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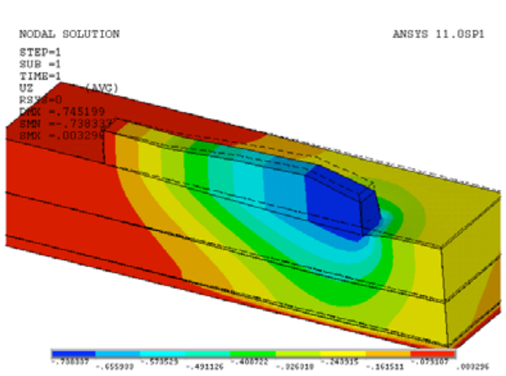
Thematic Priority 6:
Sustainable Development,
Global Change and Ecosystems

D2.2.6 GL
Guideline for subgrade reinforcement with geosynthetics

Part 1: Enhancement of track using under-ballast geosynthetics
Part 2: Improvement of transition zones on conventional lines



INNOTRACK GUIDELINE

Deliverable D2.2.6

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Executive Summary

In the Part 1 results of a campaign oriented to the enhancement of railway track using under-ballast geosynthetics are presented. This technique was tested in laboratory conditions, by Finite Element Modelling and also in-situ. Physical and numerical modelling comprised both short-term and long-term loading.

The objective of Part 2 is to assess the efficiency of innovative techniques to improve the behaviour of platforms in conventional railway lines. The section summarizes the results obtained in two track stiffness measurement campaigns before and after the improvement of 64 m of a transition zone. The improvement consisted in replacing the 2 upper meters with a well-compacted QS-3 type of material reinforced with 2 layers of high elastic modulus geogrid.

Part 1:

Enhancement of track using under-ballast geosynthetics

1. Introduction

Reinforcement of the railway structure with inserted geosynthetic material to mitigate the effects of high dynamic loading is studied in a detailed Finite Element analysis based on lab measurements. Apart from settlements reduction, the reinforcing effect of a geosynthetic material can be expressed in terms of the thickness reduction of the good-quality sub-ballast material.

The research objective was to explore the possibilities of numerical modelling to assess the reinforcing effects of a particular type of geosynthetic material used in the construction of railway structure. For this purpose the Finite Element Method (FEM) was chosen as the most suitable numerical method. FEM enables a proper simulation of the interaction between the ballasted material and the particular type of reinforcing geosynthetic material and also to properly describe the complex stress state in the three-dimensional test box. The aim was not only to prepare one FE model but to assess the complete methodology for quick and reliable assessment of the reinforcing effect both from the short and long term point of view.

To enable a comparison between the numerical modelling and performed laboratory tests, a set of FE models of the test box with the particular design of the railway construction were designed and deformation responses studied for short term and long term loading. Several parameters were evaluated for each model; in particular those were also measured experimentally (vertical displacements at 12 locations). However, the FE models also provides insight into other important physical quantities, e.g. strains and stresses in the geosynthetic.

The methodology described in the following is quite general and can be used for any type of geosynthetic material provided proper material properties have been identified. These material properties are easy to obtain from uniaxial of biaxial loading tests and for most of the geosynthetics are provided by the manufacturer. Since it is not possible to prepare a completely general tool for estimation of the reinforcing effect of any type of geosynthetic material, only general requirements and recommendations are drawn.

Further details may be found in INNOTRACK deliverable D2.2.9.

2. Practical use of Geosynthetics

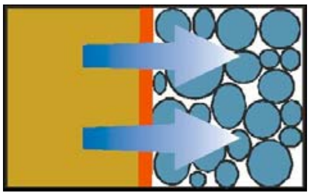
2.1. Geosynthetic functions

The operation method of Geosynthetics can be characterized by the following functions:

- FILTRATION
- DRAINAGE
- SEPARATION
- REINFORCEMENT

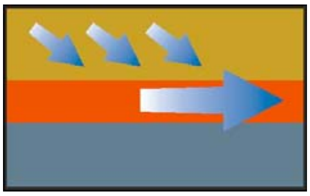
2.1.1. Filtration

Geosynthetics have to retain fine particles when water passes from fine-grained to coarse-grained soil layers. Its function is to provide permanent mechanical and hydraulic filter stability for the period of operation.



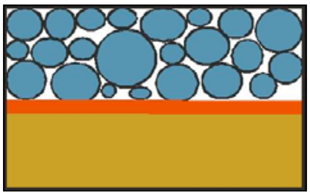
2.1.2. Drainage

Geosynthetics have to drain off liquid and gaseous media (drainage and venting of soil). Its function is to provide water drainage and gas venting in the plane of the geosynthetic.



2.1.3. Separation

Geosynthetics have to separate two layers of soil with different physical soil properties like grain size distribution, consistency and density. The mixing of the two materials under intensive dynamic and static traffic loading has to be prevented permanently.



2.1.4. Reinforcement

Geosynthetics promote an improved soil quality and thus increase the structural stability. The geosynthetic function is to increase soil shear strength of the railway sub-structure by providing a bonding mechanism of the geosynthetic-soil system.

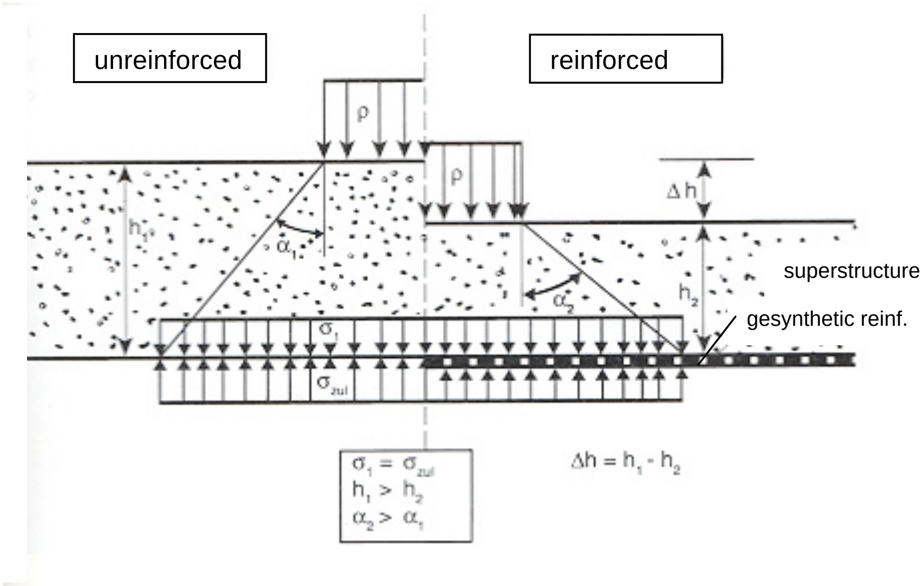


Fig. 1: Comparison of reinforced and unreinforced superstructure

2.2. Approved experiences – State of the art

Since more than 35 years geosynthetics are used in railway tracks to improve the structural stability and expand service life time. The main location of the geosynthetics, to provide the required functions is between the subgrade and the filter/protection blanket.

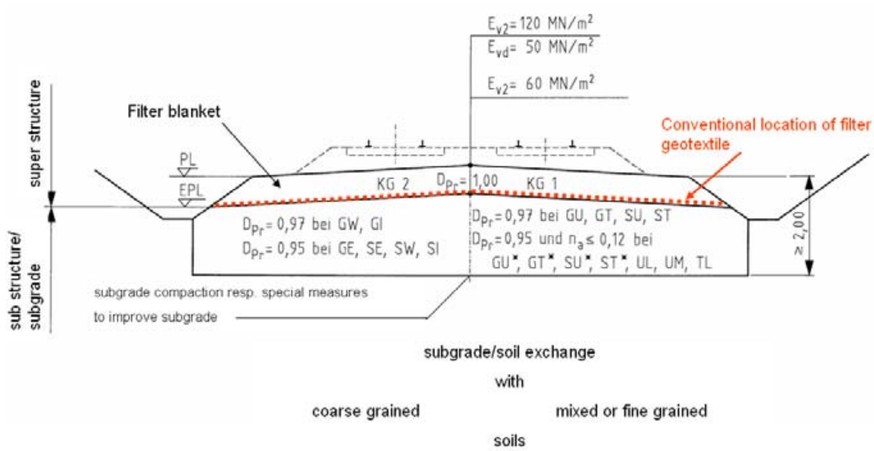


Fig. 2: Example of using geosynthetics between the subgrade and the filter/protection blanket

Mainly multifunctional geotextiles, that provide separation, filtration and reinforcement, are in use. These have proven their long term performance. Also geogrids are installed in this location if additional stiffness/reinforcement is needed, for instance to avoid differential settlements.

The mechanical and hydraulic properties of the suitable Geosynthetics are defined with required test standards in the EN 13250 'Required characteristics for geotextiles and geotextile-related products used in the construction of railways'. National specifications determine the required product values for the specified tests.

2.3. Under ballast use

Especially for secondary rail tracks the use of geosynthetics directly under the ballast to fully substitute a formation, protection or filter blanket is being considered.

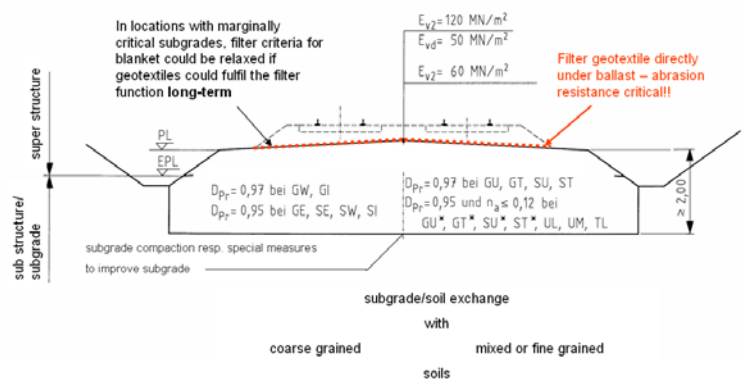


Fig. 3: Example of using geosynthetics under the ballast

As standard geotextiles are not stable enough to withstand the higher dynamic forces, special products were developed. These feature improved roughness and abrasion resistance as compared to standard geotextiles.

These products are under observance to prove their suitability for service life times ≥ 20 years.

2.4. Installation – Handling

2.4.1. Conventional

Fig. 4: Preparation of geotextile roll

Fig. 5: Placement below ballast

2.4.2. Mechanised

Fig. 6: Fully mechanized maintenance work

Fig. 7: Geotextile is fixed on the maintenance machinery

3. Investigating the effect of geosynthetics on the bearing capacity of the substructure

3.1. Computational methods

In order to determine the effect of geosynthetics inserted into the substructure construction, analytical or numerical simulations may be used. These consider the substructure as layers of elastic materials loaded by the superstructure and the weight of railway vehicles. The results of computations and their usability depend on the selection of input parameters characterizing the material properties of individual substructure layers, (see Burrow [2]). Another possibility is to use the Finite Element Method to calculate the reinforcing effect of a particular type of geosynthetics as presented in this guideline.

3.2. Experimental measurements in laboratory

In order to specify the effect of geosynthetics inserted into the substructure construction, scaled models of substructure constructions may be built in laboratory conditions (see Tebay [3], Sharpe [4] etc.) or as a full-scale test box for a half-sleeper (see Göbel [5], [6]). A full-scale model consisting several sleepers may also be arranged in a test room (see Thom [7]). Substructure models allow the specification of characteristics of individual layers and bearing capacity values by direct measurement.

The evaluation of the effect of geosynthetics on the substructure bearing capacity values always requires the establishement of a reference model without geosynthetics. It is very difficult to guarantee constant moisture content of the subgrade soil (if a fine-grained soil is used) during measurements on models in laboratory conditions as measurements on one model may take several days depending on the required load conditions.

In the construction of measuring model, the substructure may be exposed to static or cyclic loading. During cyclic loading, the evolution of the characteristics of the structural layers of the substructure may be investigated. Examples of such characteristics are settlement, bearing capacity, etc.

3.3. Experimental measurement in field

In order to specify the effect of geosynthetics inserted into the substructure construction, a section with geosynthetics and a reference section without geosynthetics may be built on an actual railway track under identical conditions. After their consolidation by railway traffic, both sections allow direct bearing capacity measurements (using the plate load tests) to determine the effect of the inserted geosynthetic. See Vanggaard [8].

4. Laboratory investigations of the effect of geosynthetics on the bearing capacity of the substructure

To determine the effect of geosynthetics inserted into the substructure construction, a series of experimental investigations of substructure model constructions in an test box were performed by static loading. In a second series of experimental measurements, the models were exposed to cyclic loading by a force corresponding to the weight acting on one wheel.

4.1. Experiment setup

Model constructions of substructures were loaded in a test box, which was made up of welded sections with removable walls of timber beams. To minimize the model friction against the box walls, these were panelled with galvanized plates. To allow the measurement of modulii of deformation at different levels of the substructure, a movable load frame was designed for the load tests. Basic dimensions of the test box are displayed in Fig. 8. Details may be found in INNOTRACK report D.2.2.9.

4.2. Model constructions

The substructure construction was composed of four layers. The subgrade was simulated by a two-layer system consisting of rubber plates, overlaid with a thin layer of dry sand. The rubber plates allowed simulating a constant low bearing capacity of the subgrade during the whole experiment (E_{V2} = 11.0 MPa), whilst the thin layer of fine sand allowed the interlocking of crushed stone mixture in model constructions with geogrids.

A sub-ballast of crushed stone mixture with a grain size of 0 – 32 mm and a thickness of 20 cm after compaction was employed. This layer was overlaid with a layer of ballast with a grain size of 32 – 63 mm and a thickness of 35 cm after compaction.

On the ballast layer surface, an instrumented B 91 S/1 half-sleeper of pre-stressed concrete with a 50 cm long piece of UIC 60 rail was mounted. The rail was mounted on a rubber pad of WU 7 type with a thickness of 7 mm and fixed onto the sleeper using fastening without soleplates and Vossloh Skl 14 clips.

During experimental measurements, various geosynthetic materials were inserted into the substructure constructions at the interface between the simulated subgrade and the sub-ballast, and between the sub-ballast and the ballast.

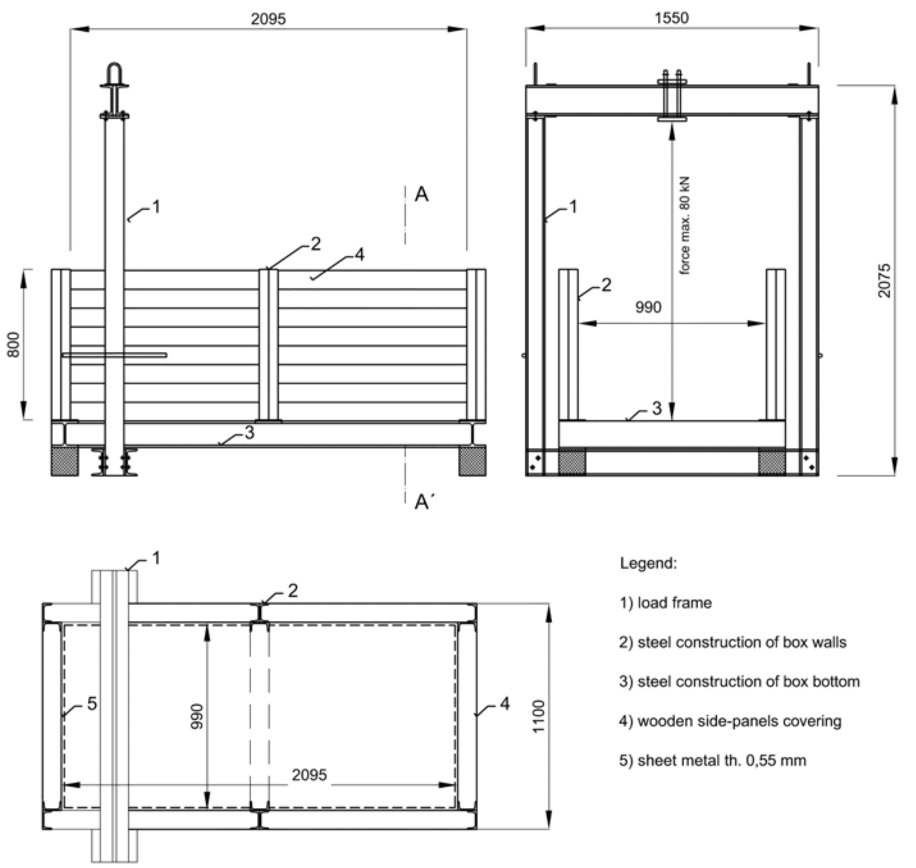


Fig. 8: Basic dimensions of the test box

4.3. Geosynthetics studied

In order to evaluate the effect of geosynthetic materials applied in a railway track construction primarily regarding the enhancement of the bearing capacity of substructure, the following types of geosynthetics were inserted into the model track bed constructions:

- NONWOVEN SEPARATION GEOTEXTILE,
- WOVEN REINFORCING GEOTEXTILE,
- REINFORCING GEOCOMPOSITE (CONSISTING OF NONWOVEN GEOTEXTILE AND GEOGRID COMPOSED OF CORDS OF POLYESTER FIBRES),

- 3 TYPES OF REINFORCING GEOGRIDS (KNITTED, WELDED, EXTRUDED).

These choises represent common types of geosynthetics produced. The selected materials had similar declared reinforcing characteristics (in particular tensile strength) thus allowing a mutual comparison of the results of measurements.

A total of 8 model constructions were assembled and measured. One without any reinforcement, four with geosynthetics on the silica sand-crushed stone mixture interface (nonwoven geotextile, woven geotextile, reinforcing geocomposite and geogrid) and three with a reinforcing geogrid at the ballast/sub-ballast interface. While placing the reinforcing geogrid on the sub-ballast layer, a separation geotextile was simultaneously inserted between the silica sand and the crushed stone mixture for practical reasons (see Fig. 9).

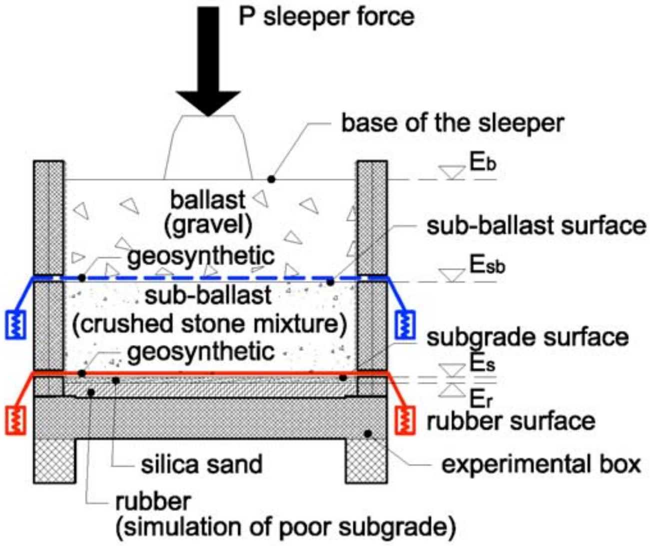


Fig. 9: Placing the geosynthetics

The geosynthetics were inserted into the test box so as to allow the free motion of overlapping ends through a gap left in the box walls and the box faces. The overhanging ends of geosynthetics were loaded by steel tie rods to ensure their adequate stretching corresponding to the force of manual stretching applied during mounting in field.

4.4. Loading

All model constructions were loaded by a short-term static load force corresponding to a standard axle load of 22.5 t (i.e. 42 kN) repeated 50 times. This repeated loading represented the initial consolidation of the whole multi-layer system. The load was applied by a manual hydraulic device with a set of cylindrical adapters (see Fig. 10).

Fig. 10: Short-term loading

Five models (four with geosynthetics), were subjected to long-term cyclic loading was perform applied by an actuator mounted on a rigid steel frame (see Fig. 11).

Fig. 11: Long-term cyclic loading

In the cyclic loading, the rail head was loaded with a force ranging from 2 to 42 kN with a sinusoidal pattern and a frequency of 3 Hz. Individual model constructions were loaded by a total of 250 000 cycles.

4.5. Monitored parameters

During the consolidation process (short-term loading), the magnitude of loading was continually recorded by a digital manometer and the settlement at 12 points were recorded. Settlements were measured at 3 different levels – on the sleeper surface, on the ballast-crushed stone mixture interface and under the crushed stone mixture. At each level the settlements were measured at 4 points placed at identical distances from the intersection point of the rail axis and the sleeper axis (see Fig. 12).

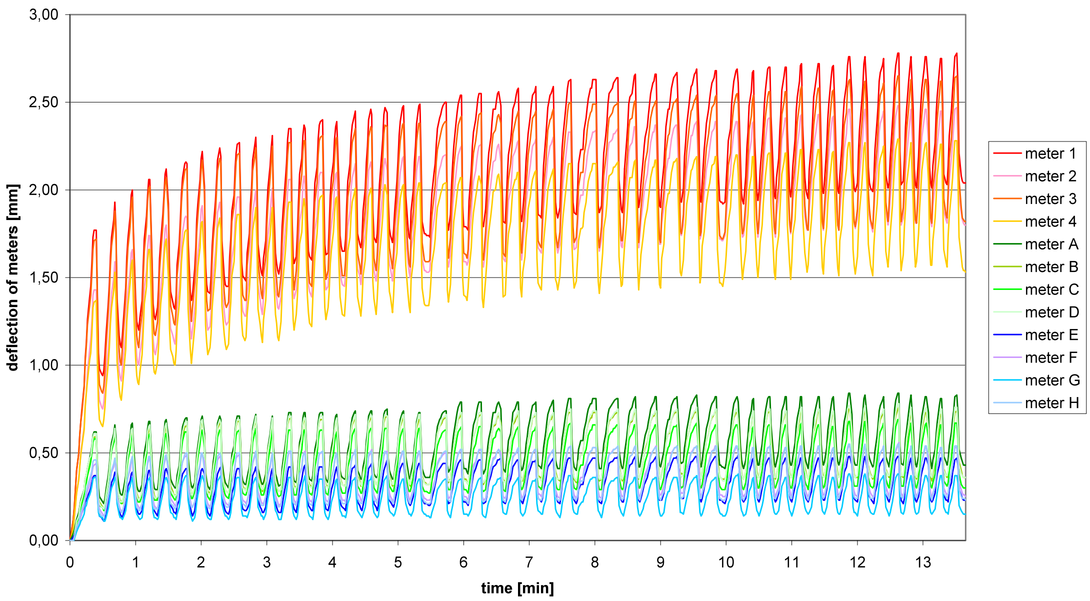


Fig. 12: Course of settlement – short-term loading

The settlements in the interfaces of the track bed layers were measured by settlement meters consisting of a steel plate and a vertical steel rod embedded in the track bed. The vertical rods were protected by steel tubes. To compare the behaviour of individual model constructions, the differences in settlements of the meters between zero and maximum loading were used.

During the long-term loading, settlements of 12 points (rail, sleeper surface and ballast- crushed stone mixture interface) were measured. Settlements were recorded before loading and after 100, 1 000, 10 000, 50 000, 100 000 and 250 000 cycles, in 5 steps (Fig. 13).

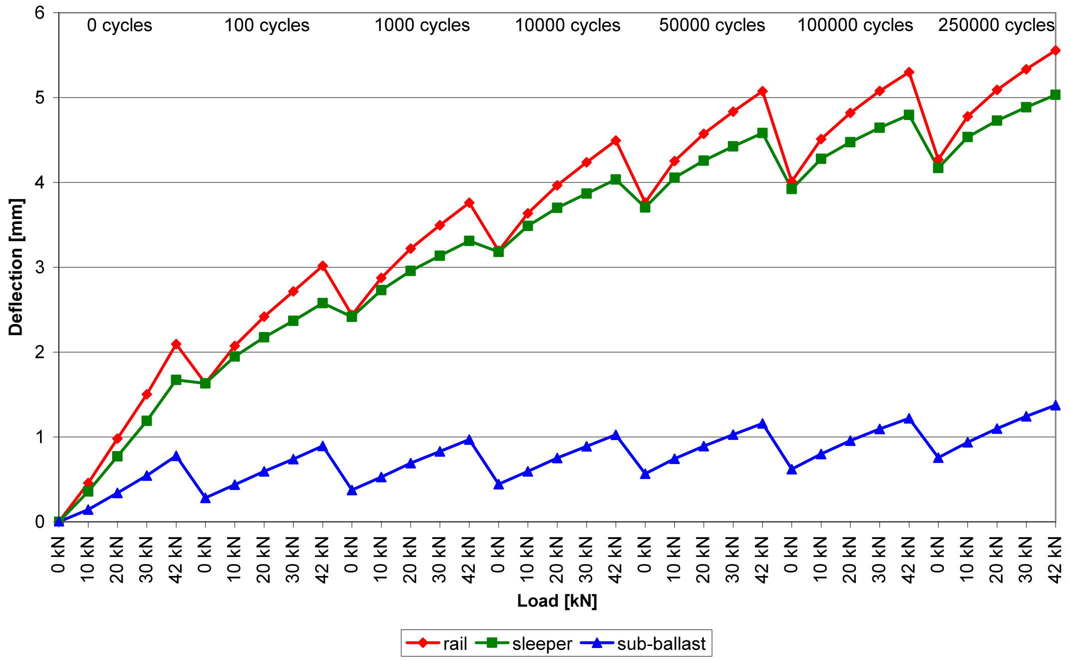


Fig. 13: Course of settlement – long-term loading

After short- and long-term loading, static plate load tests and impact load tests were carried out on the surface of the ballast layer, the crushed stone mixture and the silica sand (Fig. 14, Fig. 15). The load tests were performed and evaluated in accordance to DIN 18 134 " Plattendruckversuch".

Fig. 14: Static plate load test

Fig. 15: Impact load test

4.6. Outcomes from lab investigations

Experimental measurements performed on model constructions showed that comparative measurements carried out after a relatively low degree of track bed consolidation (50 loading cycles) allow a mutual comparison of results. From this optimum uses of geosynthetics could be specified.

The results also imply that after short-term cyclic loading reinforced model constructions showed larger track bed settlements as compared to the non-reinforced construction. This is caused by incomplete track bed consolidation. In order to achieve complete track bed consolidation, i.e. the establishment of a desirable contact of the ballast or crushed stone grains and the geosynthetic, manual compaction in the order of tens of cycles is insufficient for reinforced constructions.

In model constructions with inserted geosynthetics there is a marked difference between the constructions with one and two geosynthetics. In general, the latter show higher settlements. Very good results in terms of bearing capacity were achieved in the case of placing a geocomposite on the subgrade and also in the case of placing a welded geogrid on the sub-ballast layer in combination with a separation nonwoven geotextile placed on the subgrade (see Fig. 16).

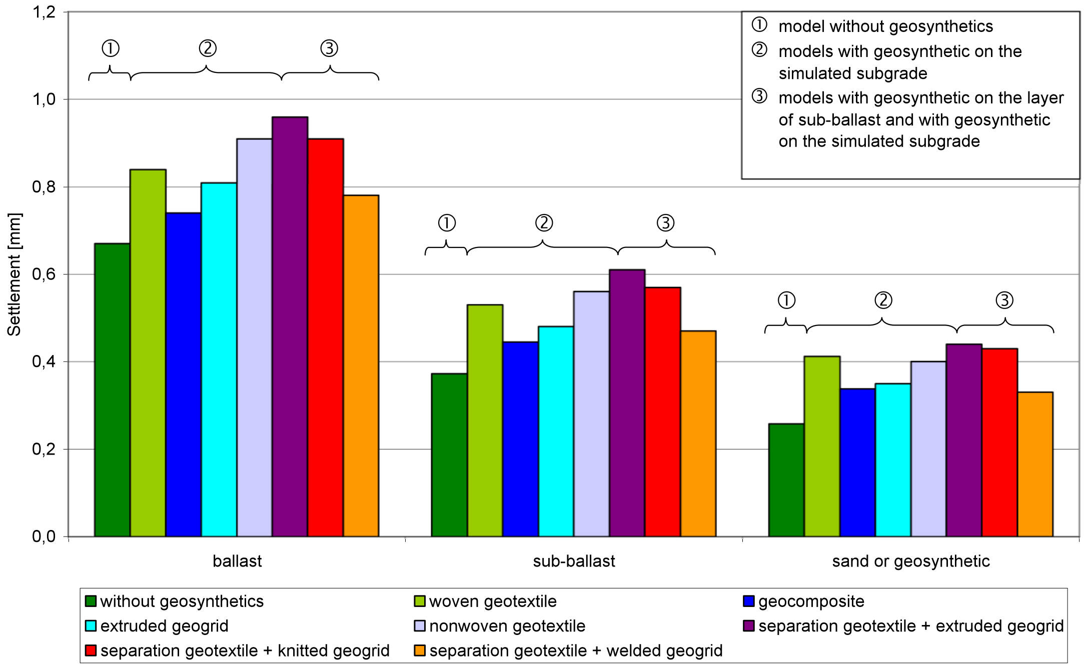


Fig. 16: Summary results of mean settlements at individual height levels (short-term loading)

The results of long-term cyclic loading imply that the elastic settlement values after 250 000 loading cycles fluctuate between 0.84 mm and 0.86 mm for all tests. This confirms that the substructure in all models had been sufficiently compacted, and no significant difference due to increased rigidity of the subgrade under the sleeper was obtained from in using various types of geogrids.

The analysis of determined values of the modulii of deformation on the ballast surface implies that by inserting an extruded geogrid under the ballast the total bearing capacity of the substructure increases about 15 % in comparison to the case when the geogrid is laid under the sub-ballast layer of crushed stone mixture.

The results of measurements on substructure models led to a conclusion that the use of geosynthetics on the sub-ballast surface has a greater influence on the enhancement of its bearing capacity than the use of geosysnthetics under the sub-ballast layer.

5. Finite Element Modelling

5.1. FE Convergence study

Since the Finite Element Method gives an approximate solution of a set of partial differential equations it is essential to assess the accuracy of the solution. For all the models used in the FE simulations a convergence study was performed (see delivereable D2.2.9). From the convergence study the optimal mesh size based on the observed magnitudes of selected values is determined as the maximal value of all the element sizes:

$$esize = \max\{esize(\sigma_1), esize(\tau_1), esize(w_{max})\}$$

This approach gives the resulting FE model consisting of 8240 nodes.

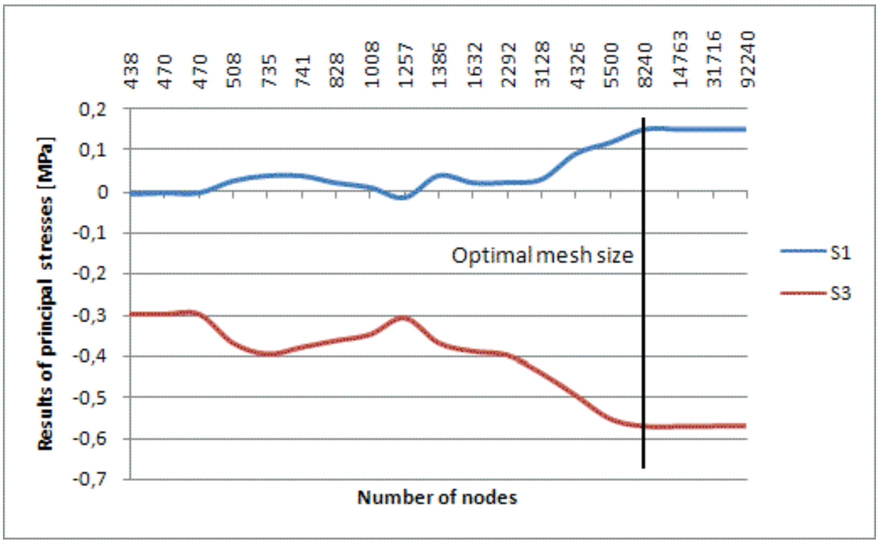


Fig. 17: Convergence study, principal stresses for various mesh size

5.2. FE Loading

The FE model is loaded according to the load in performed experiments. Symmetry in the longitudinal axis is utilised. The concentrated load P_1 is applied on a half of concrete sleeper according to the axle load $2Q$ equal to 22.5tonnes (see Fig. 18).

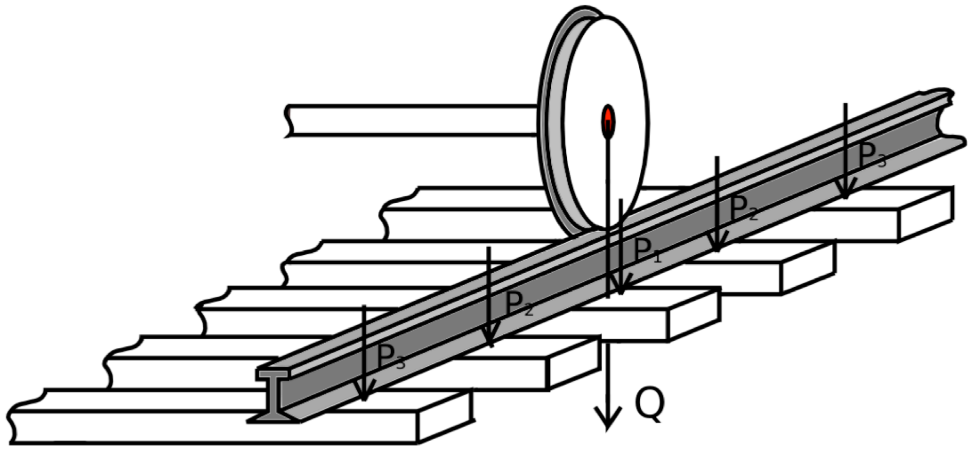


Fig. 18: Concentrated load acting on the sleeper

The magnitude is evaluated using the calculation of beam resting on resilient supports according to Zimmermann (see deliverable D2.2.9). For the chosen dynamic coefficient (1.25), the force acting on the sleeper is calculated as:

$$P_1 \approx 42.0\,\mathrm{kN}$$

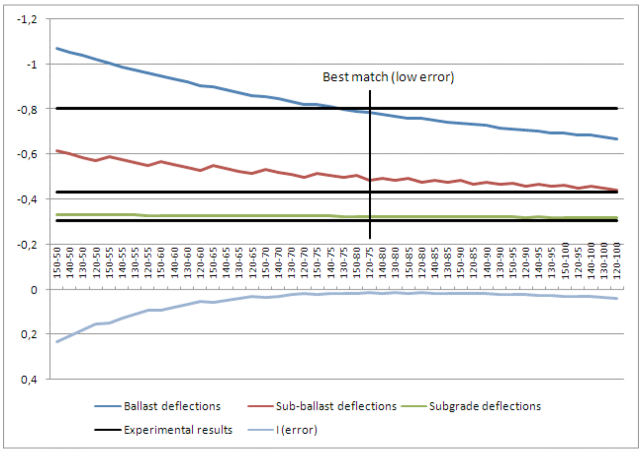
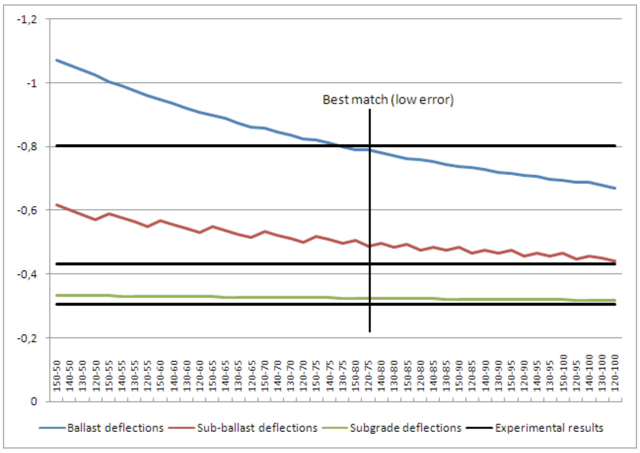
This value of P_1 is used in the experiments and also as the static load in the numerical simulations. Two configurations in terms of load application have been adopted:

No	Loading	Remarks
1	The load is applied on the selected nodes with coupled degrees of freedom. The sleeper is modelled as perfectly rigid solid by coupling vertical degrees of freedom at the sleeper.	
2	The sleeper is assumed as elastic solid; simplified geometry is modelled and divided into finite elements. Linear elastic isotropic material properties of reinforced concrete are prescribed.	

5.3. Material model for the ballast

Linear elastic material properties of ballast, sub-ballast and subgrade material are identified from experimental results using a stepwise identification method. The material properties are set to a starting values and optimized to a best-fit for the calculated deflections at the 12 measured positions. For this stepwise approximation the reference experiment without inserted geosynthetic is used. For the whole investigation static monotonic loading is used. The approximation is summarized in the flow-chart below:

No	Material properties estimation	Remarks
1	Material properties are estimated based on the results of the experiments. The deflections in layers of interest y_{measured} are measured.	
2	Values of Young's modulus of the ballast and the ratio between ballast and sub-ballast moduli are varied for the expected range	
3	The error of estimation of unknown values of Young's modulus is represented by: $$I = \sum_{k=1}^{3} (y_{\text{estimated}} - y_{\text{measured}})^2$$ where $y_{estimated}$ and $y_{measured}$ are vertical displacements obtained from FE simulations and experimental tests.	The estimation procedure should be to used for all modelled interfaces separately
4	Minimizing function *I* gives the optimal values for the modulii of all layers in the model.	
5	Young's modulus for the sub-ballast is expressed in a term proportional to the properties of ballast obtained in the previous step.	

No	Material properties estimation	Remarks
	  *Fig. 19: Stepwise approximation of Young's modulii of studied materials*	

5.4. *Material characteristics of geosynthetics*

Interaction between the geosynthetics and the ballast has to be modelled properly. Different approach in the FE modelling of geosynthetics have to be used to correctly describe the reinforcing and separating functions. For the separation function the interaction between soil and geotextiles is modelled by contact with friction. The reinforcing effect of the geogrid is modelled through inserted boundary conditions. This special effort is required since the use of contact elements gives an insufficient reinforcing effect. The interaction between the individual grains and the grid material is solved using an inserted boundary condition in the FE layer where the geogrid is placed.

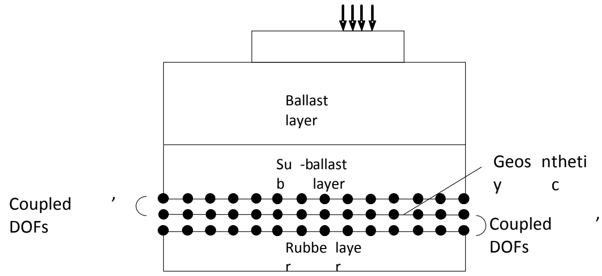
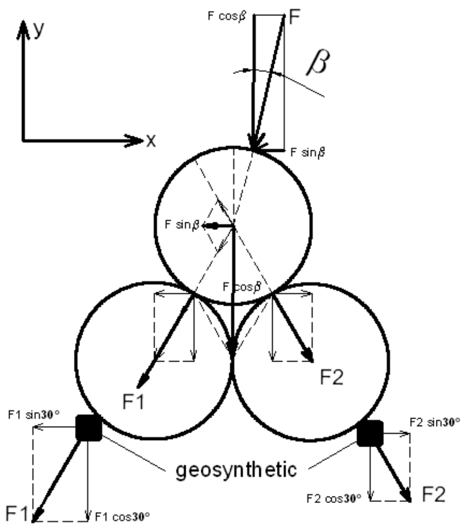
From geosynthetic materials used in civil engineering only three types are employed for their reinforcing function. These geosynthetics are listed in following table.

Tab. 1: Primary functions of geosynthetics

Type	Separation	Reinforcement	Filtration
Geotextile	×	×	×
Geogrid		×	
Geocomposite	×	×	×

5.5. Modelling of geosynthetic-soil interaction

No	Geosynthetics modeling	Remarks
1	Engineering material properties are obtained from the manufacturer of the particular geosynthetic. In the range of small deformations the material is considered linear.	
2	Geosynthetic is modelled using shell or if possible solid-shell (solid with zero bending stiffness) elements with separated nodes on the upper and lower surface.	
3	Interaction between geosynthetic and soil is represented by coupling of the degrees of freedom (DOF) of the corresponding nodes or by contact which is defined by a friction coefficient.	

No	Geosynthetics modeling	Remarks
	 *Fig. 20 Geotextile approach – coupled DOFs*	
4	The reinforcing effect of geogrids and geocomposites is modelled by inserted boundary conditions (IBCs) according to uniaxiality or biaxiality of the geogrids simulating interlocking of individual grains (see deliverable D2.2.9).  *Fig. 21 Forces acting on grains and geosynthetic*	
5	The reinforcing effect is increased by pre-stressing the geosynthetics. This is modelled by a pre-stress of the geosynthetic elements.	

6. FE simulation of the deformation behaviour

6.1. Estimation of elastic and permanent deflection

<table>
<tr><th>No</th><th>Total deflection estimation</th><th>Remarks</th></tr>
<tr><td>1</td><td>

The elastic deflection Δw is estimated from the experimental measurements. After the first 50 cycles it is considered constant.

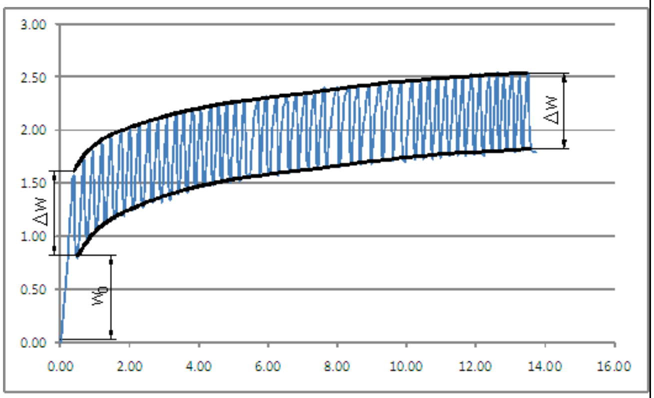


Fig. 22: Graph of consolidation during the first 50-cycles

The consolidation behaviour of the loaded laboratory model can be seen in *Fig. 22*.

</td><td></td></tr>
<tr><td>2</td><td>

The permanent deflection w_0 is calculated from the measured values as:

$w_0 = w - \Delta w$

where w_0 is the permanent deflection, w is total deflection in the first load cycle and Δw is value of elastic deflection.

</td><td></td></tr>
<tr><td>3</td><td>From FE-simulation results the model</td><td></td></tr>
</table>

No	Total deflection estimation	Remarks
	without geosynthetic, the elastic deflection $\Delta w_{\text{FE,non-reinforced}}$ is obtained.	
4	The elastic deflection $\Delta w_{FE,reinforced}$ is evalued from particular model of reinforcing geosynthetic.	
5	The ratio k between the elastic deflections for the reinforced and non-reinforced case is evalued as: $k = \frac{\Delta w_{\text{FE,reinforced}}}{\Delta w_{\text{FE,non-reinforced}}}$	
6	The permanent deflection is evaluated for the given case as: $w_{0FE,reinforced} = k \cdot w_0$	
7	The total initial deflection is evaluated as: $w_{\text{FE,reinforced}} = w_{\text{0FE,reinforced}} + \Delta w_{\text{FE,reinforced}}$	

6.2. Approximate expressions for ballast settlements

Prediction of the response of a reinforced railway substructure subjected to cyclic loading is based on the results of experimental investigations in a test box regarding long term behaviour. The aim is to find the proper function to approximate the measured settlements as a function of the number of applied load cycles. Three equations suitable for an approximate expression of settlement:

$$S_{\text{N}} = S_1 N^b \quad (1)$$

$$S_{\text{N}} = a + b \log_{10} N \quad (2)$$

$$S_{\text{N}} = S_1 (1 + C \log_{10} N) \quad (3)$$

where S_N is the deflection after N – load cycles, S_1 is the deflection in the first load cycle, N is the number of load cycle and a, b, C are empirical constants. The aim is to approximate these functions to the measured data (obtained from experiment) and evaluate the unknown constants based on the minimization of following expression:

$$error = \frac{\sum_{i=1}^{N} (y(i) - y_{\text{estimated}}(i))^2}{N}$$

Tab. 2: Approximation functions for ballast settlements

	Function	Function (Matlab)	Error	a	b
1	$S_N = S_1 N^a$	y(i)=y(1)*x(i)^a	0.017014	0.07657	-
2	$S_N = a + b \log_{10} N$	y(i)=a+b*log10(x(i))	0.005058	1.697559	0.502801
3	$S_N = S_1(1 + a \log_{10} N)$	y(i)=y(1)*(1+a*log10(x(i)))	0.006168	0.275	-

The best fit between the measured values and the approximation function is found for the second function. The third function ($S_N = S_1(1 + a \log_{10} N)$) is dependent on the value of deflection in the first load cycle, which is possible to obtain from FE-simulations. The first function ($S_N = S_1 N^a$) was found to be unacceptable for an approximation of the measured data.

6.3. Prediction of short term behaviour

A numerical simulation of the behaviour of the railway substructure during the first 50 cycles was compared to experimental results. The behaviour of the unreinforced substructure is compared to the behaviour when a particular geosynthetics is used. The reinforcing effect of geotextiles, uniaxial and biaxial geogrids is compared in the numerical simulations. Geogrids with a high reinforcing effect are treated differently than geotextiles which have mainly a separation and filtration function. Deflections of the models are plotted against the number of cycles. Example of the plot is given in Fig. 23 and Fig. 24. (For model labelling see deliverable D2.2.9).

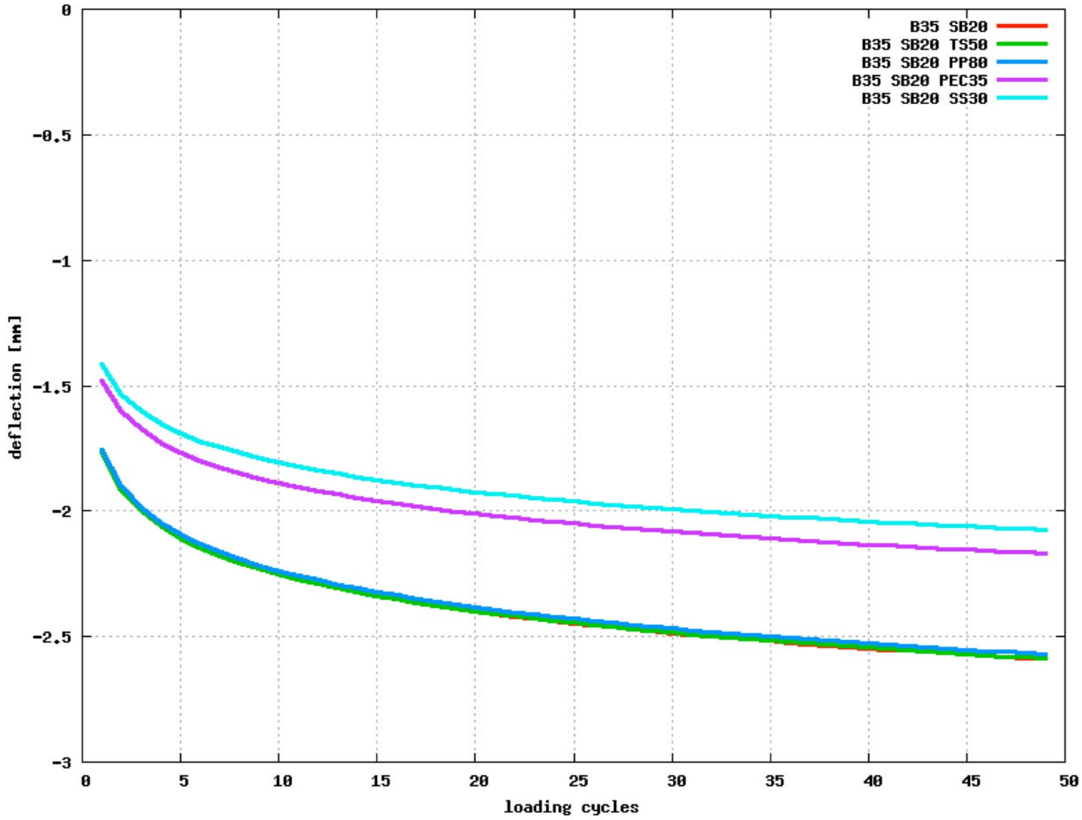


Fig. 23: Single layer model deflections

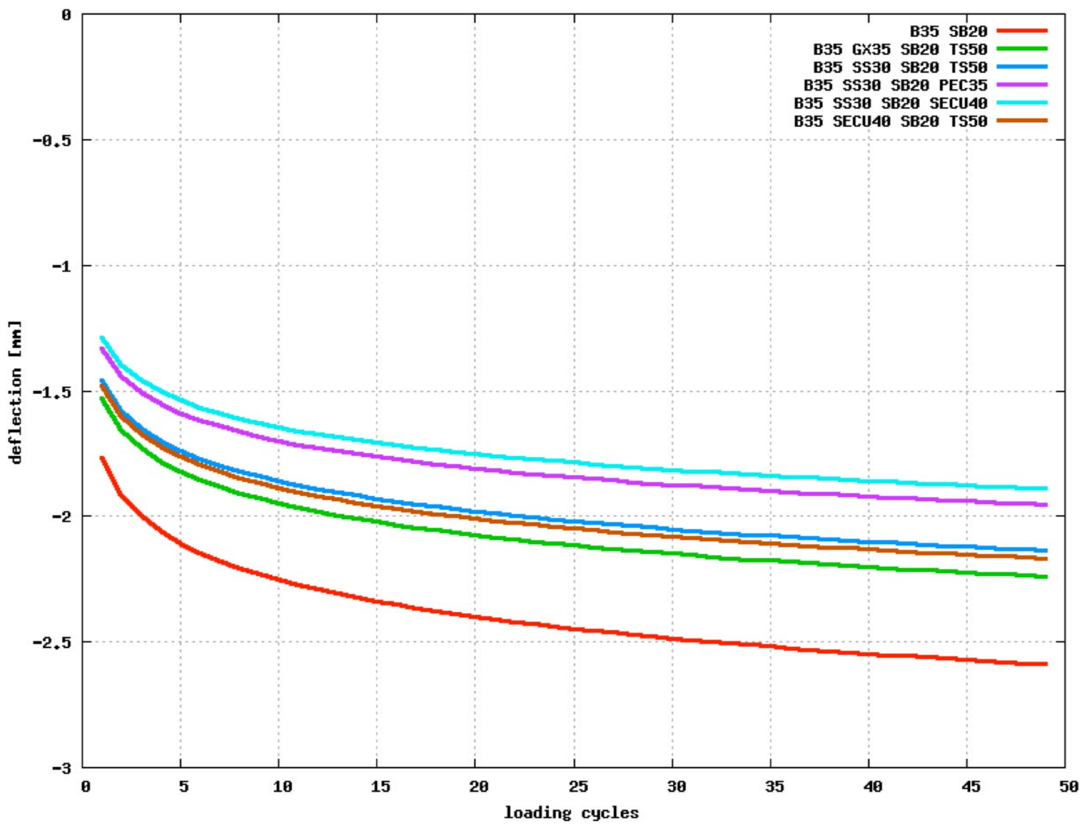


Fig. 24: Double layer model deflections

6.4. Prediction of long term behaviour

The response of a particular design of railway substructure reinforced with geosynthetic in single or double layer construction to cyclic loading is based on the results of laboratory investigations of test box in long term behaviour. Based on the experimental results following equations:

$$S_N = a + b\log_{10} N$$

$$S_N = S_1(1 + C\log_{10} N)$$

are used to approximate the reinforcing effect for up to 250,000 load cycles. These equations are likely to be able to predict settlements for up to 10,000,000 load cycles, but this heve not been studied experimentally. Examples of plots where the reinforcing effect of the studied geosynthetic is apparent for a single layer (Fig. 25) and a double layer (Fig. 26) construction are presented below (for model labelling see deliverable D2.2.9).

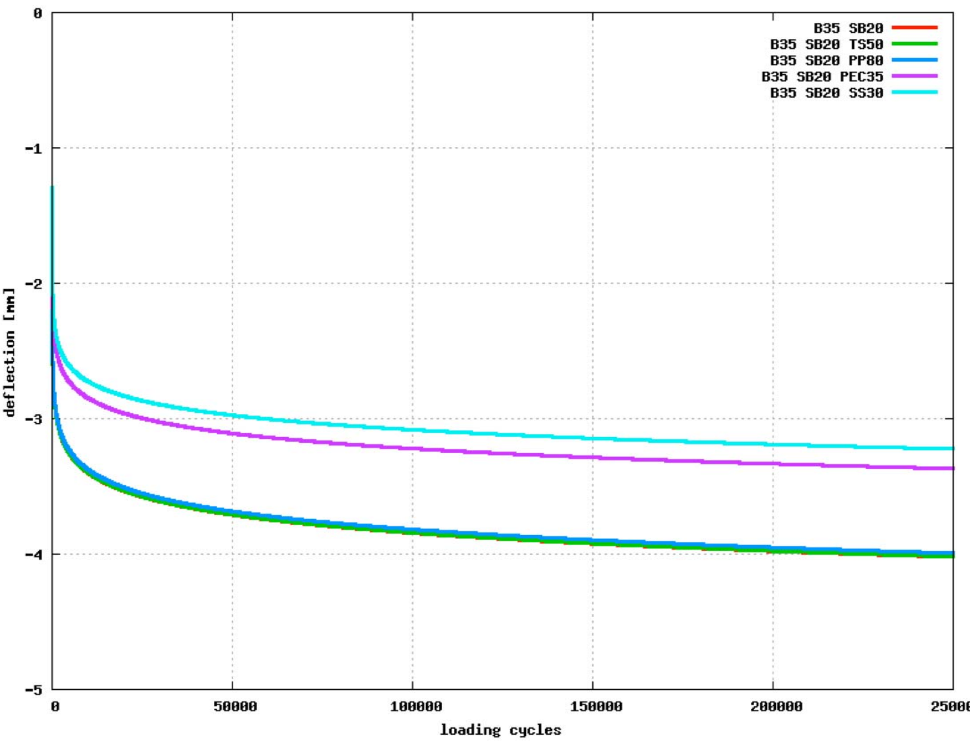


Fig. 25: Single layer model deflections

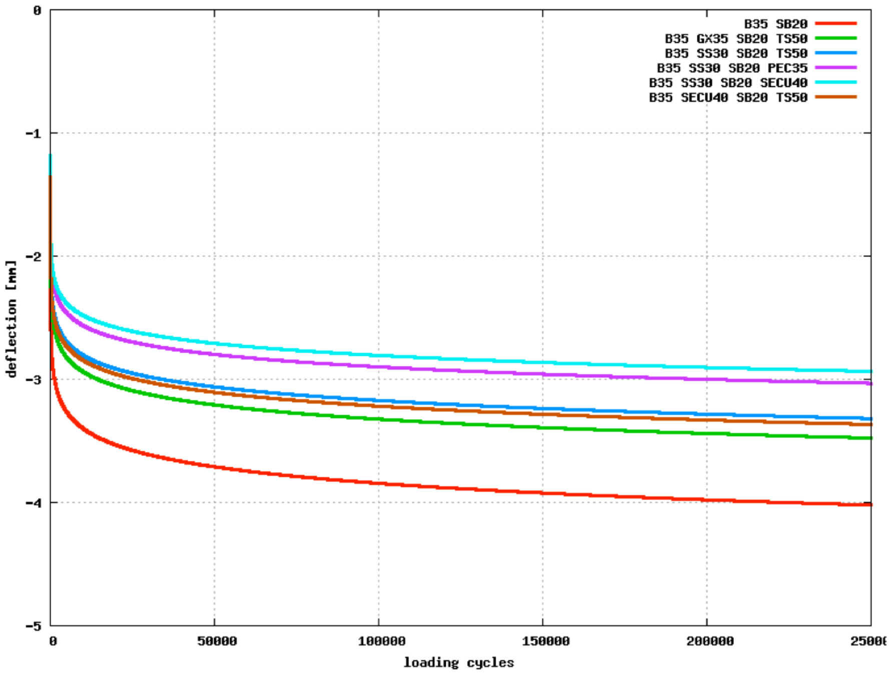


Fig. 26: Double layer model deflections

6.5. Conclusions

Numerical simulation is a powerful tool for prediction of long term deformation behaviour of railway substructure. For the relevant results of reinforcing effect of studied geosynthetics it is required to precisely describe not only the material characteristics of the geosynthetic but its interaction with surrounding material (ballast, subbalast). It is advisable to model the interaction differently for geogrids (using inserted boundary conditions (IBC)) while the woven geotextiles can be represented by membrane shell elements covered with contact elements on both their sides.

Using the approaches described in this guideline it is possible to predict the evolution of the permanent settlements during cyclic loading. Results of the modelling must be verified against larger number of experiments conducted either in laboratory conditions or in situ. The FE models can be used to optimize the reinforcing effect of a particular geosynthetic material or to study the reinforcing effect of different geosynthetics used in two separate layers.

7. Bibliography

No.	Designation of document
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2	Burrow M.P.N., Bowness D., Ghataora G.S.: A comparison of railway track foundation design methods. Proceedings of the IMechE Part F: Journal of Rail and Rapid Transit (Special Issue on Rail Research UK), 221/F1, 1-12, 2007
3	Tebay R.N., Lawrence C.A., Caddic V.R., Warvick R.G.: Geosynthetic protection of ballast and sub-grade in railway foundation structures. Geosynthetic – 7th ICG-Delmas, 2002
4	Sharpe P.: The use of geogrids in the UK. 4th Forum of Rail Transport, Bratislava, 2008
5	Göbel C., Lieberenz K., Weisemann U.: Dauerbelastungs-versuche mit Kunstoff-bewehrten Tragschichten im Eisenbahnbau. K-GEO, München, 1993
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Part 2:

Improvement of transition zones on conventional lines

8. Introduction

The objective of this study is to assess the efficiency of innovative techniques to improve the behaviour of the subgrade in conventional railway lines. To carry out this work, the transition zones between an underpass concrete block and an old 8 m high embankment at Montagut, near Lleida, on the conventional railway line Zaragoza-Barcelona in Spain were chosen. At the time the site was selected, there was a strong limitation of speed (10 km/h) for the operation of both passenger and freight trains due to the bad state of the track, which also required frequent tamping.

The methodology development for the study of the transition zone was:

- Geotechnical investigation.
    - Installation of a sliding micrometer inside a borehole as a high precision leveling tool to identify problematic zones within the embankment.
- High precision leveling campaign.
- Track stiffness measurement campaigns carried out to assess the magnitude of the problem.
- Track stiffness measurement campaigns carried out to validate the improvement method.

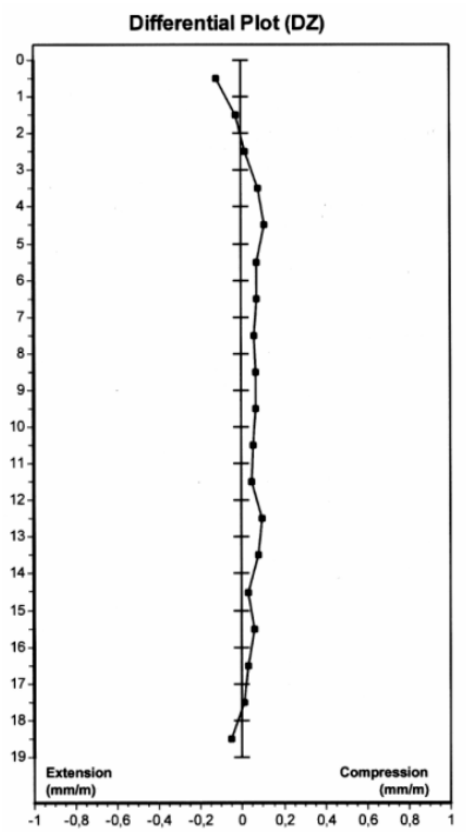
The study is here employed as a general guideline for how reinforcements using geosynthetics can be carry out. Further details may be found in INNOTRACK deliverable D2.2.7

9. Investigation methodology of the transition zone

9.1. Geotechnical investigation

In order to assess the amplitude of the treatment in the transition zone

Borehole was drilled from the top of the embankment, inside the track, at a distance of 10 m. from the concrete block, and a sliding micrometer of the TRIVEC type was installed inside, up to a depth of 21m. This affected also the embankment foundation. From the continuous core obtained from the borehole soil properties were investigated.



The sliding micrometer used inside the borehole is a high precision tool currently used in geotechnical works to identify problematic zones within earth embankments. It provides the vertical displacement ("*z*" component) of each point at the borehole sidewall to which it is connected.

The TRIVEC type provides also the horizontal displacements ("*x*" and "*y*" components) of those points.

After the borehole was drilled, a PVC tube equipped with metallic reference marks, with about 1m spacing, was installed inside.

Those reference marks, allowing small displacements, were perfectly attached to the borehole sidewall.

9.2. High precision levelling campaign

The objective of this campaign was to contribute to assess the magnitude of the on going settlements in the track that could be related to a poor mechanical behaviour of the embankment.

The vertical movements of a total of 22 control points, distributed along a section comprising 130 m. at both sides of the concrete block, were checked in three consecutive campaigns using high precision levelling instruments.

9.3. Track stiffness measurements

To implement the results obtained in the "in situ" campaigns, a measurement campaign was launched to assess the variation of track stiffness, under real operating conditions, at some representative sections on the conventional railway line near the concrete structure.

For these measurements, the three cross section indicated in Fig. 27 were chosen.

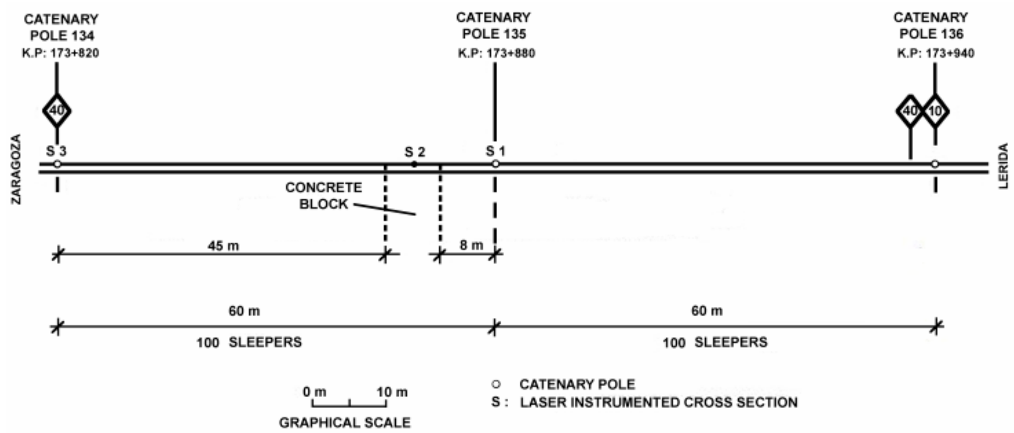


Fig. 27: Cross sections where track stiffness was measured

In the following campaign sensors were in each cross section attached, to the outer rail of the track between two consecutive sleepers:

- One laser beam receiver, to measure the vertical deflections of the rail.
- One 2 Hz geophone, with a sensitivity of 45.5 mV/mm/s, to

measure the vertical component of the rail vibration velocity.

- Two extensometer shear bands, with a sensitivity of 2.12 μV/με, to determine the loads induced by the operating traffic.

- Two additional extensometer shear bands in cross section S2 to determine sleeper reactions.

Examples of measured track stiffness values are given inFig. 28 and Fig. 29. Here, mean track stiffness values have been identified for the two types of the locotractor convoy wheel loads travelling in both directions.

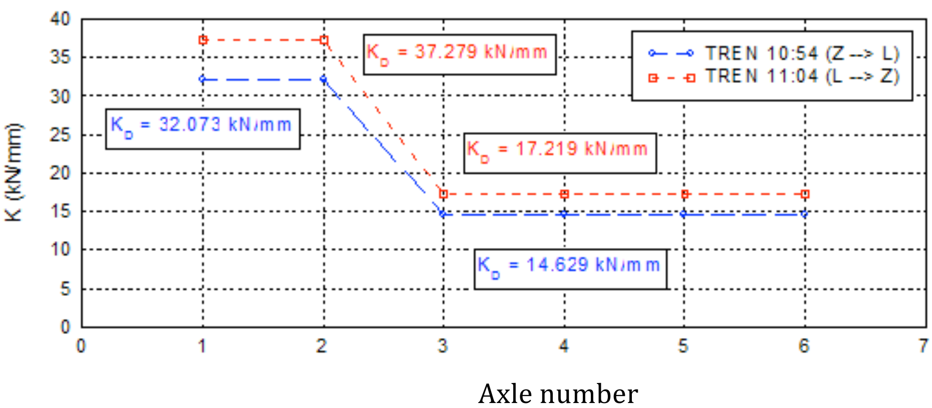


Fig. 28: Mean track stiffness with a sensitivity of values obtained at cross section S2 from locotractor convoy travelling in both directions

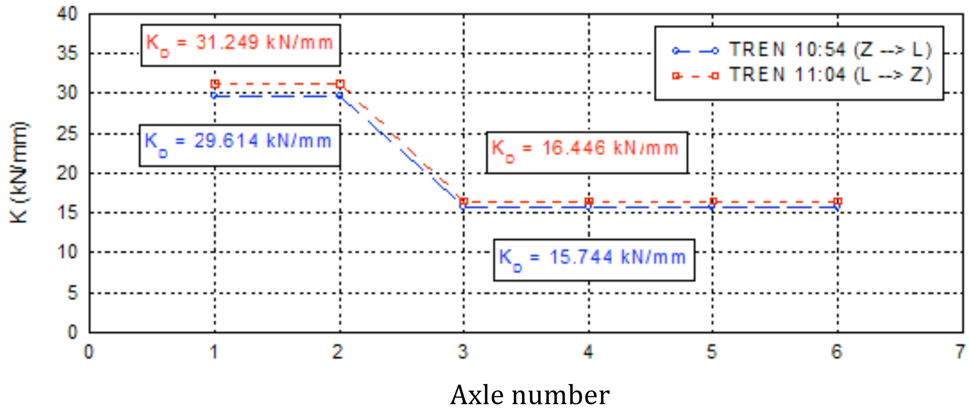


Fig. 29: Mean track stiffness values obtained at cross section S3 from locotractor convoy travelling in both directions

Summarising the results, the following set of track stiffness values have been assumed to be representative of the track condition in July 2007:

Cross section S1: K = 25 kN/mm

Cross section S2: K = 35 kN/mm

Cross section S3: K = 30 kN/mm

9.4. Adopted Solution

Based on the results provided by the works described, ADIF decided to improve only 20 m., at both sides of the concrete block, by replacing the 2.5 upper meters of the embankment by well compacted sandy gravel of the QS3 type as recommended by UIC (2008).

To reinforce the new material at the top of the embankment, two layers of shift geogrid were designed to be placed as indicated in Fig. 30: one at the bottom of the excavation that was to be filled by replacement material, and the other at mid-depth.

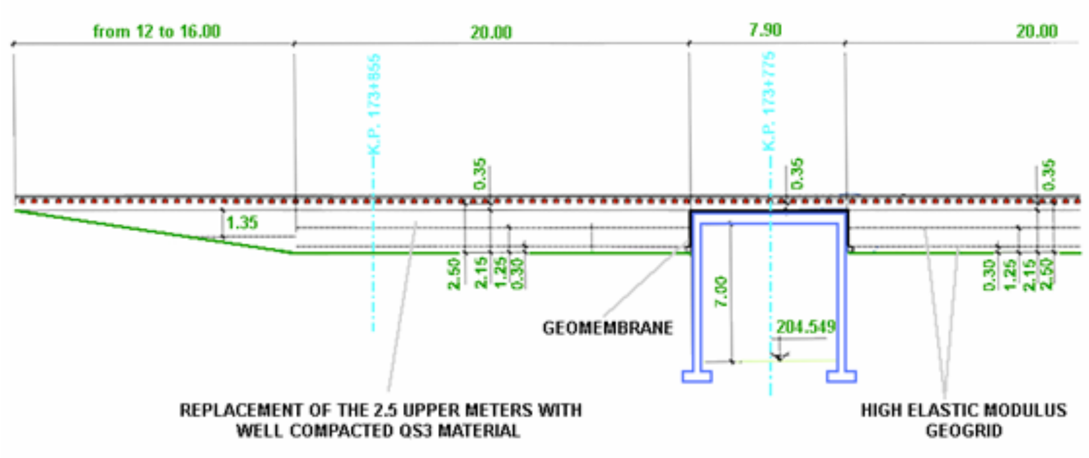


Fig. 30: Geogrid design adopted to improve the transition zones at Montagut

It was also decided to waterproof the external walls of the concrete block covering them with a geomembrane.

The existing duo-block sleepers were replaced by uni-block polyvalent PR-01 sleepers adapted to the Spanish gauge for supporting the existing UIC-54 rails.

The fastening of the rails to the sleepers (VM type), incorporated a pad with dimensions l x w x t = 180 x 158 x 7 mm.

The improvement of the ballast behaviour was considered a crucial task in the final solution. Accordingly, the ballast on top of the concrete block and on both sides of it, along sections more than 100 m long, was replaced by a 0.35 m. thick layer of high quality ballast.

The QS3 material consisted of well graded sandy gravel. It was compacted in layers 0.30 m thick to a dry density 98% of the maximum density (22.1 kN/m^3) reached in the Modified Proctor test with a water content ranging between -1.5% and + 1.0% of the optimum value (6.3%) determined in that test.

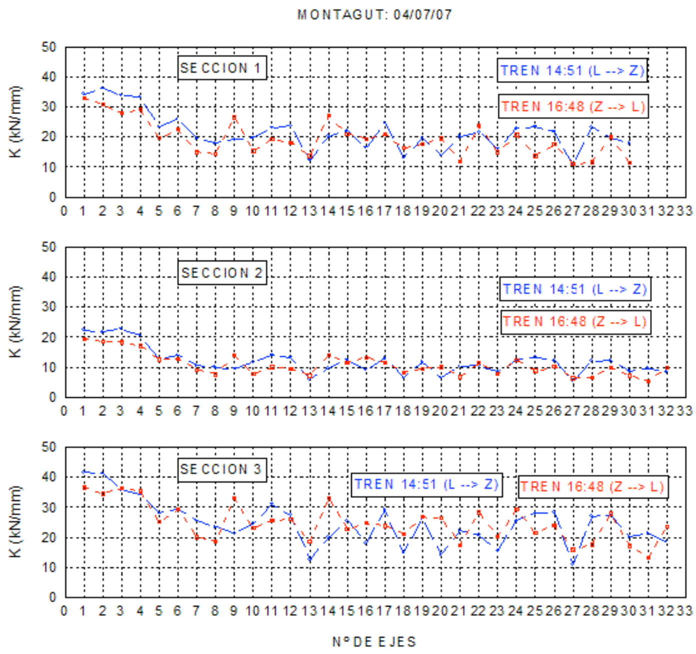
The employed geogrids, of polyvinyl alcohol (FORTRAC), were polymer coated and have the following properties:

- Unit weight (DIN EN 9864): 350 g/m^2
- Creep strain for service loading (BS 8006): ≤ 1%
- Longitudinal tensile strength at 3% extension (DIN EN ISO 10.319): ≥ 10kN/m
- Transverse tensile strength at 3% extension (DIN EN ISO 10.319): ≥10 kN/m
- Longitudinal strain for nominal tensile load (DIN EN ISO 10.319): ≤ 6%
- Transverse strain for nominal tensile load (DIN EN ISO 10.319): ≤ 6%
- The Strength is not adversely affected by hydrolysis for soil environments with pH from 2 to 13
- Mesh size: 30 x 30 mm
- Width: 5.00 m

Length: 200.00 m

9.5. Track stiffness measurements after the improvement

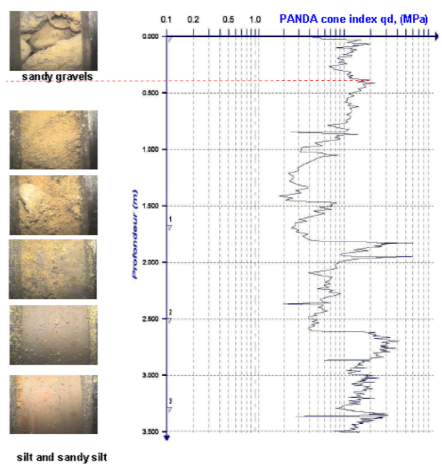
In order to assess the efficiency of the adopted solution, an in situ campaign was undertaken aimed to quantify the improvement. Cross sections S0, S1N, S2N and S3N as indicated in Fig. 27 were monitored. This time also, laser signal sources were housed in movable tripods, the equipment had been validated in other measurement campaigns.



9.6. Panda penetrometer tests

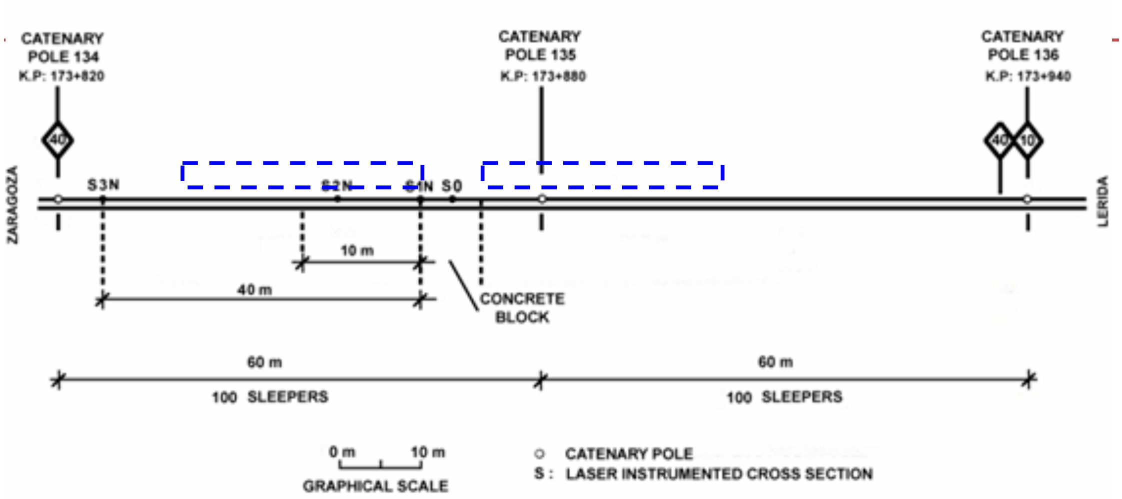
Field investigation with Panda and Geoendoscope allow identifying the different layers of the track and their thickness, furthers a cone resistance can be associated for each layer.

The analysis the data allows an evaluation of soil density based on the bank of material which has been previously tested by the Panda.



10. Results obtained

The results obtained on site before and after modification are summarized in Tab. 3 to 5.



Reinforcement area with new material at the top of the embankment and two layers of high elastic modulus geogrid

Tab. 3: Track stiffness values in Montagut before treatment (04/05-07-07)

CROSS SECTION	LOCOTRACTOR (10 km/h)	TALGO TRAINS (10 km/h)
S1: 8m from concrete block in Lleida direction	25 kN/mm	35 kN/mm
S2: in the middle of concrete block	-	20 kN/mm
S3: 45 m from concrete block in Zaragoza direction	30 kN/mm	40 kN/mm

Tab. 4: Track stiffness values at Montagut after treatment (16-10-08)

CROSS SECTION	LOCOTRACTOR (4 km/h)	TALGO TRAINS (150 km/h)
S1N1: 2m from the middle of concrete block	105 kN/mm	120 kN/mm
S1N2: at the edge of concrete block in Zaragoza side. Embankment with new material at the top of the embankment and two layers of high elastic	50 kN/mm	81 kN/mm

modulus geogrid.		
S1N3: at the edge of transition zone in Zaragoza side	35 kN/mm	58 kN/mm

Tab. 5: Track stiffness values at Montagut after treatment (17-10-08)

CROSS SECTION	LOCOTRACTOR (40 km/h)	TALGO TRAINS (160 km/h)
S0: in the middle of concrete block	92 kN/mm	86 kN/mm
S2N: 10 m from concrete block in Zaragoza direction. Embankment with new material at the top of the embankment and two layers of high elastic modulus geogrid.	105 kN/mm	95 kN/mm
S3N: 40 m from concrete block in Zaragoza direction	90 kN/mm	85 kN/mm

The geotechnical investigation carried out in the conventional railway line Zaragoza-Lleida, as described in this Guideline, turned out to be crucial in order to define the type and extension of the most adequate solution to get rid of the strong limitation of speed.

From measurements made in the line, monitoring the passage of maintenance units and commercial trains, the final solution adopted, has proved to be very efficient and confirms that problems have been mitigated.

The use of maintenance convoys providing wheel axle loads as low as 10-30 kN has been very useful for assessing the state of the ballast in the track.

Track stiffness values obtained after the treatment over the underpass concrete block and the two adjacent transition zones are 2.5 to 4 times higher than those determined before.

When implementing the type of solution described in this guideline, special attention should be focussed on the local details of the construction work carried out between the concrete block and the adjacent embankment, since a loss in the track stiffness of 30% has been detected when analysing the pass of maintenance and commercial trains from the structure to one of its transition zones.