FINAL REPORT

CONTRACT N°: G1RD-CT-2002-00777

PROJECT N°:

ACRONYM: SUPERTRACK

TITLE: Sustained Performance of Railway Tracks

PROJECT CO-ORDINATOR: Norwegian Geotechnical Institute (NGI)

PARTNERS:

- Norwegian Geotechnical Institute (NGI)
- Société Nationale des Chemins de Fer (SNCF)
- Administrador de Infraestructuras Ferroviarias (ADIF)
- Géodynamique et Structure (GDS)
- Centro de Estudios y Experimentacion de Obras Publicas (CEDEX)
- Ecole Centrale de Paris (ECP)
- Linköping University (LU)
- Swedish National Rail Administration - Banverket (BV)

REPORTING PERIOD: FROM July 1, 2002 TO September 30, 2005

PROJECT START DATE: July 1, 2002 DURATION: 39 months

Date of issue of this report: November 30, 2005

Project funded by the European Community under the ‘Competitive and Sustainable Growth’ Programme (1998-2002)
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2 Executive summary

This is the final report of the project SUPERTRACK. It gives a summary of the findings of this project together with an overview of administrative issues as requested by the European Commission (EC). The project started in July 2002 and terminated in September 2005.

The main objective of the project was to enhance the performance of ballasted tracks, hence reduction of maintenance, by better understanding the geotechnical behaviour of the elements of the track, embankment and subgrade as well as the non-linear dynamic behaviour of the complete track-train system.

To achieve this objective, a number of activities/tasks were defined; these activities can be grouped into the following:

i) Measurement of track and ground responses at several problematic sites with regards to settlement and maintenance.

ii) Characterisation and modelling the behaviour of soil and ballast using large-size specimens, and full-scale testing of track using in a track-box facility.

iii) Advanced numerical modelling of the non-linear dynamic behaviour of track/embankment and ground accounting for the train-track interaction.

iv) Implementation of innovative track retrofitting method for improving performance and reduction of maintenance.

This report summarises the activities and achievements during this project. The following summarises the main milestones:

1) Characterisation of the non-linear long-term behaviour of the embankment material (ballast, sub-ballast and platform) by testing large-scale samples in the laboratory.

2) Geotechnical characterisation and extensive instrumentation of three sections on a newly-built high-speed line between Madrid and Zaragoza. Regular measurements at this site provided data on the evolution of track settlement.

3) Construction of the large-scale testing facility track box. The facility was used to perform physical modelling of the above test site and validation of the numerical models developed in the project.

4) Geotechnical characterisation and extensive instrumentation of two test sites in France: i) a section on the French high-speed line at Beugnâtre. This site was grouted in the last year of the project and its data before and after the grouting were used both to calibrate the numerical models and to assess of the effectiveness of the grouting method; ii) a section on the conventional line at Zoufftgen. The track at this site was renewed during the project and its performance (settlement history and dynamic response) was monitored. The measurements served to calibrate/verify the numerical model mentioned above.

5) Compilation of data for a number of lines in France, Sweden and Spain with regards to mechanical, geometrical and maintenance. The database was analysed with respect to maintenance.

6) Development of advanced numerical models for i) linear dynamic response of track-ground system including train-track interaction, and ii) non-linear response of the track under long-term traffic loading.
3 Objectives

The majority of railway tracks in Europe rest on ballast. Low speed trains (LST) with speeds of around 200 km/h or less have been operating for a long time on such tracks without any major problem. These trains have been used for transporting people and freight. High speed trains (HST) with speeds more than 300 km/h, have been operating on certain segments. The experience with HST in the recent years has demonstrated that even with lower axle loads, HST have caused track settlement problems at certain sections and have burdened railway companies with expensive maintenance work. To make the HST more competitive, it is necessary to reduce the maintenance cost of high-speed lines.

The main objective of the project was to enhance performance of ballasted tracks, hence reduction of maintenance, by better understanding the geotechnical behaviour of the elements of the embankment and subgrade as well as the dynamic behaviour of the embankment-ground-train system. Developing innovative grouting techniques for retrofitting existing lines for improved performance and less maintenance is the other objective of this research.

The maintenance operations of a railway company represent about 6% of its annual turnover. This corresponds to a mean value of 15000 Euros per km per year. Among them 10% concerns the maintenance of the geometrical quality of the lines, which is mainly realised by tamping operations. Reducing the global volume of tamping operations could largely reduce the corresponding costs, and increase the availability of the lines. This should have great financial and operational influences, particularly on sites for which the tamping is not effective for a long period.

Better and more time-efficient means of transport of people and goods is a growing demand of the present world. The cross-border travel, trade and commerce activities within Europe are increasing even more with the expansion of the European Union. Traffic congestion creates adverse impact on the environment mainly by air pollution and losses of man-hours. Airlines operating within Europe cannot solve these problems partly because they are ineffective on short distances. Having removed most of these limitations and weaknesses, trains are already playing an important role in providing cost-effective, comfortable and environment-friendly transportation through hearts of cities. Moreover, a healthy railway network through Europe will contribute to achieving regional cohesion and European uniformity.

4 Scientific and technical description of results

This section gives an account of the activities and findings of this research work. The eight technical reports of the project provide more details of the individual activities. Summaries of these studies are also presented on the project’s website www.supertrack.no.

The research work in this project can be grouped in the following four activities:

i) Field testing and measurement
ii) large-scale material and model testing
iii) Development of numerical models
iv) Implementation of track retrofitting for improved performance
Section 4.1 presents the large-scale testing programs on ballast and sub-ballast samples. Section 4.2 introduces the extensive work on data collection from a large number of lines in Europe. Sections 4.3 and 4.4 present details of the site characterisation, instrumentation, and measurements at two of the four tests sites in France and Spain. Section 4.5 gives an overview of the track-box facility for full-scale testing of a track section in Spain. Sections 4.6 and 4.7 give accounts of the numerical models developed in the project for simulation of non-linear track response and train dynamics. Sections 4.8 and 4.9 present the soil improvement methods (grouting procedures) implemented at two sites in France and Spain. These sections also give details of the site characterisation and monitoring work at both sites.

### 4.1 Large-scale testing of embankment material

Large-scale triaxial tests were performed in order to obtain the data required for the constitutive modelling of the embankment material. These tests were performed using the vacuum-triaxial apparatus at NGI and the triaxial cell at CEDEX. The following gives an account of the tests performed at NGI.

A vacuum triaxial works on the principle that a controlled vacuum is applied internally to the sample that is confined in the membrane; the atmospheric pressure in the laboratory thereby supplies the confining pressure. The sample size in this apparatus is 62.5 cm in diameter and 125 cm in height, thus allowing testing of rather coarse-grained materials.

Four tests were performed on materials similar to those used at the French test section at Beugnâtre (Sec. 4.8). The ballast material used in Test 1 and Test 2 was 25 mm/80 mm diorite gravel from the French quarry Roy. The form-layer material used in Test 3 was 0/80 mm well graded crushed gravel referenced SECAB-Bellignies. The sub-ballast material used in Test 4 was 0/31.5 mm quartzite sandstone well graded gravel from the French quarry Vignats.

Figure 4.1.1 shows the test setup used in the vacuum triaxial tests. The following instrumentation was used:

- **Force** – Load cell transducer on the piston of the MTS actuator.
- **Stroke** – LVDT that measures the displacement of the piston in the MTS actuator.
- **Axial 1** – Vertical displacement transducer on the left side of the sample.
- **Axial 2** – Vertical displacement transducer on the right side of the sample.
- **Radial** – Circumferential displacement transducer at mid-height of sample.
- **Differential Pressure** – Transducer that measures vacuum relative to atmosphere.

In addition, three measuring tapes were stretched around the sample, one at mid-height of the sample (next to the radial transducer) and one at one-quarter and one at three-quarter sample height (Fig. 4.1.1).
The triaxial tests were performed with a closed-loop MTS servo hydraulic loading system. The capacities of the loading system are 1000 kN in force (tension or compression) and about 200 mm in displacement. Control of the system is achieved through a dedicated PC based operating system.

The specifications for the tests were provided by GDS. The objective was to represent the long-term cyclic loading of the material under train traffic. The load program consisted of several stages of cyclic loading followed by a monotonic loading.

The cyclic stages were performed in force control and at a cyclic frequency of 1 Hz. The monotonic stages were performed in displacement control at a displacement rate of 1 mm/min or a sample axial strain rate of 0.08 %/min.

Figure 4.1.2 shows a sample in the triaxial apparatus after it was loaded to failure under monotonic loading.

Figure 4.1.3 shows a sample of the measurements during the cyclic loadings. The plots in this figure show the results for the cyclic stages of each test with 1,200,000 cycles. The first four plots are deviator stress, axial strain, radial strain and volumetric strain versus number of cycles (time). Due to the large number of data, the results are plotted at only selected times. The fifth plot in Fig. 4.1.3 displays the shear stress versus shear strain during the last series of cycles.

Fig. 4.1.2 Testing of railway sub-ballast by vacuum tri-axial setup
Similar tests were performed on micro-ballast, sub-ballast and embankment material obtained from the Guadalajara site in the 9 inches diameter tri-axial cell available at CEDEX's Geotechnical Laboratory. Four tests, of 1.000.000 cycles each, were carried out on macro-ballast using smaller size pressure cells. Figure 4.1.4 shows a typical result from tri-axial tests by CEDEX on sub-ballast.

**Fig. 4.1.3 Typical results from cyclic testing of ballast by vacuum tri-axial setup**

**Fig. 4.1.4 Deviator versus axial strain obtained in 1000000 cycles of test on sub-ballast.**
4.2 Network investigation

A work package was devoted to collection and analysis of railway network data on the large space and time scales. The data consisted of a wide range of parameters including track design, geotechnical, maintenance, and dynamic measurements. A major objective of this study was to use the collected data to identify the parameters that influence the performance of the tracks.

The data was collected for a large number of railway lines in France, Spain and Sweden. The following is a summary of the work on the analysis of the data.

Data collection: Sixteen sites of the Swedish, Spanish and French networks were selected. For each of them, data concerning track, traffic and geotechnical context were collected:

- **Track design**: plan design (curvature, alignment), slope, profile (embankment, excavation)
- **Superstructure and geotechnical properties**: Type and mechanical properties of rail, pad, fastners, sleepers, sleepers spacing, ballast thickness, “Los Angeles values and grain size distribution of ballast, sub-ballast layer, form layer, underlaying soil and substratum characteristics,
- **Maintenance**: maintenance operations type (tamping, re-railing, grinding…), geometrical quality,
- **Traffic**: maximum speed, Cumulated tonnage on a given time span,
- **Dynamic and static measurements on several local sites**: displacement, speed, acceleration and constraints under train passing and long term settlement evolution

Correlation seeking: Correlation seeking was based on three different types of statistical analysis:

1) Person’s analysis method in which correlations coefficients are calculated between numerical values of the variates.
2) Spearman’s analysis method in which correlations coefficients are calculated between ranks of variates: the lowest value of the variate gets rank #0, the following one ranks #1, and so on.
3) Multivariable factor analysis

For the bi-variate analysis, the results showed that, in spite of some high values of correlation coefficients, relationships between descriptive and maintenance variates are not significant. Reasons may be found in:

- Limited number of the chosen track sites and compared to their high heterogeneity
- Important number of variates
- Insignificance of the “tamping frequency” as maintenance indicator
4.3 Field measurements at Guadalajara, Spain
Extensive field measurements were carried out at this site by CEDEX and ADIF. This section presents an overview of the instrumentation and field testing as well as typical measurements.

4.3.1 Site characteristics
A straight sector was selected near Guadalajara, around K.P. 69+500 of the Madrid-Zaragoza high-speed line, comprising three zones, two on embankment, and one in trench. The geological formation at this location is Paramo limestone.

Within the embankment, two line sections were chosen for instrumentation, Section 1, reaching about 18 m height, and Section 2, modelled in real scale in the track box (Sec. 4.5), reaching about 4 m height. Section 3 was located within in trench zone. Sections 2 and 3 were further divided into three subsections.

Figures 4.3.1 and 4.3.2 give perspectives of the instrumented section, and Fig. 4.3.3 show a plan view of the locations of the sections.

The instrumentation was designed in order to measure the following response quantities:
1) Stresses at the bed layers
2) Contact stresses sleeper-ballast layer
3) Rail stresses
4) Permanent settlements and deflections caused by circulating trains
5) Vibrations
6) Temperature and water content
7) Meteorological parameters

Table 4.3.1 gives the types of control parameters at each of the instrumented subsections, and Figures 4.3.4 to 4.3.7 illustrate some of the installed sensors.

Table 4.3.1 Control parameters at subsections of experimental sector

<table>
<thead>
<tr>
<th>Instrumentation subsection</th>
<th>Control parameters</th>
</tr>
</thead>
<tbody>
<tr>
<td>S-1B</td>
<td>Settlements and deflections, sleeper accelerations</td>
</tr>
<tr>
<td>S-2A</td>
<td>Compressive and shear stresses at the sub-ballast layer, Rail stresses</td>
</tr>
<tr>
<td>S-2B</td>
<td>Settlements and deflections Accelerations at rail, sleeper, sub-ballast and form layer Vibration velocities</td>
</tr>
<tr>
<td>S-2C</td>
<td>Vertical stresses at the contacts sleeper-ballast and ballast-sub-ballast layers Temperature and water content in sub-ballast</td>
</tr>
<tr>
<td>S-3A</td>
<td>Compressive and shear stresses at the sub-ballast layer</td>
</tr>
<tr>
<td>S-3B</td>
<td>Settlements and deflections Accelerations at rail, sleeper sub-ballast and form layer</td>
</tr>
<tr>
<td>S-3C</td>
<td>Vertical stresses at the contacts sleeper-ballast and ballast-sub-ballast layers Temperature and water content in sub-ballast</td>
</tr>
</tbody>
</table>
Figure 4.3.1 Perspective of Sections 1 and 2 along with their connection boxes.

Figure 4.3.2 Perspective of Section 3 along with its connection box.
Figure 4.3.3 Location of the three instrumented sections.
4.3.2 Testing campaigns

Measurements were made in three different campaigns:
- April 2003: Tests with a 120 ton diesel-electric powered unit (Fig. 4.3.8), including removable external instrumentation
- November 2003: Monitoring of commercial trains passing at speed of 200 km/h, including removable external instrumentation
- July-October 2004: Monitoring of commercial trains passing at speed of 200 km/h, including removable external instrumentation.

The following is a summary of the measurements. For details see dedicated reports by CEDEX.

Tests in April: These tests were aimed at verifying the performance of all the instrumentation placed within the sector, and estimating the magnitude of some of the parameters for proper adjustment of the sensors. The removable external sensors mentioned above consisted of potentiometric systems for displacement registration and a laser system for measuring the absolute movements of the rail. The latter includes an emitting element, fixed to a support embedded in the ground that aims at a receiver fixed to the rail base (Fig. 4.3.9). The potentiometer needs to be rigidly fastened to a fixed base.
Measurements in November 2003: These measurements were made after the railway line opened to commercial traffic with two different types of trains, AVE and TALGO, running at 200 km/h. The objective was to obtain the displacements of the main components of the track structure at Section 2. Figure 4.3.10 shows a global scheme of the external instrumentation. The sleepers are shown numbered: sleeper number 6 corresponds to Section 2A; sleeper number 14 to Section 2B, and sleeper number 22 to Section 2C. A combination of potentiometers and PSD Laser displacement transducers were used to measure the absolute and relative displacements in the track structure. The following is the average values of maximum displacements for Section 2 (S2A, S2B and S2C):

- Absolute displacement of rail: \( (\delta_{aR}) = 3.5 \text{ mm} \)
- Relative displacement rail with respect to sleeper: \( (\delta_{r, \text{R-T}}) = 0.3 \text{ mm} \)
- Relative displacement of sleeper with respect to form layer: \( (\delta_{r, \text{T-CF}}) = 2.6 \text{ mm} \)
- Absolute displacement of embankment: \( (\delta_{aT}) = 0.6 \text{ mm} \)
Measurements in July-October 2004: For these tests the following instrumentation was used: 1) displacement transducers to measure relative displacements between different track elements, 2) extensometric bands for determination of bending moment and shear force in the rail, 3) accelerometers and geophones at the rail and at the sleeper, and 4) PSD Laser transducer for absolute displacement measurement at Section 2B. Table 4.3.2 lists the different sensors used during these tests and their locations.

The main objectives of the test were to measure the dynamic loads produced by the passage of the trains for replication in the track box, and to measure the absolute displacements of the rail and the relative displacements between the track elements.

Figures 4.3.11 and 4.3.12 are two examples of measurements. Figure 4.3.11 displays the record of the dynamic loads produced by a train, derived from the measurements of two extensometric shear bands separated at 25 cm. Figure 4.3.12 plots the time history of the absolute displacements of the rail under an Altaria (Talgo) train at section 2B.

According to Figure 4.3.12 (and other measurements), the rail displacements produced by the passage of trains at 200 km/h are in the order of 2.5 mm. The corresponding values during the measurements in November 2003 were of the order of 3.5 mm. One explanation for this difference is that after one year of operation, the high speed line at the Guadalajara site has consolidated, resulting in increased track stiffness.

Finally, Figure 4.3.13 presents a synthesis of the average values of the maximum vertical pressures measured at the different layers of the track structure. The recorded values are in good agreement with those obtained in previous tests.
Table 4.3.2 Instrumentation for tests with train speed 200 km/h in the period Jul-Oct. 2004

<table>
<thead>
<tr>
<th>SECTION</th>
<th>DISPL. TRANSDUCERS</th>
<th>STRAIN-GAUGES (IN RAIL)</th>
<th>ACCELEROMETERS</th>
<th>GEOPHONES</th>
</tr>
</thead>
<tbody>
<tr>
<td>S1B</td>
<td>POTENTIOMETER (R-S) + POTENTIOMETER (S-FL)</td>
<td>SHEAR + BENDING MOMENT</td>
<td>RAIL + SLEEPER</td>
<td>SLEEPER</td>
</tr>
<tr>
<td>S2A</td>
<td>-</td>
<td>SHEAR + BENDING MOMENT</td>
<td>-</td>
<td>-</td>
</tr>
<tr>
<td>S2B</td>
<td>POTENTIOMETER (R-S) + POTENTIOMETER (S-FL) + LASER (RABS) + POTENTIOMETER (R-S) + POTENTIOMETER (S-Fl) + POTENTIOMETER (R-S) + POTENTIOMETER (S-SB) + POTENTIOMETER (S-FL)</td>
<td>SHEAR + BENDING MOMENT</td>
<td>RAIL + SHEAR, BENDING MOMENT (RAIL HEAD) + SHEAR</td>
<td>RAIL + SLEEPER</td>
</tr>
<tr>
<td>S2C</td>
<td>-</td>
<td>SHEAR + BENDING MOMENT</td>
<td>RAIL + SLEEPER</td>
<td>SLEEPER</td>
</tr>
<tr>
<td>S3C</td>
<td>POTENTIOMETER (R-S) + POTENTIOMETER (S-FL)</td>
<td>SHEAR + BENDING MOMENT</td>
<td>RAIL + SLEEPER</td>
<td>SLEEPER</td>
</tr>
</tbody>
</table>

Figure 4.3.11 Dynamic loads derived from measurement of extensometric bands

\[ Q_{\text{MAX}} = 112.295 \text{ kN} \]
Figure 4.3.12 Absolute displacements of rail by laser measurement at Section 2B

Figure 4.3.13 Synthesis of vertical pressures at different levels of Section 2C.
4.4 Field measurements at Zoufftgen, France

Extensive measurements were carried out at this site by SNCF at two points in time after renewal of the track. This section presents an overview of the instrumentation and field testing along with typical measurements.

4.4.1 Site characteristics

Zoufftgen is located in Lorraine, on the Metz-Luxembourg railway line, 4 km south from the border of Luxembourg. The site is located in a cultivated terrain dip, drained by the Kiesel River. The measurement site lies at the level crossing #10, on the road D56 between Kanfen and Zoufftgen, at kilometric point 200.361 on track #1.

The soil is highly plastic saturated marls overlying marno-calcareous levels (Hettangian period) which contain a local bank of sandstone up to 25 m high at Hettange. Silty and argilaceous soils, resulting from the weathering of marls, overlie this substratum.

A seismic characterization tests (SASW) was done a few meters away from the test site, in order to determine the seismic P and S waves velocities in the top 15 meters of the natural ground. The results of these SASW are displayed on Figure 4.4.1.

Identification tests were performed in laboratory on the preserved samples extracted from the boreholes. Results of the tests are displayed in Table 4.4.1.

![Seismic profile established by SASW test](image)

**Fig. 4.4.1 Seismic profile established by SASW test**

<table>
<thead>
<tr>
<th>Sample #</th>
<th>Reference</th>
<th>W %</th>
<th>$\rho$ (g/cm$^3$)</th>
<th>$\rho_d$ (g/cm$^3$)</th>
<th>$\rho_s$ (g/cm$^3$)</th>
<th>WL</th>
<th>WP</th>
<th>IP</th>
</tr>
</thead>
<tbody>
<tr>
<td>PS 1</td>
<td>0.71 to 1.01 m</td>
<td>20.0</td>
<td>2.03</td>
<td>1.69</td>
<td>2.31</td>
<td>52</td>
<td>25</td>
<td>27</td>
</tr>
<tr>
<td>PS 3</td>
<td>3.55 to 3.82 m</td>
<td>9.7</td>
<td>2.34</td>
<td>2.13</td>
<td>2.46</td>
<td>42</td>
<td>23</td>
<td>19</td>
</tr>
<tr>
<td>PS 4</td>
<td>7.00 to 7.26 m</td>
<td>11.8</td>
<td>2.25</td>
<td>2.01</td>
<td>2.47</td>
<td>41</td>
<td>25</td>
<td>16</td>
</tr>
<tr>
<td>-</td>
<td>8.62 to 8.90 m</td>
<td>4.8</td>
<td>2.38</td>
<td>2.27</td>
<td>2.51</td>
<td>38</td>
<td>22</td>
<td>16</td>
</tr>
<tr>
<td>PS 5</td>
<td>11.47 to 11.71 m</td>
<td>8.9</td>
<td>2.36</td>
<td>2.17</td>
<td>2.49</td>
<td>36</td>
<td>21</td>
<td>15</td>
</tr>
</tbody>
</table>
4.4.2  Railway data and maintenance

The Metz-Zoufftgen line has been used since 1859. Traffic is quite heavy, mixing both freight and passenger trains at a 140 km/h maximum speed. This double track line ranks as UIC group # 3 with an average cumulated tonnage of 700 000 tons per year and per track (over 800 000 t in 2000 and 2001) for 7633 trains (in 2002 on track #1).

Several types of trains, including passenger « intercity » trains, regional trains and freight trains can be observed on the line.

While there are many different types of wagons and passenger cars running on this line, the locomotives are primarily: French locomotives “BB 15000” and “BB 16500” (SNCF), Belgian locomotives « B1300 » (SNCB), and locomotives « 3000 » from Luxembourg (CFL). Other types of locomotives include: BB 25500, BB 26000, Z 11500, Z 6500 and 1800 diesel from Luxembourg.

The rail is U50, and rail fasteners are elastic “Nabla” with a 20 kN preload and 9mm rubber rail pad. The sleepers are monoblock concrete sleepers BONNA U 41 with the following data: Length = 2500 mm, Height = 232 mm, Spacing: 600 mm, Mass = 283 kg.

The line at the site had a rather poor quality for a long time, requiring a high level of geometry maintenance. During its last years, tamping operations were not sufficient to guarantee a durable geometrical quality. The ballast layer was fouled by clay in many places, experiencing pumping and severe attrition. The clayey subgrade was supposed to be uneven with ballast pocket entrapping a high quantity of water. In October 2003, a complete renewal of ballast and under-ballast layers was undertaken in order to restore the track quality. The “Puscal” maintenance train was used to remove the old ballast and under-ballast layers, to shape the soil surface and to lay new ballast and under-ballast on geotextile. Sleepers and rails, which exhibited unacceptable corrugation, were renewed during the same operation.

Figure 4.4.2 illustrates the process of laying geotextiles and the new track. The ballast is 25/50 mm grain size with a thickness 25 cm under the inner (left) rail and 36 mm under the outer rail. Under ballast is a 20 cm thick 0/20 mm cement-treated gravel.

Fig. 4.4.2 Laying geotextile after removal of old track (left), new track (right)
4.4.3 Site instrumentation

The different types of measurements on/inside and alongside the track consisted of:

- Acceleration on a single sleeper (“reference sleeper”) in three directions,
- Acceleration on the ground surface at two different distances from the track, in three directions,
- Vertical acceleration inside the natural ground, under the instrumented sleeper
- Vertical acceleration inside the sub-ballast layer, under the instrumented sleeper
- Total track deflection by opto-electronic camera mounted far from the track
- Rail pad deformation by no-contact laser sensor
- Pressure level under ballast, along the instrumented sleeper
- Wheel load by strain gages mounted on the rail

Figure 4.4.3 shows the cross section of the track and positions of some of the sensors.
4.4.4 Measurements

Two measurement campaigns were made after the track renewal, the first was on 26-27 Nov. 2003 and the next one was on 29 June – 2 July 2004. One of the objectives of the measurements was to monitor possible changes in the track mechanical properties with time and the track settlement. The following gives a sample of the measurements. For details see dedicated reports by SNCF.

Measurements in November 2003: The first measurements were performed on track #2, a few weeks after the track renewal works when the optimum track geometry and normal track speed were restored. Track works were not finished yet on track #1. Consequently track #2 was under two-way traffic.

A synthesis of the averaged values of deflections and sleeper accelerations are listed in Tables 4.4.2 and 4.4.3.
Table 4.4.2 Whole track (Zimmer) deflection and rail pad deformation

<table>
<thead>
<tr>
<th>Vehicle</th>
<th>BB16500 (100 km/h)</th>
<th>1300, 3000 (75 km/h)</th>
<th>BB 15000 (120 km/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wheel</td>
<td>front back</td>
<td>front back</td>
<td>front back</td>
</tr>
<tr>
<td>Zimmer (mm)</td>
<td>1.8 1.7 1.6 13.0 2.0 2.1</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Rail pad (mm)</td>
<td>0.3 0.4 0.3 0.6 0.4 0.5</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.4.3 Average sleeper acceleration values

<table>
<thead>
<tr>
<th>Vehicle</th>
<th>BB16500 (100 km/h)</th>
<th>1300, 3000 (75 km/h)</th>
<th>BB 15000 (120 km/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wheel</td>
<td>front back</td>
<td>front back</td>
<td>front back</td>
</tr>
<tr>
<td>Sleeper end - #1 (g)</td>
<td>0.34 0.59 0.23 0.33 0.78 0.86</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Sleeper end - #2 (g)</td>
<td>non exploitable 0.42 0.38 0.86 0.76</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Measurements in June-July 2004: A synthesis of the averaged values of deflections and sleeper accelerations are listed in Tables 4.4.4 and 4.4.5.

Table 4.4.4 Average deflection of rail

<table>
<thead>
<tr>
<th>Engine</th>
<th>BB 16 500 (100 km/h)</th>
<th>1 300 and 3 000 (75 km/h)</th>
<th>BB 15 000 (120 km/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wheel</td>
<td>front back</td>
<td>front back</td>
<td>front back</td>
</tr>
<tr>
<td>Deflection rail 1 (mm)</td>
<td>1.4 1.4 1.7 1.4 2.0 1.8</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Deflection rail 2 (mm)</td>
<td>1.5 1.7 2.0 1.9 2.1 1.9</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Table 4.4.5 Average sleeper acceleration

<table>
<thead>
<tr>
<th>Engine</th>
<th>BB 16 500 (100 km/h)</th>
<th>1 300 and 3 000 (75 km/h)</th>
<th>BB 15 000 (120 km/h)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Wheel</td>
<td>front back</td>
<td>front back</td>
<td>front back</td>
</tr>
<tr>
<td>Vert. acc rail 2 (ext) - g</td>
<td>0.35 0.94 0.48 0.44 0.45 0.45</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Vert. acc rail 1 (int.) - g</td>
<td>0.63 0.69 0.51 0.48 0.71 0.61</td>
<td></td>
<td></td>
</tr>
</tbody>
</table>

Figure 4.4.4 summarises the measurements of pressure under the sleeper, and Figure 4.4.5 displays the variation with time of the pressures at the centre and under the sleeper for train type 3600 CFL.
Figure 4.4.4 Distribution of ballast pressure under sleeper

Figure 4.4.5 Ballast pressures under rail (left) and centre of sleeper (right)
4.5 Large scale testing in Track box

A major achievement of SUPERTRACK is the construction of a full-scale track box testing facility by ADIF and CEDEX on the CEDEX’s premises. Track box is a steel box with dimensions $4 \times 5 \times 21$ m which is filled with soil (from the desired site) and is capped by the track structure. A number of pressure cells, displacement transducers, accelerometers and geophones were installed inside the soil and on the various elements of the track. The train load was simulated by out of phase loads at 6 points on the rails by hydraulic actuators.

The track box was used in the project to reproduce the soil/track conditions at Section 2 of the Guadalajara test site (see Sec. 4.3). Figure 4.5.1 show the filling of the track box with the soil transported from the test site, and compaction operations at the sub-ballast and ballast levels. Along with the filling of the box, and with control tests of the placement conditions (density and water content), sensors were placed at different levels below the track. Figure 4.5.2 presents a layout of the track box at two instrumented sections. Figure 4.5.3 shows a longitudinal section of the box with the results of compaction control by means of SASW tests, and Figure 4.5.4 displays the track box from different angles during one of the tests.

![Figure 4.5.1 Filling of track box (left), compaction of ballast and mounting of rails (above)]
Numerical modelling of the track-box was performed by ECP with the intention of establishing the effect of the track-box’s metallic frame on the dynamic behaviour of the track-ground inside the track box. Figure 4.5.5 presents a typical result of this study showing the first few modes of vibration of the track box. This study revealed that, except for areas close to the box’s ends, the track box represents fairly well the real ground conditions.
Figure 4.5.4 Track box during testing

Fig. 4.5.5 Computed first two vibration modes of track box
4.6 Numerical modelling of dynamic train/track interaction

The dynamic interaction of train and track systems generate forces and vibrations leading to track deterioration, such as track settlements, railhead corrugation growth, damage to track components (rail-pads, sleepers, ballast), etc. This section presents a summary of the results of the research carried out by LU on this subject.

4.6.1 Validation of numerical model

Two track models were developed in the project. The “small” model (Figure 4.6.1) assumes longitudinal symmetry of a single track and of the loading. Only half a wheelset, one rail, and half of the sleepers and the ballast/sub-ground bed were included in that model. The “large” model contains a full double-track railway line loaded with one or several moving wheelsets on one track or on both. The track models can be used to simulate the short-term behaviour of the dynamic train and track systems when a wheel or a train runs on the track. The long-term behaviour of the track due to many wheel passages can (so far) be studied in the small model only. A simple material model to simulate the permanent ballast and subground deformation was implemented into this model.

![Figure 4.6.1 Symmetrical track model with ballast bed of elastic and elasto-plastic material](image)

The models were built-up of three-dimensional solid elements using the pre-processor TrueGrid, and the commercial finite element program LS-DYNA solved the dynamic train-track interaction problem. LS-DYNA is a general-purpose finite element code for analysing large deformation dynamic responses of structures, including structures coupled to fluids. The solution methodology is based on explicit time integration. The contact force is calculated by a penalty method (contacts appear between wheel and rail and between sleeper and ballast). The code is equipped with non-reflecting boundaries which absorb the outgoing shear and pressure waves so that no reflections occur at the boundaries. However, the bending wave in the rail is still reflected.

The smaller model was composed of half a wheelset, one rail (symmetry with respect to the centre line of the track was assumed), rail-pads, sleepers, and ballast bed. The ballast bed was modelled as a continuum with elastic or elastic-plastic material properties. The wheel was modelled as rigid, and the sleepers could be either rigid or flexible. The rail-pads were modelled using a predefined rubber material.
The loading of the track came from a wheelset of a train moving on the track. A constant force representing the dead load of the car body and the bogie frame loaded the wheelset. The mass of the wheelset, i.e., the wheel mass and half of the axle mass, was taken into account. This means that the inertia force from the unsprung mass, i.e., from the wheel and the axle, was taken into account. The passage of a full train can then be simulated as the passage of a sequence of wheelsets. In this study, only one wheelset representing the train load was used. Only in a study of the long-term behaviour of the track several wheelset passages were applied to the track. When studying the short-term behaviour of the track, normally only the maximum track deflection and maximum stresses in rails, sleepers, and ballast, and other extreme values, are of interest. In that case it is sufficient to load the track with one wheelset only. The influence of one wheelset of a bogie on the deflections, etc, at the nearby wheelset is negligible. This can be concluded from the fact that much of the track deflection rebounds between the two wheels. The loading from one wheelset on the track is spread out over a few (say three or four) sleepers only below the wheelset, but the distance to the next wheelset is several sleeper spans. Therefore the influence of one wheelset at the location of the other wheelset can be neglected and it is concluded that one wheelset represents well the local loading of the train on the track.

Two models of double-track lines were developed. One of them, the model of the Madrid-Zaragoza line (Fig. 4.6.2), was used to simulate measurements that were performed during the project. Some calculations with moving wheelsets were performed to verify that the models perform well. Single wheelsets were on the two tracks (one track at a time). Also, two wheelsets, running simultaneously in opposite directions or in parallel on the two tracks were tested, and the results were satisfactory.

![Finite element model of a double-track Spanish line.](image)

The double-track finite element model was verified against the measured data from the Spanish test site at Guadalajara at a newly built state. Track deflections were measured when
a six-axle locomotive was loading the track. The results are presented in Figure 4.6.3. The axles pass the sleeper where the measurements were performed. The axles arrive in groups of three, with three axles on the first bogie, and after a while another three axles on the second bogie pass the measurement site. When performing these measurements, the six-axle locomotive loaded the track statically; it passed the measurement site by advancing just one sleeper distance at a time. The measurements gave a maximum track deflection of 1.9 mm (Figure 4.6.3). Calculations performed for this site gave displacements of the same size (1.5 mm). The difference between the measured and the calculated deflections as seen in the figure is partly due to the fact that the track was very new so that the track structure (rails and sleepers) had not yet settled firmly into the ballast. Most probably there were some small gaps between the sleepers and the ballast bed so that even at very low loading of the track, the track had some deflection due to closing of these gaps.

![Graph](image)

*Figure 4.6.3 Validation of track model against data from Spanish track*

At a later measurement campaign on the Madrid-Zaragoza line stresses in the ballast bed and in the interface between ballast and sub-ballast were measured. The computational model resembled the measurement site as close as possible. There were five different material layers: ballast layer, sub-ballast, form layer, embankment, and natural ground. Characteristics of each layer are given in Table 4.6.1.

<table>
<thead>
<tr>
<th>Layer</th>
<th>Depth (m)</th>
<th>Mod. of elasticity (MPa)</th>
<th>Poisson’s ratio</th>
<th>Weight (kN/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>ballast</td>
<td>0.35</td>
<td>70</td>
<td>0.15</td>
<td>15</td>
</tr>
<tr>
<td>Sub-ballast</td>
<td>0.225</td>
<td>70</td>
<td>0.3</td>
<td>20.5</td>
</tr>
<tr>
<td>form layer</td>
<td>0.60</td>
<td>60</td>
<td>0.3</td>
<td>20.8</td>
</tr>
<tr>
<td>embankment</td>
<td>3.0</td>
<td>50</td>
<td>0.3</td>
<td>21</td>
</tr>
<tr>
<td>natural ground</td>
<td>13.0</td>
<td>700</td>
<td>0.3</td>
<td>20</td>
</tr>
</tbody>
</table>
The computer model used for this validation had a length of 30 sleepers. Five sleepers in the centre of the model were elastic and the remaining sleepers were rigid. All results were extracted from the centre of the model close to sleeper 15 (sleepers 13 to 17 were elastic). One wheelset moved over the model at a speed of 200 km/h. The static (dead) load on the wheelset was 123.911 kN (which was the maximum wheel load measured). The stiffness of the rail-pad was adjusted so that its deformation became 0.4 mm when the wheel with the load given above passed. The comparisons which included rail deflection, stresses below the centre and outer end of the sleeper, stress below the centre of the sleeper, and stress at the interface between ballast and sub-ballast showed satisfactory performance of the LS-DYNA model.

Once the models were verified against measured data, the following a comprehensive research work was performed on the track settlement due to non-elastic deformation of the ballast bed and the sub-ground, hanging sleepers, and a smooth transition between two parts of the track with different stiffness. The following gives a summary of these studies.

4.6.2 Track settlement

The elastic-plastic continuum material model used in the present study was predefined in the computer program (material MAT SOIL AND FOAM was used). This material, which behaves like a fluid, should be used only in situations “when soils and foams are confined in the structure”. When loading the material, it behaves like a linear elastic, deformation hardening material. If the yield limit of the material is exceeded, permanent plastic deformations will remain in the ballast after it has been unloaded. When the track bed is composed of several layers with different functions and characteristics (ballast, sub-ballast, formation), different material models for the different layers should be used. As an example, the SOIL AND FOAM material was used for the ballast layer in Figure 4.6.1. The yield limit of the ballast material was selected so that the stress beneath sleepers 11 to 20 (30 sleepers in the model) exceeded the yield limit of the ballast material when the wheel passed. In the calculations it was found that the contact force between the ballast and sleeper 11 (the first sleeper in the section with “bad” ballast, i.e., a low yield strength) became zero after the first wheel passage. This indicates that the ballast bed has undergone so large plastic deformation that Sleeper 11 has become hanging in the rail. Sleeper 12 gets a lower (static) sleeper/ballast contact force after the first wheel passage, and the contact disappears after the second wheel passage. Sleepers 13 to 18 never lose contact during these cycles, but the permanent deformation (settlement) increases during the loading (Figure 4.6.4). The settlement rate after each wheel passage is, however, decreasing. Figure 4.6.4 also shows how the rail and sleeper move upwards a small amount just before the wheel arrives and after the wheel has passed.

4.6.3 Dynamic forces due to unsupported sleeper

This section presents a study of the influence of an unsupported (“hanging”) sleeper on the train/track dynamics. In an ideal case, the sleepers should rest on the ballast bed to provide support for the rails to transmit vertical, lateral and longitudinal forces from the rail down to the ballast bed. In the calculated results presented here, one sleeper was not in contact with the ballast. A gap of 0.5 mm or 1mm was introduced between the sleeper and the ballast in the model in Fig. 4.6.1, and the increase of dynamic forces due to this gap was investigated. The hanging sleeper and the two adjacent sleepers were made flexible, while the other sleepers of the track model were rigid.
Some calculated results for a track with one hanging sleeper (No 15) are shown in Figures 4.6.5 and 4.6.6. The wheelset loading the track moved at speed of 90 m/s going from left (sleeper 1) to right (sleeper 30). The gap between sleeper 15 and the ballast bed was 0.5 mm and 1 mm. For comparison, the contact force when there was no gap at sleeper 15 is also shown.

The sleeper-ballast contact force at sleeper 14, when sleeper 15 was hanging, increased by about 20 percent. The size of the gap at sleeper 15 (0.5 mm or 1.0 mm) did not make any difference. At sleeper 15 there was some contact between the sleeper and the ballast if the gap was 0.5 mm, but at a gap of 1 mm no contact occurred between the sleeper and the ballast. Sleeper 16 became the most loaded sleeper when sleeper 15 was hanging. In Figure 4.6.5 the sleeper-ballast contact force at sleeper 16 is shown. It can be seen in the figure that the sleeper-ballast contact force at this sleeper increases considerably (at vehicle speed of 90 m/s). The figure shows that the sleeper-ballast contact force at sleeper 16 increased by 70 percent when sleeper 15 was hanging in the rail with a gap of 1 mm down to the ballast. Such a large overload of the ballast bed might result in non-elastic deformations of the ballast and sub-ground at that sleeper. A process that results in track settlements might therefore be initiated.

In Figure 4.6.6 the wheel-rail contact force is displayed. It is seen that the contact force goes down when the wheel passes sleeper 15 (and also the wheel moves downwards) and it is also seen that there is a large impact when the wheel reaches sleeper 16 (where the wheel moving downwards should be turned to move upwards). These analyses and results clearly illustrate that a hanging sleeper will induce large dynamic forces in the track potentially leading to gradual deterioration of the track.
4.6.4 Optimal track stiffness

The stiffness of a track varies along a track: an embankment-to-bridge transition is an example. Therefore, a study on how to select the track stiffness at a transition zone between two parts with different stiffness was performed. The study focused on optimization of the track stiffness at the transition area (booted sleepers, for example, can be used to control track stiffness). In the model, the transition area was divided into five sections with one stiffness to be optimized in each section (Figure 4.6.7).

![Figure 4.6.7 Track stiffness transition zone from stiff to soft track, or vice versa](image)

The wheel-rail contact force was selected as the objective of the optimization. The force variation (i.e. irregularity of the force) should be as small as possible when the wheel passes the transition area. Optimal track stiffness variations in the transition area, as obtained from the optimization procedure, are shown in Figure 4.6.8. The track stiffness at one end of the
track model was 90 kN/mm and at the other end the stiffness was 45 kN/mm (different stiffness values were obtained by changing the modulus of elasticity of the ballast material). It is seen in Figure 4.6.8 that the optimal stiffness variation depends on the travelling direction of the train.

![Normalized Youngs modulus](image)

Figure 4.6.8 Optimum track stiffness variation along transition area when wheelset is moving from stiff to soft track (left figure), and when moving from soft to stiff track (right figure)

The improvement of the wheel-rail contact force can be discerned in Figure 4.6.9. The results show that by selecting the track stiffness suitably, a wheel-rail contact force can be achieved that does not differ very much from the force obtained at the constant-stiffness track. This will, of course, improve the dynamic performance of the train and less damage and deterioration will be induced to the track.

![Wheel/rail contact force](image)

Figure 4.6.9 Wheel-rail contact force at track stiffness transition area before optimization and after optimization for transition from stiff to soft track (left) and from soft to stiff track (right)
4.7 Numerical modelling of short-term and long-term dynamic response of track

This research was carried out by ECP. The objective was to develop and validate 1) an efficient and rigorous numerical model for the dynamic analysis of track-ground systems, and 2) a numerical model for prediction of long-term deformation of tracks.

4.7.1 Short-term dynamic response

The developed model takes advantage of the spatial periodicity of the track-soil system. The periodic formulation, which was implemented by using the solution advanced by Floquet, allows one to reduce the analysis of the overall system to that of a generic cell. The principle of this method is shown schematically in Fig. 4.7.1.

For the dynamic analysis of the generic cell the computer code MISS3D, which was developed at ECP, was used. This software is based on a domain decomposition method whereby the three-dimensional domain considered (the generic cell in the present case) is decomposed into two sub-domains, the track-structure and the soil, that are coupled at their interface. Each sub-domain can be independently modelled. In MISS3D, the boundary element method (BEM) with special Green’s functions is used for the soil while the finite element method (FEM) is used to represent the track-structure.

The developed model was validated against the measured data at the Beugnâtre test site (see Sec. 4.8 for site information). Figure 4.7.2 displays a comparison between the simulated and measured data at the test site. The geotechnical parameters of the site are given in Table 4.7.1.

<table>
<thead>
<tr>
<th>Layer Parameter</th>
<th>lime treated loam</th>
<th>clayey loam</th>
<th>loamy silt</th>
<th>loamy silt</th>
<th>loamy silt</th>
<th>loamy silt</th>
<th>silty clayey</th>
<th>fractured chalk</th>
<th>fractured chalk</th>
<th>fresh chalk</th>
</tr>
</thead>
<tbody>
<tr>
<td>$C_p$ (m/s)</td>
<td>935</td>
<td>224</td>
<td>468</td>
<td>281</td>
<td>187</td>
<td>224</td>
<td>281</td>
<td>2040</td>
<td>1102</td>
<td>1225</td>
</tr>
<tr>
<td>$C_s$ (m/s)</td>
<td>500</td>
<td>120</td>
<td>250</td>
<td>150</td>
<td>100</td>
<td>120</td>
<td>150</td>
<td>400</td>
<td>450</td>
<td>500</td>
</tr>
<tr>
<td>$\rho$ (kg/m$^3$)</td>
<td>1990</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1800</td>
<td>1900</td>
<td>2000</td>
</tr>
<tr>
<td>$\beta$ (damping)</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
<td>0.01</td>
</tr>
<tr>
<td>h (m)</td>
<td>0.8</td>
<td>1.5</td>
<td>1.1</td>
<td>1.2</td>
<td>1.00</td>
<td>1.30</td>
<td>2.60</td>
<td>2.50</td>
<td>5.70</td>
<td>-</td>
</tr>
</tbody>
</table>
4.7.2 Long-term dynamic response

The methodology used for the analyses was based on decomposing the problem into two parts where the linear and non-linear effects are uncoupled. The overall behaviour was considered to be linear and could be calculated with Miss3D (Sec. 4.7.1); the non-linearity was assumed to have a local effect and was studied at a given section using the ECP’s software Gefdyn.

4.7.2.1 Modelling non-linear material behaviour

The ECP’s elasto-plastic multi-mechanism model, commonly called Hujeux model was used to represent the soil behaviour. This model is written in terms of effective stress and can take into account the soil behaviour in a large range of deformations. The representation of all irreversible phenomena is made by four coupled elementary plastic mechanisms; namely, three plane-strain deviatoric plastic deformation mechanisms in three orthogonal planes and an isotropic one. The model uses a Coulomb type failure criterion and the critical state concept. The evolution of hardening is based on the plastic strain (deviatoric and volumetric strain for the deviatoric mechanisms and volumetric strain for the isotropic one). To take into account the cyclic behaviour a kinematical hardening based on the state variables at the last load reversal is used.

The model is written in the framework of the incremental plasticity, which assumes the decomposition of the total strain increment in elastic and plastic parts. The elastic part is supposed to obey a non-linear elasticity behaviour, where the bulk (K) and the shear (G) moduli are functions of the mean effective stress \((p')\). Through appropriate choice of parameters one can control the form of the yield surface in the \((p', q)\) plane to vary from a Coulomb type surface to a Cam-Clay type one. Furthermore, an internal variable \(r_k\), called degree of mobilized friction, is associated with the plastic deviatoric strain. This variable introduces the effect of shear hardening of the soil and permits the decomposition of the behaviour domain into pseudo-elastic, hysteretic and mobilized domains. Finally, an associated flow rule in the deviatoric plane \((k)\) is assumed, and the Roscoe’s dilatancy law is used to obtain the increment of the volumetric plastic strain of each deviatoric mechanism.

Fig. 4.7.2 Measured (blue line) and simulated (green line) vertical acceleration (m/s²) with respect to time (s) for sensors on sleeper (right) and in sub-ballast (left) at Beugnâtre
4.7.2.2 Parameters identification

Calibration of the model parameters was based on the two triaxial tests performed on ballast by NGI. The two samples with different initial unit weights (Test 1: 1550 Kg/m³ and Test2: 1621 Kg/m³) were subjected to cyclic q/p constant stress paths during 100000 cycles. The samples were then monotonically deformed up to 14% vertical strain. The confining pressures p₀' was the same in the two tests (40 kPa). Figure 4.7.3 summarises the results from Test 1. The following aspects are noticed on these results:

- Radial strains are negligible in the beginning of the tests for axial strains up to 0.18%
- Irreversible volumetric strain is negligible during the reloading paths
- The q-Epsz curves present a very steep (almost infinite) slope at the beginning of each unloading.

The first phenomenon can be either due to the anisotropy of the initial state or due to the performance of the radial measurement ring, or even both. Even though it was possible to take into account the initial anisotropy of the material, this aspect was not investigated because it was not relevant to the objectives. The second observation encouraged the assumption that the isotropic mechanism was probably inactive as no significant volumetric plastic deformation was obtained during the cyclic loading. The last phenomenon which can be attributed to the relaxation of the contact forces at each unloading was not integrated in the constitutive model. Thus, it was decided to model the overall behaviour of each loop in the simulations. The strategy to choose the model parameters was to fix the measurable parameters using the monotonic response of the model on the two samples.

![Figure 4.7.3 Observed behaviour of experiment Test1 on ballast obtained by NGI](image-url)
Figure 4.7.4 displays the response of the model for the first cycles of loading compared to the experimental results. The overall behaviour is well modelled, though the volumetric and axial strains are underestimated. However, it is worth noting that the magnitude of simulated strain variation during one cycle is comparable to the experimental observations and they have almost the same form. The evolution of the form of the stress-strain loops is given in Figure 4.7.5. Finally, the simulation of the static response of the test after one million loading-unloading cycles is shown in Figure 4.7.6.

Figure 4.7.4 Comparison between simulated (red) q/p constant test and experimental (green) in Test 1 on ballast: \( q - \varepsilon_1 \) and \( \varepsilon_v - \varepsilon_1 \) planes
Figure 4.7.5 Comparison between simulated (magenta) q/p constant test and experimental (blue) in Test 1 on ballast: $q - \varepsilon_l$ and $\varepsilon_v - \varepsilon_l$ planes

Figure 4.7.6 Comparison between simulated (red) and experimental (green) results of final stage of Test 1 on ballast, Dashed red: Only monotonic loading simulated, Solid red: cyclic loading followed by monotonic loading simulated: $q - \varepsilon_l$ and $\varepsilon_v - \varepsilon_l$ planes
4.7.3 Estimation of irreversible track deformation

Once the model parameters were identified, stress paths similar to those encountered under the rail in the track during the passage of one axle under isotropic elastic hypotheses were simulated and the evolution of the vertical deformation with the number of the loading-unloading cycles up to 1 million cycles were studied. In order to prevent the irreversible horizontal deformations in the direction of the track, plane strain condition in this direction were applied. The computed $q = \sigma_x - \sigma_y$, and $p = \text{tr}(\sigma)/3$ stresses during the application of the cycles are plotted in Figure 4.7.7. It can be noted that the change is not very significant.

![Figure 4.7.7 Evolution of stress path with increasing number of cycles (10^6 cycles)](image)

The predicted variation of the vertical strain with the number of loading cycles with the plane strain hypothesis is plotted in Figure 4.7.8. Based on this simulation, the evolution of the vertical strains with the number of cycles can be divided into several parts: an initial significant evolution, followed by a slower increasing phase ending up with an accelerated evolution at high number of cycles. Table 4.7.2 gives the value of the vertical deformation for several stages of loading-unloading cycle and the permanent deformation at the end of each cycle for different numbers of cycles.

Table 4.7.2 Evolution of vertical strain with number of cycles

<table>
<thead>
<tr>
<th>Number of cycles</th>
<th>$\Delta \varepsilon$/dN</th>
<th>$\delta \varepsilon_p$/dN</th>
<th>$\varepsilon_z$</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 000 000</td>
<td>4.3946e-004</td>
<td>4.9300e-006</td>
<td>0.0188</td>
</tr>
<tr>
<td>100 000</td>
<td>4.3944e-004</td>
<td>1.2000e-008</td>
<td>0.0067</td>
</tr>
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<td>10 000</td>
<td>4.4439e-004</td>
<td>1.4000e-008</td>
<td>0.0055</td>
</tr>
<tr>
<td>1000</td>
<td>4.3945e-004</td>
<td>4.9280e-006</td>
<td>0.0053</td>
</tr>
<tr>
<td>100</td>
<td>4.4907e-004</td>
<td>1.1641e-005</td>
<td>0.0024</td>
</tr>
</tbody>
</table>
4.8 Grouting operation and field measurements at Beugnâtre, France

The dynamical behaviour of the track at the site of Beugnâtre was studied by SNCF through measurements since December 2003. Beugnâtre is located in the region Nord-Pas-de-Calais, on the North Europe high speed line between Paris and Lille at 140 km North from Paris. Two sections of the track were instrumented and monitored: section #1 was a reference section with a light instrumentation, whereas section #2, which was grouted during the project, was more extensively instrumented. Grouting works took place in April 2004 in order to reinforce the subsoil and prevent the weakest loess horizons from collapse. Accelerations on track and ground were measured both under running trains (TGV, Eurostar) at about 300 km/h and in response to hammer impact and falling weight. The main intention with the latter tests was careful calibration of the track models.

4.8.1 Site characteristics

The Beugnatre site is located in the heart of the Parisian Basin. In this area, a 5-8 m thick loess formation overlies the chalk substratum. Loess is a silty aeolian quaternary deposit. In Northern France, it exhibits a sigmoidal grain size distribution with a predominant 20-50 µm fraction with a mineral content dominated by quartz (70%). Carbonates usually contribute to less than 12 %. The clay fraction of the extracted samples varies from 16 to 20 %. An 8m fractured and weathered chalk layer, at the top of fresh chalk substratum constitutes a transition layer between loess and chalk, with altered mechanical properties.

Soil samples were extracted at various depths (1.2 to 4.9m) at 10 m from the TGV track. Although the samples were extracted from levels above the actual track support layers, their data are worth presenting. Table 4.8.1 summarises the parameters of the extracted samples.
Table 4.8.1 Geotechnical properties of loess samples extracted from Beugnâtre

<table>
<thead>
<tr>
<th>Depth (m)</th>
<th>$\rho_s$ (ton/m$^3$)</th>
<th>$% &lt; 2\mu$m</th>
<th>$W_L$ (%)</th>
<th>$W_p$ (%)</th>
<th>$I_p$</th>
<th>$\rho_d$ (ton/m$^3$)</th>
<th>$W_{nat}$ (%)</th>
<th>$S_{rNat}$ (%)</th>
<th>$%$ Ca Suction (kPa)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1.2</td>
<td>2.719</td>
<td>20</td>
<td>30</td>
<td>21</td>
<td>9</td>
<td>1.52</td>
<td>18.9</td>
<td>66</td>
<td>10</td>
</tr>
<tr>
<td>2.2</td>
<td>2.714</td>
<td>16</td>
<td>28</td>
<td>22</td>
<td>6</td>
<td>1.39</td>
<td>18.1</td>
<td>53</td>
<td>12</td>
</tr>
<tr>
<td>3.5</td>
<td>2.713</td>
<td>16</td>
<td>26</td>
<td>20</td>
<td>6</td>
<td>1.54</td>
<td>16.6</td>
<td>55</td>
<td>11</td>
</tr>
<tr>
<td>4.9</td>
<td>2.712</td>
<td>18</td>
<td>30</td>
<td>21</td>
<td>9</td>
<td>1.55</td>
<td>23.7</td>
<td>82</td>
<td>16</td>
</tr>
</tbody>
</table>

Both the non-grouted (section #1) and grouted (section #2) zones were characterised by the SASW test. Figures 4.8.1 presents the results of the SASW measurements in section #1 together with the profiles of the interpreted mechanical parameters at this section.

The measured S-wave velocity profile in section #2 before grouting exhibited generally lower values than in section #1. In particular, a very low velocity zone (slightly over 80 m/s) was clearly detected at -7m, probably corresponding to the horizon of highly plastic brown loess. A second low speed layer was observed between -10 and -12m which correspond to top of the highly fragmented chalk horizon. These weak zones were considerably stiffened by the grouting. Figures 4.8.2 presents the results of the SASW measurements in section #2 after grouting together with the profiles of the interpreted mechanical parameters at this section.
Figure 4.8.2 S- and P-wave velocity profiles in grouted section #2 (top is at ballast/sub-ballast interface)

4.8.2 Railway data
The North European High-speed line is run at 300 km/h. The traffic is dedicated to passenger transport and mixing different types of trains in single or double unit configuration. This double track line is ranged as UIC group 3, with an average cumulated tonnage of $30 \times 10^6$ tons per year and per track. The test site is run daily by about 115 trains (15 Eurostar trains, 50 single unit TGV and 50 double unit TGV) corresponding to an average of 4500 axles.

Only passenger high-speed trains are allowed to run on this line. The three main types of train that were observed and recorded during the measurement campaigns were TGV Atlantique, Thalys trains and TGV réseau, and Eurostar.

The rails are UIC 60, and the fasteners are elastic “Nabla” with a 20 kN preload and 9mm rubber rail-pad. The sleepers are twin-block concrete sleepers “VAX U41” with the following data: Length 2415 mm; Height 220 mm under rail; Spacing 60 cm; and Mass 250 kg.

The ballast layer is 65 cm thick under the sleeper blocks. It is 25/50 mm grain size ballast. Under ballast is a 25 cm thick layer of 0/31.5 mm non-treated gravel. Under the gravel is a 50 cm form layer of compacted gravelly materials (0/70 mm) overlying the foundation loamy soil, the top 80 cm of which has been lime treated.

4.8.3 Instrumentation
Two track sections were instrumented as indicated below and shown in Figure 4.8.3.

- Section 1 at Pk 140.906, around sleeper # 29/30,
- Section 2 at Pk 140.934, around sleeper # 115/116.
The instrumentation consisted primarily of 3D accelerometers on and inside the track and on the ground surface. In addition, strain gages were installed on the rails to compute the wheel loads.

Sleeper accelerations were measured in the three directions on sleeper-blocks # 29, 113 and 115 (see Fig. 4.8.3). Accelerometers were glued on a cube mounted on the centre of the sleeper-end, on the external side of the rail.

Ground surface accelerations were measured in three directions. The accelerometers were mounted on a cube that was fixed on a 7 kg metallic plate and was sealed on the ground surface.

The underground accelerometers were placed in a steal tube and sealed with resin to ensure protection and tightness. Each sensor was placed inside a borehole, sealed with cement and loaded by sand. Boreholes were then filled up with clay pellets. The depths of the 12 underground accelerometers varied between 1.0m to 2.5 from the top of the track.

**Figure 4.8.4 Setting of underground sensors**
Table 4.8.2 Chronology of measurements at Beugnâtre.

<table>
<thead>
<tr>
<th>Date</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>November 24 to December 5, 2003</td>
<td>Boreholes and track instrumentation</td>
</tr>
<tr>
<td>December 18, 2003</td>
<td>Preliminary measurements for running trains on sec. 1 &amp; 2</td>
</tr>
<tr>
<td>January 24-28, 2004</td>
<td>Replacement of broken down accelerometers</td>
</tr>
<tr>
<td>February 5-6, 2004 (before grouting)</td>
<td>Measurements under running trains</td>
</tr>
<tr>
<td>February 5, 2004</td>
<td>Falling weight test</td>
</tr>
<tr>
<td>February 6, 2004</td>
<td>Impact hammer test</td>
</tr>
<tr>
<td>April 2004</td>
<td>Grouting of section 2</td>
</tr>
<tr>
<td>June 2004</td>
<td>Measurements under running trains</td>
</tr>
</tbody>
</table>

The hammer used for impact testing was of type 086D50 PCB. This hammer has a 5.5 kg impulsion mass with a built-in force cell, is used with a very soft cap and generates a low frequency impact (< 250Hz). Figure 4.8.5 indicates the locations of impacts and recording.

The drop-weight tests were performed by using the tri-pod shown in Fig. 4.8.4. A 62 kg weighting mass, suspended to the tripod was dropped on the ground from a height of 1.7 m to 1.8 m on the footpath at a distance of 3 m from the outer rail.
**Measurement campaign 5-6 February 2004**

This section presents a summary and a few samples of the measured data during this campaign.

**Axle loads:**
TGV wheel load measurements exhibited an average value of 83.7 kN, with a standard deviation of 8.5 kN. Axle loads showed an average value of 163 kN, with slightly lower scattering (Std dev. 13.4 kN = 8.3 %). This wheel load scattering includes natural differences between axle static loads (the front bogies of the extreme passenger cars are less loaded than the others) and the dynamic effect of unrounded wheels.

**Accelerations:**
Accelerations were measured on the sleepers, on the track surface and inside the sub-grade. On sleeper #115, the average peak vertical acceleration value for all wheels was about 0.78 g, which is indicative of good sleeper support conditions. The three types of trains exhibit fairly similar accelerations (0.75 g for single unit TGV; 0.80 g for double unit TGV and 0.77 g for Eurostar) and standard deviations of about 0.11 g. Negative (downward) accelerations were about half the positive (upward) ones. Figure 4.8.6 presents a summary of the recordings.

![Sleeper Z acceleration positive peak values vs wheel loads](image1)

![Sleeper Z acceleration negative peak values vs wheel loads](image2)

*Figure 4.8.6 Wheel load vs maximum generated sleeper acceleration*
Figure 4.8.7 presents a plot of the average peak acceleration values versus depth, and Figures 4.8.8 to 4.8.11 present samples of recorded load and acceleration time histories.

![Figure 4.8.7 Mean peak values of Z acceleration vs depth](image)

![Figure 4.8.7 Mean peak values of Z acceleration vs depth](image)
Similar measurements were made after the grouting of section 2 in April 2004. This section presents a comparison of some of the measurements before and after grouting.

Table 4.8.3 compares the average values of sleeper accelerations before and after grouting. The table indicates a considerable decrease (in the range 20-30 %) of the sleeper acceleration after grouting.
Figure 4.8.9 Accelerations on ground surface at 5m from rail in three directions

Table 4.8.3 Sleeper accelerations before and after grouting

<table>
<thead>
<tr>
<th>Train type</th>
<th>TGV - single unit</th>
<th>TGV - double unit</th>
<th>Eurostar</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sleeper-end #</td>
<td>113</td>
<td>115</td>
<td>113</td>
</tr>
<tr>
<td>Peak acceleration: pre-grouting (g)</td>
<td>0.78</td>
<td>0.8</td>
<td>0.8</td>
</tr>
<tr>
<td>Peak acceleration: post-grouting (g)</td>
<td>0.6</td>
<td>0.65</td>
<td>0.55</td>
</tr>
<tr>
<td>Decrease</td>
<td>23%</td>
<td>19%</td>
<td>31%</td>
</tr>
</tbody>
</table>
A similar reduction in acceleration levels was observed inside the embankment. Figure 4.8.10 compares the power spectral densities of accelerations in the sub-ballast for pre-grouting (red curve) and post-grouting (green curve) measurements. These results confirm the stiffening of the ground, hence effectiveness of grouting. The measurements also showed that some resonances emerged at 50 and 150 Hz.

![Figure 4.8.10 Comparison of mean power spectral densities in sub-ballast before and after grouting for Eurostar (top) TGV double unit (middle) and TGV single unit (bottom)](image)

### 4.8.5 Settlement monitoring

During the whole project period, altimetric measurements were performed at regular intervals (about every four months) to monitor the track settlement in Beugnâtre and compare the settlement rate before and after grouting. A 90 m section of track 1 was thus equipped with reference pins sealed onto the sleepers according to the pattern presented on Fig. 4.8.11 where the dark sleeper noted “T” is the reference sleeper #113.

Before grouting, three measurements were performed in June 2003, November 2003 and January 2004. The measured track profiles, corrected from the track slope (from Paris to Lille) are presented on Figure 4.8.12. During the 7 months period before grouting, a global average settlement of 0.72 mm was recorded with altimetric measurements. The grouting works induced an average 2mm uplift of the track.
After grouting, three measurements were made in August 2004, December 2004 and June 2005. Between August and December 2004, a settlement of 1.6 mm (i.e. higher than the pre-grouting settlement rate) was recorded (Fig. 4.8.13). During the first six months of 2005, however, no significant settlement was recorded.

The settlements experienced in Beugnâtre are probably in the limit of validation of the measurement method, which ensures, in ideal conditions, an accuracy of 0.1 mm. As measurements were performed at night and as the results were compared between several months time laps, this optimal accuracy might have somewhat been altered.

![Figure 4.8.12 Evolution of track profile with time before grouting](image)

![Figure 4.8.13 Evolution of track profile with time after grouting](image)
4.9 Soil improvement operation and field measurements at Amposta, Spain

One of the objectives of the project was assessment of a special grouting technique which can be carried out under normal operations, i.e. without traffic interruption. To this end a site was selected by CEDEX and ADIF and retrofitted during the project. This section presents the site conditions, methodology and operational process for grouting, and the measured results. In addition, a guideline was drafted for the implementation of this type of soil improvement.

4.9.1 Site characteristics

The retrofitting work was carried out on the southern embankment of the viaduct over the Ebro River on the conventional line Valencia-Barcelona of the Mediterranean Corridor. The embankment is located in Amposta, at the K.P. 180 of the above line. The embankment is made of compacted fill with a height of about 20 m and 2H:1V slopes (Figure 4.9.1).

The nature of the embankment materials is variable, with sand, silt and clay fractions being predominant at various levels, always accompanied by gravels and boulders that may be the predominant fraction locally. According to the identification tests, the plasticity of the embankment materials is low, with values of the liquid limit in the range of 18 to 23%, and of the plasticity index between 5 and 9%. The material has an average dry density of 1.86 ton/m³ and a water content of about 10%.

![Figure 4.9.1 General view of embankment at transition to viaduct](image)

4.9.2 Soil improvement principles

The main objectives with soil improvement/retrofitting were to achieve the following:

- Increasing and regulating the modulus of deformation of the embankment in the zones where it appeared less compact according to the results of geophysical explorations. It was believed that the lack of compaction of those zones had caused the need for frequent ballasting the track in this section.
- Making the soil improvement compatible with the normal service of the railway line.

It was essential to carry out the soil improvement from a working platform disconnected from the railway line. Therefore, a berm was built on the side of the embankment, so that the grouting could be carried out without interference to the railway traffic (Fig. 4.9.2). The selected grouting technique, with the use of sleeve tubes, could provide the following:

- Possibility of improving a predefined volume of embankment, by means of fans of sleeve tubes, installed from the working platform and oriented in such a way as to cover the grouting of that volume.
• Application, at each grouting sleeve, of the necessary number of passes of treatment until the required grouting pressure would be reached.

• Use, at each grouting pass, of a grouting mixture of necessary characteristics (viscosity, setting time, etc.) placed at the convenient rate and volume, in order not to move the rails in excess of the comfortable circulation deformation limit.

• A safe criterion to correlate the grouting pressure and the final parameters of shear strength in the treated soil so that the process of grouting could be properly controlled.

Figure 4.9.2 General view of working platform for grouting operation

4.9.3 Grouting procedure

Figure 4.9.3 shows the distribution of sleeve tubes used in different sections of the ground treatment. The transition zone to the viaduct abutment was treated by grouting from the upper part of the platform, down to a depth of 7 m, within a 20 m horizontal distance to the abutment. For each 10 m along the track, the improvement was carried out by decreasing the depth up to 2.5 m, in order to create a transition between the treated and untreated sections.

Figure 4.9.3 Key features of grouting operations along embankment
Theoretical formulas were developed to correlate the final grouting pressure and the corresponding parameters of shear strength of the treated soil based on Coulomb’s plasticity theory. By using this relationship, the final grouting pressures to be reached at different embankment levels were established. Figure 4.9.4 illustrates two phases of the soil improvement: drilling holes and grouting through the sleeve tubes.

The condition of maximum allowable track deformation for comfortable operation established vertical movements of the rails not larger than 3 mm between sections 5 m apart. This was achieved through different methods of levelling control.

![Image](image1.png)

*Figure 4.9.4 Steps of soil improvement: Drilling holes and grouting through sleeve tubes*

The improvement in the embankment condition achieved by means of the grouting treatment was verified using two different techniques: 1) direct comparison of physical parameters of the fill before and after treatment; 2) indirect control by measuring the dynamic response of the track to daily traffic before and after grouting.

Before grouting, PS-logging and cross-hole techniques were used to evaluate the variation with depth of the shear-wave velocity in the embankment. Similar measurements were made after grouting. Figure 4.9.5 displays a comparison between the two measurements. The data in the figure indicate a noticeable increase in the embankment stiffness (of the order of 2.5 – note: stiffness is proportional to square of shear wave velocity).

![Image](image2.png)

*Figure 4.9.5 Measured shear-wave velocities before (red) and after grouting (blue)*
In relation to the indirect control of fill improvement, Figure 4.9.6 presents a comparison between the measured deformations of the tracks for Euromed train passages before and after grouting. The ratio between the maximum deformations in this measurement was 1.26. In assessing the effectiveness of grouting, however, one should take into consideration that the treatment was basically applied to the platform (i.e. form layer and embankment) without any effect on the track structure (i.e. ballast and sub-ballast). To isolate the stiffening effect of the platform a simple analysis was made using the data from the Guadalajara test site (Sec. 4.8.3) to compute the contribution of the various element of the track-embankment to the total track deformation. The following gives the elements of this approximate computation.

- Settlement of rail due to track structure deformation = 1.0 mm
- Settlement of rail due to platform deformation before grouting = 1.54 – 1.0 = 0.54 mm
- Settlement of rail due to platform deformation after grouting = 1.23 – 1.0 = 0.23 mm
- Ratio of settlement of top of platform, before and after grouting = 0.54/0.23 = 2.3

Consequently, it was shown through both direct and indirect methods of estimation that the modulus of deformation of the grouted fill amounted to about 2.5 times the initial modulus.

Figure 4.9.6 Track deformation measurements for Euromed train before (left plot) and after (right plot) grouting
5 Deliverables

Table 5.1 gives a list of the deliverables according to the project’s DoW (Note: deliverables starting with “D” are those meant mainly for internal use during the project and for project management. Deliverables starting with “MD” are the **Main (official) Deliverables** of the project).

**Table 5.1 List of Deliverables according to project’s DoW**

<table>
<thead>
<tr>
<th>Deliverable</th>
<th>Date (month)</th>
<th>Relevant WP</th>
<th>Nature of deliverable and brief description</th>
</tr>
</thead>
<tbody>
<tr>
<td>D1A</td>
<td>6</td>
<td>WP1</td>
<td>Preliminary report on data obtained from different sites on ballast track performance, maintenance problems and available sub-grade data</td>
</tr>
<tr>
<td>D1B</td>
<td>27</td>
<td>WP1</td>
<td>Final report on correlations developed with regard to existing track and geotechnical data and track performance.</td>
</tr>
<tr>
<td>D2A</td>
<td>12</td>
<td>WP2</td>
<td>Report on laboratory testing of materials (ballast, sub-grade layer)</td>
</tr>
<tr>
<td>D2B</td>
<td>18</td>
<td>WP2</td>
<td>Midterm report on validation of physical model tests</td>
</tr>
<tr>
<td>D2C</td>
<td>36</td>
<td>WP2</td>
<td>Final report &amp; lab results: Physical model tests on ballasted railway tracks</td>
</tr>
<tr>
<td>MD3</td>
<td>18</td>
<td>WP3</td>
<td>Software including documentation</td>
</tr>
<tr>
<td>D4A</td>
<td>18</td>
<td>WP4</td>
<td>Midterm Report on geotechnical investigations and dynamic measurements</td>
</tr>
<tr>
<td>MD4B</td>
<td>36</td>
<td>WP4</td>
<td>Final Report on validation of numerical simulation of track performance from in situ measurements and physical model tests</td>
</tr>
<tr>
<td>MD5</td>
<td>36</td>
<td>WP5</td>
<td>Final Report on leading parameters affecting track behaviour</td>
</tr>
<tr>
<td>MD6</td>
<td>36</td>
<td>WP6</td>
<td>Final Report on new design guidelines and maintenance procedure</td>
</tr>
<tr>
<td>MD7</td>
<td>30</td>
<td>WP5</td>
<td>Final Report on specification of grouting procedure.</td>
</tr>
</tbody>
</table>

While keeping the same themes, it was decided to split the original deliverables and publish the main deliverables under different titles. This resulted in 8 Main Deliverables (as opposed to the original 5 MDs) as listed below. Both digital and printed copies of these reports were submitted to the European Commission.

- MD1: Track measurements in Beugnâtre (corresponding to original MD4B)
- MD2: Track measurements in Zoufftgen (corresponding to original MD4B)
- MD3: Instrumentation, monitoring and physical modelling of high-speed line in Spain (corresponding to original MD4B)
- MD4: Numerical modelling of short-term and long-term track response (corresponding to original MD3)
- MD5: Numerical simulation of train-track dynamics (corresponding to original MD3 and MD5)
- MD6: Numerical modelling of track-box behaviour
- MD7: Improved design and cost-benefit analysis (corresponding to original MD6)
- MD8: Retrofitting of track sections (corresponding to original MD7)
5.1 Comparison of original planned activities and work actually undertaken

Figure 5.1 displays the original schedule of the activities according to the DoW, The same figure indicates the updated schedule at the end of the 2nd year by marking the extended activities in Red. In addition, the Commission approved a 3 month extension of the project.

At the end of the project, the consortium achieved all the tasks in the DoW, except Task 1.3 in the way it as was originally envisaged. The objective of this task was to use the collected network data to establish a correlation between maintenance and network data. Despite an extensive research on this topic, the data did not point to a clear correlation. This was partly because one of the important parameters, namely the track stiffness variation, which through work in other tasks turned out to be a key governing parameter, was not available for the selected network locations. However, measurements and numerical simulations revealed the significance of this parameter, and methods were proposed to improve track maintenance by properly designing (or retrofitting) tracks with due considerations for this parameter.

6 Management and co-ordination

The activities of the project were coordinated by NGI, and the management decisions were made by the Project Coordination Committee or Steering Committee (SC). Each partner had a representative in SC, as listed below:

- Amir M. Kaynia (NGI) - Coordinator: amir.m.kaynia@ngi.no
- Laurent Schmitt (SNCF): laurent.schmitt@snf.fr
- Antonio Lozano (ADIF): alozano@adif.es
- Alain Pecker (GdS): alain.pecker@geodynamique.com
- Vicente Cuéllar (CEDEX): Vicente.Cuellar@cedex.es
- Didier Clouteau (ECP) – Chairman of SC: clouteau@mss.ecp.fr
- Tore Dahlberg (LU): torda@ikp.liu.se
- Eric Berggren (BV): eric.berggren@banverket.se

The project had the kick-off meeting in Madrid on July 10, 2002. The SC held 6 project meetings (2 per year) in Paris (twice), Oslo (twice), Madrid (once) and Borlänge, Sweden (once). The final meeting of the project was held in Oslo on Sept. 23, 2005 with the presence of the project officer, Mr. H. McBryan. In addition, five smaller meetings were held in Paris, Madrid and Borlänge for the follow-up/coordination of the local activities.

Other than meetings, the partners communicated regularly by Email/Telephone, while keeping the coordinator informed of all correspondences. The partners exchanged data/reports through the project’s limited-access web site (http://extranet2.ngi.no) maintained by NGI. In addition, a public domain web site was established by NGI (http://www.supertrack.no). This site provides general information about the project and the results to the public.

The partners signed a Consortium Agreement which defines the partners’ roles and responsibilities towards the project and each other, process of decision making, handling of financial matters and payments, liabilities and access rights.

The partners enjoyed a very friendly and fruitful cooperation throughout the project.
Fig. 5.1 Original (green) and Revised (red) Schedules of activities

<table>
<thead>
<tr>
<th>Workpackage Description</th>
<th>Partner Manmonths*</th>
<th>1st year</th>
<th>2nd year</th>
<th>3rd year</th>
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</thead>
<tbody>
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<td></td>
<td>NGI</td>
<td>SNCF</td>
<td>RENFE</td>
<td>GIF</td>
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<td>WP1 Network investigations</td>
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<td>1.1 Data collection</td>
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<td>6.2</td>
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<tr>
<td>1.2 Site identification</td>
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<td>1.3 Correlations</td>
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<td>WP2 Physical modelling</td>
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<td>2.2</td>
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<td>2</td>
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<tr>
<td>2.1 Material test</td>
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<td>2.2 Short-term cross-section test</td>
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<td>2.3 Long term cross-section test</td>
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<td>WP3 Numerical modelling</td>
<td>3</td>
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<td>3.1 Train-track interaction</td>
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<td>3.2 Short-term models</td>
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<td>3.3 Long-term models</td>
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<td>4.4 Validation</td>
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<td>WP5 Performance enhancing</td>
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<td>5.1 Leading parameters</td>
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<td>5.2 Demonstration</td>
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<td>WP6 Exploitation</td>
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<td>6.1 Exploitation</td>
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<td>WP7 Management</td>
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<td>1.6</td>
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<td>TOTAL MAN-MONTHS*</td>
<td>14</td>
<td>20.4</td>
<td>19.5</td>
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* Excluding subcontractors, which represent about 27 man-months
7 Results and conclusions

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 shown that an important factor contributing to track deterioration is the non-homogeneity 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 or hanging sleepers. This research has vividly shown that proper design/ construction of a line with due consideration for track homogeneity ensures a healthy track with minimum maintenance and sustained satisfactory performance.

The following is a summary of the results and achievements from this study:

1) The cyclic tri-axial tests on large scale samples at NGI and medium-scale samples at CEDEX represent 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.

2) 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. This state-of-the-art facility provides the opportunity to examine the behaviour of full-scale tracks under repeated train loading and under controlled environmental conditions. The construction of the track box was heavily financed by external sources provided by ADIF in Spain. This is considered an added value of the project to the European Community.

3) The numerical models for simulation of non-linearities and non-homogeneities in the track and their effects on the long-term response of the track represent state-of-the-art researches in this field. There are hardly any solutions in the market that can compete with these models in terms of detail and scale of analysis. These studies will motivate the university partners to educate new doctoral students, create post-doc opportunities and generate new research projects for their institutions.

4) The measurements at the test sites were made with state-of-the-art sensors and instrumentation set-ups. The collected data, which have extremely high quality, will provide valuable databases for future research and calibration of numerical tools by the research community. Moreover, the experiences gained in these studies will help streamline future measurements and improve their qualities.

5) The innovative grouting technique implemented successfully at a site in Spain is a valuable contribution of this project to the state of practice. The main advantage of this method is that it does not require interruption of train traffic; therefore, it can potentially represent a cost-effective retrofitting method. Besides the railway industry, the method has applications in the construction industry. Manufacturing of new machines, grouting equipment, and other related machinery are potential economic spin-offs of this work.

While the railway administrative partners of the project can directly incorporate the results of the project in their routine design, the educational and R&D partners have the possibility of exploiting the results of their research in railway as well as other related projects.
The project has had an ambition for highly scientific and original research. Most of the dissemination of the research work is expected to be disseminated to the engineering community through articles in seminars, workshops and international conferences as well as refereed journals. The following is a list of articles published at the time of writing this final report.


8 Acknowledgement

This research was performed under Contract G1RD-CT-2002-00777 in the GROWTH Programme of the European Commission’s 5th Framework Program.

The participants of the project would like to to express their appreciation to the European Commission for the partial financial support to this research. The participants would also like to thank the Project’s officer, Mr. Hugh McBryan who provided timely administrative advices as well as encouragement and guidance throughout the project execution.
9 References

This report is based on the internal documents/notes produced during the project (Sec. 4) as well as the final Deliverables of the project as listed below:

1. Final Report 1 by SNCF: Track measurements in Beugnâtre
2. Final Report 2 by SNCF: Track measurements in Zoufftgen
3. Final Report by CEDEX: Instrumentation, monitoring and physical modelling of high-speed line in Spain
5. Final Report by LU: Numerical simulation of train-track dynamics
7. Final Report by ADIF: Improved design and cost-benefit analysis
8. Final Report by CEDEX: Retrofitting of track sections