the cohesion value. In this work, cohesion parameter is assumed to be zero.

The friction law implemented in ANSYS is given by equation 4.20.

\[ \tau_{lim} = \mu p + b \]  \hspace{1cm} (4.20)

For each node of the numerical model, exists a limit value \( \tau_{lim} \), that depends on the friction coefficient \( \mu \) and on the normal force on the contact surface.
4. Methodology in the dynamic interaction analysis

![Diagram of the Coulomb's friction law used in ANSYS](image)

Figure 4.17: Diagram of the Coulomb's friction law used in ANSYS

If the above mentioned tangential stress $\tau_{\text{lim}}$ is reached, contact status of the interface will change from sticking to sliding.

Four simple examples are presented in order to validate the general contact problem in ANSYS, when frictional behaviour in the contact interface is considered.

**Example 1. Validation of Coulomb's law of friction with two solids in contact**

With this example, a simple validation of the mechanical behaviour of the contact elements used in this work has been developed.

For this purpose two different solids in contact are created. These two solids have a different size, as can be seen in Figure 4.18:

![Layout of the solids in contact for Example 1](image)

Figure 4.18: Layout of the solids in contact for Example 1

The smaller solid A is on the top and it rests directly on the larger solid B. Solid B is supported at the base and lateral sides, as it is depicted in Figure 4.18.

Both solids have a common contact surface, $S_C$. 
4.7 Mechanical behaviour of the contact elements used in this work.

Mechanical characteristics for both solids as well as the contact formulation between them, are presented in Tables 4.4 and 4.5.

Table 4.4: Mechanical characteristics of the solids in contact. Example 1

<table>
<thead>
<tr>
<th>Solid</th>
<th>$E$ (MPa)</th>
<th>$\rho$ ($kg/m^3$)</th>
<th>$\nu$ (−)</th>
<th>Element type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid A</td>
<td>10E8</td>
<td>2200</td>
<td>0</td>
<td>PLANE 182</td>
</tr>
<tr>
<td>Solid B</td>
<td>30E14</td>
<td>2500</td>
<td>0</td>
<td></td>
</tr>
</tbody>
</table>

Table 4.5: Main features of the finite element software to analyze the contact problem. Example 1

<table>
<thead>
<tr>
<th>Contact formulation</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Finite Element Software</td>
<td>ANSYS</td>
</tr>
<tr>
<td>Contact type</td>
<td>Point-Line</td>
</tr>
<tr>
<td>Approach</td>
<td>Contact elements: CONTA175-TARGE169</td>
</tr>
<tr>
<td>Contact algorithm</td>
<td>Lagrange multipliers</td>
</tr>
<tr>
<td>Tangential behaviour</td>
<td>Coulomb’s law</td>
</tr>
</tbody>
</table>

It was decided to apply a vertical concentrated load of 100kN on the top of the solid A. In order to simplify this analysis, it has been decided to ignore transverse deformation due to Poisson effect.

Therefore in this example, Poisson coefficient was assumed to be zero. Example 2 shows the effect of accounting for transverse deformation in the analysis.

As it can be seen in Figure 4.18, contact surface is discretized in ten different elements, with a total number of eleven nodes.

Vertical load applied on the solid A is homogeneously distributed in the contact nodes. Consequently, each node receives a vertical force of 10kN, except for the extreme nodes that will receive half of the force resulting in the internal nodes: 5kN.

In Figure 4.19, it can be observed the distribution of the vertical load in the contact nodes belonging to the contact surface $S_C$.

Thus, in each node of the contact surface, there is a normal force that will be different in the internal nodes and in the nodes at the edges of the contact surface.
4. Methodology in the dynamic interaction analysis

Figure 4.19: Normal force distributed in the contact nodes

A change in the normal force between the internal and external nodes implies also a change to the sticking/sliding conditions given by Coulomb’s law.

In equation 4.20, \( \tau \) and \( p \) could be either stresses, if they are related to an area, or forces if they are related to a single point as a node in the contact surface.

Taking into account a friction coefficient \( \mu = 0.3 \), sticking/sliding condition for external nodes is: \( \tau_{lim} = \mu p = 0.3 \times 5 = 1.5 \text{kN} \).

Proceeding in the same manner for the internal nodes the sticking/sliding conditions are given by \( \tau_{lim} = \mu p = 0.3 \times 10 = 3 \text{kN} \).

Bearing in mind the previous limits, it is decided to apply an horizontal force \( H \) (in the tangential direction of the contact surface) in the external node \( N_E \), on the right, of solid A.

A detail of the contact surface, with the external node \( N_E \) and internal node \( N_I \), is shown in Figure 4.20.

Figure 4.20: Detail of the contact surface for Example 1

Sticking and Sliding are obtained when an horizontal force of 1.4 kN and 1.5 kN are applied in the external node \( N_E \), see Table 4.6.
4.7 Mechanical behaviour of the contact elements used in this work.

Table 4.6: External load applied on external node

<table>
<thead>
<tr>
<th>Normal force (kN)</th>
<th>μ</th>
<th>η_{lim}</th>
<th>Horizontal force H (kN)</th>
<th>Contact status</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>0.3</td>
<td>1.5</td>
<td>1.4</td>
<td>Sticking</td>
</tr>
<tr>
<td>5</td>
<td>0.3</td>
<td>1.5</td>
<td>1.5</td>
<td>Sliding</td>
</tr>
</tbody>
</table>

Graphic results, obtained in ANSYS, show the contact status in the contact interface, for the previous load case, see Figure 4.21.

![Contact Status](image)

(a) (b)

Figure 4.21: Contact status when an horizontal force $H$, is applied on the external node $N_E$: (a) $H = 1.4kN$ (sticking); (b) $H = 1.5kN$ (sliding)

When different values of the horizontal force are applied in the internal node $N_I$, sticking/sliding status changes, as shown in Table 4.7.

Table 4.7: External load applied on internal node

<table>
<thead>
<tr>
<th>Normal force (kN)</th>
<th>μ</th>
<th>η_{lim}</th>
<th>Horizontal force H (kN)</th>
<th>Contact status</th>
</tr>
</thead>
<tbody>
<tr>
<td>10</td>
<td>0.3</td>
<td>3</td>
<td>1.4</td>
<td>Sticking</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>1.5</td>
<td>Sticking</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.0</td>
<td>Sticking</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>2.5</td>
<td>Sticking</td>
</tr>
<tr>
<td></td>
<td></td>
<td></td>
<td>3.0</td>
<td>Sliding</td>
</tr>
</tbody>
</table>

Graphic results, obtained in ANSYS, show the contact status in the contact interface, for the previous load case in the internal node, see Figure 4.22.

With this simple example, mechanical behaviour of the contact elements in the interface between two different solids, has been validated.

Furthermore, it has been proved that sticking/sliding condition in ANSYS program, is applied and evaluated in each node of the model.
4. Methodology in the dynamic interaction analysis

Figure 4.22: Contact status when an horizontal force $H$, is applied on the internal node $N_I$: (a) $H = 0 - 2.5$ kN (sticking); (b) $H = 3$ kN (sliding)

In the next example, a similar validation is made but in this case the Poisson effect is taken into account, which introduces an extra complexity to the contact problem.

**Example 2. Validation of Coulomb’s law of friction with two elastic solids in contact** This example is a complement to Example 1, in which transverse deformation, due to Poisson effect, was ignored.

In this case, both solids can experiment transverse deformations according to Poisson elastic theory.

Geometrical and mechanical characteristics of the solids are the same (except for the Poisson coefficient) than those taken in Example 1. Poisson coefficient, in this case is considered $\nu = 0.3$, for both solids.

In this situation, the contact analysis and distinction between the sticking/sliding conditions is not so easy and intuitive as before. This is because the stress distribution along the contact interface, is more complex if solids are allowed to deform in the transverse direction.

In this example, two different load cases are analyzed. The objective is to show the behaviour of contact interface when different load levels act on the solids.

In order to carry out the analysis, two different types of forces are applied in the system. A vertical punctual load applied on the top of solid A, and horizontal forces applied on the nodes of the contact surface.

**Load case I: Vertical load $V=100$ kN, Horizontal load $H =0$ kN**

In this first load case, a vertical load of 100 kN is applied on solid A, as in Example 1.

A distribution of the vertical force in each node of the contact interface is depicted in Figure 4.23
4.7 Mechanical behaviour of the contact elements used in this work.

In this case, values of force distribution are lightly different of the distribution shown in Example 1.

In Example 1, the value of vertical force in the internal nodes, $N_I$ was 10 kN, while for a external node as $N_E$, the value of the vertical force was 5 kN.

In this case, due to the the effect of the transverse deformation phenomenon, when a vertical load is applied in solid A, it deforms in the transverse direction as well. As a consequence, a field of tangential stresses is created in the contact surface, as shown in Figure 4.24.

![Figure 4.23: Normal force distributed in the contact nodes. Load case I](image1)

![Figure 4.24: Tangential force distributed in the contact nodes. Load case I](image2)

In this particular load case, no horizontal force is applied in the contact nodes. Equilibrium condition is given by the above mentioned Coulomb’s law of friction. In a external node, $N_E$:

$$\tau_{\text{lim}} = 0.3 \times 6.835 = 2.050 \text{kN}.$$  

If tangential stress in the contact interface were higher than 2.05 kN, sliding status would be reached. This does not occur as it can be seen in the contact status depicted in Figure 4.25.
4. Methodology in the dynamic interaction analysis

In this case, the contact status is sticking and this means the existence of constrain conditions on every contact nodes of the solids.

This constraint generates horizontal reactions in the contact interface, that do not allow the smaller solid A to deforms freely. Consequently, the contact surface of solid A suffers a compression stress instead of a tension stress. The above mentioned value of compression, leads to a reduction of the contact interface and thus distribution of vertical forces in contact surface, is higher than in Example 1.

Normal forces were, in this case, shown in Figure 4.23:

**Load case II:** Vertical load \( V=100 \text{ kN} \) and Horizontal load \( H=0.5 \text{ kN} \)

In this case, an horizontal force is applied on every nodes of the contact surface. Value for this horizontal load is \( H = 0.5 \text{ kN} \). The fact of applying this horizontal force, make more complex the problem. In this situation is different to predict if surfaces in contact have reached relative displacements or stay in a sticking state.

Figure 4.26 shows, for this particular case, the value of normal distribution forces in the contact surface.

---

Figure 4.25: Contact status for load case I. Example 3

Figure 4.26: Normal force distributed in the contact nodes. Load case II
From the previous normal force distribution, Coulomb’s law gives the value for the tangential limit stress in the external node $N_E$.

$$\tau_{\text{lim}} = \mu p = 0.3 \times 6.739 = 2.021 \text{kN}.$$  

Figure 4.27, shows the value of tangential stress distribution in the contact surface for this particular load case.

![Figure 4.27: Tangential force distributed in the contact nodes. Load case II](image)

In this case, tangential force in $N_E$ is higher than $\tau_{\text{lim}} = 2.021 \text{kN}$. This implies that at this node, surfaces slide relative to each other.

Application of an horizontal force in the right direction, contributes to reach the sliding state in the external node $N_E$ on the right. On the other hand, application of the horizontal force, contributes to relax tangential force on the external node on the left. This is because at this node, the value of the applied horizontal force and the value of tangential stresses have a different sign.

Contact status for this particular case is shown in Figure 4.28:

![Figure 4.28: Contact status for load case II.](image)

In the next example, an assessment of the contact stresses field, using two different contact algorithms is made.
Example 3. Assessment and validation of the contact stresses using different contact algorithms: Penalty method and Lagrange multiplier method  In this work, two different algorithms have been used with the purpose of analyzing the contact problem. In order to have the guarantee of having similar results using either Penalty method or Lagrange multipliers method, it is decided to validate the stress field in the contact surface of two different contact solids.

In this example, a bidimensional numerical model consisting of two different contact solids A and B, plus a third solid C located above solid A and solid B, has been used. See Figure 4.29

![Figure 4.29: Layout of the three solids involved in Example 2](image)

The whole model is supported in the lateral sides as well as in the bottom as it can be observed in Figure 4.29.

Solids A and B represent a concrete structure and a wedge-shaped backfill respectively, these kind of elements are typical at transition zones. Solid C models the ballast layer, located above the solids A and B.

Mechanical characteristics of the solids in contact as well as the contact formulation between them, are presented in Tables 4.8 and 4.9.

<table>
<thead>
<tr>
<th>Solid</th>
<th>$E$ (MPa)</th>
<th>$\rho$ (kg/m$^3$)</th>
<th>$\nu$ (%)</th>
<th>Element type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Solid A</td>
<td>30E3</td>
<td>2500</td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td>Solid B</td>
<td>10E3</td>
<td>2200</td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td>Solid C</td>
<td>120</td>
<td>1530</td>
<td>0.20</td>
<td>PLANE 182</td>
</tr>
</tbody>
</table>

In this example, a vertical distributed load of 400 kN is applied on the solid C. Friction coefficient in the interface between solids A and B is equal to $\mu = 0.3$.

Results of the normal stresses field, in the contact interface, are depicted in Figure 4.30, for two different contact algorithms.

In Figure 4.30 (a) normal contact stresses in are obtained using the Penalty method while in Figure 4.30(b), Lagrange multipliers methodology has been used to obtain contact stresses.
4.7 Mechanical behaviour of the contact elements used in this work.

Table 4.9: Main features of the finite element software to analyze the contact problem. Example

<table>
<thead>
<tr>
<th>Contact formulation</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Finite Element Software</strong></td>
</tr>
<tr>
<td><strong>Contact type</strong></td>
</tr>
<tr>
<td><strong>Approach</strong></td>
</tr>
<tr>
<td><strong>Contact algorithm</strong></td>
</tr>
<tr>
<td><strong>Tangential behaviour</strong></td>
</tr>
</tbody>
</table>

For this particular load case, stress field obtained from Lagrange method are approximately 5% higher than using Penalty method. This is an indicator of the great similarity, in terms of stresses, that is reached when using Penalty or Lagrange multipliers methodologies.

In Figure 4.31, values of tangential stresses obtained from both the Penalty and Lagrange multipliers methods, are obtained.

In the case of tangential stresses, there is a difference around 6% higher using the Lagrange method. Once again there is a great similarity between the two algorithms, in terms of the tangential stress module.

From Figure 4.31, one can observe that the distribution of tangential stress in the contact surface is a little bit different using Penalty method or Lagrange methods. This difference is due to the particular ways of carry out contact analysis with each methodology.

In the Penalty method, as it was seen in Section 4.2.2, a small penetration between the contacting solids is allowed.

The fact of considering these small penetrations in Penalty method, modifies the value of stresses in the contact surface as well as its distribution if compares with Lagrange methodology.

A layout of the transition state, from open contact to closed contact is depicted in Figures 4.32, for each of the two methodologies used in this work.

With the purpose of using both methodologies and obtained the same results, it is
4. Methodology in the dynamic interaction analysis

Figure 4.30: Normal stress field in the contact interface for Example 3: (a) Penalty method; (b) Lagrange multipliers method

Figure 4.31: Tangential stress field in the contact interface for example 2: (a) Penalty method; (b) Lagrange multipliers method

decided to impose, in the Penalty methodology, a zero penetration between the two solids in contact, allowing the relative displacement to each other in the tangential direction.

This modification is done by changing the «keyoptions» of the contact elements available in ANSYS.

Values of normal contact stresses and tangential stresses, in the contact interface, are shown in Figure 4.33. In this case, limitation of zero penetration between solids in contact is taken into account in Penalty method.

In this case, differences between Penalty and Lagrange multipliers algorithms are very small, not only in the module of the normal and tangential stress field but also on the distribution along the contact interface.

Example 4. Numerical validation using two different Finite Elements programs: ANSYS & ABAQUS   Due to the importance of contact elements in the frame of
4.7 Mechanical behaviour of the contact elements used in this work.

Figure 4.32: Transition from closed to open state: (a) Lagrange multipliers; (b) Penalty method

Figure 4.33: Stress distribution for Penalty method when penetration is restricted: (a) Normal stress distribution; (b) Tangential stress distribution

In the present work, it is decided to validate numerical results obtained in ANSYS with results obtained in a different finite element program: ABAQUS.

A simple bidimensional structural system composed of two different rectangular solids in contact has been considered.

Support conditions of the system consist on constraining vertical displacements at the lower side, as well as horizontal displacements at the right side of the system.

A layout of the structural system is shown in Figure 4.34.

In ANSYS, finite elements of four nodes are used: the particular element of ANSYS, PLANE 182 has been used to define them.

In ABAQUS, bi-linear finite elements solids of four nodes were used. Particular denomination of this elements in ABAQUS is CPS4.

In both softwares, a plane stress condition is used to solve the problem.

Meshing of the solids were the same for ANSYS and ABAQUS, using square elements with a size of 10 cm. The total number of nodes in this example is 242.
In order to simulate the contact surface between the two solids, available algorithms in both ANSYS and ABAQUS have been used.

A description of contact formulations, is shown in Table 4.10.

Table 4.10: Main features of the finite elements softwares to analyze the contact problem

<table>
<thead>
<tr>
<th>Contact formulations</th>
<th>ANSYS</th>
<th>ABAQUS</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Finite element software</strong></td>
<td>Point-Line</td>
<td>Edge-to-edge contact</td>
</tr>
<tr>
<td><strong>Contact type</strong></td>
<td>Contact elements: CONTA175 -TARGE169</td>
<td>Contact surface pairs: master-slave algorithm</td>
</tr>
<tr>
<td><strong>Approach</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td><strong>Contact algorithm</strong></td>
<td>Lagrange multipliers</td>
<td>Lagrange multipliers (direct method)</td>
</tr>
<tr>
<td><strong>Contact pressure interpenetration ratio</strong></td>
<td>zero</td>
<td>'Hard contact'</td>
</tr>
<tr>
<td><strong>Tangential behaviour</strong></td>
<td>Coulomb’s law</td>
<td>Coulomb’s law</td>
</tr>
</tbody>
</table>

To carry out this analysis, two different forces have been applied on the solids. The
first one was a vertical force applied on the top side of the system. The component of this force is normal to the contact surface.

The second one was an horizontal force applied on the left side of the element located on the top, as seen in Figure 4.34. The component of this force is tangential to the contact surface.

In order to have a better understanding of the frictional problem, this force has been applied in a total number of eleven load steps.

Mechanical characteristics of the solids in contact, and main features of the applied loads, are depicted in Table 4.11

<table>
<thead>
<tr>
<th>2D solids in contact</th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Mechanical characteristics</td>
<td>$E \left( \frac{N}{m^2} \right)$</td>
<td>2.1E11</td>
</tr>
<tr>
<td></td>
<td>$\nu (-)$</td>
<td>0.3</td>
</tr>
<tr>
<td></td>
<td>$\rho \left( \frac{kg}{m} \right)$</td>
<td>7850</td>
</tr>
<tr>
<td></td>
<td>$\mu (-)$</td>
<td>0.3</td>
</tr>
<tr>
<td>Applied load</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vertical load ($kN$)</td>
<td>1000</td>
<td></td>
</tr>
<tr>
<td>Horizontal load ($kN$)</td>
<td>900</td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.35 - Figure 4.39, the results given by ANSYS and ABAQUS are presented. With the intention of facilitating the comparison between both softwares, each of the next figures contain results from ANSYS on the left and results from ABAQUS on the right.
4. Methodology in the dynamic interaction analysis

Figure 4.36: Vertical stresses comparison

Figure 4.37: Horizontal displacements comparison
Finally, in Figure 4.40 a tangential stress-load step diagram, obtained in ANSYS, is shown for two different friction coefficients: $\mu = 0.3$ and $\mu = 0.4$. In these diagrams both states: sliding and sticking can be perfectly identified. To carry out this analysis, the horizontal load has been applied in a total number of 10 sub steps in order to capture the exact moment in which the contact state changes from sticking to sliding.

With this simple example, it was proved the great accuracy of the response obtained in the structural system depicted in Figure 4.34, using two different finite elements codes: ANSYS and ABAQUS.
4. Methodology in the dynamic interaction analysis

![Graph showing tangential stress vs load step for different friction coefficients]

Figure 4.40: Horizontal load step-tangential stress diagram for different friction coefficients

4.8 Track Irregularities

4.8.1 Introduction

Track irregularities are an important variable to take into account in this study. Its importance is based on the influence of track irregularities in the dynamic component of the interaction forces between the train and the track. This variation of the dynamic component is associated to an increment of interaction forces, more accentuated with higher values of speed. The increments of interaction forces may lead to an increment of the acceleration in the track degradation process as duly concluded by Ribeiro (2012). Furthermore, the fact of taking into account track irregularities affects the behaviour of the train, increasing acceleration on the axles as it moves along the transition zone.

Summarizing, it can be concluded that the presence of irregularities in the track, causes vibration and noise. Longitudinal level irregularities may also affect the dynamic response as well and accelerate track degradation in transition zones, which in turn increases the maintenance costs. According to Garg (1984) track irregularities can be classified into four different groups:

- Vertical surface profile: Average elevation of the two rails
- Alignment irregularity: Average of the lateral positions of two rails
- Gage irregularity: Horizontal distance between two rails
- Cross level irregularity: Difference between the elevations of the two rails

Attending to the shape of irregularities along the track, these can be divided into two main groups: isolated irregularities and distributed irregularities.

Isolated irregularities occur mainly due to differences in the vertical alignment in transition zones, rail corrosion, rail gaskets...
On the other hand, distributed irregularities are caused mainly due to geometrical deterioration of different components of the track.

The effect of irregularities has also been studied in other fields as road bridges, such as the works developed by Calçada (2001) in which the author analyzed the effect of pavement irregularities in a road bridge, coming to the conclusion that the level of pavement irregularities has a great influence in the dynamic effects induced by the traffic.

Steenbergen (2006) showed that application of certain simplified contact models may underestimate the contact forces in cases in which track irregularities are taken into account.

Vale (2010) evaluated the effect of track irregularities in the dynamic interaction response between the vehicle and the track, coming to the conclusion that dynamic amplification factor of the dynamic load increases significantly with moving speeds, in cases in which irregularities of the track are considered.

Nguyen & Galbadón (2012) generated several vertical track irregularities, concluding that dynamic effects of high-speed traffic loads on the ballast tracks are sensitive both to the track irregularities and the vehicle speed.

Youcef et al. (2013) analyzed the effect of parameters such as depth and position of a imperfection of the track. Results obtained show that the rail irregularities affect the vertical acceleration of the train which conflicts with the passenger’s comfort limit state criterion.

Rocha et al. (2014) evaluated the existence of track irregularities. These have been randomly generated using power spectral density functions. In this regard, an important conclusion that can be drawn is that the use of track irregularities, in the numerical model developed, leads to an important increase of the dynamic amplification of the wheel-rail contact forces.

To take into account the effect of longitudinal level track irregularities, a modification of the regular rail profile geometry, in the numerical models, is introduced.

In the next section, a brief description of isolated and distributed irregularities is made, introducing the main concepts, related with each type of irregularity, that will be used in subsequent sections. This concepts will be useful to generate profiles of distributed irregularities that lead us to analyze dynamic interaction behaviour and degradation process of the track at transition zones.

### 4.8.2 Isolated irregularities

Isolated irregularities are a very common type of track imperfections and may arise from various and different sources like: variation of foundation stiffness, the presence of unsupported sleepers...This type of irregularity occurs in different sections of the
track as: rail joints, railway embankments, rail switches and bridges approaches. This phenomenon is well referred in Fryba (1996).

In the same work, the author described seven different forms of isolated irregularities. They are expressed analytically by different equations based on experimental parameters. These parameters are based on the results of measurements carried out in the USA, for different track categories and they define the quality of the track.

Application and effects of this particular type of irregularities has not been studied in the present work as it would probably widen too much the scope of this thesis.

### 4.8.3 Distributed irregularities

Distributed irregularities along the track can be of two different types: random irregularities or periodically irregularities: These imperfections are defined by two factors: Amplitude and wavelength.

Random irregularities are given by the sum of a certain number of periodic defects with different values of amplitude and wavelength. Because of this, when a wheel of the train moves along a track with this type of imperfections, the interaction force that is generated, contains a wide range of frequencies, Vale (2010).

Distributed irregularities are defined geometrically by the loss of alignment of the rails in both, vertical and horizontal directions.

The causes of this type of irregularity are: degradation of the track platform, excessive train speeds and weight, existence of differential settlements of the subgrade, the effect of longitudinal forces due to the train’s variation in velocity when braking or accelerating. See Figure 4.41. This type of irregularities, particularly longitudinal level irregularity profiles, have been considered in Chapters 5 and 6 in order to evaluate the effect of track irregularities in the dynamic interaction analysis between train and track.

![Distributed irregularities examples](image)

**Figure 4.41:** Distributed irregularities examples: (a) track with wooden sleepers; (b) track with concrete sleepers
4.8.4 Modeling of artificial track irregularities profiles

With the purpose of assessing train-track interactions as well as predict track degradation phenomenon, in the subsequent sections, an analytical description of the track geometry is required. From a general point of view it is almost impossible to define track geometry through an analytical expression. This impossibility leads to the use of a statistical representation.

Several inspection campaigns, carried out by some railway Administrators have shown that track irregularities can be properly simulated by a stationary stochastic process. Stationary and stochastic processes may be described by spectral density functions. In this case a power spectral density function $G(\omega)$ is used to define the track irregularities. Function $G(\omega)$ depends on the cyclic spatial frequency of the irregularity, $\gamma$, given by equation 4.21:

$$\gamma = \frac{2}{r}$$

where $r$ represents the wavelength of the irregularity. Depending on different railway Administrators, function $G(\omega)$ may be different. In the following, some proposals, given by the SNCF, DBAG and FRA railway Administrators are presented.

**SNCF (Société Nationale des Chemins de Fer)**

SNCF is the French Railway Administrator. It proposes a power spectral density function that is given by equation 4.22:

$$G(\omega) = \frac{10^{-6}A}{1 + \frac{\omega}{\Omega_r}}$$

where: $A$ is a parameter related to the quality of the track. It varies from 160 to 550 according to a good or bad quality of the track, respectively and $\omega = 0.307 \text{ m}^{-1}$.

It should be noted that last equation is valid only for wavelengths of the irregularities from 2 m to 40 m.
4. Methodology in the dynamic interaction analysis

Figure 4.42: Generated profile according to SNCF: (a) longitudinal profile; (b) PSD validation

**FRA (Federal Railroad Administration)**

FRA is the responsible Railway Administrator of the USA railway lines. It developed several measurements to obtain information about the observed irregularities on the track. Power spectral density function recommended in this case is based on the the statistical treatment of the above mentioned observed results.

The FRA suggests the use of different expressions for the power spectral density function, depending on the type of irregularity. For elevation irregularities, it provides the power spectral density function given by equation 4.23:

\[
G(\Omega) = \frac{A_{1} \Omega_{2}^{2} (\Omega_{1}^{2} + \Omega_{2}^{2})^{\frac{3}{2}}}{(\Omega_{1}^{2} + \Omega_{2}^{2})^{\frac{5}{2}}} \quad \text{(4.23)}
\]

In a similar way, for both the cross level and gauge irregularities, equation 4.24 must be used to define power spectral density:

\[
G(\Omega) = \frac{A_{2} \Omega_{2}^{2}}{(\Omega_{1}^{2} + \Omega_{2}^{2})^{\frac{5}{2}}} \quad \text{(4.24)}
\]

Values for \( A \) parameter depend on the quality of the track. They vary from \( 1.5E-6 \) rad.m to \( 2.39E-5 \) rad.m, \( \Omega_{1} \) and \( \Omega_{2} \) parameters take values of 0.0206 rad/m and 0.8246 rad/m respectively, Fryba (1996).
4.8 Track Irregularities

Figure 4.43: Generated profile according to FRA: (a) longitudinal profile; (b) PSD validation

DBAG (Deutsche Bahn AG)

DBAG is the German railway company. It developed a great number of experimental measurements to capture existent irregularities in the German railway lines. Its definition of the power spectral density function is made depending on the type of irregularity considered.

Alignment irregularities are defined by the power spectral density function given by equation 4.25.

\[ G(\Omega) = \frac{A}{\Omega^2(\Omega^2 + \frac{2}{r})(\Omega^2 + \frac{2}{c})} \]  

(4.25)

Where \( A \) is a scale factor for irregularities. It varies from \( 0.59233e - 6 \text{rad.m} \) to \( 1.58610e - 6 \text{rad.m} \) and, \( r \) and \( c \) parameters take values of \( 0.0206 \text{rad/m} \) and \( 0.8246 \text{rad/m} \) respectively Claus & Schiehlen (1998).

Cross level irregularities are defined through a different power spectral density function, in this case given by equation 4.26.

\[ G(\Omega) = \frac{A}{\Omega l^2(\Omega^2 + \frac{2}{r})(\Omega^2 + \frac{2}{c})(\Omega^2 + \frac{2}{s})} \]  

(4.26)

where \( l \) is the half of the track width and \( s \) is a constant parameter that take the value of \( 0.438 \text{rad/m} \) Claus & Schiehlen (1998).

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4. Methodology in the dynamic interaction analysis

![Graphs showing generated profile and PSD validation](image)

Figure 4.44: Generated profile according to DBAG: (a) longitudinal level profile; (b) PSD validation

### 4.8.5 Generation of distributed irregularities profile

As described in Claus & Schiehlen (1998), the numerical process to generate distributed irregularities is given by equation 4.27.

\[
r(x) = \frac{2}{\pi} \sum_{i=1}^{N} A_i \cos(\Omega_i x - \theta_i)
\]  

(4.27)

To obtain the previous expression, power spectral density is taken into account. It is assumed the the irregularity profile can be defined through a sum of several sinusoidal functions.

In equation 4.27, \(N\) is the number of harmonic functions that are considered, \(\Omega_i\) is the frequency of the harmonic function. This frequency is defined in the range between \(\Omega_0\) and \(\Omega_f\). In this work it is assumed that \(\Omega_0 = \frac{2\pi}{25}\) rad/m and \(\Omega_f = \frac{2\pi}{3}\) rad/m. \(A_i\) correspond to the amplitude of the harmonic function and \(\theta_i\) is the phase angle that is uniformly distributed in the range 0 to \(\pi\).

Function \(r(x)\) satisfies the main conditions for an stochastic, ergodic and stationary process.

The amplitude factor \(A_i\) can be determined using the power spectral density function, \(G(\cdot, \cdot)\), following the equation 4.28.

\[
A_i = \frac{1}{2} \frac{G(\cdot, \cdot) \Delta \Omega_i}{\Delta_i}
\]  

(4.28)

where \(\Delta \Omega_i\) corresponds to the frequency increment considered.
Validation of the generated artificial profiles, for each railway Administrator, can be seen in Figures 4.42, 4.43 and 4.44. As it can be appreciated in this figures, the PSD of the generated artificial profiles is very similar to the PSD, given by the analytical expression in each railway Administrator, for the frequencies range under consideration in this work.

### 4.8.6 Experimental irregularities

Irregularities coming from measurements of the railway track are other data source that will be considered in this work. These irregularities are measured through monitoring operations, carried out by special vehicles named track recording coaches TRC.

The maximum sampling distance has been stipulated as 0.5 m but most TRC use a shorter sampling distance.

Track irregularities are estimated from the vertical and lateral acceleration of the car body.

In this work, irregularities profiles coming from two different railway Administrators such as REFER (Portugal) and ADIF (Spain), have been used. Because of this, a brief description of track monitoring for both the Portuguese and Spanish cases is here made.

#### REFER (Portugal)

Monitoring data of Portuguese line correspond to the north line called “Linha do Norte”. Monitoring operations are carried out by the Portuguese railway Administrator REFER. Track recording vehicle used is EM 120, see Figure 4.45.

Average periodicity inspection of the north line is 90 days. It is important to remark that this recording vehicle is available only for Iberian gauge tracks (1688 mm) and not for high-speed standard gauge (1435 mm).

Maximum speed for this recording vehicle is 120 km/h. In the next it is presented a list of the main devices, contained in this vehicle, used for monitoring track irregularities and other particular features:

- Laser system to check the position of the cross section of both, track and catenary.
- Optical scanner place on a inertial platform to record geometrical track irregularities.
- Optical system (KLD) to check the wear on head of the rails.
- GPS system to locate geographically the obtained records.
- Video cameras in front and back of the vehicle, to film the track surface.
It should also be noted that monitoring, in this case, has been made with a spatial discretization of 0.25 m.

**ADIF (Spain)**

In Spain, geometrical inspection of the track is carried out each 6 months for both regular and high-speed lines. Monitoring is performed by “Dirección Ejecutiva de Mantenimiento de Infraestructuras de ADIF”.

Track recording vehicle used in this case is SIV-1002, see Figure 4.46.

SIV-1002 coach belongs to the 8000 series of RENFE. It can be adopted to record both: Iberian gauge (1688 mm) and high-speed standard gauge (1435 mm) tracks. The length of this coach is 26.4 m and the total weight is 53 t. It is equipped with different individual transducers and with gyroscopes accelerometers, optical systems, laser systems, etc...

Signals coming from the transducers are analyzed through the TRANS-HEADLINE system, developed by the collaboration of AEA technology and RENFE. See Villalmanzo (2008). With these systems, several parameters of the track can be measured as: rail geometry, track geometry (elevation, alignments, gauge and cross level), track tracing...

Monitoring of the track, in this case, has been carried out with a spatial discretization of 0.125 m.

### 4.8.7 Experimental irregularities adjustment

In order to adapt the longitudinal level rail profiles, coming from monitoring, to the rail mesh of the bidimensional numerical model, an adjustment of the above mentioned profiles is necessary. The objective is to adapt experimental values so that they can be used as a geometrical input to define the new geometry of the rail, in the numerical model.
4.8 Track Irregularities

As it was commented in previous sections, spatial discretization for the REFER and ADIF are 0.25 m and 0.125 m respectively. On the other hand, spatial discretization of the rail in the numerical model was taken as 0.025 m. Thus it is necessary to apply an artificial interpolation to the monitoring rail profiles to achieved a spatial discretization (for REFER and ADIF profiles) of 0.025 m, a suitable distance to work with in the numerical model.

For this reason, it has been decided to use a cubic data interpolation given by the Spline function, available in the MATLAB software.

An example of a cubic spline passing through $n$ data points is illustrated in Figure 4.47

![Cubic spline interpolation](image)

Figure 4.47: Cubic spline interpolation, adapted from Wolberg (1988)

Applying the cubic spline interpolation, to the monitoring profiles provided by REFER and ADIF, next profiles are obtained for a spatial discretization $ds = 0.025$ m. See Figure 4.48
4. Methodology in the dynamic interaction analysis

4.8.8 Assessment of the longitudinal level profiles quality

The goal of this Section is to evaluate the quality of the longitudinal level rail profiles coming from both, the statistical analysis of the above mentioned railway Administrators: SNCF, FRA, and DBAG, and those coming from the track monitoring operations carried out by REFER and ADIF.

In this case, two control parameters have been considered:

- Standard deviation of the longitudinal level EN13848-5 (2010).
- Maximum value of peak, relative to the average value of the longitudinal level, TSI (2008).

The range of train speeds that is taken into account is $v = 200 - 300$ km/h.

According to EN13848-5 (2010) and TSI (2008), the range of speeds considered, provides the values of the control parameters given in Table 4.12.

Table 4.12: Control parameters to assess the quality of the rail profiles

<table>
<thead>
<tr>
<th>Speed (km/h)</th>
<th>Standard deviation (mm)</th>
<th>Maximum peak value (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>200-300</td>
<td>1-1.5</td>
<td>good quality</td>
</tr>
<tr>
<td></td>
<td></td>
<td>4</td>
</tr>
</tbody>
</table>

In the case of artificial profiles, ten different longitudinal level profiles will be generated for each Railway Administrator. Since in every railway Administrator it is possible to generate both good and bad quality profiles, depending on the scale parameter $A$, five profiles will be generated according to a good quality condition of the track and the other five, generated according to a bad quality condition of the track.
4.8 Track Irregularities

On the other hand, in the case of the real monitoring profiles, two profiles will be assessed for each Railway Administrator: REFER and ADIF.

In every rail profile, artificial and experimental, both the standard deviation and maximum peak value will be obtained. The idea is to compare these profiles with the values given in Table 4.12.

Table 4.13: Standard deviation and maximum peak value for artificial DBAG profiles

<table>
<thead>
<tr>
<th>profile</th>
<th>Good quality</th>
<th>Bad quality</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$p$</td>
<td>$p$</td>
</tr>
<tr>
<td>1</td>
<td>0.491</td>
<td>1.505</td>
</tr>
<tr>
<td>2</td>
<td>0.392</td>
<td>1.123</td>
</tr>
<tr>
<td>3</td>
<td>0.353</td>
<td>0.952</td>
</tr>
<tr>
<td>4</td>
<td>0.382</td>
<td>1.310</td>
</tr>
<tr>
<td>5</td>
<td>0.461</td>
<td>1.032</td>
</tr>
<tr>
<td>mean value</td>
<td>0.415</td>
<td>1.184</td>
</tr>
</tbody>
</table>

Table 4.14: Standard deviation and maximum peak value for artificial FRA profiles

<table>
<thead>
<tr>
<th>profile</th>
<th>Good quality</th>
<th>Bad quality</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$p$</td>
<td>$p$</td>
</tr>
<tr>
<td>1</td>
<td>0.772</td>
<td>2.105</td>
</tr>
<tr>
<td>2</td>
<td>0.901</td>
<td>2.532</td>
</tr>
<tr>
<td>3</td>
<td>0.781</td>
<td>2.201</td>
</tr>
<tr>
<td>4</td>
<td>0.623</td>
<td>1.774</td>
</tr>
<tr>
<td>5</td>
<td>0.562</td>
<td>1.533</td>
</tr>
<tr>
<td>mean value</td>
<td>0.727</td>
<td>2.029</td>
</tr>
</tbody>
</table>

1According to TSI (2008), standard deviation for every rail profile is analyzed over a rail length of 200m
Methodology in the dynamic interaction analysis

<table>
<thead>
<tr>
<th>profile</th>
<th>Good quality</th>
<th>Bad quality</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>( \sigma )</td>
<td>( p )</td>
</tr>
<tr>
<td>1</td>
<td>1.053</td>
<td>2.701</td>
</tr>
<tr>
<td>2</td>
<td>1.253</td>
<td>3.122</td>
</tr>
<tr>
<td>3</td>
<td>1.100</td>
<td>3.219</td>
</tr>
<tr>
<td>4</td>
<td>0.938</td>
<td>2.787</td>
</tr>
<tr>
<td>5</td>
<td>0.803</td>
<td>2.320</td>
</tr>
<tr>
<td>mean value</td>
<td>1.029</td>
<td>2.829</td>
</tr>
</tbody>
</table>

In Tables 4.13, 4.14 and 4.15 and \( p \) are the standard deviation and the maximum peak value respectively. Analyzing the previous results it can be concluded that rail profiles generated according to DBAG presents the lowest values for both the standard deviation as well as the maximum peak value relative to the average value of the longitudinal level. It is proved also that for every Railway Administrator, generated profiles, according to a good quality of the track, are lower than those belonging to profiles generated with a bad quality of the track.

Note that for both the SNCF and FRA cases and for rail profiles generated with a bad quality of the track, the control parameters (in bold letter) does not comply with the limits given in Table 4.12.

<table>
<thead>
<tr>
<th>profile</th>
<th>( \sigma )</th>
<th>( p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.399</td>
<td>0.981</td>
</tr>
<tr>
<td>2</td>
<td>0.470</td>
<td>1.16</td>
</tr>
<tr>
<td>mean value</td>
<td>0.434</td>
<td>1.07</td>
</tr>
</tbody>
</table>

Table 4.17: Standard deviation and maximum peak value for monitoring ADIF profiles

<table>
<thead>
<tr>
<th>profile</th>
<th>( p )</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.661</td>
</tr>
<tr>
<td>2</td>
<td>0.737</td>
</tr>
<tr>
<td>mean value</td>
<td>0.699</td>
</tr>
</tbody>
</table>

Analyzing the previous results it can be concluded that theoretical profiles generated according to requirements of DBAG, are those that have a better quality of the track, in accordance with EN13848-5 (2010) and TSI (2008) specifications. Because
of this, artificial profiles generated following DBAG requirements, have been used to carry out some of the dynamic interaction analysis in the subsequent sections of the current work. On the other hand, as it can be observed in Tables 4.16 and 4.17, values for both the standard deviation and peak value, are in accordance with the limits proposed by EN13848-5 (2010) and TSI (2008). This demonstrates the good quality of the analyzed profiles coming from both REFER and ADIF. As in the case of artificial profiles from DBAG, experimental profiles will be used also in the subsequent sections, with the purpose of proving the importance of consider the longitudinal level irregularity profiles in the dynamic interaction analysis carried out at transition zones. Every longitudinal level irregularities used herein: artificial and experimental profiles are below the Alert Limit level described in EN13848-5 (2010) and TSI (2008).

### 4.8.9 Validation of the dynamic interaction

A particularly important aspect within the dynamic analysis of structures is that referred to the multi-body contact analysis. In the particular case of dynamic interaction forces analysis between track and vehicle, special importance is given to the wheel-rail contact analysis.

Due to the need of using contact elements to properly analyze the mentioned dynamic problem, a sensibility evaluation has been made, to validate the use of contact elements developed in the ANSYS commercial finite element software.

For the validation, the well-known beam proposed by Yang & Yau (1997), is taken, as illustrated in Figure 4.49.

![Figure 4.49: Beam with moving sprung mass: (a) without irregularities; (b) with irregularities, adapted from Yang & Yau (1997)](image)

The structure represents a simple beam of span length \( L = 25 \text{ m} \), subjected to a moving sprung mass. The properties of both the mass and the beam are listed in the following Table 4.18.
4. Methodology in the dynamic interaction analysis

Table 4.18: Characteristics of the beam and the sprung mass presented by Yang & Yau (1997)

<table>
<thead>
<tr>
<th>Mass</th>
<th>M (kg)</th>
<th>5750</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>K (N/m)</td>
<td>1595E3</td>
</tr>
<tr>
<td>Beam</td>
<td>E (N/m²)</td>
<td>2.87E9</td>
</tr>
<tr>
<td></td>
<td>ν (−)</td>
<td>0.20</td>
</tr>
<tr>
<td></td>
<td>I (m⁴)</td>
<td>2.90</td>
</tr>
<tr>
<td></td>
<td>M_b (kg/m)</td>
<td>2303</td>
</tr>
</tbody>
</table>

With the purpose of comparing and validating the dynamic response of the system, two different tools are used. The first one is the well-known ANSYS commercial FE software. The other is a software developed by Faculty of Engineering of the University of Porto, FEUP, called FEMIX.

FEMIX code was used in the current version 4.0 to perform the validation developed in this section. It is a finite element computational code developed in FEUP by means of an integrated formulation. This formulation includes equilibrium and compatibility equations, with unknowns that consist on displacements and interaction forces.

Both programs ANSYS and FEMIX incorporate algorithms that consider contact between bodies. Further information of the contact methodology implemented in FEMIX can be find in Neves et al. (2012) and Azevedo (2003).

Results will be compared and analyzed for two different cases of study: Case ‘a’ perfect geometry of the rail and case ‘b’ rail with irregularities. In the latter, a modification of the mentioned beam is introduced, bearing in mind a random profile of irregularities proposed by the German regulations (DBAG):

In Figure 4.50, the real profile of irregularities, used in this case, is depicted:

![Figure 4.50: Irregularity profile, from DBAG (good quality) adapted to the beam proposed by Yang & Yau (1997)](image_url)

In both cases, the following outputs will be analyzed.
4.9 Settlements of the embankment

- Vertical displacement of the mass.
- Acceleration of the mass.
- Vertical displacement of the mid-span of the beam.
- Contact force.

Contact behaviour in ANSYS is given by the Lagrange multipliers method, already explained in previous sections.

The present study is evaluated for a moving speed of the mass of \( v = 27.8 \text{ m/s} \).

Results depicted in Figures 4.51 - 4.54, show the values of: mass acceleration, mass vertical displacement, mid-span vertical displacement and contact forces respectively. In each Figure, values on the left (a) are obtained when no irregularity is considered. On the other hand, values on the right (b), are obtained when the generated artificially irregularity depicted in Figure 4.50, is considered.

Analyzing the previous results, it can be perfectly seen that the fact of considering irregularities in the beam, affects much more the behaviour of the sprung mass system than the behaviour of the beam. The reason for this is the fact that these irregularities do not significantly influence the stiffness of the beam and consequently the response.

On the other hand, stiffness and properties of the sprung mass make it more sensitive to the presence of track irregularities.

4.9 Settlements of the embankment

4.9.1 Introduction

In this section methodology used to obtain settlements of embankments for HSL at transition zones, is explained.

These settlements will be entered in the 2D numerical model, analyzing their influence on the dynamic interaction analysis between the train and the track. At this point it is important to note that to carry out such an analysis, the use of contact elements in the different interfaces, in the vicinity of the structure, is fundamental to simulate the effect of differential settlements, as it was explained in Section 4.5.

In this section, only settlements of the embankments are analyzed. It is considered that subgrade soil is of a good quality, very rigid, and consequently does not experiment any settlement.

A detailed description of the methodology that has been applied to obtained the settlements of the embankments is given below. In it, also a description of the model and the main parameters used to obtain the above settlements is done. Results of settlements are obtained depending on two main parameters: the height of the embankment \( H \) and the deformability modulus of the material that makes up the
4. Methodology in the dynamic interaction analysis

Figure 4.51: Sprung mass acceleration: (a) without irregularities; (b) with irregularities

Figure 4.52: Mass displacement: (a) without irregularities; (b) with irregularities
4.9 Settlements of the embankment

Figure 4.53: Mid-span displacements: (a) without irregularities; (b) with irregularities

Figure 4.54: Contact force: (a) without irregularities; (b) with irregularities
embankment $E$. A more detailed information of the above mentioned methodology can be found in De la Fuente & Bermejo (2009).

### 4.9.2 Settlements of embankments in HSL lines

Available data of settlements in embankments of HSL are very limited and, in some cases deficient, regarding to to material characteristics.

In Melis (2006), there is a detailed information of the post-constructive settlements in embankments for both the Madrid-Seville and Madrid-Barcelona HSL. See Tables 4.19 and 4.20.

Table 4.19: Post-constructive settlement in 10 years. Madrid-Seville AVE railway line, from Melis (2006)

<table>
<thead>
<tr>
<th>Height $H$ (m)</th>
<th>number of embankments</th>
<th>settlement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>50</td>
<td>2</td>
<td>11-46</td>
</tr>
<tr>
<td>40-41</td>
<td>2</td>
<td>35-252</td>
</tr>
<tr>
<td>30-33</td>
<td>6</td>
<td>25-450</td>
</tr>
<tr>
<td>20-22</td>
<td>3</td>
<td>100-285</td>
</tr>
<tr>
<td>15-18</td>
<td>5</td>
<td>19-149</td>
</tr>
<tr>
<td>8-10</td>
<td>6</td>
<td>18-152</td>
</tr>
</tbody>
</table>

Table 4.20: Post-constructive settlement in 10 years. Madrid-Barcelona AVE railway line, from Melis (2006)

<table>
<thead>
<tr>
<th>Height $H$ (m)</th>
<th>number of embankments</th>
<th>settlement (mm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>40-45</td>
<td>2</td>
<td>77-89</td>
</tr>
<tr>
<td>30-36</td>
<td>6</td>
<td>84-490</td>
</tr>
<tr>
<td>20-25</td>
<td>8</td>
<td>71-187</td>
</tr>
<tr>
<td>15-19</td>
<td>9</td>
<td>60-482</td>
</tr>
<tr>
<td>10-14</td>
<td>7</td>
<td>72-319</td>
</tr>
<tr>
<td>4-8</td>
<td>3</td>
<td>30-257</td>
</tr>
</tbody>
</table>

Values of Tables 4.19 and 4.20 show creep settlements that are in the range of $0.1 - 1.5\%$ of the height of the embankment.

Authors as Escario (1980) and Justo et al. (1988) show in their works that embankments with a good compaction level, have creep settlements in the range of $0.1 - 0.3\%$ of the height of the embankment.

Although there is a good knowledge about the value of settlements in embankments, knowledge of geotechnical properties of the used materials is still poor, as well as the information available about compacting conditions.
Regarding to the settlement limits in embankments, some railway agencies of different countries recommend different limitation values:

- Railway Technical Research Institute (Japan, 2001) limits the settlement of the embankment to 100 mm for a ballasted track and 30 mm for slab track.
- In Germany DB, maximum settlement is 60 mm for embankments in slab track. Other limitation given by DB does not allow differential settlements higher than 20 mm in lengths lower than 10 m.
- In Sweden, maximum settlement permitted is 100 mm. In this case, no distinctions are made between ballasted track and slab track.

### 4.9.3 Parameters of the study and main results

In this numerical analysis, it is necessary to define the temporal behaviour of the embankment deformability. To this end, a creep coefficient $C_\alpha$ is defined. This coefficient related with the compression index $C_c$, Vermeer & Neher (1999), Mesri & Castro (1987). Other relationships between various creep parameters can be found in Medina & Melis (2003), Olsson (2010) and PLAXIS (2011).

In this study, the range of heights taken into account for the different embankment is (2-15 m).

Other parameter that was bear in mind in this study was the compacting condition, given by the deformability modulus parameter. This stiffness parameter takes values of 40 MPa, 60 MPa and 80 MPa depending on compacting conditions: bad, medium and good conditions respectively.

For the creep coefficient $C_\alpha$, two different values were adopted: $C_{\alpha}^{max} = \frac{C_c}{15}$ and $C_{\alpha}^{min} = \frac{C_c}{25}$, PLAXIS (2011). Other authors considered different limits for this values as Mesri & Castro (1987) $C_{\alpha}^{max} = \frac{C_c}{10}$ and $C_{\alpha}^{min} = \frac{C_c}{50}$.

The creep model used to obtain settlements, was implemented in PLAXIS (2011). It is formulated according to the visco-plasticity field, following the law given by equation 4.29.

$$\varepsilon = \varepsilon_c - C_B \log \left( \frac{t}{t_c} \right) \quad \text{for } t > t_c$$

(4.29)

where $\varepsilon$ is the temporal value of strain, $\varepsilon_c$ is the strain value at the end of the primary consolidation, $t_c$ time at the end of the primary consolidation and $C_B$ is a material coefficient that is given by equation 4.30.

$$C_B = \frac{C_\alpha}{1 + \varepsilon_0}$$

(4.30)

In the previous equation, $\varepsilon_0$ refers to the initial void ratio.
4. Methodology in the dynamic interaction analysis

Table 4.21: Main characteristics entered in the numerical model of the embankment

<table>
<thead>
<tr>
<th>Geometric parameters</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Crown width (m)</td>
<td>14</td>
</tr>
<tr>
<td>Slope</td>
<td>2 (H) : 1 (V)</td>
</tr>
<tr>
<td>Height (m)</td>
<td>6-10-15</td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Geotechnical parameters</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Deformability modulus (MPa)</td>
<td>40,60, and 80</td>
</tr>
<tr>
<td>Cohesion (KPa)</td>
<td>15</td>
</tr>
<tr>
<td>Friction angle</td>
<td>32°</td>
</tr>
<tr>
<td>Dilatance angle</td>
<td>0</td>
</tr>
<tr>
<td>Void ratio</td>
<td>0.60</td>
</tr>
<tr>
<td>$C_c$</td>
<td>0.040</td>
</tr>
<tr>
<td>$C_{\alpha}$</td>
<td>$\frac{C_c}{15}$</td>
</tr>
</tbody>
</table>

The rest of the parameters, used to obtain the settlement of the embankments, are in Table 4.21.

Through the geotechnical and geometric values shown in Table 4.21, settlements for embankments of different types have been obtained. The total period of time in which settlements were collected is 10 years.

Settlements curves obtained, through the described methodology, are presented in Figure 4.55. Values of settlement are given regarding to the height of the embankment as well as its deformability modulus.

Analyzing the previous results, it is possible to draw some conclusions:

For all scenarios, the higher the value of the height of the embankment, the higher the value of the settlement. Although there is no linear relation between the height of the embankments and the value of the settlement.

Also, in all cases, the higher the value of the deformability modulus, the lower the value of the settlement.

Particularly, for a deformability modulus of 40 MPa, creep settlements are in the range of 0.22 – 0.45 % of the height of the embankment (this values are coincident with the higher limits of the measured settlements in the high-speed lines Madrid-Seville and Madrid -Barcelona).

When deformability modulus is 60 MPa, settlements are into the range of 0.18 – 0.32 % of the height of the embankment. Finally, for a deformability modulus of 80 MPa, settlements are into the range of 0.14 – 0.24 % of the height of the embankment.
4.10 Conclusions

This Chapter has presented the methodology, through which, numerical results of Chapters 5 and 6 have been obtained.

Methodology has been focused mainly in two different features. The first one is the analysis of contact elements located in different interfaces of the bidimensional numerical model. The second one is the assessment of the well known track irregularities phenomena, very common in the HSL field.

Regarding to contact elements, it was proved that in some cases of the wheel-rail interaction problem, contact forces obtained through Penalty method may contain an error, due to the penetration value allowed in this methodology. Moreover, in some cases in which irregularities of the rail are considered, the dynamic interaction analysis does not converge. This problem has been solved by changing the contact algorithm from Penalty to Lagrange Multipliers, despite of a higher value of computation effort.

In sleeper/ballast contact, Penalty method was used. No convergence problems were
detected when Penalty algorithm was used in this type of contact elements.

In the rest of contact cases: soil/structure, ballast/ballast and sub-ballast/sub-ballast, Penalty method with restrictions in the penetration value was used, as it was explained in 4.7.1.

It was also explained that the frictional behaviour in contact elements, was implemented in ANSYS following the Coulomb’s law of friction, proving the sticking/sliding condition, provided by this law, is applied and evaluated in each pair of contact nodes belonging to the contact surfaces.

Through Example 4, it was shown the great accuracy of the response obtained for a contact problem in a structural system, using two different finite elements codes: ANSYS and ABAQUS.

Regarding to track irregularities, both theoretical and experimental longitudinal level profile were evaluated. The quality of the profiles was checked having into account the particular specifications of the EN13848-5 (2010) and TSI (2008). Within the theoretical profiles, it was proved that irregularity profiles generated in accordance with the DBAG requirements, were those which had a best quality. On the other hand experimental profiles, coming from the monitoring works carried out by both the Portuguese and Spanish Railway Administrators, comply also the limits of track quality provided by EN13848-5 (2010) and TSI (2008).

Finally, from the long-term numerical results of the embankment settlements study, it can be concluded that the magnitude of the settlements in the embankments of a HSL depends directly on the values of the height as well as on the embankment deformability modulus. This fact would justify a particular assessment in which different values of embankments settlements may be considered in a dynamic train-track interaction analysis, Section 5.13.
5 Short-term analysis

5.1 Introduction

At this stage, it is important to analyze the dynamic interaction effects at a real transition zone. The transition zone that is the focus of this study, is located in the Portuguese railway line “Linha do Norte” and it is made up by a culvert with two wedge-shaped backfills on either side of it.

A detailed description of this transition zone has been provided in Ribeiro (2012). The main objective of the current section is to use the bidimensional numerical model developed by Ribeiro (2012) and adapt it, to take into account a more realistic scenario in which some variables as: the effect of frictional behaviour in different interfaces, the irregularities of the track and differential settlements of the embankments are considered.

Firstly, a brief description of the main components of the transition zone, is presented. Next, experimental results obtained by Ribeiro (2012), at the transition zone, will be described. These results allow for a better understanding the global behaviour of the transition zone, and on the other hand to calibrate and validate the numerical model developed and used to assess the behaviour of transition zones in a ballasted track. The experimental results are divided into three different categories, depending on the type of measurement obtained: track receptance, moving train measurements and train-track synchronized measurements.

Then, a detailed description, of the particular bidimensional model used to assess the dynamic interaction between the train and the track, will be made, referring also to the numerical calibration carried out in the model.

Some experimental validations, of the bidimensional model, carried out in the work Ribeiro (2012), are also shown in this Chapter.

Finally, dynamic results corresponding to different scenarios: track irregularities and embankment settlements are described, analyzing the main variables involved in the dynamic interaction between the train and the track, at the transition zone.

5.2 Description of the transition zone

The transition zone which consists the case study for this work is located in the Portuguese railway line “Linha do Norte”. The available information of this transition
zone is abundant and that is one of the reasons to select it as the primary case study of this thesis. Furthermore, the information available about the culvert and the surrounding soil was very important to calibrate properly the models developed by Ribeiro (2012), one of which has been taken and adapted to the particular evaluations that will be made, not only in this Chapter 5, but also in Chapter 6.

The selected transition zone, aim of this study, is made up by a concrete box culvert and the corresponding wedge-shaped backfill 1 on either side of the culvert.

The necessary information to develop this study was provided by the Portuguese Railway Administrator, REFER.

The concrete structure is located at km 40+250, along the the sub-section Alhandra-Setil, belonging to the section Vila Franca de Xira (north)-Azambuja of the “Linha Norte” line.

Technical denomination used for this structure is PH126-A.

![Figure 5.1](image1.png)

**Figure 5.1**: Location of the culvert in the Portuguese Railway line: (a) general view of the Portuguese rail net; (b) real picture of the culvert

Location of the transition zone as well as real picture of the culvert are depicted in Figure 5.1.

### 5.3 Characteristics of the culvert

The so-called PH126-A culvert consists of a rectangular frame made of reinforced concrete.

Geometric characteristics of the culvert are shown in Table 5.1. A detail of the culvert and its foundation is illustrated in Figure 5.2.

This is the typical configuration for culverts at this location of the Portuguese railway line, where alluvial soils are abundant.

---

1This is the typical configuration that culverts have in this railway line
5.4 Description of the wedge-shaped backfill

Table 5.1: Geometric characteristics of the culvert PH126-A, from Ribeiro (2012)

<table>
<thead>
<tr>
<th>Ground level [m]</th>
<th>Top level [m]</th>
<th>Height [m]</th>
<th>Width [m]</th>
<th>Difference between the top of the culvert and the rail head [m]</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.62</td>
<td>5.98</td>
<td>2.70</td>
<td>1.50</td>
<td>1.28</td>
</tr>
</tbody>
</table>

5.4 Description of the wedge-shaped backfill

The soil wedges are typical solutions for this type of structure. In this case, transition between the embankment and the culvert is made through a treated soil wedge, also named as wedge-shaped backfill, located on either sides of the culvert. The maximum wedge length is 12 m and the total width is 8 m. Both wedges laid on the embankment layer, right beneath the sub-ballast layer.

A detail of the wedge is shown in Figure 5.3.

The treated soil should have 150 kg/m³ of cement content and the height of the wedge tends to decrease gradually as we move away from the structure.

5.5 Characteristics of the track and the foundation

Every track layers were designed and built according to the specified requirements in UIC (2005a).

The brand of the railpads, used in the railway line, is Vossloh and technical specifications are depicted in Table 5.2.

As referred in Ribeiro (2012), in the building process of the transition zone, geological drillings were carried out.

The SPT test made identified that top layers of the soil are made up of clay and silty clay.

A further information of the transition zone elements can be found in Coba (1994).
5. Short-term analysis

![Figure 5.2: Culvert PH126-A, from Ribeiro (2012)](image)

5.6 Geometry and track layers

According to the project design and a experimental campaign carried out\(^2\), it was verified that the layout of the transition zone is similar to the illustration depicted in Figure 5.4.

It exists a sub-ballast layer above the culvert. The thickness of this layer, above the culvert, is 0.30 m. Elsewhere in the transition zone, it exists a sub-ballast layer, at these points the thickness of this layer is 0.55 m.

Ballast layer was also measured. Results showed the the depth of this layer is 0.45 m with a slope angle of 45°.

5.7 Experimental measurements

In this section, a brief description of the experimental measurements, carried out in Ribeiro (2012), is made.

\(^2\)The experimental campaign consisted on the digging of a hole to check: the type of materials used in the track layers and the dimensions of each layer.
5.7 Experimental measurements

These measurements were used mainly to calibrate the numerical model, that has been used in Chapters 5 and 6, to assess both the short-term and long-term behaviour of the transition zone respectively, when excited by a moving train.

Experimental results have been divided into three different categories: Track receptance measurements, measurements of the track with moving trains and train-track synchronized monitoring.

The track receptance measurements allow to have a better knowledge of the dynamic behaviour of the transition zones at different points. Resonant frequencies of the track were identified through the receptance functions. Dynamic stiffness tends to be higher over the culvert and it presents lower values as we move away from the structure.

Regarding the measurements with moving trains, a special attention was paid to the train that reaches the maximum speed (220 km/h) in this railway line: Alfa Pendular. With these measurements it was possible to validate the dynamic component of the

Table 5.2: Mechanical properties of the Vossloh railpad used in this study

<table>
<thead>
<tr>
<th>Features</th>
<th>Vossloh railpad</th>
</tr>
</thead>
<tbody>
<tr>
<td>Type</td>
<td>Zw – 687 a</td>
</tr>
<tr>
<td>Material</td>
<td>EVA: ethylene-vinylacetate</td>
</tr>
<tr>
<td>Size</td>
<td></td>
</tr>
<tr>
<td>Length</td>
<td>165 ± 2</td>
</tr>
<tr>
<td>Height</td>
<td>148 ± 1.5</td>
</tr>
<tr>
<td>Width</td>
<td>6 ± 0.22</td>
</tr>
<tr>
<td>Shore hardness D</td>
<td>32 – 47</td>
</tr>
<tr>
<td>Density</td>
<td>0.932 – 0.952 g/cm³</td>
</tr>
<tr>
<td>Static stiffness</td>
<td>450 kN/mm</td>
</tr>
</tbody>
</table>
5. Short-term analysis

Train loads in the transition zone.

Train-track synchronized monitoring was necessary to measured dynamic effects in the train track system. With this methodology was proved that accelerations inside the coach, when the trains move along the transition zone, comply with the normative limits explained in Chapter 2.

5.8 Numerical model of the transition zone

In this section, the bidimensional numerical model used to carry out the dynamic interaction analysis is described.

It has been developed in the finite element code ANSYS, in a plane stress state. This bidimensional model provides a good approximation, after calibration process, of the train/track interaction results with reduced computation time than the more complex 3D model. A more general description of this bidimensional model was already provided in Section 3.3.3.

Figure 5.5 shows a complete layout of the transition zone analyzed in this Chapter.

The rail has been modeled using beam elements and railpads were modeled using spring-damper elements. The rest of the track elements have been modeled using 4 – node finite elements in a plane stress state. These type of elements allow to have a certain width that will be assigned after complete the calibration process of the model.

The characteristics of the different materials that made up the track are presented in Tables 5.3 - 5.6.
5.9 Calibration of the numerical model

Before using the bidimensional model, it was necessary to calibrate it in order to ensure a better approach in the numerical results analysis. The calibration procedure has the purpose of establishing the width for the 4 – node finite elements, that have been used to model the track layers. For this purpose, a static calibration is made.

In the same way, it is necessary to modify the values of some mechanical parameters of track layers to obtain a good approach between numerical results and the experimental measurements carried out in the track. For this purpose, a dynamic calibration has been performed by fitting receptance curves coming from the numerical results to the curves obtained by experimental test.

With the purpose of developing both the static and dynamic calibrations, a reference 3D numerical model was developed in LS-DYNA code. More detailed and technical information of the 3D model can be found in Ribeiro (2012).
5. Short-term analysis

Table 5.3: Mechanical characteristics of the rail

<table>
<thead>
<tr>
<th>Rail UIC-60</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$E$ (MPa)</td>
<td>$\nu$ (-)</td>
</tr>
<tr>
<td>200E9</td>
<td>0.35</td>
</tr>
</tbody>
</table>

Table 5.4: Mechanical characteristics of the railpad

<table>
<thead>
<tr>
<th>Railpad</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>$K_{rp}$ (N/m)</td>
<td>$C_{rp}$ (Ns/m)</td>
</tr>
<tr>
<td>700E6</td>
<td>30E3</td>
</tr>
</tbody>
</table>

The basic features of the 3D FE-model were already explained in Section 3.3.2. It is important to note that the above mentioned 3D model was successfully calibrated through the data obtained in the experimental tests campaigned develop at the transition zone under consideration.

The calibration process is applied according to the diagram depicted in Figure 5.6.

As it can be seen from the previous layout, the 3D model developed in LS-DYNA is calibrated with the help of the experimental data, obtaining a next 3D calibrated model. This last is will be used as the reference tool that help us to calibrate the 2D model developed in ANSYS, with which most of the dynamic analysis of the current Thesis have been obtained.
5.9 Calibration of the numerical model

Table 5.5: Mechanical characteristics of the sleepers

<table>
<thead>
<tr>
<th>Sleepers</th>
<th>$E$ (Pa)</th>
<th>$\nu$ (−)</th>
<th>$\rho$ (kg/m$^3$)</th>
<th>Element type</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>30E9</td>
<td>0.3</td>
<td>2400</td>
<td>PLANE182</td>
</tr>
</tbody>
</table>

Table 5.6: Mechanical characteristics for the rest of the layers

<table>
<thead>
<tr>
<th>Layer</th>
<th>$E$ (MPa)</th>
<th>$\rho$ (kg/m$^3$)</th>
<th>$\nu$ (−)</th>
<th>Element type</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ballast</td>
<td>120</td>
<td>1530</td>
<td>0.20</td>
<td>PLANE 182</td>
</tr>
<tr>
<td>Sub-ballast</td>
<td>160</td>
<td>1935</td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td>Embankment</td>
<td>80</td>
<td>1900</td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td>Soil cement soil</td>
<td>10E3</td>
<td>2110</td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td>Foundation soil</td>
<td>285</td>
<td>1900</td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td>Culvert foundation</td>
<td>150</td>
<td>1900</td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td>Structure (culvert)</td>
<td>30E3</td>
<td>2500</td>
<td>0.30</td>
<td></td>
</tr>
<tr>
<td>Waterproof layer</td>
<td>3000</td>
<td>2300</td>
<td>0.30</td>
<td></td>
</tr>
</tbody>
</table>

5.9.1 Static calibration

Methodology followed for the static calibration was already presented in Section 3.3.3. This calibration is based on the determination of the finite elements width, depending on both the depth and location of the finite elements in the bidimensional model.

Along the transition zone, the longitudinal profile of the track is variable and thus it has been decided to make the static calibration at three different track sections: at the culvert section $CS$, at the wedges section $WS$ and at the embankment section $ES$. See Figure 5.7.

Figure 5.7: Three different sections ES, WS and CS in which the static calibration is carried out

In order to carry out the static calibration, a vertical static load of 100 kN is applied on the rail. Below, iterative methodology described in Section 3.3.3 is applied to obtain the width of each one of the finite elements that are part of the bidimensional numerical model. The foregoing width of each element at different sections of the transition zone is depicted in Figure 5.8.
One of the main conclusions drawn by Ribeiro (2012) is that, after having concluded the static calibration, it is possible to adopt the same width variation, coming from the different depths of finite elements in the model, obtaining a good approach to the results provided by the 3D model.

### 5.9.2 Dynamic calibration

With the purpose of carrying out this calibration, it is necessary to simulate a track receptance test.

As in the static calibration case, it is important to note that dynamic calibration of the bidimensional model is based on the results provided by the more complete 3D numerical model, developed in LS-DYNA software.

In order to develop the experimental calibration, only values of deformability modulus of the ballast and sub-ballast layers as well as the stiffness of the railpads, were considered.

Results obtained from the dynamic calibration of the 3D model show that the above mentioned mechanical features suffer a change in their modulus, see Table 5.7.

<table>
<thead>
<tr>
<th>Element</th>
<th>Initial value</th>
<th>Calibrated value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ballast</td>
<td>E=130 MPa</td>
<td>E=120 MPa</td>
</tr>
<tr>
<td>Sub-ballast</td>
<td>E=120 MPa</td>
<td>E=160 MPa</td>
</tr>
<tr>
<td>railpads</td>
<td>E=450E6 N/m</td>
<td>E=700E6 N/m</td>
</tr>
</tbody>
</table>

Table 5.7: Calibrated values of the 3D model, from Ribeiro (2012)
5.10 Numerical model of the train

Considering the experimental calibrated values shown in Table 5.7, a numerical adjustment of the receptance curves of the 3D and 2D models has been done, in order to calibrate the bidimensional model used in this work. This adjustment allowed to determine the values for the Rayleigh damping matrix used in the 2D model developed in ANSYS.

Table 5.8 shows the Rayleigh damping calibrated values for the different track materials.

Table 5.8: Values adopted in the Rayleigh damping matrix in 2D numerical model

<table>
<thead>
<tr>
<th>Layer</th>
<th>(s(^{-1}))</th>
<th>(s)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ballast</td>
<td>15</td>
<td>1.8E-4</td>
</tr>
<tr>
<td>Sub-ballast</td>
<td>15</td>
<td>1.0E-3</td>
</tr>
<tr>
<td>Embankment</td>
<td>15</td>
<td>3.0E-3</td>
</tr>
<tr>
<td>Foundation soil</td>
<td>15</td>
<td>3.0E-3</td>
</tr>
<tr>
<td>Culvert foundation</td>
<td>15</td>
<td>3.0E-3</td>
</tr>
</tbody>
</table>

Taking into account the adjusted damping values shown in Table 5.8, receptance curve of the rail is illustrated in Figure 5.9, for both 2D and 3D models.

The obtained results show a good agreement between 2D and 3D model. For this reason, 2D model is taken, as a reference, to carry out the dynamic interaction analysis in the subsequent sections of the current work.

Figure 5.9: Adjustment of the receptance curves in the 2D and 3D models, numerical results from Ribeiro (2012)

5.10 Numerical model of the train

With the purpose of analyzing both the short-term and long-term behaviour of the train-track interaction at the transition zone and in the frame of a HSL track, it has
been decided to consider the Eurostar train.

The Eurostar train runs, with certain exceptions, at a maximum speed of 300 km/h. The service provided by this train, connects United Kingdom with France and Belgium, crossing the Channel Tunnel.

Locomotives, coaches and a bogie of this train is illustrated in Figure 5.10.

![Eurostar train](image)

Figure 5.10: Eurostar train: (a) General view; (b) Detail of the locomotives, coaches and bogie

The length of the Eurostar train is 386.67 m, it consists of a locomotive at each end of the train and three different types of coaches: R1, R2-R8 and R9. Located in one of the symmetry axes of the train. See layout in Figure 5.11, where $D$ is the total length of the coach and $d$ is the distance between two axles of the bogie.

![Layout of the Eurostar train](image)

Figure 5.11: Layout of the Eurostar train, with distances between axles and bogies.

Geometric and mechanical characteristics of the locomotives and coaches of the Eurostar train, are depicted in Table 5.9.

In order to clarify each one of the parameters presented in Table 5.9, it is depicted the general layout of a coach model in Figure 5.12. It includes the mass of the wheels, the mass of the bogie, the mass of the coach, the stiffness and the damping value of the primary suspension and the stiffness and the damping value of the secondary suspension.
5.10 Numerical model of the train

![Model of an Eurostar coach](image)

Figure 5.12: Model of an Eurostar coach

Table 5.9: Geometric and mechanical characteristics of an Eurostar train, ERII-D 214/RP9 (1999)

(a) Locomotive

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_c$</td>
<td>51500 kg</td>
</tr>
<tr>
<td>Secondary suspension</td>
<td></td>
</tr>
<tr>
<td>$K_s$</td>
<td>3.26E6 N/m</td>
</tr>
<tr>
<td>$C_s$</td>
<td>9.00E6 N.s/m</td>
</tr>
<tr>
<td>$M_b$</td>
<td>2200 kg</td>
</tr>
<tr>
<td>Primary suspension</td>
<td></td>
</tr>
<tr>
<td>$K_p$</td>
<td>2.60E6 N/m</td>
</tr>
<tr>
<td>$C_p$</td>
<td>1.20E4 N.s/m</td>
</tr>
<tr>
<td>$M_e$</td>
<td>1700 kg</td>
</tr>
<tr>
<td>Load per axle</td>
<td>170 kN</td>
</tr>
<tr>
<td>$d$</td>
<td>3.0 m</td>
</tr>
<tr>
<td>$D$</td>
<td>22.50 m</td>
</tr>
</tbody>
</table>

(b) Coach R1

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_c$</td>
<td>35860 kg</td>
</tr>
<tr>
<td>Secondary suspension</td>
<td></td>
</tr>
<tr>
<td>$K_s$</td>
<td>0.90E5 N/m</td>
</tr>
<tr>
<td>$C_s$</td>
<td>2.00E4 N.s/m</td>
</tr>
<tr>
<td>$M_b$</td>
<td>2200 kg</td>
</tr>
<tr>
<td>Primary suspension</td>
<td></td>
</tr>
<tr>
<td>$K_p$</td>
<td>2.60E6 N/m</td>
</tr>
<tr>
<td>$C_p$</td>
<td>1.20E4 N.s/m</td>
</tr>
<tr>
<td>$M_e$</td>
<td>1700 kg</td>
</tr>
<tr>
<td>Load per axle</td>
<td>170 kN</td>
</tr>
<tr>
<td>$d$</td>
<td>3.0 m</td>
</tr>
<tr>
<td>$D$</td>
<td>21.845 m</td>
</tr>
</tbody>
</table>

(c) Coaches R2-R8

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_c$</td>
<td>22525 kg</td>
</tr>
<tr>
<td>Secondary suspension</td>
<td></td>
</tr>
<tr>
<td>$K_s$</td>
<td>5.80E5 N/m</td>
</tr>
<tr>
<td>$C_s$</td>
<td>2.00E4 N.s/m</td>
</tr>
<tr>
<td>$M_b$</td>
<td>2900 kg</td>
</tr>
<tr>
<td>Primary suspension</td>
<td></td>
</tr>
<tr>
<td>$K_p$</td>
<td>2.00E6 N/m</td>
</tr>
<tr>
<td>$C_p$</td>
<td>1.20E4 N.s/m</td>
</tr>
<tr>
<td>$M_e$</td>
<td>1900 kg</td>
</tr>
<tr>
<td>Load per axle</td>
<td>170 kN</td>
</tr>
<tr>
<td>$d$</td>
<td>3.0 m</td>
</tr>
<tr>
<td>$D$</td>
<td>18.70 m</td>
</tr>
</tbody>
</table>

(d) Coach R9

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>$M_c$</td>
<td>27122 kg</td>
</tr>
<tr>
<td>Secondary suspension</td>
<td></td>
</tr>
<tr>
<td>$K_s$</td>
<td>2.50E5 N/m</td>
</tr>
<tr>
<td>$C_s$</td>
<td>2.00E4 N.s/m</td>
</tr>
<tr>
<td>$M_b$</td>
<td>2900 kg</td>
</tr>
<tr>
<td>Primary suspension</td>
<td></td>
</tr>
<tr>
<td>$K_p$</td>
<td>1.32E6 N/m</td>
</tr>
<tr>
<td>$C_p$</td>
<td>1.20E4 N.s/m</td>
</tr>
<tr>
<td>$M_e$</td>
<td>1900 kg</td>
</tr>
<tr>
<td>Load per axle</td>
<td>170 kN</td>
</tr>
<tr>
<td>$d$</td>
<td>3.0 m</td>
</tr>
<tr>
<td>$D$</td>
<td>21.965 m</td>
</tr>
</tbody>
</table>

According to the requirements given by TSI (2008), analysis of the previous parame-
sters confirm that Eurostar train comply with the particular specifications established for the European high-speed rail network:

- Load per axle is not higher than 170 kN.
- Total length of the train is lower than 400 m.
- Total weight of the train is lower than 10000 kN.
- Non-sprung mass is lower than 2000 kg.
- Geometric limits are also complied in Eurostar train.

To carry out this study, only a bogie of an Eurostar train is model. A layout of the bogie, corresponding to a Coach R1-Eurostar, is illustrated in Figure 5.13.

![Figure 5.13: Layout of the Eurostar (Coach R1-Eurostar) bogie used in the analysis](image)

In Table 5.10, main features of the above mentioned bogie, are presented.

Table 5.10: Mechanical characteristics of the Bogie

<table>
<thead>
<tr>
<th>Characteristic</th>
<th>Value</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bogie mass $M_b [kg]$</td>
<td>2200</td>
</tr>
<tr>
<td>Wheel/Rail mass $M_e [kg]$</td>
<td>1700</td>
</tr>
<tr>
<td>Contact stiffness $K_h [N/m]$</td>
<td>1.35E9</td>
</tr>
<tr>
<td>Stiffness of the primary suspension $K_p [N/m]$</td>
<td>2.60E6</td>
</tr>
<tr>
<td>Damping coefficient of the primary suspension $C_p [Ns/m]$</td>
<td>1.20E4</td>
</tr>
</tbody>
</table>

5.11 Static and dynamic validation of the 2D numerical model

In order to have a certain degree of confidence in the proposed 2D model, two simple validations have been here carried out.

Firstly, a static validation analysis in which the vertical displacement of the rail is obtained when a vertical punctual load is applied on the track. Secondly, a dynamic validation is made. This validation consist on obtaining the receptance curve in which the dynamic properties of the track system can be analyzed.
5.11 Static and dynamic validation of the 2D numerical model

Static validation

A simple static test has been carried out in order to obtain the vertical deflection of the rail. In this case, a vertical punctual load is applied on the rail between two consecutive sleepers in a track section over the culvert. This vertical load corresponds to half of the maximum force acting on an axle of an Eurostar train: 170/2 kN, see Table 5.9. A similar validation example can be found in Nguyen (2013).

Figure 5.14: Comparison of vertical displacement of the rail for a static load of 85 kN

Figure 5.14 shows the maximum vertical deflection of the rail for this particular case, 0.2 mm. This value is slightly smaller than that obtained in Nguyen (2013) for the same load case, applied on a more flexible regular track section: 1 mm.

Dynamic validation

In this case, two different track receptance curves have been obtained through a typical harmonic analysis in which an unitary vertical punctual load has been applied on the rail. The application points of this vertical punctual loads are placed over the central sleeper and in the middle point of the rail between two consecutive sleepers. These two points are located in a track section over the culvert.
5. Short-term analysis

Figure 5.15: Receptance curves obtained from the harmonic analysis in which an unitary vertical punctual load is applied on the rail

Figure 5.15 shows the receptance curves obtained when the harmonic analysis is carried out. Note that when the load is applied between two consecutive sleepers the pin-pin frequency can be captured an its value is approximately 1350 Hz. Other fundamental vibration frequencies of the track have been also here obtained as: Full track resonant frequency: 80 Hz, anti-resonant frequency of the sleepers: 190 Hz and vibration frequency of the rail on the sleepers: 700 Hz

5.12 Dynamic results

5.12.1 Introduction

In this section, a dynamic interaction analysis between the track and the vehicle is presented. The main objective of this assessment is to evaluate the influence of some of the key features, in both the short-term and long-term behaviour of the transition zone.

Transition zones have been defined as those sections of the track in which an abrupt change of the track stiffness occurs. As a consequence of this change in the track stiffness, these zones are critical points in which track degradation is much more accused than in other sections of the track. As a result, if a special attention is not paid to the degradation process, safety conditions of passing trains may be exceeded, causing major problems.

Historical data shows that deterioration process at transition zones is strongly correlated with some parameters as: magnitude of interaction forces between the wheel and the rail, presence of differential settlements and abrupt changes in the stiffness of the track coming from different backgrounds.

Taking as a reference the transition zone, made up by the presence of a culvert, a dynamic analysis is carried out analyzing the influence of different variables that
affect the above mentioned parameters. The variables considered, in the current Chapter, to develop the sensitivity analysis are:

- Frictional behavior of soil/structure, ballast-ballast and sub-ballast/sub-ballast interfaces.
- Track irregularities.
- Differential settlements at different sections of the transition zone.

With the purpose of having a better understanding of each of the previous variables, a sensitivity analysis has been developed, evaluating the main response parameters that have a significant influence in both the short and long-term behaviour of the track. These response parameters are: contact force between the wheel and the rail, acceleration magnitude on the axles and the value and distribution of the deviatoric stress magnitude, along the transition zone, in the ballast layer.

Two control parameters have been considered in this study, on one hand the value of the contact forces: a negative value of this parameter indicates a loss of contact between the wheel and the rail. On the other hand, other control parameter defined in this study is the value of accelerations on the axles of the vehicle. Accelerations higher than 30 m/s² indicate that the normal control level of the track quality has been exceeded, and the track needs to be followed up with intense monitoring. This limit is actually a recommended action provided by López Pita (2006), according to the acceleration levels in dynamic inspection of the AVE Madrid-Seville.

Value of deviatoric stresses in the ballast layer is used, in this case, as a good indicator to know which sections of the transition zone are more sensitive to suffer a major degradation. Moreover, this parameter is included in the formulation provided by ORE (1970), to simulate the track degradation phenomena that will be analyzed later on, in Chapter 6.

### 5.12.2 Different scenarios of analysis

In order to achieve a better understanding the effect of some variables as: frictional behaviour in different interfaces, track irregularities and differential settlements, four different scenarios have been taken into account:

- Scenario $C_0$: Correspond to the original case presented in (Ribeiro, 2012). In this scenario, neither contact elements are considered in the interface between the soil and the culvert nor track irregularities are taken into account in the rail geometry definition. This scenario is considered as a reference case to compare results with.
- Scenario $C_1$: In this case, contact elements have been considered but only in the soil/structure interfaces. For this particular case of study it has been considered a friction coefficient $\mu = \tan(2/3.) = 0.36$ on the interface between the soils and the concrete structure (culvert).
5. Short-term analysis

- Scenario $C_2$: In this case, contact elements have been considered not only in the soil-structure interfaces, as in the previous case, but also in the sub-ballast/sub-ballast and ballast/ballast interfaces located in the upward vertical extension of the soil/structural interfaces as well. In this case, the use of contact elements in ballast and sub-ballast layers aimed to model the real lack of shear strength of ballast and sub-ballast layers when a settlement of the embankment occurs. In this case, friction coefficient considered was equal to $\mu = \tan(\phi) = 0.83$.

- Scenario $C_3$: In this scenario, imperfections of the rail have been considered in order to simulate the effect of track irregularities. Particularly, longitudinal level irregularity profiles have been taken into account to develop the dynamic interaction analysis in the scenario. Both the theoretical profiles, generated as explained in 4.8.5, and experimental profiles are used in this scenario to carry out the simulations.

An overview of the different scenarios, that have been taken into account in this Chapter, is depicted in Figure 5.16.

![Figure 5.16: Overview of the different scenarios that have been considered in the dynamic interaction analysis](image)

### 5.12.3 Influence of contact elements

In this section, an assessment of the contact elements in different interfaces of the transition zone, is made. Scenarios $C_1$ and $C_2$ are involved in this evaluation analysis. In this case, Scenario $C_0$ is taken as the reference to compare results with.

In the first part, a evaluation of contact forces between the wheel and the rail is made. To do so, a dynamic interaction analysis for speeds of the train of 300 km/h, 250 km/h
and 200 km/h, is made. Later on, a similar evaluation is also made, considering another different parameters as vertical accelerations on the axles of the bogie and deviatoric stresses along the transition zone.

**Interaction forces**

The forces between the rail and the wheel are known to be parameters of great importance in dynamic analysis of train-track interaction.

Its value is important for both the train and track. Regarding to the track, contact forces have a great influence in the track components, causing damage to the rail and other track components.

It should also be noted that interaction force provides several fundamental functions for the train as the support action for the vehicle load. Depending on its magnitude, accelerations on the vehicle may be exceeded and important damages can be caused in the wheels.

Summarizing it can be said, that interaction forces between the wheel and the rail are an important parameter regarding to the analysis of both the safety and comfort conditions.

![Graphs showing contact forces between the wheel and the rail](image)

**Figure 5.17:** Contact forces between the wheel and the rail, for a speed of 200 km/h: (a) Scenarios $C_0$ and $C_1$; (b) Scenarios $C_0$ and $C_2$

Results from Figure 5.17 correspond to the front axle of the bogie. Similar results are depicted in Figures 5.18 and 5.19 for speed of the train of 250 km/h and 300 km/h respectively.

In Figures 5.17 - 5.19, it can be observed that contact force between the wheel and the rail oscillates, along the transition zone, around the static load of a half axle, that is $170/2=85$ kN. Such oscillation is more homogeneous in scenario $C_0$, in which slight variations of interaction forces are detected when the train moves above the embankment/wedge and the wedge/culvert limits. In scenarios $C_1$ and $C_2$, variation of contact forces are higher right above the culvert.
5. Short-term analysis

Figure 5.18: Contact forces between the wheel and the rail, for a speed of 250 km/h: (a) Scenarios $C_0$ and $C_1$; (b) Scenarios $C_0$ and $C_2$

Figure 5.19: Contact forces between the wheel and the rail, for a speed of 300 km/h: (a) Scenarios $C_0$ and $C_1$; (b) Scenarios $C_0$ and $C_2$

In Table 5.11, values of the maximum contact force are shown. These values are given for both front and back axles of the bogie.

As seen on the previous table, variation of contact forces are not very significant. Comparing to scenario $C_0$, scenario $C_2$ is those in which variation of interaction forces is the highest (around 3.7% highest for a speed of 300 km/h).

On the other hand, differences of contact forces due to a variation of the speed are not very significant, as well, as Table 5.11 shows.

In conclusion it is evident that, for all scenarios, the magnitude of the interaction force between the wheel and the rail increases with the train speed. Furthermore, scenario $C_2$, that is appropriate to simulate the effect of differential settlements, is the scenario that presents the highest value of contact forces, for every speeds taken into consideration.

In the following, frequency content of interaction force is shown, for some of the scenarios previously analyzed: $C_0$, $C_1$ and $C_2$. For every cases, a comparison is
5.12 Dynamic results

<table>
<thead>
<tr>
<th>Speed [km/h]</th>
<th>Max. contact force [kN]</th>
<th>Scenario $C_0$</th>
<th>Scenario $C_1$</th>
<th>Scenario $C_2$</th>
</tr>
</thead>
<tbody>
<tr>
<td>200</td>
<td>86.00</td>
<td>86.06</td>
<td>0.97%</td>
<td>86.78</td>
</tr>
<tr>
<td>250</td>
<td>86.38</td>
<td>86.61</td>
<td>0.26%</td>
<td>87.75</td>
</tr>
<tr>
<td>300</td>
<td>86.77</td>
<td>87.42</td>
<td>0.75%</td>
<td>90.01</td>
</tr>
</tbody>
</table>

always made taken as a reference scenario $C_0$ and a speed of the train of 300 km/h.

Analyzing Figures 5.20 and 5.21, in scenario $C_0$ the interaction force is influenced by the parametric frequency given by the distance of the sleepers. In scenarios $C_1$ and $C_2$ parametric frequency given by the distance of the sleepers has also a certain influence, but in these cases a low frequencies band arises when contact elements are taken into account in the different interfaces.

Figure 5.20: Interaction forces between the wheel an the rail for scenarios $C_0$ and $C_1$ and $v=300$ km/h: (a) time values and (b) frequency values
5. Short-term analysis

Figure 5.21: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_2$ and $v=300$ km/h: (a) time values; (b) frequency values

**Acceleration on the axles**

Acceleration on the axles is used in the current work as a control parameter. An extreme value of this parameter indicates a decrease in terms of track quality, as referred in López Pita (2006). The reference values for this control parameter are shown in Section 2.8, Table 2.13 in which different control levels depend on the value of the vertical acceleration value given on the axles of the bogie.

In Figure 5.22, maximum value of vertical accelerations on the axles of the bogie are depicted. It has been shown that values of these vertical accelerations on the axles are well under the reference limit of $30 \text{ m/s}^2$, in which the track need to be followed up with intense monitoring, López Pita (2006).

Figure 5.22: Values of the maximum vertical acceleration on the axles of the bogie given for different scenarios and speeds

As in the previous case of contact forces, vertical accelerations of the axles are greater, the higher the speed of the train. Furthermore, scenario $C_2$ have the higher values
in terms of vertical accelerations, for every speeds taken into consideration.

**Deviatoric stress evolution in ballast layer**

This parameter has a great influence in the global vertical deformation of the track, given by settlement of the ballast layer. According to Holtzendorff & Gerstberger, for a certain level of the deviatoric stress, the frictions resistance at the contact points between the ballast particles is exceeded. Consequently the particles start to move against each other. There is a limit of deviatoric stress that, if it is exceeded, the ballast layer becomes destabilized, causing significant settlements in the track.

![Figure 5.23](attachment:deviatoric_stress_evolution.png)

**Figure 5.23:** Deviatoric stress evolution in the top of the ballast layer along the transition zone: (a) $v=200$ km/h; (b) $v=250$ km/h; (c) $v=300$ km/h

Figure 5.23 shows the deviatoric stress value in the top of the ballast layer, along the transition zone. Values of deviatoric stress are given right under the sleepers when the vehicle moves from the left to the right. As it can be seen, for every scenarios, the value of the deviatoric stress is higher above the wedges than over the regular track. Furthermore, the value of deviatoric stress is higher above the culvert than above the treated soil wedges. Distribution of deviatoric stresses along the transition zone, gives an idea of which sections of the track are going to suffer a major degradation over the time.
The influence of both the speed of train and scenario taken into account is not very important neither in the magnitude of deviatoric stress nor in the distribution of stresses along the transition zone. Even so, it is important to note that Scenario $C_2$ is that one that presents the higher value of deviatoric stresses, for every speeds analyzed.

5.12.4 Influence of track irregularities, scenario $C_3$

With the objective of performing an accurate and representative model of the physical behaviour of the train-track system, track irregularities are taken into account. For a fixed track design, the geometry of the rails is in constant evolution, which is mostly due to the interaction between the train, the track and the substructure. As a consequence, a new track is gradually damaged and needs to be regularly subjected to maintenance works.

In this section, the influence of track irregularities in the dynamic train-track interaction, at transition zone, is carried out. Furthermore, an assessment between theoretical and experimental irregularity profiles will also be made, evaluating the short-term dynamic behaviour of the transition zone with each one of the above mentioned profiles. The knowledge of the influence of each longitudinal level irregularity profile, in a short-term analysis, will provide an idea of its importance in long-term analysis carried out in the next Chapter.

5.12.4.1 Theoretical profiles

These profiles are generated as explained in section 4.8.5. In the quality sensitivity analysis of longitudinal level profiles, carried out in Section 4.8.8, it was concluded that profiles generated according the German Railway Administrator DBAG, were less aggressive, presenting lowest values for both standard deviation and peak value of the distribute irregularity profiles. It was also proved that theoretical profiles coming from DBAG, were into the limits given by TSI (2008) and EN13848-5 (2010). In this Chapter, theoretical profiles generated according to the DBAG requirements, are used to evaluate the short-term behaviour of the train-track dynamic interaction.

In the first part of this analysis, a evaluation of the dynamic interaction between train and track is made. Two different theoretical profiles are generated, taking into account a bad quality of the track. After performing the dynamic analysis, both the values of interaction forces between the wheel and the rail and accelerations in the axles of the bogie are determined.

After that, a evaluation of the dynamic interaction between train and track is carried out, but in this case by considering three different profiles, generated according to the DBAG requirements. In this case a good quality of the track is considered to carry out the dynamic interaction analysis.
5.12 Dynamic results

Finally, the same experience is done with real irregularity profiles coming from the monitoring of two different railway Administrators: REFER: profiles are coming from a conventional Portuguese railway line and ADIF: profiles are coming from a Spanish HSL.

Scenario $C_3$ DBAG bad quality profiles

Two different profiles are generated taking into account a bad quality of the track. Geometry of this profiles is depicted in Figure 5.24.

![Figure 5.24: Theoretical irregularity profiles generated according to a bad quality of the track: (a) profile 1; (b) profile 2](image)

**Interaction forces**

Interaction forces for each profile are depicted in Figure 5.25, for a speed of the vehicle of 300 km/h. Note that in these diagrams, the value of interaction force for Scenario $C_3$, are compared with the values of contact forces coming from scenario $C_0$.

![Figure 5.25: Contact forces between the wheel and the rail, for a speed of 300 km/h: (a) profile 1; (b) profile 2](image)
Table 5.12: Values of maximum contact forces between the wheel and the rail

<table>
<thead>
<tr>
<th>Speed [km/h]</th>
<th>Max. contact force [kN]</th>
<th>Scenario C₀</th>
<th>Scenario C₃</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td>profile 1</td>
<td>profile 2</td>
</tr>
<tr>
<td>300</td>
<td>86.77</td>
<td>206.2</td>
<td>137.6%</td>
</tr>
<tr>
<td></td>
<td></td>
<td>193.8</td>
<td>123.35%</td>
</tr>
</tbody>
</table>

Analyzing the contact forces depicted in Figure 5.25, it can be concluded that the value of contact forces is extremely high for both profiles. In two cases, contact forces have a negative value which means a loss of contact between wheel and rail.

**Acceleration on the axles**

In Figure 5.26, values of maximum accelerations on the axles of the bogie are depicted. In this case, values of the current analyzed scenario C₃ are also included, for the two irregularity profiles taken into account. Limits of acceleration on the axles, provided by López Pita (2006) are shown in order to have a reference of the excessive value of vertical accelerations, obtained when bad quality profiles are considered. Thus, it has been decided to discard in the current work, theoretical profiles generated according to a bad quality of the track.

![Figure 5.26: Values of the maximum vertical acceleration on the axles of the bogie given for different scenarios and speeds. Scenario C₃ is evaluated considering bad quality profiles](image)

**Scenario C₃ DBAG good quality profiles**

In this section, a dynamic interaction analysis has been performed considering three different theoretical profiles. These profiles have been generated in accordance with the requirements, provided by the German railway Administrator DBAG, for a good quality of the track. With the purpose of having a better understanding of the influence of the speed of the train, three different speeds are considered in this section: 200 km/h, 250 km/h and 300 km/h.
5.12 Dynamic results

Geometry of the generated artificial profiles are shown in Figure 5.27.

As in the previous cases, a evaluation of contact forces between the wheel and the rail is here assessed, developing a dynamic interaction analysis for different speeds of the train. Later on, an evaluation of the influence of track irregularities is made, paying a special attention to variables that have a strong relationship with the track degradation phenomena: accelerations on the axles and evolution of deviatoric stresses on the top of ballast layer, along the transition zone.

**Interaction forces**

Values of interaction forces, for the different irregularity profiles, are depicted in Figures 5.28 - 5.30. As it can be seen, the magnitude of the contact force is lower, than the corresponding values of the profiles generated according to a bad quality of the track condition.
5. Short-term analysis

Figure 5.28: Contact forces between the wheel and the rail, for a profile 1 (good quality): (a) \( v=300 \text{ km/h} \); (b) \( v=250 \text{ km/h} \); (c) \( v=200 \text{ km/h} \)

Figure 5.29: Contact forces between the wheel and the rail, for a profile 2 (good quality): (a) \( v=300 \text{ km/h} \); (b) \( v=250 \text{ km/h} \); (c) \( v=200 \text{ km/h} \)

Figure 5.30: Contact forces between the wheel and the rail, for a profile 3 (good quality): (a) \( v=300 \text{ km/h} \); (b) \( v=250 \text{ km/h} \); (c) \( v=200 \text{ km/h} \)

From the analysis of the previous contact forces, it is observed that a loss of contact occurs for profile 3 at a speed of 300 km/h. Even though contact forces values have
improved, by using profiles generated according to a good quality of the track, the values of interaction forces are still very high which may induce loss of contact between the wheel and the rail.

A comparison of the maximum contact forces between scenario $C_0$ and $C_3$ (with 3 different profiles) is made in Table 5.13. Note that increment in $\%$, for each profile of scenario $C_3$ is related to the corresponding value of scenario $C_0$ with the same speed value.

Table 5.13: Values of maximum contact forces between the wheel and the rail

<table>
<thead>
<tr>
<th>Speed [km/h]</th>
<th>Max. contact force [kN]</th>
<th>Scenario $C_0$</th>
<th>Scenario $C_3$ theoretical profiles DBAG</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td>profile 1</td>
</tr>
<tr>
<td>200</td>
<td>86.0</td>
<td>123.8 44%</td>
<td>142.9 66%</td>
</tr>
<tr>
<td>250</td>
<td>86.3</td>
<td>145.2 68%</td>
<td>174.9 102%</td>
</tr>
<tr>
<td>300</td>
<td>86.7</td>
<td>160.6 85%</td>
<td>195.4 125%</td>
</tr>
</tbody>
</table>

Some conclusions can be drawn from the analysis of interaction forces:

- The use of these profiles means a significant increment in the value of interaction forces. This increment increases with speed of the train.

- No significant variations of contact forces when the train passes right over the different transitions have been recorded. This variation did occur, as it has already been presented in Section 5.12.3, when the train moved along the transition zone and passed over sections with different stiffness: embankment-wedge transition and wedge-culvert transition. Moreover, such variation in contact forces was more pronounced with higher train speeds. In this case, the contact force is clearly influenced by the track irregularity parameter.

The influence of the frequency content in the track irregularity geometry is very significant, as previously demonstrated. This fact has been proven by analyzing the frequency content of the contact force. Frequency content of interaction force is shown, for different rail profiles of scenario $C_3$: $p_1$, $p_2$, and $p_3$, see Figures 5.31 - 5.36. For every cases, a comparison is always made, taken as a reference scenario $C_0$ and a speed of the train of 300 km/h.

Analyzing Figures 5.31, 5.32 and 5.33 it can be seen how in this case, the frequency content of the rail irregularity has a very significant influence in the magnitude of contact forces between the wheel and the rail. For every cases, the parametric frequency given by the distance between the sleepers is negligible compares with the influence of track irregularities. These values are given for a speed of the train of 300 km/h but it can be interesting to show the frequency content for the contact forces at a lower speed of 200 km/h. Theoretically in this last case, the influence of the parametric frequency, given by the distance of the sleepers, should be more significant than for a speed of 300 km/h.
5. Short-term analysis

![Graph](image1)

Figure 5.31: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_{3p1}$ and $v=300$ km/h: (a) time values; (b) frequency values

![Graph](image2)

Figure 5.32: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_{3p2}$ and $v=300$ km/h: (a) time values; (b) frequency values

In Figures 5.34, 5.35 and 5.36, frequency content of the contact forces for the different rail profiles of scenario $C_3$: $p1$, $p2$, and $p3$ is depicted. Note that as it was done before, a comparison is always made taken as a reference scenario $C_0$ and a speed of the train of 200 km/h.

From the previous figures analysis it can be seen, as expected, how for a speed train of 200 km/h, contribution of the parametric frequency of the sleeper is more important than in the previous case.

It can be concluded that theoretical track irregularities have a great influence in the dynamic response of the system. This influence is more pronounced the higher the value of the speed. For lower speeds (e.g. 200 km/h) influence of track irregularities in the dynamic response is still important but at this point the effect of other parameters started to take on certain relevance.
5.12 Dynamic results

Figure 5.33: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_{3p3}$ and $v=300 \text{ km/h}$: (a) time values; (b) frequency values

Figure 5.34: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_{3p1}$ and $v=200 \text{ km/h}$: (a) time values; (b) frequency values

**Acceleration on the axles**

Regarding the vertical accelerations on the axles of the bogie, for all profiles, the limit of $30 \text{ m/s}^2$ is exceeded. In Figure 5.37 it can be observed the growth of the maximum vertical acceleration on the axles as the speed of the train increases.

Regarding the values of vertical accelerations, it can be concluded that the use of these theoretical profiles, generated according to the DBAG specifications for a good quality of the track, is not adequate to analyze degradation phenomenon. Values of acceleration analyzed, already point out an advanced state of degradation of the track, in which track maintenance operations are necessary, as suggested in López Pita (2006).
Figure 5.35: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_{3p2}$ and $v=200\,\text{km/h}$: (a) time values; (b) frequency values

Figure 5.36: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_{3p3}$ and $v=200\,\text{km/h}$: (a) time values; (b) frequency values
5.12 Dynamic results

**Figure 5.37:** Values of the maximum vertical acceleration on the axles of the bogie, in Scenario $C_3$ (DBAG theoretical profiles), given for different profiles and speeds

**Deviatoric stress evolution in ballast layer**

**Figure 5.38:** Deviatoric stress evolution in the top of the ballast layer along the transition zone:
(a) $v=200$ km/h; (b) $v=250$ km/h; (c) $v=300$ km/h
5. Short-term analysis

Deviatoric stress evolution in the top of the ballast layer, is in this case, highly influenced by the track irregularities, see Figure 5.38. For the whole spectrum of train speeds considered herein, the pattern of deviatoric stress obtained for scenarios $C_0$, $C_1$ and $C_2$ (with higher values above the wedge-shaped backfill and even higher values above the culvert) disappear. When theoretical profiles are taken into account, the pattern of evolution of deviatoric stress is given by the geometry of the irregularity considered for the speeds of 200 km/h, 250 km/h and 300 km/h. This fact is important, because shows that track irregularities coming from a theoretical profile (generated according to a good quality of the track) are more important to the track settlement, than any other factor: as speed of the train or stiffness variations in the different transitions.

5.12.4.2 Experimental irregularities

In this section, the influence of experimental irregularities coming from two different Railway Administrators, has been analyzed.

Firstly, an assessment with two different longitudinal level profiles provided by the Portuguese Administrator REFER, is made. Later, the same assessment is carried out but with two different longitudinal level profiles, provided by the Spanish Administrator ADIF. These experimental profiles, coming from REFER and ADIF, have been included in this study in order to have another complementary source of irregularity data to assess the dynamic interaction behaviour between the train and the track. The scope of this thesis is not to characterize the quality of both the Portuguese and the Spanish track sections from which the longitudinal level irregularity profiles have been taken.

As it was made in the previous cases, a dynamic interaction analysis is carried for the speeds of 200 km/h, 250 km/h and 300 km/h. Contact forces, vertical accelerations and evolution of deviatoric stress along the transition zone, the response parameters analyzed. From its analysis, a better understanding of the influence of real irregularity profiles, in both the train-track dynamic interaction process and the track degradation phenomena, is provided.

REFER profiles

These profiles are provided by the Portuguese railway Adminsitrator REFER. The irregularity data come from the monitoring operations, carried out by REFER, at the Portuguese railway line “Linha do Norte” were the transition zone is located.

A detail explanation of this Portuguese line can be found in Vale (2010).

The north line of 336 km length connects the cities of Lisbon and Oporto. It is an electrified double-track line built for a mix traffic of passengers and freight.

The maximum speed in this line is 220 km/h and it is reached by the Alfa-Pendular train.
5.12 Dynamic results

In Table 5.14, different sections of the line regarding to the maximum speed reached by the trains, are depicted.

Table 5.14: Number of km of the Portuguese railway line, according to the maximum speed reached from Vale (2010)

<table>
<thead>
<tr>
<th>Speed [km/h]</th>
<th>Track length [km]</th>
</tr>
</thead>
<tbody>
<tr>
<td>v</td>
<td>80</td>
</tr>
<tr>
<td>80 &lt; v</td>
<td>120</td>
</tr>
<tr>
<td>120 &lt; v</td>
<td>160</td>
</tr>
<tr>
<td>160 &lt; v</td>
<td>220</td>
</tr>
</tbody>
</table>

Geometry of the obtained profiles has been post-processed in order to adapt it to the mesh of the numerical model. The procedure to adapted the experimental profiles was already explained in Section 4.8.6.

Figure 5.39 illustrates the two irregularity profiles analyzed in this Chapter. These profiles come from the monitoring operations carried out by REFER in December 2007.

![Figure 5.39: Experimental irregularities profiles of the north line, REFER: (a) profile 1; (b) profile 2](image)

**Interaction forces**

Values of interaction forces are depicted in Figures 5.40-5.45. In this case, results are shown for both the time domain and the frequency domain. Values in frequency domain show the influence of track irregularities in the dynamic component of contact force between the wheel and the rail.
Figure 5.40: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_3$: REFER profile 1 and $v=300$ km/h: (a) time values; (b) frequency values

Figure 5.41: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_3$: REFER profile 1 and $v=250$ km/h: (a) time values; (b) frequency values

Figure 5.42: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_3$: REFER profile 1 and $v=200$ km/h: (a) time values; (b) frequency values
Figure 5.43: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_3$: REFER profile 2 and $v=300$ km/h: (a) time values; (b) frequency values

Figure 5.44: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_3$: REFER profile 2 and $v=250$ km/h: (a) time values; (b) frequency values

Figure 5.45: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_3$: REFER profile 2 and $v=200$ km/h: (a) time values; (b) frequency values
A comparison of the maximum contact forces between scenario $C_0$ and $C_3$ (with 2 different experimental profiles) is made in Table 5.15. It should be mentioned that, as in previous cases, increment in $\%$, for each profile of scenario $C_3$ is related to the corresponding value of scenario $C_0$ with the same speed. This comparative analysis takes into account also three different speeds of the train: 300 km/h, 250 km/h and 200 km/h.

As it can be seen, the magnitude of interaction forces has a maximum increment for the higher value of speed: 300 km/h. The increment in this case is 27 $\%$, a value much lower than 125 $\%$, given in the case of theoretical profiles generated according to DBAG requirements.

In this case, no loss of contact occurs in the dynamic interaction between the wheel and the rail along the transition zone. This indicates that these experimental profiles are not so aggressive, in terms of the dynamic interaction, than those theoretical profiles generated according to the DBGA specifications.

Analyzing the frequency values of interaction forces, parametric frequency induced by the distance of the sleepers, has a major influence with this type of irregularities. Moreover, the influence of track irregularities is lower the smaller the value of the speed of the train.

Table 5.15: Values of maximum contact forces between the wheel and the rail

<table>
<thead>
<tr>
<th>Speed [km/h]</th>
<th>Max. contact force [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td>Scenario $C_0$</td>
<td>Scenario $C_3$ experimental profiles REFER</td>
</tr>
<tr>
<td>profile 1</td>
<td>profile 2</td>
</tr>
<tr>
<td>---</td>
<td>---</td>
</tr>
<tr>
<td>200</td>
<td>86.0</td>
</tr>
<tr>
<td>250</td>
<td>86.3</td>
</tr>
<tr>
<td>300</td>
<td>86.7</td>
</tr>
</tbody>
</table>

**Acceleration on the axles**

At this point it is interesting to analyze the magnitude of the vertical accelerations on the axles of the bogie. This parameter provides an indicator of the track quality. As it can be seen in Figure 5.46, values of the maximum vertical acceleration on the axles are always within the Normal control level suggested by López Pita (2006).

**Deviatoric stress evolution in ballast layer**

In this case, the evolution of deviatoric stress along the transition zone presents, for the two different analyzed profiles, a similar pattern than scenarios $C_0$, $C_1$ and $C_2$, with higher values above the wedge-shaped backfill than above the embankment section and even higher values above the culvert than above the wedge, see Figure 5.47. The above mentioned pattern is more accentuated the lower the value of the train speed. The last proves that, for high speeds of the train, the influence of experimental
5.12 Dynamic results

Figure 5.46: Values of the maximum vertical acceleration on the axles of the bogie, for Scenario C3 (REFER profiles), given for different profiles and speeds

track irregularities is more important than the effect of change of stiffness given by the transition zone. It should also be mentioned that for lower values of the strain speed, the influence of the transition zone, on the dynamic interaction behaviour, is more accentuated than the effect of the experimental track irregularities.

**ADIF profiles**

These profiles have been kindly provided by the Spanish Railway Administrator ADIF and the Computational Mechanics Group of the School of Civil Engineering (UPM). The reason for the inclusion of these profiles in this work is to have another source of knowledge of experimental irregularities in a dynamic interaction analysis between the train and the track. In this case, profiles belonging to a Spanish conventional railway line, are used. The selected ADIF profiles comply with the requirements already explained in Section 4.8.8.

Irregularity profiles are coming from the monitoring operations carried out by ADIF in the Mediterranean HSL. A detailed explanation of this Spanish HSL can be found in Nguyen (2013).

The track section, to which the monitoring profiles belong to, is Les Valls-Chilches. This section is included within the Mediterranean Spanish high-speed line Valencia-Sant Vicent de Calders.

Les Valls-Chilches section has a total length of 8.2 km. This lines was remodeled according to the Railway Transport Plan of 1987. It is a double, electrified track suitable for mix traffic (passengers and freight). The maximum speed reached in this section of the Mediterranean HSL is 220 km/h.

As in the previous case of longitudinal level irregularities provided by REFER, geometry of the monitoring profiles given by ADIF, has been post-processed in order to adapt it to the mesh of the numerical model.
Figure 5.47: Deviatoric stress evolution in the top of the ballast layer along the transition zone:
(a) v=200km/h; (b) v=250km/h; (c) v=300km/h

Figure 5.48 shows the two longitudinal level irregularity profiles, coming from the monitoring works of ADIF, that have been considered in the current work.

**Interaction forces**

A comparison of the maximum contact forces between scenario $C_0$ and $C_3$ (with 2 different experimental profiles) is presented in Table 5.16. Note that, as in previous cases, increment in %, for each profile of scenario $C_3$ is related to the corresponding value of scenario $C_0$ with the same speed. This comparative analysis takes into account also three different speeds of the train: 300 km/h, 250 km/h and 200 km/h.

As it can be seen, the magnitude of interaction forces has a maximum increment for the higher values of speed: 300 km/h. The increment in this case only a 7.8% higher than values of scenario $C_0$, a value much more softer than those of 125% given in the case of theoretical profiles (DBAG) and 27% given in experimental profiles given by REFER.

The above mentioned results of contact force magnitude for experimental irregularities given by ADIF show a very good behaviour of the dynamic interaction along the transition zone. As it can be seen by analyzing the frequency values, track ir-

<table>
<thead>
<tr>
<th>Position [m]</th>
<th>Deviatoric stress [kPa]</th>
<th>$C_0$</th>
<th>$C_{3p1}$</th>
<th>$C_{3p2}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>-10</td>
<td>-70</td>
<td>Red</td>
<td>Blue</td>
<td>Green</td>
</tr>
<tr>
<td>0</td>
<td>-60</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>10</td>
<td>-50</td>
<td></td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

...
regularity influence, given by the value of the amplitude is much lower than the case of experimental irregularities coming from REFER. This indicates that the influence of the frequency content of the rail irregularity have a higher effect on the contact forces than the amplitude of the irregularity.

As it can be seen from the frequency values of interaction forces, the parametric frequency induced by the distance between the sleepers, has even a major influence with this type of irregularities given by ADIF. Finally, it is worth noting that the influence of track irregularities is lower the smaller the value of the speed of the train.

**Acceleration on the axles**

In this case, values of maximum vertical acceleration on the axles of the bogie are, for both profiles, far below the limit between the Normal control level and the Internal control level suggested by López Pita (2006).

Values of maximum vertical acceleration are almost similar for the two experiemen-
5. Short-term analysis

Figure 5.50: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_3$: ADIF profile 1 and $v=250 \text{ km/h}$: (a) time values; (b) frequency values

Figure 5.51: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_3$: ADIF profile 1 and $v=200 \text{ km/h}$: (a) time values; (b) frequency values

tal profiles as well as for all values of speed considered, see Figure 5.55. This fact highlights the good quality of the profiles analyzed.

**Deviatoric stress evolution in ballast layer**

Values of deviatoric stress evolution along the transition zone are depicted in Figure 5.56.

In this case, the evolution of deviatoric stress along the transition zone presents, for the two different analyzed profiles, a similar pattern than scenarios $C_0$, $C_1$, $C_2$ and $C_3$ (with the experimental longitudinal level irregularity profiles from REFER), with higher values above the t-wedge-shaped backfill than above the embankment section and even higher values above the culvert than over the wedge. The above mentioned pattern is even more similar in this case to those given for scenarios $C_0$, $C_1$ and $C_2$. 

180
5.13 Effect of differential settlements in the transition zone

5.13.1 Introduction

Analyzing different designs of transition zones, it is remarkably how some Railway Administrators presents solutions in which treated materials or wedge-shaped backfill do not reached the base of the wall or abutment, see Figure 5.57(a).

In other cases, a bad compacting level of the backfill may occur because of different factors: use of complex geometries of the abutment, the presence of inappropriate levels of the material humidity, human errors in the tamping procedure, Paixão (2014).

Every one of the previous described cases may contribute to cause differential settlements between the soil and the structure. Below, some examples of this phenomena are depicted:
5. Short-term analysis

Figure 5.54: Interaction forces between the wheel and the rail for scenarios $C_0$ and $C_3$: ADIF profile 2 and $v=200\,$km/h: (a) time values; (b) frequency values

Table 5.16: Values of maximum contact forces between the wheel and the rail

<table>
<thead>
<tr>
<th>Speed [km/h]</th>
<th>Max. contact force [kN]</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Scenario $C_0$</td>
</tr>
<tr>
<td></td>
<td></td>
</tr>
<tr>
<td>200</td>
<td>86.0</td>
</tr>
<tr>
<td>250</td>
<td>86.3</td>
</tr>
<tr>
<td>300</td>
<td>86.7</td>
</tr>
</tbody>
</table>

The transition zone, analyzed in the current Chapter due to the presence of a culvert, could be also a good example of this type of solutions.

As it can be seen in Figure 5.57 (a), the wedge-shaped backfill does not reach the top of the culvert foundation soil. This fact may induce a more remarkable differential settlement in the vicinity of the culvert if a settlement of the embankment occurs.

Importance of differential settlements at transition zones and its relation with the degradation process, were already explained in Chapter 2. Having this in mind, it is decided to assess the effect of differential settlements in the dynamic train-track interaction at a transition zone.

5.13.2 Introduction of embankment settlements in the bidimensional model

In this Section, a parametric study with three different values of embankment settlement has been developed. Selected values for embankment settlements, $s$ are: 0.5 cm, 1.0 cm and 1.5 cm. Two main considerations have to be mentioned at this point: Settlements are entered into the bidimensional model as imposed vertical displacements in the nodes on the top of the embankment layer. On the other hand, distribution
of vertical settlements along the transition zone is made according with the layout of Figure 5.58, in which it is intended to related the value of the settlement of the embankment layer with its height along the transition zone. For this purpose, four different zones are defined. In each zone a proportional value of the embankment settlement is taken into account.

Below, the value of each settlement is given depending on the zone: A, B, C or D.

- Zone A: settlement = value of the considered settlement, $s$.
- Zone B: settlement = $(2/3) s$
- Zone C: settlement = $(1/2) s$
- Zone D: settlement = $(1/3) s$

In Figure 5.59 it can be clearly seen the value of vertical displacements, when a maximum settlement $s = 0.015 \text{ m}$ is entered in the embankment.

A detail of vertical displacements in the embankment on the left side is illustrated in Figure 5.60.

The fact of using contact elements in the different interfaces, leads to the presence of a different field of stresses and displacements in the surrounding of the culvert. An example of this fact may be observed in Figure 5.61, in which a comparison of scenarios $C_0$ and $C_2$ is made, when a certain settlement is entered in the embankment layer.

Introducing a settlement of $s = 0.015 \text{ m}$ the global deformed mesh of the transition zone, for scenario $C_2$ is illustrated in Figure 5.62.

Within this part, a detail study of the introduction of embankment settlement in the numerical model, has been carried out. A particular way of entered vertical displacements in the nodes on the top of the embankment layer allows to simulate the settlements explained and obtained in Section 4.9. Moreover, the approach in
5. Short-term analysis

![Graphs showing deviatoric stress evolution](image)

Figure 5.56: Deviatoric stress evolution in the top of the ballast layer along the transition zone:
(a) \( v = 200 \text{ km/h} \); (b) \( v = 250 \text{ km/h} \); (c) \( v = 300 \text{ km/h} \)

which embankment settlements are entered allows to bear in mind the effect of the embankment height: the higher the embankment height, the higher the value of the settlement.

As was seen, the use of contact elements in the different interfaces allows to simulate an independent behaviour between the surrounding soil and the culvert in terms of both the stress and displacement fields. This fact is important to simulate differential settlements in the vicinity of the structure as well as the presence of hanging sleepers, as it can be noted from Figure 5.62.

### 5.13.3 Dynamic results

In order to have a better understanding of the influence of contact elements in the dynamic behaviour of the train-track system, a simple comparison between the scenarios: \( C_0 \), \( C_1 \) and \( C_2 \) was developed in Section 5.12.3. This comparison was made taking into account the effect of the wheel-rail interaction force when a train is moving along the transition zone. This study was carried out for three different values of train speed: 300 km/h, 250 km/h and 200 km/h.
In the above mentioned assessment no settlements were considered. The purpose was only, to capture the effect of contact elements in the dynamic interaction analysis.

It is interesting to develop a dynamic interaction analysis in which, through the use of contact elements (scenario $C_2$), different settlements of the embankments on both sides of the culvert are considered.

Values of these settlements are: 0.5 cm, 1.0 cm and 1.5 cm and its distribution along the transition zone is illustrated in Figure 5.58. In this case, two different values of the train speed are taken into account: 300 km/h and 200 km/h.

As it was made in foregoing sections, some important response parameters of the dynamic interaction analysis are here presented as: contact forces and vertical accelerations in the axles of the bogie. Furthermore in this case, due to the fact that permanent deformations of the embankments have been taken into account, other parameters linked to the hanging sleepers phenomena are also here presented. Such parameters are: evolution of the gap existing between the sleepers and the ballast layer in the vicinity of the culvert and temporal values of vertical displacements and
5. Short-term analysis

![Figure 5.58: Different zones depending on the values of embankment settlement considered](image)

![Figure 5.59: Values of vertical displacements when a maximum settlement $s=0.015$ m is entered in the embankment](image)

accelerations of the sleepers located nearby the culvert.

**Interaction forces**

Below, interaction forces between the wheel and the rail are shown. In all cases diagrams present two different values of interaction forces for scenario $C_2$. In the first one, no settlements of the embankment have been considered $s = 0.0$ cm. In the second one, a certain settlement that can be: $s = 0.5$ cm, $s = 1.0$ cm or $s = 1.5$ cm, has been considered.

As it can be seen in Figures 5.63 and 5.64, the value of interaction forces is clearly influenced by the value of the settlement taken into account. This influence is even more remarkable above the embankment/wedge transition and above the soil-cement wedge/culvert transition.

It must be noted that, unsurprisingly, the interaction force increase with the settlements. The maximum value of contact force is given, for all cases, when the first axle of the bogie is placed right after the culvert.

The influence of the speed is also very important as it can seen in Figures 5.63 and 5.64. The greater the values of the speed, the higher the magnitude of the contact
5.13 Effect of differential settlements in the transition zone

Figure 5.60: Progressive value of settlements along the embankment: (a) value of vertical settlements and (b) undeformed + deformed mesh. Scale factor: 50

force, especially in the vicinity of the culvert where the effect of the differential settlement is more pronounced.

Table 5.17 shows the maximum values of dynamic interaction force as well as the percentage, for the different settlement values considered in Scenario C₂.

Note that the increment in %, for each settlement of scenario C₂ is related to the corresponding value of scenario C₂, in which settlement is s = 0.0 cm, with the same speed.

Table 5.17: Values of maximum contact forces between the wheel and the rail, scenario C₂

<table>
<thead>
<tr>
<th>Speed (km/h)</th>
<th>Interaction forces (kN)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Scenario C₂</td>
</tr>
<tr>
<td></td>
<td>s=0.0 cm</td>
</tr>
<tr>
<td>200</td>
<td>86.7</td>
</tr>
<tr>
<td>300</td>
<td>89.3</td>
</tr>
</tbody>
</table>

At this point it is interesting to analyze the deformed mesh of the components of the numerical model for two different situations: maximum value of contact force (point A) and minimum value of contact force (point B), see Figure 5.65. Both of them, located in the vicinity of the culvert. The maximum value of contact force corresponds to a load state, while the minimum value of contact force is due to an unload state between the wheel and the rail.

For this evaluation, contact force curve from Figure 5.63(b) is taken as a reference.
5. Short-term analysis

Figure 5.61: Comparison of vertical displacements in the vicinity of the culvert: (a) without contact elements in the interface, scenario \( C_0 \); (b) using contact elements in the interface, scenario \( C_2 \)

Figure 5.62: Global deformed mesh of the transition zone for scenario \( C_2 \) when a settlement of 0.015 is entered in the embankment. Scale 75:1

For this particular case, \( v = 300 \text{ km/h} \) and the settlement taken into account was \( s = 1.0 \text{ cm} \).

Figures 5.66 and 5.67 illustrate the instant, in which the maximum and the minimum values of contact forces respectively, takes place.

**Hanging sleepers development**

Evolution of hanging sleepers, when settlements are entered, is depicted in Figure 5.68. Hanging sleepers phenomenon is achieved when a gap or void exists between the base of the sleeper and the top of the ballast layer. A total number of nineteen sleepers have been chosen in order to illustrate this phenomenon: eight sleepers right before the culvert (T1-T8), three sleepers right over the culvert (T9-T11) and finally, eight sleepers located after the culvert (T12-T19).

As can be seen in Figure 5.68, the higher the value of the entered settlement, the the bigger the value of the void developed under the sleepers. This effect is specially accused in the sleepers located before and after the culvert. For example, sleepers T1-T4, T16-T19 that do not have any void beneath for \( s = 0.5 \text{ cm} \). For a settlement \( s = 1.0 \text{ cm} \), sleepers T4 and T16 have a minimum void that is more evident for \( s = 1.5 \text{ cm} \). Even sleeper T10, located in the middle of the culvert, changes its contact status with the ballast layer when the settlement of the embankment increases from \( s = 0.5 \text{ cm} \) to \( s = 1.5 \text{ cm} \).
5.13 Effect of differential settlements in the transition zone

Figure 5.63: Contact forces, for different values of settlements: (a) s=0.5 cm; (b) s=1.0 cm; (c) s=1.5 cm. Speed of the train: 300 km/h

Accelerations on the axles

In Figure 5.69, maximum values of acceleration on the axles of the bogie are depicted. These acceleration values are compared with the track quality limits provided by López Pita (2006).

As it can be seen, the recommended value of vertical acceleration on the axles of $30 \, m/s^2$ given by the mentioned author, is exceed for all cases, except the one in which $s = 0.5$ cm and speed, $v = 200 \, km/h$.

Values of maximum acceleration on the axles, provide an idea of the negative effect that permanent settlement of the embankments have in the dynamic behaviour of the train-track interaction. As one would expect, the negative effect increases as the value of both the embankment settlements and speed of the train get higher and higher.

Displacements of the wheels

Since no loss of contact occurs between the wheel and the rail, vertical displacement of the wheels adjusts to the vertical displacement of the rail in the contact point. Thus, vertical displacements of the wheel provides in each case, the evolution of the
vertical geometry of the track as the train passes along the transition zone. In this case this parameter is analyzed for both the front and back axles of the bogie. As in the previous case, an assessment of vertical displacements of the wheels is carried out, taking into account different values of settlements and train speeds.

As it can be seen, the transition of the wheel of the first axle is softer the lowest the value of settlement in the embankments. For every settlements, the vertical displacement of the wheel is almost zero, when the first axle is over the culvert, as it can be seen in Figure 5.70. This is due to the fact that the section of the track, coincident with the location of the culvert, does not experiment any settlement. As a consequence, the more sudden the transition in the vicinity of the culvert (due to the magnitude of the settlements) the greater the value of contact forces between the track and the train.

At this point it is interesting, to assess the behaviour of the two axles of the bogie, in terms of vertical displacements, when the bogie moves along the transition zone, closer to the culvert.

Analyzing Figure 5.71, it can be seen that vertical displacement for both the front and back axles of the bogie are almost similar when it moves over the culvert, for the
5.13 Effect of differential settlements in the transition zone

![Graph showing contact force diagram with position and contact force values](image)

**Figure 5.65:** Contact force diagram with the maximum and minimum peak values, A and B respectively.

![Images showing time step and deformed mesh](image)

**Figure 5.66:** Time step in which maximum contact force is reached, point A: (a) location of the first axle; (b) deformed mesh of the system.

speeds of 200 km/h and 300 km/h and a settlement \( s = 0.5 \text{ cm} \).

In Figure 5.72, when the settlement taken into account is \( s=1.0 \text{ cm} \), there is a slight perturbation in the back axle that is coincident with the location of the front axle right above the beginning of the structure. This perturbation is more evident for a speed of 300 km/h than for a speed of 200 km/h.

Finally, in Figure 5.73, when the settlement that has been considered is \( s = 1.5 \text{ cm} \), the above mentioned perturbation is still more evident and it starts to be significant for both speeds of the train: 200 km/h and 300 km/h.

**Vertical displacements of the sleepers**

Below, a particular assessment of vertical displacements an accelerations in different sleepers of the transition zone is carried out. The sleepers chosen to developed such an
5. Short-term analysis

Figure 5.67: Time step in which minimum contact force is reached, point B: (a) location of the first axle; (b) deformed mesh of the system

analysis are depicted in Figure 5.74. According to Figure 5.68, the above mentioned selection is made taking into account: a sleeper with a high void (sleeper T5), a sleeper with a medium void (sleeper T3) and a sleeper practically without any void (sleeper T7). However this last sleeper T7 may experiment a void under it when settlements around 1.5 cm are considered, see Figure 5.68(c).

From Figures 5.75 and 5.76, it can be concluded that vertical displacement of sleeper T7 is slightly higher in the case in which speed of 300 km/h is taking into account. For every cases, displacements of sleeper T7 (located in the middle of the culvert) are lower than displacements observed in sleepers T3 and T5.

T5 (located right before the culvert) is the sleeper that has the highest values of vertical displacements, while for settlements of \( s = 1.0 \) cm and \( s = 1.5 \) cm, vertical displacements in sleeper T3 tend to approach to the displacements in sleeper T5. This fact indicates that the effect of hanging sleepers is more accused, as the values of embankment settlement increases.

For every cases, an increment of the train speed means an increment of the vertical displacements value in the sleepers. Although, for the analyzed range, this effect have a less impact, on the vertical displacement magnitude than the effect of consider increasing values of the embankment settlement.

**Vertical accelerations of the sleepers**

From Figures 5.77, 5.78 and 5.79, it can be observed that in contrast to the displacements of the sleepers case, here the effect of the velocity, influence clearly the value of accelerations. For the same value of the considered settlement, the higher the value of the speed, the bigger the difference of vertical acceleration in the sleeper. Particularly, for sleeper T7 and for settlements in the range 1.0-1.5 cm, vertical accelerations suffer an important amplitude when the speed of the train changes from 200 km/h to
300 km/h. This increment is due to two main reasons: From settlements of 1.0 cm, sleeper T7 becomes a voidedsleepers, as it can be seen in Figures 5.68 (b) and 5.68 (c). On the other hand, the increased effect of the speed makes that a peak value of acceleration take place in sleeper T7 when the speed increases from 200 km/h to 300 km/h, as can be seen in Figures 5.78(a) and 5.79(a).

**5.14 Conclusions**

This Chapter has presented a complete assessment of the dynamic train-track interaction at a particular transition zone of the Portuguese North Railway line. The analysis performed have considered the short-term dynamic behaviour for both the train and the track, when a bogie of an Eurostar train moves along the transition zone at different speeds.

In this Chapter different scenarios were considered. These scenarios have the goal of analyzing the effect of different variables in the general dynamic interaction between the train and the track at transition zones. The above mentioned variables are: Frictional behaviour of the soil-structure interfaces, the track irregularities (longitudinal
level profiles) effect and the differential settlements phenomena on both sides of the structure.

Another purpose of this Chapter was to analyze the main variables that may have a significant influence in a long-term behaviour of the transition zone. To accomplish this objective, a short-term dynamic interaction analysis has been done for different values of train speed: 200 km/h, 250 km/h and 300 km/h.

Regarding the effect of the frictional behaviour of different interfaces: soil/structure, sub-ballast/sub-ballast and ballast/ballast, it was concluded that the increase of contact forces between the wheel and the rail are only 3.73 % higher for a speed of 300 km/h when frictional behaviour is considered in the interfaces. Vertical acceleration in the axles do not experiment significant changes.

In all cases: scenario $C_0$, $C_1$ and $C_2$, values of deviatoric stress along the transition zone have a defined pattern, in which maximum values are above the culvert and decrease gradually, when we move away from the culvert.

In the case of track irregularities, the influence in the train-track dynamic interaction is considerably larger. Concerning this point, a distinction between the influence of artificial and experimental profiles has to be made. Interaction analysis results, in terms of wheel-rail contact forces, acceleration on the axles of the train and deviatoric stress along the transition zone on top of the ballast layer, are clearly higher when artificial profiles are considered if compared with results of experimental profiles. This was expected because, on one hand artificial profiles are generated according to a power spectral density function, that usually contains a wide range of frequencies. Some of these frequencies may have a great influence in the dynamic interaction behaviour by amplifying the response of the dynamic analysis.

On the other hand, artificial profiles provided by different railway Administrators are generated in order to have into account the worse case regarding to safety and
5.14 Conclusions

![Graphs showing vertical displacement of the wheel for different settlement values](image)

Figure 5.70: Vertical displacement of the wheel $v=300\text{km/h}$ (a) $s=0.5\text{cm}$, (b) $s=1.0\text{cm}$ and (c) $s=1.5\text{cm}$

...passengers comfort conditions.

Notwithstanding, results for artificial profiles are far above the dynamic interaction results obtained with experimental profiles. For speeds of the order of $300\text{km/h}$, values of wheel-rail interaction forces are too high and may, in some cases, occur loss of contact between the train and the track.

In the extreme case, dynamic interaction values for the experimental profiles coming from ADIF, are even lower than results coming from REFER profiles. Acceleration on the axles are far below the limit of $30\text{m/s}^2$, López Pita (2006).

Finally, the influence of differential settlements in the vicinity of the culvert were analyzed. It can be concluded that contact elements are fundamental to simulate the effect of differential settlements between two sections of the model.

The value of contact forces increases as the value of the entered embankment settlement, on both sides of the culvert, is higher. Moreover, for each value of settlement analyzed, the effect of train speed is also very important, increasing notably the value of contact force as the speed changes from $200\text{km/h}$ to $300\text{km/h}$.

Values of vertical acceleration on the axles, suffer also an important variation when...
5. Short-term analysis

Figure 5.71: Vertical displacement of the wheels when the settlement is \( s = 0.5 \) cm: (a) \( v = 300 \) km/h; (b) \( v = 200 \) km/h

Figure 5.72: Vertical displacement of the wheels when the settlement is \( s = 1.0 \) cm: (a) \( v = 300 \) km/h; (b) \( v = 200 \) km/h

the train moves along the transition zone. Almost for every analyzed cases, values of acceleration exceeded the limit of 30 \( m/s^2 \). This indicates how important is the fact of consider the effect of differential settlements of the embankment in a dynamic interaction analysis at transition zones.
Figure 5.73: Vertical displacement of the wheels when the settlement is s=1.5 cm: (a) v=300 km/h; (b) v=200 km/h

Figure 5.74: Location of sleepers T3, T5 and T7 along the transition zone
5. Short-term analysis

Figure 5.75: Vertical displacements of the sleepers: T3, T5, and T7 for a speed \( v = 300 \text{ km/h} \): (a) \( s = 0.5 \text{ cm} \); (b) \( s = 1.0 \text{ cm} \) and (c) \( s = 1.5 \text{ cm} \)
Figure 5.76: Vertical displacements of the sleepers: T3, T5 and T7 for a speed $v=200 \text{ km/h}$: (a) $s=0.5 \text{ cm}$; (b) $s=1.0 \text{ cm}$ and (c) $s=1.5 \text{ cm}$

Figure 5.77: Vertical acceleration of the sleepers when the settlement is $s=0.5 \text{ cm}$: (a) $v=300 \text{ km/h}$; (b) $v=200 \text{ km/h}$
5. Short-term analysis

Figure 5.78: Vertical acceleration of the sleepers when the settlement is $s=1.0\,\text{cm}$: (a) $v=300\,\text{km/h}$; (b) $v=200\,\text{km/h}$

Figure 5.79: Vertical acceleration of the sleepers when the settlement is $s=1.5\,\text{cm}$: (a) $v=300\,\text{km/h}$; (b) $v=200\,\text{km/h}$


6 Long-term simulation

6.1 Introduction

This section presents the results of long-term assessment of the transition zone described in Chapter 5 in which the effects of track degradation have been considered. Such analysis is carried out with the degradation program already described in Section 3.6.

The first part of the current Chapter focus on the influence of two parameters, in the long-term behaviour of the transition zone. The first of these parameters is the initial number of cycles to discount, \( N_i \). This allows to take into account the ballast stabilization process. The second one is the ballast porosity \( n_p \), which varies according to the degree of ballast compaction.

A summary of the characteristics of the long-term simulations is then presented. This part is related to the Section 3.6.4 in which data inputs of the “LongTermSim” program are described.

Finally the long-term analysis has been performed considering several different scenarios. Particularly, the scenarios considered in this chapter are, Scenario \( C_0 \), and a mix of Scenarios \( C_1 \) and \( C_3 \) that is called scenario \( C_4 \). The option for considering scenario \( C_4 \), has been motivated by the need to account both track irregularities (longitudinal level profiles) and frictional Coulomb law, in the interface between the soil and the walls of the culvert. Exclusion of scenario \( C_2 \) of this Chapter was motivated by the fact that this scenario had as an objective to evaluate the effect of the settlements in the embankments on both sides of the culvert. This problem was already evaluated in Section 5.13. Furthermore, implementation of scenario \( C_2 \) in the “LongTermSim” program is still being developed in the Faculty of Engineering of the University of Porto, and is expected to be concluded in a very near by future.

In the current Chapter, only settlement of the ballast layer has been considered, analyzing the influence of each scenario in the global behaviour of the transition zone, described in Chapter 5, in a long-term assessment.

Track response parameters are analyzed in order to have a better understanding of the global behaviour of the track at the transition zone when subjected to multiple loading cycles. These response parameters are: the contact force between the wheel and the rail, acceleration in the axles, vertical displacements of the wheels, settlement
of the ballast layer, evolution of the void under the sleeper with the number of loading cycles.

Using a similar procedure to the one in Chapter 5, two control parameters were considered in this study. Firstly, the value of the contact forces, in which a negative value of this parameter indicates a loss of contact between the wheel and the rail. Secondly, the accelerations on the axles of the bogie, in which values higher than $30\text{m/s}^2$ indicate that the normal control level of the track quality has been exceeded.

6.2 Sensitivity analysis of parameters $N_i$ and $n_p$

In this section, the influence of two parameters in the global degradation behaviour of the track at the transition zone, is assessed. These parameters are included in the mathematical formulae used to obtained vertical deformations in the ballast layer:

$$N = 1 \quad C \log \frac{N+N_i}{N_i}$$

where $1$ refers to the permanent deformation of the ballast layer due to the first loading cycle. The value of this parameter is given by $1 = 0.082(100n_p - 38.2) \left( \frac{1}{3} \right)^2$ as it was duly mentioned in Section 3.6.3.

The first of the above mentioned parameters is the number of loading cycles in the first phase in which the ballast layers is stabilized, $N_i$. The second parameter is the ballast layer porosity parameter that varies in the range $0.4 - 0.5$ depending on the degree of compaction of the ballast layer. According to ORE (1970) and Ionescu (2004) values for $N_i = 100000$ cycles represent a good approach to take into account the ballast stabilization process. Both ORE (1970) and Ionescu (2004) suggest values for $n_p = 0.4$, if a good compaction level of the ballast layer has been assumed.

With the objective of comparing results of ballast settlement, for different values of $N_i$ and $n_p$, a degradation analysis has been carried out, simulating the passage of the same Eurostar bogie that was considered in Chapter 5. The speed of the train considered in this case was $v = 300\text{km/h}$.

The number of simulations considered in this analysis was $N = 5$. It has been considered that each simulation corresponds with a total number of 100000 passages of the bogie, along the transition zone.

Three different cases are considered in this analysis, depending on the value of $N_i$ and $n_p$ parameters:

- Case 1: $N_i = 50000$ and $n_p = 0.40$
- Case 2: $N_i = 100000$ and $n_p = 0.40$
- Case 3: $N_i = 100000$ and $n_p = 0.41$

Values of ballast settlement are depicted, for each one of the previous cases, in Figure 6.1.

Comparing diagrams from Figure 6.1, it can be concluded:
6.2 Sensitivity analysis of parameters $N_i$ and $n_p$

![Graphs showing settlement vs position for different values of $N_i$ and $n_p$](image)

Figure 6.1: Ballast settlement for different values of $N_i$ and $n_p$ parameters: (a) Case 1; (b) Case 2; (c) Case 3

In cases 1 and 2, with the same value of $n_p$ and different values for $N_i$, ballast settlements are higher the lower the value of loading cycles taken into account in the first phase. This underlines the importance of taken into consideration a proper value of $N_i$ parameter so that results adjust to reality of the problem.

In cases 2 and 3, with the same value of $N_i$ and different values of $n_p$, ballast of settlements are higher the greater the value of ballast porosity. It should be remembered that higher values of the porosity parameter means a lower quality in the compaction of the ballast layer. Hence, it is reasonable that settlements of the ballast layer increased as the compaction quality decreases.

The choice of these parameters must be carefully done and adapted to each case of study. These values have a remarkable influence on the value of the ballast settlement as well as on the settlement ratio of the track.
6.3 Characteristics of the long-term simulation

In the current section an assessment of the long-term behaviour of a ballasted track, when a bogie of the Eurostar train is moving along the transition zone described in Chapter 5, is presented.

To carry out such an analysis, it is necessary to establish some characteristics of the long-term assessment, that allow to capture and identify the main features of the degradation phenomena.

To this end, it is decided to carry out a total number of 20 simulations. Each simulation of the degradation process, correspond to 100000 loading cycles of the bogie of the Eurostar train.

Characteristics of the bogie of the Eurostar train were already described in Section 5.10.

To take into account the effect of the stabilization of the ballast layer, it is assumed that $N_i = 100000$ cycles. It is also assumed that the degree of compaction of the ballast layer is good, hence the value of ballast layer porosity is $n_p = 0.4$.

In this analysis, two different velocities of the train are considered $v = 300$ km/h and $v = 200$ km/h. It has been decided not included the speed of $v = 250$ km/h in this analysis, mainly due to the low difference existing between this speed and the speeds of 200 km/h and 300 km/h in the dynamic analysis developed in Chapter 5.

Three different scenarios are considered in the degradation phenomenon: Scenario $C_0$, in which neither the contact elements in the soil-structure interface nor the irregularities of the rail are taken into consideration. Scenario $C_1$, in which contact elements between the soil and the structure are considered, in the interface, to model the Coulomb frictional law. A new scenario $C_4$ is included in this study, as it was said before. The latter is the result of adding scenario $C_3$, in which track irregularities are considered, to scenario $C_1$.

In the case of track irregularities, two different type of profiles are here considered: the theoretical profile $p_3GQ$, generated according to the requirements of the DBAG for a good quality of the track. This profile was already analyzed in Chapter 5, for a short-term analysis.

On the other hand experimental profiles are also analyzed in this study. These profiles come from the monitoring operations carried out by the Portuguese and the Spanish Railway Administrators, REFER and ADIF respectively.

Profiles here analyzed are: profile 1: $p1$ and profile 2: $p2$ from REFER and profile 1: $p1$ from ADIF.

In Table 6.1, a summary of the irregularity profiles considered in this study, is summarized.
Table 6.1: Summary of the irregularity profiles considered in the long-term simulation

<table>
<thead>
<tr>
<th>Irregularity profiles</th>
<th>Theoretical</th>
<th>Experimental</th>
</tr>
</thead>
<tbody>
<tr>
<td>DBAG (Germany)</td>
<td>p3CQ</td>
<td></td>
</tr>
<tr>
<td>REFER (Portugal)</td>
<td>p1</td>
<td></td>
</tr>
<tr>
<td>ADIF (Spain)</td>
<td>p2</td>
<td></td>
</tr>
</tbody>
</table>

### 6.4 Long-term analysis

#### 6.4.1 Scenario $C_0$

The methodology used, to present the long-term results, will be the same for every scenarios. As already indicated, the variables of the long-term dynamic analysis that are here analyzed are: Settlement of the ballast layer, given by the vertical displacements of the nodes located on the top of the ballast layer. Vertical displacement of the wheel along the transition zone, given by the the dynamic displacements experimented by the wheel of the first axle of the bogie when this is moving along the transition zone. Interaction forces between the wheel and the rail. Vertical acceleration experimented by the axles of the bogie, given by the maximum value of the dynamic acceleration existing when the bogie is moving along the transition zone. Values for both interaction forces and vertical acceleration of the axles will be shown every 500000 cycles. Finally, the evolution of the hanging sleepers phenomenon, in the simulation analysis, is shown. To assess the effect of the speed of the train in the degradation analysis, all the cases have been analyzed for two different speeds: $v = 300 \text{ km/h}$ and $v = 200 \text{ km/h}$, being the total number of simulations equal to 20, which implies, taking into account 2000000 loading cycles in the degradation process.

#### Ballast Settlement

![Figure 6.2: Ballast settlement for scenario $C_0$: (a) $v=300 \text{ km/h}$; (b) $v=200 \text{ km/h}$](image-url)
Figure 6.2 shows the evolution of the ballast settlement for different speeds. As it can be seen, values of settlement lightly higher for the speed of 300 km/h. In both cases, the settlement ratio decreases as the number of simulations grows.

For all simulations, the higher values of the ballast settlement are located right under the culvert, decreasing gradually from the section under the treated soil wedge to the embankment sections.

**Vertical displacement of the wheel**

![Graph showing vertical displacements](image)

As it can be seen in Figure 6.3, in this scenario, vertical displacements of the wheel present a shape pattern very similar, along the transition zone, to that obtained for the ballast settlement.

The higher the speed of the train, the greater the value of the vertical displacements experimented by the wheels.

**Interaction forces**

Contact forces evolution, along the transition zone are depicted in Figure 6.4. Variation of interaction forces is more evident over the track sections where there is a change in the stiffness. Such variation is also more important the higher the cycle analyzed.

It is important to note also that for every simulations analyzed, contact forces are always positive, which means that there is always contact between the wheels and the rail.

**Evolution of maximum accelerations on the axles**

Evolution of the maximum acceleration on the axles is depicted in Figure 6.5. As it can be seen for every cycles, the value of the maximum acceleration is far below the acceleration limit between the internal and the normal control levels: $30 \text{ m/s}^2$. Last
6.4 Long-term analysis

Figure 6.4: Interaction forces between the wheel and the rail in the first axle of the bogie, for scenario $C_0$: (a) $v=300$ km/h; (b) $v=200$ km/h

Figure 6.5: Evolution of the maximum acceleration on the axles, for scenario $C_0$: (a) $v=300$ km/h; (b) $v=200$ km/h

data, combined with the fact that in every cycles no lost of contact occurs between the wheel and the rail, proved the good dynamic behaviour of the transition zone when the passage of an Eurostar train is simulated.

**Evolution of hanging sleepers**

Figures 6.6 and 6.7 show the evolution of the hanging sleepers phenomenon as the number of loading cycles increases. Location of the voids under the sleepers, is located in the zones where a significant variation of interaction forces occurs: embankment-treated soil transition and treated soil-culvert transition. See Figure 6.4. The magnitude of the voids is small, in this scenario it is lower than 0.15 mm for 2000000 cycles and a speed of 300 km/h.

Differences of hanging sleepers evolution is notably depending on the speed of the train. For a speed of 200 km/h hanging sleepers are located for every cycles in the zones where exists a variation of stiffness. On the other hand, for a speed of 300 km/h,
hanging sleepers phenomena is located in the zones where exists a variation of stiffness, in the first cycles, and it arises in the adjacent zones for a higher number of loading cycles.

6.4.2 Scenario $C_4$

In this case and as it has already been explained, both track irregularities and contact elements in the soil-structure interface are considered.

Regarding to track irregularities, this study distinguish between theoretical profiles and the experimental profiles. Theoretical profiles are generated according requirements provided by the German Railway Administrator DB. On the other hand, experimental profiles are coming from two different sources: monitoring of the Portuguese railway line, provided by REFER and monitoring of a Spanish conventional railway line, provided by ADIF.
In this section a total number of four different irregularity profiles are analyzed: one (theoretical) from DB, two (experimental) from REFER and the last one (experimental) from ADIF. Two different speeds of the train were considered: 300 km/h and 200 km/h for each analyzed profile.

6.4.2.1 Theoretical profiles DBAG

In this case, results are presented only for a speed of 200 km/h. The speed of 300 km/h was considered excessive for this type of profiles as it was seen in the analysis carried out in Chapter 5.

Ballast Settlement

Settlements of the ballast are depicted in Figure 6.8. In this case, values of ballast settlement are clearly influenced by the irregularities of the track considered. The
maximum value of the settlement is located over the wedge-shaped backfill section and not over the culvert as in the case analyzed in scenario $C_0$. The maximum value of the ballast settlement changes also when track irregularities are considered. It varies from $3.80\,\text{mm}$ in scenario $C_0$ and $v = 200\,\text{km/h}$ to $11.05\,\text{mm}$ in scenario $C_4$ (theoretical profile) and the same speed.

**Vertical displacement of the wheel**

Figure 6.9 shows the evolution irregularity of the rail as the number of the cycles increases. In black colour, the initial and last cycles are depicted.

**Interaction forces and Evolution of maximum accelerations on the axles**

Figure 6.10 shows the dynamic interaction evolution in terms of both contact forces and maximum value of vertical accelerations on the axles of the bogie. As it can be
seen, values of contact forces and accelerations do not change significantly. Regarding the values of contact forces, no loss of contact occurs between the wheel and the rail in all the simulation process. On the other hand, vertical accelerations on the axles are well above the reference limit of $30 \text{ m/s}^2$ in which the track need to be followed up with intense monitoring. Accelerations above 50 and $70 \text{ m/s}^2$ require, respectively planned maintenance and immediate intervention operations, López Pita (2006).

The higher values of accelerations and interaction forces, even for speeds of 200 km/h, it is demonstrated that this type of profiles are very aggressive to develop a degradation analysis. In view of these circumstances, it is decided to continue this assessment with the longitudinal level irregularity profiles coming from the monitoring of the Portuguese and Spanish railway lines.

### 6.4.2.2 Experimental profiles from REFER

In this section two different profiles coming from the monitoring operations carried out by REFER, are analyzed. A total number of 20 simulation have been analyzed for speeds of 200 km/h and 300 km/h.

**Ballast Settlement**

Figures 6.11 and 6.12 show ballast settlement evolution for the two experimental profiles considered in this analysis. In this case, the influence of track irregularities in the degradation phenomenon is lower than in the case of theoretical profiles. Ballast settlement evolution along the transition zone depends not only on the stiffness variations, as in scenario $C_0$ but also on the irregularity profile entered in the model. Note that for speeds of 200 km/h, ballast settlement evolution depicted in 6.11(b) and 6.12(b) are very similar, when different irregularity profiles have been considered. Differences between ballast settlement evolution depicted in 6.11(a) and
6. Long-term simulation

![Graph](image1)

Figure 6.11: Ballast settlement for scenario $C_4$, profile p1: (a) $v=300$ km/h; (b) $v=200$ km/h

![Graph](image2)

Figure 6.12: Ballast settlement for scenario $C_4$, profile p2: (a) $v=300$ km/h; (b) $v=200$ km/h

The results in Figure 6.12(a) are more clear. This highlights the importance of the speed of the train, in cases in which track irregularities are taken into account. The higher the value of the speed, the greater the influence of track irregularities in the ballast settlement phenomena if it is compared with the influence of the track stiffness variations.

**Vertical displacement of the wheel**

Figures 6.13 and 6.14 show the evolution of the vertical displacement of the wheel of the first axle, along the transition zone. Since no loss of contact occurs between the wheel and the rail, these vertical displacements can provide an idea of the track irregularity profile evolution along the transition zone. In these figures, results for both the first and the last cycle are highlighted in a different color (black).

In this case, vertical displacement of the wheel seem to be influenced, in a remarkably way, by the track irregularity profile. In this case the relationship between vertical displacement of the wheel and ballast settlement evolutions, existing in scenario $C_0$, does not exist.
Figure 6.13: Vertical wheel displacements of the first axle of the bogie, for scenario $C_4$, profile $p1$: (a) $v=300$ km/h; (b) $v=200$ km/h

Figure 6.14: Vertical wheel displacements of the first axle of the bogie, for scenario $C_4$, profile $p2$: (a) $v=300$ km/h; (b) $v=200$ km/h

**Interaction forces**

Regarding the interaction forces evolution, Figures 6.15 and 6.16 show the contact forces between the wheel and the rail when the experimental profiles $p1$ and $p2$ are considered. As it can be seen, the value of contact forces does not change significantly as the number of cycles grows. Furthermore the value of contact forces, along the transition zone, does not experiment significant changes when the axles move over the section in which a variation of track stiffness occurs, as in scenario $C_0$. This means that the value of contact forces between the wheel and the rail is influenced mainly by the track irregularity profile considered in each case. This is particularly important, because the value of accelerations transmitted to the axles and coach will depend on the value of the above mentioned contact forces, affecting directly passengers comfort.

For every cycles analyzed, no loss of contact has been detected between the wheel and the rail for the speeds of $200$ km/h and $300$ km/h. This indicates that this type of profiles is less aggressive, in terms of the dynamic interaction behaviour, than the
Figure 6.15: Interaction forces between the wheel and the rail in the first axle of the bogie, for scenario $C_4$, profile $p1$: (a) $v=300\text{ km/h}$; (b) $v=200\text{ km/h}$

Figure 6.16: Interaction forces between the wheel and the rail in the first axle of the bogie, for scenario $C_4$, profile $p2$: (a) $v=300\text{ km/h}$; (b) $v=200\text{ km/h}$

At this point, it is interesting to evaluate the maximum value of vertical accelerations in the axles. This parameter is related with the track quality and gives an idea of the influence of the experimental profiles in the track quality when a degradation analysis is carried out.

**Evolution of maximum accelerations on the axles**

Figure 6.17 shows the evolution of the maximum acceleration in the axles. For a speed of 300 km/h and vertical accelerations, for both profiles, are very close to the reference limit of $30\text{ m/s}^2$ in which the track need to be followed up with intense monitoring. As it can be seen in Figure 6.17(a), values of acceleration tends to stabilized after 1000000 million cycles for both profiles.

For a speed of 200 km/h, values of vertical accelerations, for profiles $p1$ and $p2$, are far below of the reference limit of $30\text{ m/s}^2$, which indicates the good dynamic
6.4 Long-term analysis

Figure 6.17: Evolution of the maximum acceleration on the axles, for scenario \( C_4 \), profiles \( p_1 \) and \( p_2 \): (a) \( v=300 \text{ km/h} \); (b) \( v=200 \text{ km/h} \)

behaviour of the train-track interaction with this type of experimental profiles at speed of 200 km/h.

**Evolution of hanging sleepers**

In contrast with the evolution of hanging sleepers, in scenario \( C_0 \), here distribution of this phenomenon is strongly influenced by the track irregularity parameter. This happens also in the case of interaction forces, as already said.

Evolution of this degradation phenomenon depends also on the speed of the train. For speeds of 300 km/h, magnitude of the voids tend to increase until a certain number of loading cycles. After that, the magnitude decreases and it achieves a stability for the last loading cycles. In the last cycles, distribution of the voids is more regular along the transition zone (lower than 0.5 mm), see Figures 6.18 and 6.20.

For speeds of 200 km/h, see Figures 6.19 and 6.21, distribution of hanging sleepers tend to be located in the sections between the beginning and the end of the wedge-shaped backfill. In this case the magnitude of the voids are clearly below the 0.5 mm, almost reached for speeds of 300 km/h.

**6.4.2.3 Experimental profile from ADIF**

**Ballast Settlement**

Figures 6.22 show ballast settlement evolution for the experimental profile, provided by ADIF, considered in this analysis. In this case, the influence of track irregularities in the degradation phenomenon is even lower than in the case of the experimental profiles provided by the Portuguese Administrator, REFER. Notwithstanding, the influence of the track irregularities is still evident for the speed of 300 km/h as it is depicted in Figure 6.22(a). For a speed of 200 km/h, the effect of track irregulari-
ties, in terms of ballast settlement, is very low and almost coincident with ballast settlement given in Figure 6.2(b).

**Vertical displacement of the wheel**

As in the previous cases, vertical displacement of the wheel is influenced by the initial irregularity profile considered. In this case, no loss of contact happens between the wheel and the rail. For this reason, the vertical displacement of the wheel, provides also an idea of the geometry evolution of the rail, along the transition zone, as the number of cycles of the simulating process increases.

The greater the value of the speed, the higher the magnitude of the track irregularity at the end of the long-term simulation, as shown in Figure 6.23.
6.4 Long-term analysis

Interaction forces

Regarding the interaction forces evolution, Figure 6.24 shows the contact forces between the wheel and the rail when the experimental profiles $p_1$, provided by ADIF, is considered. As in the previous case of irregularities provided by REFER, the value of contact forces does not change significantly as the number of cycles grows. Also in this case, the value of contact forces, along the transition zone, does not experience significant changes when the axles move over the section in which a variation of track stiffness occurs.

For every cycles analyzed, no loss of contact has been detected between the wheel and the rail for the speeds of 200 km/h and 300 km/h. Dynamic component of the interaction forces are slightly lower, in this case, than if they are compared with those obtained for REFER profiles.

As it was done before, it is interesting to evaluate the maximum value of vertical
accelerations on the axles. This parameter gives an idea of the track quality, in terms maintenance works. Indirectly, this parameter is also related with the passengers comfort since the lower the value of acceleration in the axles, the less the magnitude of acceleration inside the coach.

**Evolution of maximum accelerations on the axles**

Regarding the vertical accelerations on the axles, values in this case are far below the above mentioned limit of $30 \, m/s^2$, see Figure 6.25. This indicates the good dynamic behaviour obtained with the longitudinal level irregularity profile from ADIF, for both speeds of the train: 200 km/h and 300 km/h.
Figure 6.21: Evolution of the hanging sleeper phenomenon, for scenario $C_4$, profile $p2$ and $v$=200 km/h

**Evolution of hanging sleepers**

Figures 6.26 and 6.27 show the evolution of the hanging sleepers phenomena, along the transition zone, as the number of loading cycles increases, for the speeds of 300 km/h and 200 km/h respectively.

For a speed of 300 km/h the influence of track irregularities is higher than the effect of track stiffness variations. Distribution of the voids along the transition zone is homogeneous as it can be seen in Figure 6.26. The maximum void between the sleeper and the ballast layer is lower than 0.3 mm at the end of 200000 cycles.

On the other hand, for a speed of 200 km/h, the influence of track irregularities is lower than the effect of track stiffness variations. In this case, distribution of the voids is concentrated in the zones where a change of track stiffness occurs: embankment to wedge-shaped backfill transition and wedge-shaped backfill to culvert transition, see Figure 6.27. The maximum void between the sleeper and the ballast layer is not
higher, in this case, than 0.1 mm at the end of 2000000 cycles.

6.5 Conclusions

This Chapter analyzed the degradation process to which the track is subjected due to train passage at a transition zone. This degradation process has been assessed considering different scenarios characterized by the fact of taking into account variables as: track irregularities and frictional behaviour in the soil-structure interfaces.

Within the track irregularities, two types of profiles were considered in this study: theoretical profiles, provided by the established requirements of the German Railway Administrator DB and experimental profiles, provided by the monitoring operations carried out by the Portuguese and Spanish Railway Administrator, REFER and ADIF, respectively.

In the first part, it was highlighted the importance of taking into consideration both
6.5 Conclusions

Figure 6.24: Interaction forces between the wheel and the rail in the first axle of the bogie, for scenario \( C_4 \), profile \( p1 \): (a) \( v=300 \) km/h; (b) \( v=200 \) km/h

Figure 6.25: Evolution of the maximum acceleration on the axles, for scenario \( C_4 \), profile \( p1 \): (a) \( v=300 \) km/h; (b) \( v=200 \) km/h

Dynamic train-track interaction as well as degradation of the track, given by ballast settlement, are analyzed for each scenario. When no track irregularities are considered: scenario \( C_0 \), both ballast settlements and dynamic interaction between the train and the track, depend strongly on the variation of track stiffness. Higher settlements of the ballast are located over the culvert, decreasing gradually in sections of the track that move away from the culvert.

Regarding the acceleration values, these comply the limits of \( 30 \) m/s\(^2 \) proposed by López Pita (2006).

Through the analysis of vertical displacements in both the sleepers and ballast layer, hanging sleepers phenomenon can be displayed. For a speed of 200 km/h, hanging sleepers are located in the zones where a variation of the vertical track stiffness exists.
On the other hand, for higher speeds: 300 km/h, hanging sleepers phenomenon is also located in the zones where the vertical track stiffness changes, for the first cycles. This degradation phenomenon arises in the adjacent sections, as the number of cycles of the degradation analysis increases.

Regarding to track irregularities, differences between artificial and experimental profiles is very accentuated, as it was seen in Chapter 5.

When track irregularities are considered in a long-term analysis, evolution of ballast settlement is clearly influenced by the speed of the train.

Differences in the rail profiles evolution are also evident when the speed of the train changes from 200 km/h to 300 km/h.

Vertical acceleration on the axles are far above the normal control limit when theoretical profiles, coming from REFER, are considered. In the case in which experimental
proposed profiles were considered, vertical acceleration on the axles are very close to the reference limit of $30 \text{m/s}^2$, when the speed is $300 \text{km/h}$. On the other hand, for speeds of $200 \text{km/h}$, the value of vertical accelerations stabilizes around $12 \text{m/s}^2$.

Evolution of the hanging sleepers is a more chaotic phenomena when track irregularities are considered. It is strongly influenced by the track irregularity parameter, specially when the speed of the train is $300 \text{km/h}$. For speeds of $200 \text{km/h}$, distribution of hanging sleepers tends to be more concentrated in the sections located between the beginning and the end of the wedge-shaped backfill.

When experimental irregularities coming from ADIF are considered, dynamic effects and degradation process tend to be equal to the results obtained for scenario $C_0$, specially the lower the value of the the analyzed speed. This fact show the good quality of the analyzed profile coming from ADIF, not only in terms of the geometry as it was analyzed and proved in Chapter 4, but also in terms of the dynamic interaction,
as it was proved in both the short-term and long-term evaluations carried out in this work.

Notwithstanding it is interesting to note that even in the case of irregularity profiles belonging to the Spanish conventional railway line, hanging sleepers phenomena has a different pattern distribution than those obtained for scenario $C_0$, specially for speeds of 300 km/h.
7 Conclusions and future research

7.1 Introduction

This Chapter presents the main conclusions obtained from the research, carried out in this work, and the subjects opened for future research works.

The main objective of this thesis was to analyze the effects of certain parameters such as: track irregularities, frictional behaviour of the soil/structure interface and embankment settlements, in the dynamic interaction existing between train and track at transition zones. Such effects have been analyzed in the context of both short-term and long-term depending on the number of loading cycles that are considered.

Transition zones are commonly defined, in railway track, as locations of discontinuity in the support or with abrupt changes in the track support conditions. Special importance is given to transition zones by railway Administrators since at these locations, additional maintenance costs are required to preserve the railway line, level and ride quality.

The idea to develop this work arises in the LESE (Laboratorio de Engenharia Sismica e Estructural) of the Technical University of Oporto, after a broad experience in the field of railway research.

Among the outstanding achievement of this group about transition zones, have been the recent works of Ribeiro (2012) and Paixão (2014). In these works it is worth noting the great importance of interaction forces magnitude, the track stiffness variations and embankment settlements in the long-term behaviour of the track at transition zones.

In order to account accurately the above mentioned effects, it has been decided to consider the effect of: soil/structure frictional behaviour, track irregularities and embankment settlements, in the different assessments developed in this work.

The conclusions of this thesis are given in Section 7.2. In this section, conclusions are presented differentiating between the conclusions derived from the aspects of the methodology followed in this work, the conclusions obtained from the short-term analysis and the conclusions drawn from the evaluation of the long-term assessment of the transition zone. Lastly, Section 7.3 shows the suggestions for future research works.
7. Conclusions

7.2 Conclusions

7.2.1 Aspects of the methodology followed in this work

Bidimensional models are commonly used in the track degradation analysis due to the great advantages in terms of computation time that these models have compared with more complex three-dimensional numerical models. It is estimated that time-consuming is 20-30 times lower in the bidimensional model, made up by 2D finite elements in plane stress, to analyze the degradation phenomenon presented in this thesis. A particular methodology has been followed in order to adapt the existing numerical model, developed in ANSYS, to the specific requirements and goals defined to carry out this work.

From the development of the methodology used in this work, following conclusions can be drawn:

- Regarding contact elements, it was detected that in cases of wheel-rail interaction problem, results obtained using Penalty method may contain an error due to the penetration value allowed in this methodology. This problem can be solved by changing the contact algorithm from Penalty to Lagrange Multipliers, despite of a higher value of computation effort.

- In the simulation of the frictional behaviour of contact interfaces, Penalty method improves not only the computational efficiency but also the convergence problem. In this cases a restriction in the penetration value must be introduced in order to obtain similar results with the two algorithms: Penalty method and Lagrange Multipliers. On the other hand, the use of these elements in soil/structure, ballast and sub-ballast interfaces, is essential to replicate the effect of differential settlements between sections over the embankments and sections over the culvert, in transition zones with a similar configuration to that of the one presented in this work.

- Frictional behaviour in a contact problem has been numerically validated, using two different finite element commercial codes: ANSYS and ABAQUS. In both cases, very similar results were obtained in terms of strains and stresses.

- Contact force between the wheel and the rail has been, as in the previous contact problem, numerically validated using two different finite element software, in this case ANSYS and FEMIX. Results of this validation show the great similarity when either ANSYS or FEMIX are used.

- Regarding track irregularities, artificial profiles generated according to different railway regulations have been evaluated following the requirements of track quality given by EN13848-5 (2010) and TSI (2008), concluding that those profiles generated following the German Railway Administrator (DBAG) experience present the best quality. On the other hand, experimental experimental profiles coming from monitoring operations carried out by REFER and ADIF
have been evaluated, in terms of track quality. It has been checked that profiles from both railway managers comply with the geometrical quality standards given by EN13848-5 (2010) and TSI (2008).

- From the long-term numerical results of the embankment settlements assessment it can be concluded that the magnitude of the settlement caused by consolidation of embankments is directly dependent on both the embankment’s height and deformation modulus. This relationship is not linear as it can be seen from the analysis of the numerical results shown in Section 4.9 and in the settlement data provided by Melis (2006) for the Spanish HSL.

### 7.2.2 Short-term analysis

This assessment is carried out with the purpose of identifying the main parameters that influence the degradation process of the track at a transition zone. A simple dynamic interaction analysis has been developed considering only one loading cycle due to the passage of a bogie of an Eurostar train over the transition zone. This assessment has been carried out considering different features as: different values of train speed, frictional behaviour in the soil/structure interface, frictional behaviour in the ballast/ballast and sub-ballast/sub-ballast interfaces, and longitudinal level track irregularities.

From the development of the short-term analysis, following conclusions can be drawn:

- From a general point of view, the speed of the train has a great influence in the train-track dynamic interaction. This is specially significant when the speed changes from 200 km/h to 300 km/h.

- Taking into account the frictional behaviour in the different interfaces: soil/structure, ballast/ballast and sub-ballast/sub-ballast is not specially relevant in the short-term dynamic interaction between train and track. However the use of contact elements to simulate the above mentioned interfaces is fundamental to account for the effects of differential settlements between sections over the embankments and sections over the culvert. Otherwise, the effect of differential settlements in a finite element program can not be fully simulated.

- Regarding to track irregularities, the influence of longitudinal level irregularity profiles is very important when short-term analysis are carried out. Concerning this point, a distinction between the influence of artificial and experimental irregularity profiles has to be made. Wheel/rail contact forces, acceleration in the axles of the bogie and deviatoric stress magnitude along the transition zone are notably greater when artificial irregularity profiles are considered. For a speed of 300 km/h, loss of contact between wheel and rail, is reached or almost reached, for all analyzed profiles. In the case of experimental profiles, response parameters are clearly lower which highlighted the better quality of the analyzed profiles coming from the monitoring operations carried out by REFER and ADIF.
This fact is very reasonable because on one hand, artificial profiles are generated according to a power spectral density function, that usually contains a wide range of frequencies. Some of these frequencies may have a great influence in the dynamic interaction behaviour between the wheel and the rail, by amplifying the response given by the dynamic interaction analysis. On the other hand, artificial profiles provided by the different railway managers, are generated in order to have into account the worse possible scenario regarding to safety and passengers comfort condition.

- Experimental profiles belonging to the Spanish conventional railway line (ADIF), have a better dynamic behaviour than profiles coming from a conventional line (REFER). In the case of REFER profiles, vertical accelerations on the axles almost reached the limit suggested by López Pita (2006) of $30 \text{m/s}^2$, for $v = 300 \text{km/h}$, in which the track need to be followed up with intense monitoring. Value of accelerations on the axles decreases as the speed of the train is lower and lower.

In the case of ADIF profiles, vertical acceleration on the axles are clearly below the limit of $30 \text{m/s}^2$, for every values of analyzed speed.

- Lastly, the influence of differential settlements in the vicinity of the culvert is very important in the dynamic interaction analysis between the train and the track. Values of contact forces increases as the magnitude of the entered embankment settlement is higher. Moreover, for each value of analyzed settlement, the effect of the train speed is also very important, increasing notably the value of contact forces an the rest of the dynamic response parameters, as the speed changes from $200 \text{km/h}$ to $300 \text{km/h}$.

Almost for every cases, values of acceleration on the axles exceeded the limit of $30 \text{m/s}^2$. This points out the importance of taking into consideration the effect of differential settlements in dynamic interaction analysis at transition zones.

### 7.2.3 Long-term analysis

This analysis has been carried out in order to evaluate the degradation process in a ballasted track when a bogie of an Eurostar train is moving along a transition zone. Several loading cycles have been used with the purpose of evaluating the behaviour of the train track interaction in a long-term period. The study has been developed considering two different values of the train speed: $200 \text{km/h}$ and $300 \text{km/h}$, as well as the influence of some parameters as track irregularities and frictional behaviour in the soil/structure interface.

The main conclusions obtained from the long-term analysis are collected in the following points:

- Parameters as: ballast layer porosity and ballast stabilization process are very important in degradation analysis, in which the ballast settlement is considered.
Sensitivity to these two parameters is very accentuated when the settlement law, provided by ORE (1970), is followed.

- When no track irregularities are considered, both the ballast settlement and dynamic interaction between train and track, depend strongly on the variation of track stiffness. In this case, higher settlements of the ballast layer take place in sections over the culvert, decreasing gradually in sections of the track that move away from the culvert.

Acceleration values on the axles are, for every analyzed speed, lower than the limit provided by López Pita (2006): 30 $m/s^2$, above which the track need to be followed up with intense monitoring.

Analyzing the evolution of hanging sleepers phenomenon, it can be concluded that for a speed of 200 km/h, voided sleepers are located in the sections where a variation of the vertical track stiffness exists. On the other hand, for higher speeds: 300 km/h, voided sleepers phenomenon takes place in the zones where the vertical track stiffness changes, for the first cycles but the phenomenon arises also in the adjacent sections as the number of loading cycles increases.

- Differences of long-term dynamic interaction behaviour are quite notable if artificial or experimental profiles are considered. In both cases, the evolution of ballast settlements is clearly influenced by the speed of the train.

Differences in the rail profiles evolution are very evident when the speed of the train changes from 200 km/h to 300 km/h when track irregularities are considered. This difference is almost negligible when no track irregularities are considered. This fact points out the importance of the speed of the train when track irregularities, particularly longitudinal level irregularity profiles, are included in the train-track interaction assessments.

- As it was shown in short-term analysis (Chapter 5), acceleration value on the axles are far above the normal control limit of 30 $m/s^2$ when theoretical profiles, generated according DBAG requirements, are considered.

- In the case in which dynamic interaction analysis is carried out with REFER experimental profiles, vertical accelerations on the axles are very close to the reference limit of 30 $m/s^2$ when the speed is 300 km/h. On the other hand, for a lower speed: 200 km/h, the value of vertical acceleration stabilizes around 12 $m/s^2$.

Evolution of voided sleepers in this case, is a more chaotic phenomenon. It is strongly influenced by the longitudinal level irregularity profile, specially for a speed of 300 km/h. For speeds of 200 km/h, distribution of hanging sleepers tend to be more concentrated in the sections located from the beginning to the end of the wedge-shaped backfills.

- When the experimental profile coming from the Spanish conventional railway line is considered, dynamic effects and degradation process tend to be similar
to the results presented when no track irregularities are beard in mind. This fact shows the good quality of the irregularity profile coming from ADIF, not only in terms of the geometry requirements but also in terms of the dynamic interaction between train and track.

Notwithstanding it is interesting to note that even in the case of a very good quality profiles, hanging sleepers phenomenon has a different pattern distribution, if compared with the case of a perfect geometry of the rail profile, specially for higher values of the train speed.

- The methodology here presented allows to have into account track irregularities (longitudinal level profiles) and contact elements in different interfaces. This would have a great interest in order to predict and program the proper maintenance works of the track at those zones in which track degradation is especially accelerated.

### 7.3 Future work

From the assessment developed in this work, the following research subjects may be suggested as interesting areas for future works:

- Analyze the effect of considering different laws of ballast settlement in which the value of the settlement depends on the vertical accelerations in this layer.

- Include both the embankment settlements and ballast settlement in a long-term analysis. This may allow to have interesting results in which degradation phenomena at transition zone could be better understood.

- More theoretical and experimental rail irregularity profiles need to be studied in order to have a complete knowledge of the influence of these profiles at transition zones.

- Due to the differences existing in the dynamic interaction results between experimental and artificial profiles, a new artificial profile is suggested to be proposed. Results obtained with this profile would be closer to that of the observed experimental profiles, in those cases in which a degradation analysis and a estimation of the maintenance costs are carried out.

- The methodology presented in this thesis, to simulate track degradation at the described transition zone, could be also adapted and implemented in other different track critical sections as bridge transitions, tunnel transitions and switches and crossings.

- The study of the particular transition zone analyzed in this thesis, consisting on a short-span concrete culvert, could be extended to different types of concrete culverts, even with different dimensions, very common in any railway network. These studies would provide a useful guideline with practical recommendations for transition zones belonging to the European network rail.
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