Guidelines on the Use of Novel Construction and Maintenance Techniques within the Operational Railway Environment

D4.1

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Executive Summary

The DESTination RAIL project aims to support railway infrastructure managers by developing a number of novel techniques and systems for identifying, analyzing, predicting and remediating the critical rail infrastructure. As its final goal, DESTination RAIL aims to integrate the different techniques into a decision support system that will allow for the holistic management of all assets of a rail network.

This deliverable provides a review of novel techniques for construction and remediation of critical railway tracks, embankments and bridges. Particular emphasis has been given to the techniques that cause minimal interruption of railway traffic. Their features and main characteristics are discussed. In addition, an attempt to include such novel railway track improvements into numerical models is also described.

Deliverable 4.1 results from Task 4.1 Construction Techniques for Tracks and Earthworks and Task 4.2 Construction Techniques for Structures – Bridge Abutments. The main idea of the two tasks has been to give asset managers, and particularly consultancies that design and deliver railway projects, technical support for introducing novel remediation and construction techniques into their projects.

Objectives of these tasks can be grouped into three main categories:

- to demonstrate technologies (i.e. geosynthetic-reinforced soil, injection of high-pressure expansion polyurethane resins) with an ability to improve the performance of embankments, bridge abutments and transition zones,

- to support the use of marginal, locally sourced fill materials and other alternative materials in railway structures with characterization done by large-scale laboratory tests, and

- to perform advanced finite element analysis of the materials to be incorporated into railway structures.

Those objectives were fully achieved. In addition, an extensive overview of technologies which were not demonstrated at the project test sections but have shown high potential for use is also described.
1 Introduction

A series of laboratory, field prototype-scale pilot and demonstration projects on live railways has been undertaken work package 4 of the project. These ranged from proof of concept (low TRL level), validation (medium TRL) to demonstration projects for certification (high TRL). These included testing the Ground Penetrating Radar (GPR) methodology (from Task 1.2) on a 100 km section of the Croatian main line, real-time monitoring of switches and crossings (Task 1.3) at a test-site in Norway, rock slope surveys using drones (Task 1.4) in Croatia and Ireland, non-destructive testing (NDT) dynamic test methods for assessing the integrity of rock bolts in Croatia and large-scale triaxial testing of geosynthetic reinforced soil (GRS) in Slovenia.

This deliverable maps existing novel techniques for construction and rehabilitation of critical railway tracks, embankments and bridges. Several techniques are analysed in detail and an outline of their implementation is also provided.

The novel techniques are organised into two sections as follows:

- Maintenance techniques (Section 2);
- Construction techniques (Section 3).

The aim of Section 2 was to develop novel maintenance techniques that cause minimum traffic disruption. Among them, injection of high-pressure expansion polyurethane resins has been recognised as a technology with particular potential. Therefore, this technology was analysed in greater detail and has been demonstrated at two test locations in Slovenia.

Section 3 provides an overview of techniques that can be used during the construction phase when the railway line is closed. This deliverable focuses on tracks and earthworks, as well as transition zones. Amongst other techniques, the use of geosynthetic soil reinforcement is examined. In particular, geosynthetic reinforced soil (GRS) has been researched and two possible uses on test sections in Slovenia and Croatia are presented within this deliverable.

Finally, the deliverable introduces a Finite Element (FE) modelling approach and presents two case studies, where research results (results of laboratory testing and field monitoring) of Destination Rail project have been used. As such, this deliverable provides guidance on the selection of possible intervention measures in the case of critical rail infrastructure and gives an example of how to use advanced numerical tools for their modelling.
2 Maintenance Techniques

2.1 Introduction

The railway network in Europe currently consists of 138,072 km of comprehensive railway lines (50,762 km of railway lines form the core network). The total length of railway track in the EU increased by less than 1% between 2004 and 2014, while it decreased by ~9% between 1990 and 2014 (Eurostat, 2017). Many of the rail networks in Eastern Europe and parts of Western Europe were developed more than 150 years ago. They suffer from low levels of investment and significant parts of these networks are in need of rehabilitation actions.

On the other hand, one can see from the numbers presented above, that more than one third of the total railway network (i.e. the core network) cannot afford line closure, or significant bottlenecks for long periods. Thus, rehabilitation techniques that cause minimal interruption of traffic are very important.

This section provides an overview of promising novel rehabilitation techniques. Those with a potential for reducing traffic interruption and improving performance are listed below. Attention was particularly focused on interventions on the railway network based on the injection of high-pressure expansion polyurethane resins. It has been assumed that the use of this technology on the railway line has negligible effects on railway traffic, but evidence of track improvements was not available. Thus the potential of the technology was evaluated through trials on two test sections.

2.2 Rehabilitation Techniques for Deteriorated Rail Structures

2.2.1 New strategies for ballast tamping and cleaning

Over time, railway ballast can settle and result in the need for re-alignment of the track. This can be done with a ballast tamper or tamping machine which is used to pack (or tamp) the ballast under the railway sleepers (see Figure 1). With modern machines the track as a whole can be levelled, aligned and tamped, in order to achieve a smooth train journey and to reduce the mechanical strain applied to the track by passing trains.

The ballast also becomes worn with time, and loses its angularity, becomes rounded. This hinders the interlocking or tessellation (from Latin tessera = square) of pieces of ballast with one another, and this reduces its effectiveness. Fine pieces of granite, like sand, are also created by attrition, known simply as "fines". Combined with water in the ballast, these fines stick together, making the ballast into concrete-like lumps. This hinders both track drainage and the flexibility of the ballast to constrain the track as it moves under traffic. Ballast cleaning removes the fine material in the worn ballast (Figure 2), screens it and replaces the "dirty" worn ballast with fresh ballast. The advantage of ballast cleaning is that it can be done by an on-track machine without removing the rail and sleepers, and it is therefore cheaper than a total excavation. It can be carried out with e.g. a ballast cleaner which is a machine that specialises in cleaning the railway track ballast (gravel, blue stone or other aggregate) of impurities. In total, over the whole life of track, tamping is the most frequently adopted
maintenance technique. Rail exchange or ballast cleaning is far more costly than tamping (factor up to 10 times more expensive). Together tamping and cleaning of ballast are usually the most costly processes in railway track maintenance. The strategy for how often these maintenance procedures are carried out has a vast influence on the overall cost and quality of the track (Veit and Marschnig, 2012; Automain, 2013).

Figure 1 The ballast cleaner machine (Litija, Slovenia)

Figure 2 Detail of ballast remove from the track (Litija, Slovenia)
2.2.2 Stone blowing

Stone blowing (Figure 3) is a relatively new process for track geometry adjustment; it involves adding crushed rock to the surface of ballast under the lifted tie to shim the track. This is an alternative to tamping in which the ballast particles are rearranged to fill the void under the lifted tie. It prevents sleeper damage, which very often occurs due to the tamping process (see Figure 4). To begin the stone blowing process, the existing geometry of the track is measured. The vertical adjustment at each tie required to achieve the desired geometry is subsequently calculated. Next, the volume of stone to be blown beneath each tie to achieve this adjustment is determined. The stone is then placed by lifting the ties, inserting blowing tubes, and blowing the stone under the tie. Train traffic will then seat the ties as the blowing stone particles settle in. Stones used for blowing are selected to optimise their use for shimming (Selig, 2014).

Figure 3 Stone blowing technique (Strathspey Railway, 2017)

Figure 4 Typical damage caused to concrete sleepers by tamping (Litija, Slovenia)
2.2.3 Track relaying trains without crawler

Matisa P-95 and P-190 machines and Plasser SUM-type machines are included in the category of track relaying trains without crawler. This type of track relaying trains threads the new rails into the clamp guide, the material wagons run on the old track, while the machine in the rear already runs on the new track (Mainline, 2011).

The frame of the main machine consists of two parts, which lifts up in order to provide the necessary space for the working units to pick up and lay the sleepers. The old sleepers are collected, one by one, by the sleeper collection device and transported by a conveyor belt to the sleeper conveying wagon which is running at the front, along the old track. When there are sufficient old sleepers stored, a gantry crane picks them up and unloads them onto the old sleeper platform wagons, situated at the front part of the train.

This same gantry crane is used to pick the new sleepers, which are also lying on platform wagons behind the old sleeper ones. The gantry crane unloads them onto the conveyor belt of the supply device, and then lays them onto the ballast according to the desired space between sleepers (usually 60 cm). Between the pick-up of old sleepers and the laying of new ones, an excavator chain, or a plough depending on the case, is used to obtain the correct ballast surface to receive the new sleepers.

Finally, the old rail is removed and the new one is positioned by “clamps”. The rear part of the track relaying train runs on the new rails. Figure 5 illustrates the removal of old rails and the positioning of new ones.

![Figure 5 Schematic drawing of a track relaying train without crawlers (COMSA)](image)

2.2.4 Vacuum excavating method

There are many instances where it is very difficult to remove ballast or other types of material from the track area. For example, near station platforms, level crossings, tunnels, bridge approaches switches, etc. Frequently, it is necessary to excavate small quantities of ballast and to prevent dust, as well as low noise during the application of this non-destructive method.

Various types of material such as ballast, sand, soil and weeds can be removed cost-efficiently from the track area using a vacuum scraper-excavator machine (e.g. VM170 Jumbo from Plasser & Theurer, as outlined in Figure 6). This machine can be effectively used even in places where it is otherwise only possible to use manual methods. The machine is also suitable for producing drainage ditches and for excavation of ditches for foundations, masts, cables, etc. The machine combines a high-performance vacuum device with a rotating suction nozzle. The central part of the vacuum device is a suction part consisting of two chambers to
separate materials. The rotating suction nozzle is connected to the material separators by a flexible hose (Lichtberger, 2011). Thanks to the extension of the suction range compared to earlier machines, it is possible to pick up material across the adjacent track. The machine can be fitted optionally with an additional removable funnel attachment which allows surface cleaning of the track without picking up ballast.

Figure 6 Vacuum excavator machine Jumbo (Plasser & Theurer, 2017)

2.2.5 Puscal method

The Puscal method divides a track section into 18 m long parts. The track grid of these parts is loaded onto a wagon by gantry cranes and replaced by temporary rails connected by tie rods. A spiral cutter to excavate the material and funnel wagons to transport the soil away enters this track section from the other end. After the soil has been compacted, the ballast is inserted and finally gantry cranes relay the track panels. This method achieves a performance of about 18m/h; it requires track closures of at least 20 hours (Lichtberger, 2011).

2.2.6 Subgrade improvement method for existing railway lines with soil-cement vertical columns

This method presents a technique to reinforce the subgrade of the existing tracks. The method involves building vertical soil-cement columns under the sub-ballast layer without removing the track. This technique limits the duration of maintenance work and, consequently, traffic disruption. In addition, it makes use of the existing soil, thereby reducing the amount of concrete needed, and also does not create stiff zones at the location of each column that might
accelerate the degradation of ballast. This technique can be quickly executed on site and the excavated columns have shown strong mechanical resistance. The program carried out within the INNOTRACK project (Kouby, 2010) has confirmed the feasibility of this technique in soils stiffer than those in which lime-cement columns are commonly used and has yielded preliminary findings on the economic aspects of the technique (execution times, quantities of cement, etc.).

**Figure 7** Drilling columns under the track (Kouby, 2010)

**Figure 8** Excavation of one column on the side track site (Kouby, 2010)
2.2.7 Deep soil mixing trenching for subgrade rehabilitation of railway embankments

Deep soil mixing is used for building solid and water-blocking soil-cement walls in one pass by mixing the native soil with an injected cement suspension. With that technology it is possible to achieve continuous applying of solid and dense soil-cement walls up to 25 m deep and 1.25 m wide without any soil movements. The width of the soil-cement wall is unlimited by building several stripes next to each other (Allcons, 2018). The technology was deployed by the German railway company Deutsche Bahn AG.

**Figure 9** Schematic presentation of deep soil mixing trenching (Allcons, 2018).

**Figure 10** Trencher for soil stabilisation by deep soil mixing (Allcons, 2018).
2.2.8 EKG stabilisation of a railway embankment

Electrokinetic geosynthetic (EKG) technology has numerous applications across a range of industries including stabilisation of embankments and cuttings; dewatering of wastes such as sewage sludge and mine tailings; and sports turf management.

With the advent of EKG, active geosynthetics are now a viable tool for design and process engineers and manufacturers. Traditional geosynthetics and industrial textiles are used in the civil, mining, environmental and waste engineering industries to carry out a range of functions including drainage, reinforcement, filtration, separation, containment, encapsulation and sorption. All of these functions, in one way or another, are influenced or limited by the rate at which water is able to flow through the materials with which the geosynthetics are being used to improve or treat (Electrokinetic, 2018).

An active role of geosynthetics can be achieved by combining the electrokinetic phenomena of electroosmosis, electrophoresis and associated electrokinetic functions such as electrolysis with the traditional functions of geosynthetics of drainage, filtration, containment and reinforcement to form electrokinetic geosynthetics (EKG). When ground is treated by EKG materials, electrokinetic processes such as electroosmotic flow from the anode areas towards the cathode, pore pressure reduction spreading out from anodes, cementation around the anodes and precipitation around the cathodes are activated (Figure 11).

Electrokinetic principles can also be used very effectively for slope stabilisation purposes, to stabilise existing unstable embankments and cuttings. Slope stabilisation using electroosmosis where anodes and cathodes are used in the active (electrokinetic) phase to force water out of the ground result in an improvement in the material strength of the soil. After active treatment the anodes are reinforced and converted to soil nails and benefit from a major improvement in the frictional and adhesion components of the soil / reinforcement bond strength. The cathodes are left in place in the slope to provide long term drainage.

Figure 11 EKG technology (Electrokinetic, 2018)
The electrokinetic approach has been used to successfully stabilise a failing clay embankment in London resulting in a 26% cost reduction and a 47% reduction in carbon footprint over conventional methods, reports Electrokinetic (2018). Toe weighting and/or slope regrading is commonly used to tackle the problem, but these do not address the problem of shrink-swell or pore water pressure changes and typically delay failure rather than prevent it. In addition, these methods can require large quantities of primary aggregate and energy and are becoming less environmentally and economically viable. Upon application of a DC potential (60-80V) electroosmosis forced water to flow from the soil adjacent to the anodes to the cathodes. In this particular case (Electrokinetic, 2018), the treatment took only six weeks and resulted in dewatering from the cathodes >25 times that from control drains and in a reduction in plasticity and shrinkage characteristics.

Figure 12 Railway embankment with EKG treatment (Electrokinetic, 2018)

2.2.9 Improvement of a soft embankment area with inclined lime/cement columns

It is complicated to carry out any remedial work under existing track while not restraining train operations. There are two possibilities: either close train operations and remove the track and embankment and perform strengthening or execute subsoil stabilisation without traffic disruption. It is well known that the first described option is very expensive and time consuming.

To realise the first option, inclined lime columns can be installed in soft embankments when factor of safety regarding stability for this embankment is too low. After drilling the borehole, the lime is mixed with the clay when the mixing tool is withdrawn under rotation in the opposite direction. The admixture is forced out with compressed air at a prescribed quantity per metre through a hole in the upper part of the mixing tool. The rate of withdrawal during lime mixing is normally approximately 25 mm/revolution. When mixing cement, normally columns are performed with 15-20 revolutions. Columns can be installed at a maximum inclination of 30-70° and to a maximum length of about 20 m. The admixture feed is stopped 0.2-0.5 m below the ground surface so that the admixture is not blown into the open air. Where there is a dry crust thicker than 1 m, the admixture feed is normally stopped 0.5 m above the bottom edge of the crust (Hartlen and Wolski, 1996).
Lime/cement column increases the bearing capacity of soft clay and reduces consolidation settlements under surfaces, but it often causes heaving of track during the installation and track settlements after completion of activity.

![Installation of inclined lime cement columns under embankment](image)

**Figure 13** Installation of inclined lime cement columns under embankment

### 2.2.10 Structural support with an expanding polyurethane pillar

The latest innovation, the expanding polyurethane pillar, offers a solution for soil reinforcement and structural support for buildings that have settled and are on weak ground. The pillar can be installed directly under the structure into the ground, in which it reinforces poorly compacted soil, provides support for load-bearing structures and solves settlement problems caused by water.

Expanding polyurethane pillars are also suitable for strengthening bridges, railways and roads (Keen et al., 2012). The pillars are a fast and non-disruptive alternative to traditional piling and ground strengthening solutions. They can also be used to repair failed piling. No excavation is required and the total cost is usually less than that of other solutions.
2.3 Improvement of Deteriorated Rail Structures by the Injection of High-Pressure Expansion Polyurethane Resins

The injection of high-pressure expansion polyurethane resins is frequently used in stabilising shallow foundations, such as road and railway embankments and riverbanks. The injection at a given depth of expanding polyurethane resin at high pressure generally results in notable improvement of the geotechnical properties on the surrounding soil inducing a radial compression and a reduction of the voids ratio due to permeation.

The resin is obtained from a chemical reaction of two components, which cause a quick expansion of the mixture with a high expansion pressure. The expansion pressure as well as the expansion time varies depending on the injected material. Expansion pressure is normally within the range 500 MPa up to 10 MPa, while expansion time ranges between 6 to 40 seconds.

To establish its main mechanical characteristics, the resin was subjected to vertical unconfined compression tests and swelling tests in a study outlined by Favaretti et al. (2004). The results reported in Figure 15 show how the compression resistance, $\sigma_c$, rapidly increases with the volume weight, $\gamma_r$, of the resin.

The tests made it possible also to identify the modulus of elasticity of the resin, $E_r$, with values between 15 MPa and 80 MPa, comparable to the typical moduli $E_{50\%}$ and $E_{25\%}$ of alluvial soils. This means that, in a volume of soil exposed to treatment with resin, the average stiffness of the soil mass does not undergo significant variations.

Figure 14 The expanding polyurethane pillar under the railway track
Figure 15 Unconfined compression tests: tendency of the compression resistance, $\sigma_c$, with the volume weight, $\gamma_r$, of the resin (Favaretti et al., 2004)

The values of swelling pressure measured during the tests are indicative of the pressure that the resin can generate whenever it gets injected into the soil (Figure 16). The state of stress in the ground determines the pressure of expansion to which the resin completes the reaction of polymerization. The final volume weight of the resin and the degree of volumetric expansion, measured at the end of the process, are both functions of such value of stress.

Figure 16 Swelling tests: tendency of the maximum swelling pressure, $\sigma_{sw}$, with the volume weight, $\gamma_r$, of the resin. (Favaretti et al., 2004)
The injection of high-pressure expansion polyurethane resins is intended to increase the bearing capacity of foundation soils and it is characterised by shallow compaction; the expansive action of the resins focus on the soil immediately below the foundation, in order to increase the resistance and to fill the voids. It is also possible to get a limited recovery of the subsidence that may occur.

The injections of resin continue until the verification, through a continuous laser monitoring, of an early lifting (less than 1 mm). The material of the injection consists of a bi-component polyurethane system, made of polyols and isocyanates, whose blend injected into the soil in the form of solution, polymerize, increasing its volume. The mixture of the two components of the resinous mixture occurs on the head of the injection tube, pressing the gun trigger that controls the injection pump. The resin mixture is directed at low pressure through the steel pipe to the point of injection. As soon as it leaves the pipe, the resin begins to expand developing an expansion pressure, which is transmitted directly on the soil.

In order to carry out this intervention, an injection technology has been selected with the following characteristics:

- Injections into the railway foundation (stabilised layer) right underneath the rails and sleepers, in successive steps and at regularly spaced points, in order to obtain a homogenous increase in the mechanical characteristics of the ground being treated.
- The injected material would be composed of a rigid, closed cell, bicomponent, polyurethane resin, especially formulated to undergo controlled-pressure swelling in short expansion periods.
- The injected resin would expand due to the chemical reaction, which would increase its volume more or less five times its initial volume generating pressure in all directions. This process will result in the voids filling of the surrounding ground.
- Continuous monitoring of the area interested by the treatment, using a laser level during the injection. The detection of an early lifting of the rails surface would confirm that the voids filling in the area of the injection point has been reached.

### 2.4 Demonstration projects

Two railway sections were selected to study the effect of injection of high pressure expanding polyurethane resin on track quality along a bridge transition zone and the effect on possible improvement of deteriorated railway embankment. The injection has been performed into ballast layer and subgrade in the case of transition zone, while in the case of embankment the injection has been performed deeper in the subgrade (Pasquetto, 2016). Measurements to compare track quality before and after the injection action were performed in both cases.

#### 2.4.1 Test section Dolgi most

The test section at Dolgi Most, Ljubljana is an approximately 10 m long bridge transition zone (see Figure 17 & Figure 18). Due to the restriction of time for measurements, Technical University Munich (TUM) performed the measurement only on one track and on one side of the bridge, which is the one behind the bridge with respect to direction of train runs. The track is equipped with rail profile 60, wooden sleeper and KS fastening system. Along the transition,
guardrails are connected to the sleepers. Additionally, rail welds had been observed along the outside rail, see Figure 19. Otherwise, railway track is constructed from well graded crushed material and is of proper quality.

**Figure 17** Plan and site view of test section Dolgi most

**Figure 18** Plan of intervention at test section Dolgi most
According to the replenishment layers composition (0.30 m of ballast followed by 1.20 m of gravel replenishment), it was decided to treat the layer between 0.0 and 3.0 m with 3 levels injections at a depth of 0.4 – 1.5 and 2.5 m. The radius of influence of each injection was approximately 0.6 m. As such, the spacing between injections is defined equal to 1.2 m (Figure 20).

In order to reach the ground to be treated, the replenishment has been drilled with electric hand-drillers, using boreholes of 26 mm diameter (Figure 21). Afterwards, the injection tubes were vibrated into the ground underneath the railway surface. Throughout the injection phase, the road surface has been monitored with a laser system in order to stop the injection in case of lift.
The track quality is a combination of unloaded track geometry quality and track stiffness quality. Because of uneven track settlement activated by dynamic traffic loads, both criteria are not constant in time. The injection of high-pressure expansion polyurethane resin is intended to improve the durability of track quality by increasing the stability of the track supporting structure (ballast and subgrade) along sections, which are typically affected by the problem of track stiffness variation, e.g. bridge transition zones. Based on the decisions of ZAG, TUM performed following two measurement series:

- unloaded track geometry measurements before and after the injection to study the effect on track geometry
- rail foot bending strain measurements under train runs before and after the injection to study effects on track stiffness quality

TUM used the track measurement trolley MessRegCLS manufactured by Vogel & Plötscher (see Figure 22) to record unloaded track geometry before and after the injection. Additionally, ZAG measured track deflections (digital image correlation, optical deformation measurement system GOM - ARAMIS) during the train passing before and after the intervention.
To measure rail foot bending strains, strain gauges were installed on the lower surface of the rail, mid span between rail seats. Beginning from the bridge abutment, altogether 15 strain gauges have been installed along the outside rail towards the direction of open track section. At the position of welds, it was not possible to install strain gauges due to the unevenness of rail’s lower surface.

Figure 22 Track geometry measurement trolley MessRegCLS

Figure 23 The passage of passenger train used for the rail bending strain measurements
The first measurement series was performed one day prior to the injection. One month later, the second measurement was performed. The measurement results are presented in Figure 24 to Figure 27. Rail gauge variation before and after injection (Figure 24) had been used to synchronize the measurement data of both measurement series.

From Figure 24 and Figure 25, it can be concluded that the unloaded track geometry has not been significantly changed by the injection. It should be taken into account that the change of vertical rail level (maximum about 1mm) next to the bridge might be affected by the actual situation of the bridge deck due to temperature and pre-loading.
Before and after the injection, the rail bending strain under the first axle of the same type of locomotive had been analysed. An example of measured rail bending strain from two neighbouring strain gauges is presented in Figure 26.

![Figure 26 An example of measured rail bending strains under train run from two neighbour strain gauges](image)

The running speed of a train was estimated from the time interval of first peaks measured by the first and last strain gauge (with respect to train run direction). Their space distance (rail seat distance) was taken into account. In the test section, the train speed was constantly 80 km/h.

To eliminate the possible influence of different trains (although the same type), the mean value of four train runs has been calculated both for before and after the injection, which is shown in Figure 27. The mean value and standard deviation of corresponding positions at each line are concluded in Table 1.

| Table 1 Mean value and standard deviation of the rail bending strain along transition |
|---------------------------------|-----------------|-----------------|
|                                | Before the injection | After the injection |
| Mean value [μm/m]              | 324              | 317              |
| Standard variation [μm/m]      | 67               | 70               |

Regarding the mean value and standard variation of rail bending strain distribution, no significant change of track quality is observed.
2.4.2 Test section Divača

A second implementation of high-pressure expansion polyurethane resin was performed on the embankment on the railway line Divača-Prešnica (Figure 31). The railway line in that part of the network consist of only one single track, which presents the main connection of Port of Koper with inland. Port of Koper is the main sea port not only for Slovenia, but also for some other Central European countries, i.e. Austria, Hungary, Czech Republik, Slovakia and also southern Germany and Poland. Thus, the railway line is extremely important and the maintenance work on the track is limited only to the work which does not need line closure. As the amount of the sea line cargo is constantly increasing for last few decades also the load on the track increased. The embankment settlement has been monitored for several years due to the exceeded load on few sections on this line. Figure 28 shows the amount of cargo transported per year on the railway line Ljubljana – Koper from 1999 to 2011. It can be seen that the cargo is constantly increasing since 1999, with small decrease in 2009 due to economic crisis. Similarly also the number of train passages has been increasing in the same period (Figure 29), reached almost 90 trains per day – most of them are freight trains. Therefore, line closure for maintenance work is something that asset managers want to avoid in every way.
Due to the very dense traffic on this line it was impossible to perform any drilling of boreholes on the track. Thus, GPR survey was conducted in May 2015 to find out the reason for observed settlements. Typical results are presented on Figure 30. Green colour in the central part denotes additional ballast material which has been installed over years to counteract the effect of settlements. Besides low bearing capacity subgrade layer several sinkholes were detected deep under the track. It was decided to improve bearing capacity of the embankment and to eliminate the subgrade cavities by the use of high-pressure expansion polyurethane resins locally injected into the subgrade soil under the injection pressure approximately 2 bar.

Injection of expanding polyurethane resin is a common alternative to underpinning for individual houses, buildings, and paving slabs for a wide variety of differential settlement situations. The resin is injected in such case directly under the building by means of small diameter aluminium tubes and means almost no disturbance to residents. Thus it might be also very welcome solution in the case of remediation of differential settlements and bearing capacity improvements on railway track under traffic. The pressure exerted by evolved gas during the chemical reaction between two components of polyurethane resin compacts and stiffens the surrounding soil and lifts the soil above the place of injection. Thus it helps to
improve bearing capacity of soil and eliminate differential settlements. As this chemical reaction happen in very short time (from 2 s to 30 s) it enables operator reliable control of quantity of resin to be injected.

![Diagram](https://via.placeholder.com/150)

**Figure 30** GPR results from the railway line section Divača-Prešnica, from km 4+300 to km 4+450

![Plan and site view](https://via.placeholder.com/150)

**Figure 31** Plan and site view of test section Divača, km 4+370

The main advantage of the proposed intervention is possibility to perform improvement works under almost no traffic disruption, what is also the major request of a railway managers. Figure
32 schematically represents the injection intervention, which took place at the depth of 2 to 4 meters. Boreholes were drilled inclined for 30° to 45° and the drilling work had very limited impact upon the railway traffic.

The area for the intervention was 10 m long, started at km 4+370 and finished at km 4+380. The location of boreholes is presented on Figure 33. 22 boreholes of a diameter 100 mm were conducted (11 on left side and 11 on right side) with PVC tube inserted in length 2 – 4 m. PVC tubes prevent the boreholes from collapsing. Furthermore, four metallic injection tubes of diameter 14 mm have been installed into every PVC tube. The lengths of injection tubes were 2.5 m, 3.5 m, 4.5 m and 5 m to reach different depths of the railway track. Throughout the injection phase, the road surface has been monitored with a laser system in order to stop the injection in case of lift.

Figure 32 Scheme of the intervention to improve embankment bearing capacity and to eliminate cavity

Figure 33 Drilling points for injection at test section Divača, km 4+370 to km 4+380
Figure 34 Drilling boreholes at test section Divača, km 4+370

Figure 35 Injection of high-pressure expansion polyurethane resin at test section Divača, km 4+370

Special "measuring" train measured rail deflections on the test section before and after the intervention. Figure 36 and Figure 37 sows deflections of left and right rail. One can see that the effect of intervention is positive, although not very significant. Deflections decrease with injection of high-pressure expansion polyurethane resin in order of magnitude and it can be concluded that it decreases approximately to the half of the initial value.
Figure 36 Deflections of left rail measured by measuring train before and after the intervention on railway track (test section Divača, km 4+370)

Figure 37 Deflections of right rail measured by measuring train before and after the intervention on railway track (test section Divača, km 4+370)
2.4.3 Summary

Injection of expanding polyurethane resin is a common alternative to underpinning for individual houses, buildings, and paving slabs for a wide variety of differential settlement situations. The resin is injected in such case directly under the building by means of small diameter aluminium tubes and means almost no disturbance to residents. Thus it is also welcome solution in the case of remediation of differential settlements and bearing capacity improvements on railway track under traffic. The pressure exerted by evolved gas during the chemical reaction between two components of polyurethane resin compacts and stiffens the surrounding soil and lifts the soil above the place of injection. Thus it helps to improve bearing capacity of soil and eliminate differential settlements. As this chemical reaction happen in very short time (from 2 s to 30 s) it enables operator reliable control of quantity of resin to be injected.

It seems that the dynamic track performance along the test section Dolgi most (bridge transition zone) has not been changed by the injection. In addition, track geometry shows insignificant change before and after the injection. On the other hand, injection of high-pressure expansion polyurethane resin decreased rail deflections at the test section Divača (embankment with sinkholes and marginal fill material).

Therefore, it could be concluded that the injection affects (increase) track stiffness in case of low poorly graded fill material (marginal fill material) with sinkholes, while it has no impact on the track stiffness when well graded high quality granular material is used.
3 Novel Construction Techniques

In recent years, several novel construction techniques have been developed that contribute to improved performance of railway tracks and structures. The following section provides a brief overview of promising techniques. The use of light-weight material and geosynthetic reinforced soil (GRS) has been highlighted. Light-weight material is important gravel-like construction material that can help reducing vertical pressure on low bearing capacity ground. Meanwhile, geosynthetic reinforcement can significantly increase the strength of various geomaterials, including marginal materials, and enable construction of resilient infrastructure.

Particular light-weight material has been laboratory tested with an aim to provide key characteristics for numerical modelling. Two construction techniques based on the use of GRS are also introduced. This material has been implemented in the construction of the most loaded railway geo-structures; bridge abutments and transition zones. Generally speaking, the use of GRS enables faster, environmental friendly, cheaper and more resilient infrastructure.

3.1 Construction techniques for tracks and earthworks

3.1.1 Stabilisation of frictional soil through injection using CIPS (Calcite In-situ Precipitation System)

Climate change is likely to result in intensified periods of rain and drought in various locations around the world. Saturated or partly saturated conditions in railway track subgrade or in the slope of cut-off cause sudden instabilities and slope failures for railway line infrastructure. A fairly new method of subgrade stabilisation involves the use of lime–cement columns (Larsson et al., 2012; Palmen, 2012).

The precipitation system CIPS (Calcite In-Situ Precipitation System) has been created as a permeation grouting system based on a two component fluid with the intention of slowly permeate and fill the pores. It causes cementation through a chemical reaction which bonds the soil particles together at the contact points. CIPS mimics one of the natural reactions in nature where sandstone is formed through calcite precipitation. The CIPS method has to this point mainly been used in Australia and the potential of using the CIPS-method in Swedish frictional soils with the temperature conditions was also recognised.
3.1.2 Chemical stabilisation of marginal soil

In relation to chemical soil stabilisation methods, cement can be used for a wide range of materials. However, the soil should have a plasticity index lower than 30. In the case of coarse-grained soils, the rate of grains passing a number 4 sieve (grains smaller than 4.76 mm) should be greater than 45%.

Treatment with lime is among soil stabilization methods, which are effective in medium, moderately fine, and fine-grained soils. Its application results in decreased plasticity, increased workability and strength, and reduced swelling. Soil treated with lime gains strength slowly and requires approximately 14 days under hot weather conditions and 28 days under cool weather conditions to achieve significant strength. Unsurfaced soils treated with lime abrade rapidly under traffic movements, and thus paving is needed to prevent surface deterioration.

Fly ash consists mainly of silicon and aluminium compounds and is a common additive in chemical stabilizing mixes. It is a by-product of coal fired, electric power-generation facilities. The quality of fly ash highly depends on the type of coal burned. Fly ash is categorized into two classes based on its calcium oxide (CaO) content. These are class C and class F.

Class C fly ash has significant content of CaO (i.e. more than 12%). It originates from subbituminous and lignite coal. Fly ash obtained through lignite combustion has the highest CaO content, often exceeding 30%. This kind of fly ash can be used as a stand-alone stabilizing agent (Figure 39). Class C fly ash, which has CaO content lower than 25%, needs lime to improve its performance.

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Figure 38 Stabilization of frictional soil after using CIPS
Class F fly ash has a low lime content (less than 10%). It originates from anthracite and bituminous coal. This kind of fly ash is not effective as a stabilizing agent by itself, and thus has to be mixed with either lime or lime and cement to be able to stabilize soil.

All traditional soil stabilization methods require certain soil grading and weather conditions in order to be effective, thus limiting their applicability in practice. Stabilizing agents created by Global Road Technology (GRT) do not have these kinds of limitations; they can be applied on soil of any content and are equally effective under both cold and hot weather conditions.

3.1.3 Asphalt/bituminous railway trackbeds

A layer of asphalt or bituminous paving material could be used as a portion of railway track support structures. This technology has been primarily limited to heavy tonnage freight lines in USA, however has steadily increased. It is a promising technology for design of new and maintenance of existing trackbeds. Recent findings and results of research studies are already available in USA, Italy, Japan, France, Spain, Austria and Germany (Rose et al., 2011).
3.1.4 Construction of new tracks with special construction machines

The intention to transfer large volumes of traffic from road to rail requires the reconstruction of the main European railway routes. The construction of new railways is not the only priority. Track reconstruction is also essential owing to increasing demands on track systems, such as axle load and speed, as well as the ageing of the existing tracks. This not only applies to the main railway routes but also to secondary lines and privately owned railroads.

Special construction machines allow to construct and to complete 2,000 meters per shift. The higher output of this continuous working action is the greatest advantage compared to cyclic action using gantry units. Other advantages are the careful handling of the material, particularly the rails, and better preparation of the subsoil (sleeper bed). Usually those machines are also used for the maintenance and are in detail explained in Section 2.

3.1.5 Soil improvement of a bad drainage zone using geogrid

Ballast reinforcement by geogrid is not technologically difficult and it is not expensive in comparison to other maintenance costs. However, it is important to have knowledge on when this method is effective. Consequently, a technical and conditional assessment needs to be made based on an analysis of test section results and further experience. EU research projects give possibility to test technology in various environments.

For instance, Innotrack research project (2010) included an experimental programme that has been carried out to improve a bad drainage zone using geogrid in the Czech Republic. The first test section was situated in a shallow cut. The neighbouring field were sloping towards...
the track and as no drainage was employed on the cut edge, in practice the cut behaves like a ditch. The second test section (Innotrack, 2010) was situated at a railway station.

![Figure 42 Old ballast removal with geogrid installation (Dolga Gora, Slovenia)](image)

Furthermore, rehabilitation of an existing railway embankment of the railway line Ljubljana-Maribor, section Poljčane-Dolga Gora, has been described in detail by Lenart and Klompmaker (2014). Geotechnical investigations indicated very bad ground conditions (low plasticity clay) and a high ground water level. Due to very low subgrade modulus of $E < 10$ MPa a rehabilitation by geogrid reinforcement under sub-ballast layer, 1 m below the sleeper, has been performed in 2008. The whole embankment was reconstructed at that occasion as shown in Figure 42.

Similarly, on the old ballasted railway track of the railway line Ljubljana-Maribor, section Litija-Sava, formation of mud spots has been observed (Lenart and Klompmaker, 2014). Geotechnical investigation showed that approximately 50 cm of ballast is placed directly on the ground consisting of weak subgrade material. Rehabilitation of the railway track needed to be performed with as less as possible traffic disruption. Ballast has been replaced on a short section of the track. A geocomposite reinforcement product consisting of a laid and welded geogrid and separation and filtration geotextile has been installed under the ballast. Geogrid deformation under railway traffic has been monitored over last decade indicating positive effect of reinforcement upon deformation behaviour of railway track (Lenart and Klompmaker, 2014).

### 3.1.6 Geocell reinforcement

Cellular confinement, or geocell, is a three-dimensional (3D) honeycombed shaped geosynthetic used to increase the strength and modulus of the cohesionless soils through a
mechanism of confinement. In cohesionless soils, such as sands or gravels, shear strength and modulus are generally low under small confining pressures, but can be increased using additional confinement offered by a geocell structure under small deformations. Geocell is more effective for soil confinement than other types of geosynthetics (e.g., geogrid or geotextile through interlocking or friction), making it the subject of extensive study as the complex interdependence of geometry, soil strength and loading mechanism make its composite behaviour difficult to quantify (Satyal et al., 2018).

![Image](image.png)

**Figure 43** Lateral displacement (m) of the ballast embankment at the end of 100,000 cycles for (a) Unreinforced 0.6 m embankment and (b) Reinforced 0.6 m embankment on 0.5% CBR subgrade (positive values indicate outward displacement). (Satyal et al., 2018)

Calibrated numerical simulations of the physical experiments were created using three-dimensional finite element analyses, demonstrating general agreement with experimental data. Upon this agreement, a parametric study was performed by Satyal et al. (2018) using calibrated numerical model adopted to realistic railway embankment geometry and loading, enabling observation of geocell configuration, embankment geometry and subgrade properties on railway embankment performance. Some of the conclusions inferred form the study are as follows:

1. Geocell confinement reduced the settlement of the track for all observed subgrade strengths, particularly weak subgrades. Greater subgrade strengths reduced the benefit of geocell reinforcement for additional confinement. The rate of continuous settlement due to cyclic loading was reduced through the use of geocell reinforcement.

2. The use of geocell confinement was very effective in redistributing the vertical stresses on the subgrade, resulting in larger areas of subgrade mobilizing shear strength and reducing...
plastic deformations. Improved subgrade performance from geocell confinement occurring from redistribution of vertical stresses did not vary significantly for the observed range of subgrade strengths. Finally, the redistribution of stresses occurred in the initial cycles for all cases, exhibiting an initial peak in ballast-subgrade pressures followed by relatively steady pressures. Geocell muted this behavior to some level, mainly due to the reduced pressures observed.

3. For the loading conditions used in the analyses, the strains in geocell were low and within the elastic range for typical geosynthetic materials. The maximum tensile strains were localized at the bottom corners of the cells for the railway embankment models, showing the importance of adequately durable seams. They were localized directly under the end of ties forming a band one to two cells in width along the length of track.

4. Subgrade settlement was significantly reduced from use of geocell confinement. The more uniformly distributed vertical stresses effectively reduced the vertical settlement of the subgrade observed due to an arrest of subgrade plastic deformation. With reduced subgrade settlement, geocell confinement helped to preserve the geometry of the track.

5. Lateral displacements along the side slopes of the railroad embankment were greatly reduced through use of geocell confinement. Reduction in lateral displacement was significantly higher in weaker subgrades. The confinement offered by cellular geometry of the geocell likely prevents the lateral heave of the material along with the mattress effect stemming from the confinement of the granular ballast.

Use of geocell reinforcement to improve the settlement performance of the track may be instrumental in increasing track maintenance cycles, enhancing track capacity and preventing differential settlement over localized areas of soft soil.

### 3.1.7 The use of light-weight foam concrete as subgrade bed filler of ballastless track

Controlling settlement of ballastless track subgrade in special soil areas is an ongoing scientific problem in the construction and operation of high-speed railways primarily because of the consolidation settlement of the ground soil, which is caused by additional stress from large loads. To reduce the subgrade weight and additional stress, the use of lightweight foam concrete as subgrade filler, thereby controlling subgrade settlement in special soil areas. However, it remains unknown whether the ballastless track subgrade filled with lightweight foam concrete has a long-term dynamic stability under cyclic loads. To solve this problem, compressive strength and dynamic triaxial tests are conducted to analyse the static and dynamic strength of lightweight foam concrete. Then a large-scale model of a subgrade filled with lightweight foam concrete with target density of 650 kg/m³ is established to determine its long-term performance under cyclic dynamic loads. The results show that the strength of lightweight foam concrete with target density of 500–800 kg/m³ can meet the requirements of both the static and dynamic conditions of ballastless track subgrade, and the ballastless track subgrade filled by lightweight foam concrete with target density of 650 kg/m³ has a good long-term dynamic stability under cyclic dynamic loads when a dynamic buffer layer with thickness of 0.5 m is set between lightweight foam concrete layer and foundation slab (Huang, 2017).
3.1.8 Light-weight material

High compressive soils can exhibit significant settlement due to the construction overload caused by railway embankments. Thus appropriate soil treatment (i.e. soil improvement techniques) is required to eliminate such consequences. Another possible approach is load compensation aimed to hold the stress-strain state present in the soil when undisturbed. Instead of large volumes of material replacements, also light-weight materials can be used.

3.1.8.1 Geofoam

Geofoam is defined by the ASTM D6817 Standard as large lightweight block or planar rigid cellular lightweight foam of polymeric material, expanded polystyrene (EPS) or extruded polystyrene (XPS), used in geotechnical engineering applications. The bulk density of geofoam varies from 0.1 to 0.4 kN/m³ with water absorption of about 2÷5% of the specimen volume. The use of geofoam is a suitable alternative to natural aggregates for use in the construction of embankments standing on soft subgrade (Stark et al., 2004). Geofoam offers the possibility of having a very light structure avoiding all the problems related to the settlement of the embankment foundation (Figure 45). This is especially true when rapid rail track rehabilitation is required in order to solve the severe problems related to landslide phenomena.
3.1.8.2 Light-weight granular material

Different kinds of light-weight natural and artificial granular materials can be used to decrease overloads caused by railway embankment on soft ground. Two of them, Leca (lightweight expanded clay aggregate), and Glasopor (cellular glass aggregate) (Figure 46), were tested within the Destination Rail project to assess their possible use in railway construction industry.

Granular material being used at railway embankments are subjected to specific stress conditions with confining stresses much lower than stresses applied in vertical direction. The large-scale triaxial tests performed within the project were conducted with consideration of results for practical applications in railway track design. Thus tests where in-situ stress conditions and railway traffic loads are adequately simulated were needed.
### Table 2: Test protocol for Lecca and Glasopor samples

<table>
<thead>
<tr>
<th>Loading phase</th>
<th>Confining pressure (vacuum), p'</th>
<th>Loading mode, stress/strain rate</th>
<th>Control parameter: ( q = ) deviatoric (axial) stress, ( \varepsilon = ) vertical strain</th>
<th>Number of cycles, N</th>
<th>Comment</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>5 kPa</td>
<td>stress controlled, 1 kPa/min</td>
<td>( q = ) from 0 to 0.5 kPa</td>
<td>1</td>
<td>virgin loading</td>
</tr>
<tr>
<td>2</td>
<td>5 kPa</td>
<td>stress controlled, 0.1 kPa/min</td>
<td>( \varepsilon = \pm 0.001% )</td>
<td>5</td>
<td>to determine equivalent elastic modulus</td>
</tr>
<tr>
<td>3</td>
<td>5 kPa</td>
<td>stress controlled, 0.1 kPa/s</td>
<td>( q = ) from 0.5 to 0 kPa</td>
<td>50</td>
<td>cyclic loading</td>
</tr>
<tr>
<td>4</td>
<td>5 kPa</td>
<td>stress controlled, 0.1 kPa/min</td>
<td>( \varepsilon = \pm 0.001% )</td>
<td>5</td>
<td>to determine equivalent elastic modulus</td>
</tr>
<tr>
<td>5</td>
<td>5 kPa</td>
<td>stress controlled, 1 kPa/min</td>
<td>( q = ) from 0.5 to 0 kPa</td>
<td>1</td>
<td>unloading</td>
</tr>
<tr>
<td>6</td>
<td>20 kPa</td>
<td>stress controlled, 1 kPa/min</td>
<td>( q = ) from 0 to 1.0 kPa</td>
<td>1</td>
<td>virgin loading</td>
</tr>
<tr>
<td>7</td>
<td>20 kPa</td>
<td>stress controlled, 0.1 kPa/min</td>
<td>( \varepsilon = \pm 0.001% )</td>
<td>5</td>
<td>to determine equivalent elastic modulus</td>
</tr>
<tr>
<td>8</td>
<td>20 kPa</td>
<td>stress controlled, 0.2 kPa/s</td>
<td>( q = ) from 1.0 to 0 kPa</td>
<td>50</td>
<td>cyclic loading</td>
</tr>
<tr>
<td>9</td>
<td>20 kPa</td>
<td>stress controlled, 0.1 kPa/min</td>
<td>( \varepsilon = \pm 0.001% )</td>
<td>5</td>
<td>to determine equivalent elastic modulus</td>
</tr>
<tr>
<td>10</td>
<td>20 kPa</td>
<td>stress controlled, 1 kPa/min</td>
<td>( q = ) from 1.0 to 0 kPa</td>
<td>1</td>
<td>unloading</td>
</tr>
<tr>
<td>11</td>
<td>25 kPa</td>
<td>stress controlled, 1 kPa/min</td>
<td>( q = ) from 0 to 3.0 kPa</td>
<td>1</td>
<td>virgin loading</td>
</tr>
<tr>
<td>12</td>
<td>25 kPa</td>
<td>stress controlled, 0.1 kPa/min</td>
<td>( \varepsilon = \pm 0.001% )</td>
<td>5</td>
<td>to determine equivalent elastic modulus</td>
</tr>
<tr>
<td>13</td>
<td>25 kPa</td>
<td>stress controlled, 0.6 kPa/s</td>
<td>( q = ) from 3 to 0 kPa</td>
<td>50</td>
<td>cyclic loading</td>
</tr>
<tr>
<td>14</td>
<td>25 kPa</td>
<td>stress controlled, 0.1 kPa/min</td>
<td>( \varepsilon = \pm 0.001% )</td>
<td>5</td>
<td>to determine equivalent elastic modulus</td>
</tr>
<tr>
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<td>unloading</td>
</tr>
<tr>
<td>16</td>
<td>40 kPa</td>
<td>stress controlled, 1 kpa/min</td>
<td>( q = ) from 0 to 10 kPa</td>
<td>1</td>
<td>virgin loading</td>
</tr>
<tr>
<td>17</td>
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<td>( \varepsilon = \pm 0.001% )</td>
<td>5</td>
<td>to determine equivalent elastic modulus</td>
</tr>
<tr>
<td>18</td>
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<td>stress controlled, 2 kPa/s</td>
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<td>50</td>
<td>cyclic loading</td>
</tr>
<tr>
<td>19</td>
<td>40 kPa</td>
<td>stress controlled, 0.1 kpa/min</td>
<td>( \varepsilon = \pm 0.001% )</td>
<td>5</td>
<td>to determine equivalent elastic modulus</td>
</tr>
<tr>
<td>20</td>
<td>40 kPa</td>
<td>stress controlled, 1 kpa/min</td>
<td>( q = ) from 10 to 0 kPa</td>
<td>1</td>
<td>unloading</td>
</tr>
<tr>
<td>21</td>
<td>40 kPa</td>
<td>strain controlled, 0.25 %/min</td>
<td>( q = ) from 0 to 30 kPa</td>
<td>1</td>
<td>virgin loading</td>
</tr>
<tr>
<td>22</td>
<td>40 kPa</td>
<td>strain controlled, 0.005 %/min</td>
<td>( \varepsilon = \pm 0.001% )</td>
<td>5</td>
<td>to determine equivalent elastic modulus</td>
</tr>
<tr>
<td>23</td>
<td>40 kPa</td>
<td>strain controlled, 0.25 %/min</td>
<td>( q = ) from 30 to 90 kPa</td>
<td>1</td>
<td>virgin loading</td>
</tr>
<tr>
<td>24</td>
<td>40 kPa</td>
<td>strain controlled, 0.005 %/min</td>
<td>( \varepsilon = \pm 0.001% )</td>
<td>5</td>
<td>to determine equivalent elastic modulus</td>
</tr>
<tr>
<td>25</td>
<td>40 kPa</td>
<td>strain controlled, 0.25 %/min</td>
<td>( q = ) from 90 - 150 kPa</td>
<td>1</td>
<td>virgin loading</td>
</tr>
<tr>
<td>26</td>
<td>40 kPa</td>
<td>strain controlled, 0.005 %/min</td>
<td>( \varepsilon = \pm 0.001% )</td>
<td>5</td>
<td>to determine equivalent elastic modulus</td>
</tr>
<tr>
<td>27</td>
<td>40 kPa</td>
<td>strain controlled, 0.25 %/min</td>
<td>( q = ) from 150 kPa to failure</td>
<td>1</td>
<td>virgin loading</td>
</tr>
</tbody>
</table>
Two types of light-weight aggregates were characterized with a series of large-scale repeated loading triaxial tests. A railway environment conditions were particularly considered and thus particular stress conditions were taken into account. The large-scale triaxial apparatus at the Slovenian National Building and Civil Engineering Institute (ZAG) was used. Details and the calibration procedure for the device have been described in detail by Lenart and Kaynia (2018), while similar approach has been also used by other authors (AnhDan et al. 2006, Lenart et al., 2014). The test results were validated by test repetition. Due to the low confining pressures used in order to simulate stress state of railway track, the confining pressure was applied by means of a partial vacuum, as a back pressure. While vacuum was applied at the bottom of the specimen, it was measured (controlled) at the top of the specimen.

**Figure 47** Test setup with location of local sensors

A series of tests on prismatic specimens with dimensions of 80 cm in height and 40 cm times 40 cm in cross-section was conducted. The axial loading device employs an electro-hydraulic
actuator having a capacity of 100 kN. The axial load is measured by means of two load cells of different precision (one for very small load range with possibility to be secured at large loads) attached successively at the top cap, in order to eliminate the effects of piston friction and other bedding error effects (Figure 49). Deformations were measured locally using vertical and horizontal local deformation transducers (range of 0.0001%). Results of two types of local deformation transducers (LVDT– linear variable differential transformer and LDT – local deformation transducer based on strain gauge (Figure 50)) were compared with deformations detected by image correlation analysis.

Figure 47 presents how the local transportation transducers were attached on the specimen. The same characteristic (vertical deformation, horizontal deformation) has been measured on two opposite sides (A-C for vertical direction, B-D for horizontal direction) and the average strain has been calculated from them to eliminate possible bending of specimen. Unsaturated specimens were tested in drained triaxial compression, using monotonic and repeated low frequency loading and unloading (Table 2). During the testing procedure the effects of cyclic loading (with number of cycles up to 50) and specimen preloading were tested. Confining pressure was applied by vacuum which subsequently was increased from 5 kPa, 20 kPa, 25 kPa to 40 kPa. Strength characteristics were not the main interest of the test; the stiffness properties were mainly the subject of research interests. Young’s modulus at different stress states was evaluated by using static and dynamic methods. A very small unload-reload cycles and shear wave velocity measurements were performed for this purpose.

![Figure 48](image48.png)  
**Figure 48** Light-weight aggregate specimen during vacuum triaxial testing
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Figure 49 Load cells for vertical load measurements

Figure 50 Home made and calibrated local deformation transducers (LDT) based on strain gauge measurements
Figure 51 Glasopor specimen: material (above) and installation of LVDT (below)
Specimens were compacted in means of slight tamping, achieving dry densities equal to 0.302 kN/m$^3$ in case of Leca, and 0.240 kN/m$^3$ in case of Glasopor. Figure 53 and Figure 54 show the complete loading history and deformation propagation for two tested material specimens. It can be seen that in general Leca material exhibits noticeable higher strains. Difference increase with loading range. Strain in the case of Glasopor is approximately twice (at confining stress 5 kPa) to four times (at confining stress 40 kPa) higher compared to strain measured in the case of Leca.

**Figure 52** Leca specimen: material (above) and installation of LVDT (below)
**Figure 53** Complete loading history in case of Leca specimen

**Figure 54** Complete loading history in case of Glasopor specimen
For modelling of railway track response, it is important to know the quasi-elastic behaviour of material during operational stage of a track. Thus, it was very important in case of laboratory testings of material to enable precise stress-strain measurements in the range of very small strains, i.e. $\varepsilon < 10^{-5}$. Based on the stress-strain curves in that range, the quasi elastic properties of material were evaluated. Following figures presents typical results obtained in the case of Glasopor material, which has been used also in the Case study 1: Railway embankment constructed from light-weight material on poor ground, presented in the chapter 4. Typical increase of stiffness can be observed with an increase of confining pressure (Figure 55 - Figure 58). It is also evident from the test results that the material exhibits quasi elastic behaviour with very low damping behaviour at this range of strain.

![Figure 55](image55.png)

**Figure 55** Stress-strain response of Glasopor material during cyclic loading at confining pressure 5 kPa

![Figure 56](image56.png)

**Figure 56** Stress-strain response of Glasopor material during cyclic loading at confining pressure 20 kPa
3.2 Construction techniques for transition zones

Transition zones are defined as parts of the railway track where a change of basic characteristics that define a railway structure in its entirety takes place (Esveld, 2001; Rossmann, 2009; Jenks, 2006). Under the basic characteristics following parameters are considered: substructure and superstructure stiffness, deformation of each substructure layer and each superstructure part, overall value of track deformations, geometric restraints. The transition zones in general represent the appearance of discontinuity in the track structure (UIC, 2008a).
Figure 59 Track structure is a multi-layered system, each component has its own basic characteristics

A certain structural solution is in fact hidden behind the term “transition zones”. The main role of transition zones is to prevent sudden changes in stiffness of the load-bearing structural elements of the track. The aim is to minimize or prevent the occurrence of additional negative dynamic loads over a part of a transition zone, which additionally accelerate the track geometry degradation, with reduced quality and safety of driving as an immediate consequence. This can be achieved by linearly changing certain properties of the surrounding structures at a reasonable distance by dividing one differential change into smaller steps, i.e. dynamically irrelevant intervals (Esveld, 2001; Jenks, 2006).

Ideally these inconsistencies occurring in parts of transition zones do not influence the performance of a passing train in terms of safety but rather they more often reflect upon the quality and comfort of rail services and other dynamic occurrences (Rossmann, 2009). Different railway authorities have approached the solution to the transition zones issue differently. The reason for this lies in the fact that there is no single unique solution which would adequately solve every problem. The Figures in UIC CODE 719R illustrate the exact diversity in structural solutions for transition zones divided into particular railway authorities in Europe (UIC, 2008a).

Transition zones are complex constructions which require individual approaches in design engineering in terms of their location and type (Vajdić et al, 2012). Various approaches to solve this issue have been proposed, with emphasis either on increasing track structure stiffness in a transition zone or on reducing track structure stiffness on a bridge/tunnel/culvert/modern slab track. All the proposed solutions are based on gradual linear adjustment of stiffness of the load-bearing base in the transition zone in order to avoid discontinuity in track structure rigidity, (Sasaoka and Davis, 2005).

Two characteristics are commonly considered when the transition zone issue is trying to be solved:

- Differential Settlements
- Differential Stiffness
Within this research project transition issue was trying to be solved by focusing on structural elements of the track on the open part of the railway line, such as embankments and ballast, which are most often prone to settlements.

3.2.1 GRS transition zone

Geogrids that can improve bearing capacity of formation which is very important for the railway transition zone. The treatment is achieved by replacing the upper 2 meters of the embankment by well compacted sandy gravel. The new material is reinforced with two layers of high elastic modulus geogrid. Further, a geomembrane has to be installed on the bridge. For the FP7 EU research project MAINLINE (2011) the old ballast on top of the concrete block and at sections more than 100 m on both sides of the block is replaced by a 0.35 m thick layer of high quality ballast. For solution in the picture it took only six days to carry out the reparation works. Since the implementation, in the summer of 2008, no maintenance problems have been reported from the passages of track inspection cars over the site. After the retrofitting campaign, the operation on the line was resumed with a maximum allowed operation speed of 160 km/h.

![Figure 60 Final solution adopted to improve transition zones at Montagut (Mainline, 2011)](image)

The other FP7 international EU research project SMART Rail (Pires et al, 2014) analysed problems in the transition zones. Task was focused on how to achieve smooth transition between different types of track structure where abrupt change in the rigidity of track structure and track settlement occurs between individual transverse profiles. “Buna” bridge on the railway line Zagreb - Sisak- Novska in Croatia was selected as a pilot project due to the obvious problems which were typical for transition zones:

- irregularities in the geometry → track unevenness and under ballast gaps;
- vertical displacements of the whole track structure.

Main causes of the above mentioned problems detected during the investigation works performed during the SMART RAIL project were as follows:

- differential settlements;
- dynamic impacts of the train due to the changes in the track stiffness;
- poor distribution of traffic load.
On each side of the bridge different technical solution was implemented (Vajdić et al, 2014). One of them is based on geogrid reinforced soil (GRS) and is described in the following paragraph.

A prestressed geogrid reinforced soil (GRS) is used in transition zone between "Buna" bridge and open ballasted track in direction to Sisak. The length of transition zone is 17 m. The subgrade material of controlled composition and certain degree of compaction is built into the embankment from bridge abutment to the end of the transition zone. The embankment is constructed in a slope 1:1 from the bottom to the top of the abutment foundation and in a slope 1:2 to the depth 1 m bellow the top of the embankment. A layer of geotextile is laid at the bottom of embankment. Upper 1 m of embankment is constructed by geogrid reinforced soil (GRS). Horizontal layers of geogrid have a vertical spacing of 20 cm. GRS starts behind the abutments and continue from 8 m (at the bottom) to 17 m (at the top) in the direction going away from the bridge. Subgrade material of controlled composition for the embankment construction in the transition zone is used as a backfill material in upper 40 cm of GRS, while bottom 60 cm of GRS is formed from the backfill material that is used for construction of embankments on the open track. In order to obtain higher stiffness of formation layer GRS was designed to be prestressed in vertical direction in the length of 8 m, direction going away from the bridge.

More details on the site construction can be found in SMART RAIL project deliverables (Pires et al, 2014) and in Section 3.3.3 Case study 2: Transition zone.

Figure 61 Construction of GRS prestressed in vertical direction (Pires et al, 2014)
### 3.2.2 GRS bridge abutment

On the other hand, bridge can be constructed also without particular transition zone and still being capable to eliminate “bridge bumps”. This particular advantage is enabled by bridges with GRS abutments, which allows uniform settlements of the entire bridge-embankment structure (Helwany et al., 2003). A considerable number of studies have been investigated the applicability of GRS technology on the construction of bridge support-structures over the last two decades, (Tatsuoka et al., 1997; Adams et al., 2002; Wu et al., 2006; Nicks et al., 2016; Ardah et al., 2017, Chang et al., 2017). It has been proved that bridge abutments with GRS structures have a lower costs and cause lower environmental impacts (Fifer Bizjak and Lenart, 2018).

Such a bridge with GRS bridge abutment has been constructed across the Pavlovski potok in Slovenia at the end of 2014. A reinforced concrete slab was integrated into a pair of geosynthetic reinforced soil bridge abutments. The facings of the abutments were at a distance of 5.50 m. A longitudinal cross-section of the bridge is shown in Figure 62. The geotechnical situation under the abutments was very difficult to construct because the soil was made up of clay with a low load-bearing capacity (). If a conventional bridge were built, a deep piled foundation would have been needed for a sub-soil of such high compressibility and low shear strength. Due to time and cost limitations, it was decided to construct GRS bridge abutments with full height rigid (FHR) facings. This ensured a high external stability while minimizing the potential negative effects of significant ground settlements (Lenart et al., 2016).

![Figure 62](image_url)

**Figure 62** Cross-section of a bridge supported by reinforced soil abutments at Pavlovski potok
Table 3 Geological – geotechnical data at the bridge across the Pavlovski potok site

<table>
<thead>
<tr>
<th>Depth [m]</th>
<th>Description</th>
</tr>
</thead>
<tbody>
<tr>
<td>0.0 – 0.5</td>
<td>sandy gravel</td>
</tr>
<tr>
<td>0.5 – 3.0</td>
<td>sandy clay with inclusions of gravel and sand</td>
</tr>
<tr>
<td>3.0 – 5.0</td>
<td>clayey and silty sand</td>
</tr>
<tr>
<td>5.0 – 8.0</td>
<td>silty sand</td>
</tr>
<tr>
<td>8.0 – 11.0</td>
<td>decayed stratified mart</td>
</tr>
<tr>
<td>11.0 – 17.0</td>
<td>sandy mart</td>
</tr>
<tr>
<td>17.0 – 23.3</td>
<td>sandy-silty clay</td>
</tr>
<tr>
<td>23.3 – 26.3</td>
<td>sandy mart - solid</td>
</tr>
<tr>
<td></td>
<td>Water level depth: 2.7 m</td>
</tr>
</tbody>
</table>

GRS bridge abutments were constructed in less than 10 days due to the simplicity of the selected construction process. In the first stage, the foundation of the abutments was completed. Well-compacted gravelly soil was used for the foundations of the abutments and wrapped around with geosynthetic soil (Figure 63). This step was followed by the construction of a geosynthetic reinforced soil mass for the bridge abutments. A detailed description of the process is given by Lenart et al. (2016). The geogrid was wrapped around the gabion bags. After this, the backfill material was placed and then compacted until optimal dry density was achieved (Figure 64). Additional stripes (anchors) were used for the vertical steel reinforcement and the connection of the ends of the horizontal layers inside the facing structures. The concrete facing structures above the abutments were built using appropriate formwork and cast-in-place concrete.

Minimal reinforcement was installed in the facings to prevent the occurrence of any kind of cracks in the concrete. Scouring below the level of the bottom of the bridge abutment was prevented with a foundation of reinforced concrete facings at a depth of 150 cm. This was done because of a significant flow of water, which can appear in the spring and autumn. For this reason, it was decided to construct facing structures with a minimum thickness of 16 cm. The facing elements were not used as a construction element, but only as a scour protection element. For additional protection against scouring, a total of four wing walls were built as a riprap structure (Figure 65). The construction was extended for another 5m on each side of the abutments to control the channel of the stream. The bridge deck was placed directly on top of the geosynthetic-reinforced backfill of the bridge abutments using a thin layer of bedding concrete.
Figure 63 Construction of the gravel foundation, before wrapping the foundation with geosynthetics

Figure 64 Construction of the GRS abutments

Figure 65 GRS bridge abutment after completion
For the bridge abutments, natural gravel material from the nearest quarry was used. For this type of construction there is no need to use heavy construction machinery, which results in lower construction time and costs. Piles, pile caps, steel-reinforced concrete abutments, wing walls of RC retaining structures, and approach slabs become unnecessary when the described GRS technology is used, which significantly reduces the amount of concrete needed. Based on data obtained from the nearby newly-built bridge, which had steel-reinforced concrete abutments founded on deep piled foundations, the quantity of concrete works needed when constructing such conventional bridge abutments was compared to the case of the integrated GRS bridge across the Pavlovski potok. The results showed that, in the case of steel-reinforced concrete abutments, nearly 120 m$^3$ of extra concrete would have been needed in comparison with the geosynthetic reinforced soil abutments (Table 4) or with the other words, GRS bridge abutments caused almost 70% savings in concrete.

**Table 4** Comparison of the amounts of concrete needed for bridge abutments in case of GRS bridge abutments and reinforced concrete abutments for a comparable bridges across the Pavlovski potok

<table>
<thead>
<tr>
<th>Element</th>
<th>Amounts of concrete needed [m$^3$]</th>
<th>Difference</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>RC abutments</td>
<td>GRS abutments</td>
</tr>
<tr>
<td>Piles (D=100cm L=24m)</td>
<td>75</td>
<td>-</td>
</tr>
<tr>
<td>Pile caps (120/120cm)</td>
<td>23</td>
<td>-</td>
</tr>
<tr>
<td>Abutments (d=50cm)</td>
<td>21</td>
<td>9</td>
</tr>
<tr>
<td>Wing walls (d=30cm)</td>
<td>7</td>
<td>5</td>
</tr>
<tr>
<td>Approach slabs</td>
<td>12</td>
<td>-</td>
</tr>
<tr>
<td>Superstructure</td>
<td>35.5</td>
<td>42</td>
</tr>
<tr>
<td><strong>Total</strong></td>
<td><strong>173.5</strong></td>
<td><strong>56</strong></td>
</tr>
</tbody>
</table>
4 Advanced Finite Element Analysis

This section outlines the use of numerical modelling for novel construction techniques, which is important for their implementation in practice. Two case studies were analysed, whereby Finite Element (FE) modelling was employed. FE modelling was conducted in the first case study to analyse the use of novel light-weight materials in railway track on poor ground, while it was conducted in the second case study for the design of a novel GRS transition zone.

4.1 Background

Advanced FE modelling may be employed to analyse the behaviour of rail embankments under a range of maintenance techniques and novel construction methods. Numerical modelling has gained popularity in recent decades due to its flexibility, cost effectiveness, and time efficiency. The ability of a numerical model, such as an FE model, to accurately simulate field conditions largely depends on the adopted material constitutive models and their ability to represent real soil behaviour. Soil constitutive models range from simple linear elastic material models that greatly simplify the material behaviour of soil, which is highly non-linear, to advanced models that include extended hardening concepts that can be used to capture complex soil behaviour, e.g. cyclic loading behaviour, anisotropy, etc. The associated material parameters of a soil constitutive model may be obtained from field and laboratory testing, as described by Potts et al. (2002).

Many studies have employed FE modelling to evaluate the impact of reinforcement to support embankments constructed on soft cohesive soils under undrained and partially drained conditions to ensure adequate shear strength to prevent failure, as well as ensuring that settlement values are within acceptable limits. For the analysis of embankments constructed of lightweight material and founded on soft soils, FE modelling has an advantage over alternative limit state equilibrium models since it allows the complete interaction to be simulated and does not require the mode of failure to be predetermined. Furthermore, three-dimensional (3D) FE modelling is advantageous in situations where complex geometries are required to be analysed, e.g. transition zones. However, it requires more computational effort and time to analyse than alternative 2D models that assume plane strain conditions.

Zheng et al. (2009) investigated the influence of various reinforcing conditions for an embankment founded on soft deposits with low permeability according to a two-dimensional (2D) model using PLAXIS FE software. Reinforcement of the embankment using 1) geosynthetics, and 2) a combined application of geosynthetics and piles was investigated for the embankment and compared to the situation of no reinforcement. The model was used to analyse the displacement of the embankment, as well as the stresses in the geosynthetic layer and the piles. Benmebarek et al. (2015) also employed PLAXIS FE software to investigate the effect of geosynthetic reinforcement on the settlement of an embankment over a locally weak zone. The study employed a 2D FE model, simulating plane strain conditions, and the soft soil model in PLAXIS was used to simulate the behaviour of the existing soils, including the locally weak zone. Meanwhile the embankment fill was modelled according to a Mohr-Coulomb failure criterion and the geosynthetic material was simulated according to line elements,
modelled as elastoplastic material since this material can sustain tensile forces only. Interface elements were also employed to simulate the slippage between the soil and the geosynthetic material. Both short-term and long-term consolidation phase analyses were performed using the model, and the settlement results of the reinforced embankment were compared to the unreinforced case.

Similarly, Jira et al. (2017) investigated the use of geogrids and vertical lime-cement columns to improve the bearing capacity of the subgrade system for an existing railway embankment using FE modelling. The model was generated in 3D using ANSYS FE software, whereby the geogrid was modelled as a thin plate and contact surface were implemented between the soil and the geogrid, as well as between the soil and the vertical columns. Koch and Szepesházi (2013) also numerically modelled the use of deep mixed columns as a ground treatment technology to analyse the settlement and the stability of a constructed embankment. Several studies have also employed FE modelling to analyse the impact of vertical drains in terms of the acceleration of consolidation for embankments constructed on soft soils (Borges, 2004; Krishnamoorthy and Rajan Idicula, 2012).

For transition zones along railway lines, advanced FE modelling may be employed to analyse track movements and the dynamic train-track interaction at these critical locations. For example, Paixão et al. (2014) employed 2D FE modelling to analyse a transition zone on the approach to a composite bridge structure along the Portuguese South Main Line. The model was generated in 2D using ANSYS FE software to represent the longitudinal transition section and was calibrated using field measurements. The model was subsequently used to determine the train-track interaction forces, the vertical displacement of the train wheels along the transition zone, and the elastic deformation of the subgrade. Advanced FE modelling may also be employed to determine the dynamic response of a railway track to high speed rail. For example, Hu et al. (2016) conducted a FE of a combined track, embankment and layered ground model to determine the critical velocity of high speed trains in terms of the induced vibrations. Gallego Giner et al. (2016) also conducted a similar study, whereby a 3D FE model was employed using ANSYS and a dynamic analysis was performed to determine the maximum train velocity based on limits in terms of track deflection.
4.2 Case study 1: Railway embankment constructed from light-weight material on poor ground

Lightweight filling materials, such as lightweight aggregate or foam glass are primarily used as stabilising measures to reduce the ground stresses, and for reduction of load and settlement on poor ground. In addition, these materials are used for frost insulation. However, it has also been claimed that these materials may have a vibration reducing effect, either when used in railway fillings or as a layer between the ground and basement walls of nearby buildings. Typical grain size and densities for the light fill material compared to blasted rock are shown in Table 5.

<table>
<thead>
<tr>
<th>Material</th>
<th>Grading (mm)</th>
<th>Dry density (kg/m³)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Lightweight aggregate</td>
<td>0 - 32</td>
<td>400</td>
</tr>
<tr>
<td>Foam glass</td>
<td>10 – 50/60</td>
<td>180-300</td>
</tr>
<tr>
<td>Blasted rock</td>
<td>0 -300</td>
<td>1800</td>
</tr>
</tbody>
</table>

According to the Norwegian National Rail Administration’s Technical Regulations, a filling with lightweight aggregate or foam glass should be designed as shown in Figure 66.

![Figure 66 Principle cross section of railway with filling from the Norwegian National Rail Administration’s Technical Regulations](image)

LC-columns are soil mixed with lime-cement. Figure 67 (left) shows an illustration of the process, and Figure 67 (right) shows the resulting LC-columns in the soil after hardening.
The LC-columns can be installed below the railway tracks (as reinforcement of the ground soil) or as a wall to the side of the railway tracks (to block propagation of railway vibrations), see typical geometry in Figure 68.
In this case study we have investigated the possible vibration reduction effect by lightweight filling materials, alone or in combination with lime cement columns. The analyses have been performed using the FE-program Comsol Multiphysics. Characteristics of the materials were derived from laboratory tests of lightweight filling materials performed in the Destination Rail project by ZAG, and earlier laboratory tests on conventional materials performed by NGI for the Norwegian Rail Administration. Characteristics of the soil in the models are typical for Norwegian sites with soft soil conditions.

### 4.2.1 FE modelling

The FE-model is a 2-dimensional (2D) model, describing a typical cross-section of a railway line with a 2 m high filling. Comparisons with results using a 3-dimensional (3D) model have also been performed, see section 4.2.4. The elements are quadratic (second order) Lagrange elements with a maximum element size dependent on the frequency in order to maintain good resolution also for the higher frequencies, i.e. app. 8 elements (or 16 evaluation points) per wave length. The models are equipped with transmitting boundary layers in order to allow the vibration energy to dissipate out of the models and prevent it from reflecting back from the boundaries. Since this is crucial for achieving correct results, efforts have been made to design the boundary layers and to verify that they work properly, see section 4.2.5.

The calculations are performed in the frequency domain. The models are excited by a vertical dynamic unit force, applied synchronously on both rails at the mid frequencies of the 1/3-octave frequency bands from 4.0 Hz to 125 Hz. Static weight of the train is included in the calculation model by taking into account the impact on the stiffness of the ballast and fill masses underneath the track. The dynamic load of an unsprung mass of 3000 kg is applied to the models in the form of boundary loads on the rails corresponding to the contact surface with the train.
Since the models are excited with a unit force, the calculated vibration velocities are not equal to the vibration velocities that will arise during actual train passages; however, the results show the frequencies at which the ground responds most to dynamic forces. To get an estimate of actual vibration velocity spectra the input force has also been scaled to give vibration spectra for the reference case which corresponds to measured spectra on the type of ground conditions described in the model input data.

The following configurations are described in FE-models, see Figure 69:

1. Model with a filling made of local materials, i.e. clay. This is the reference case used for evaluation of the other models. Model 1 is shown in Figure 70.

2. Similar to model 1 but with different types of filling materials, e.g. lightweight Foam glass

3. Similar to model 1 and 2 but with lime-cement columns under track bed

4. Similar to model 1 and 2 but with a screen of lime-cement columns between railway line and the nearby dwellings

The soil is modelled as five horizontal homogenous layers, with bedrock at a depth of 50 m. The undrained shear strength profile of the soil layers is shown in Figure 70.

![Model 1](image1.png)  ![Model 2](image2.png)  ![Model 3](image3.png)

**Figure 69** Cross section of a railway track on top of a filling with various measures to reduce vibrations.
To study the vibrations by train passages in a typical building foundation, the vertical vibration velocities are computed and averaged over a cross-section through the dry crust (2.5 m depth) and over the width of a typical building foundation (6 m), in the direction transverse to the track, see Figure 71.
4.2.2 Model input data

The input material properties used in the computations are shown in Table 6.

**Table 6 Material properties of soil and track**

<table>
<thead>
<tr>
<th>Material</th>
<th>Young’s modulus $E$ (MPa)</th>
<th>Poisson’s ratio $\nu$</th>
<th>Density $(\text{kg/m}^3)$</th>
<th>Loss factor $\delta$</th>
</tr>
</thead>
<tbody>
<tr>
<td>Rail</td>
<td>205E3</td>
<td>0.28</td>
<td>49 (kg/m)</td>
<td>0.02</td>
</tr>
<tr>
<td>Sleeper</td>
<td>25E3</td>
<td>0.33</td>
<td>2300</td>
<td>0.02</td>
</tr>
<tr>
<td>Ballast</td>
<td>150-180 $^\circ$</td>
<td>0.25</td>
<td>1900</td>
<td>0.3</td>
</tr>
<tr>
<td>Reinforcing layer</td>
<td>110-140 $^\circ$</td>
<td>0.25</td>
<td>1900</td>
<td>0.3</td>
</tr>
<tr>
<td>Blasted rock</td>
<td>110-140 $^\circ$</td>
<td>0.25</td>
<td>1900</td>
<td>0.3</td>
</tr>
<tr>
<td>Glasopor (Foam glass)</td>
<td>93</td>
<td>0.2</td>
<td>240</td>
<td>0.3</td>
</tr>
<tr>
<td>Clay</td>
<td>75-110 $^\circ$</td>
<td>0.49</td>
<td>1900</td>
<td>0.06</td>
</tr>
<tr>
<td>Lime cement columns</td>
<td>250</td>
<td>0.33</td>
<td>2000</td>
<td>0.1</td>
</tr>
</tbody>
</table>

*Varying with depth

For the models with lime cement columns, an undrained shear strength of 250 kPa has been assumed. Material properties of the LC-columns in clay are calculated as average values between clay and LC-columns based on the distance between the piles in the direction perpendicular to the modelled cross-section.

4.2.3 Results

In the following figures, the centre of the track is marked with a red line. The large field on the left in the figure show the results in the frequency range from 4 Hz - 125 Hz. The small field to the right of the large field shows the result for the scaled frequency-weighted RMS value summed across the entire frequency range ($v_w$). The far right field shows the colour bar scale.

Figure 72 shows results for the model with Glasopor in the filling. The figure shows the relationships (Transfer Function) between the model with Glasopor in the filling and the reference model with local masses in the filling. A transfer function equal to 1.0 means no effect of the measure on the vibrations. A transfer function higher than 1.0, corresponds to an increase in the vibration values, while a transfer function lower than 1.0 indicates that the mitigation measure reduces the vibration values.
The results indicate that the Glasopor filling may have some vibration mitigation effect in the frequency area between 40 and 63 Hz. However, as can be seen from the bar showing the frequency weighted values (vw) the effect on the overall frequency weighted RMS value seems to be negligible (the transfer function is close to 1.0).

The reason for this can be seen from Figure 73. The figure shows the estimated vibration velocity after scaling with measurement data, i.e. the input force has been calibrated so that the reference model, with local masses as filling, gives the same results 20 meter from the track as typically measured at soft ground conditions. Hence, these results captures that train passages generate vibrations differently at different frequencies. The figure shows that on soft ground the frequency range from 6.3 – 16 Hz totally dominates and very little vibration energy is located at the higher frequency. For stiffer soil conditions Glasopor used as a vibration mitigation measure may be more effective.
Clay filling - Estimated vibration velocity $v_w$ (mm/s)

Figure 73 Reference model with clay in the filling. Estimated vibration velocity weighted according to NS 8176

Results from the analyses with clay as filling materials and LC-columns with alternating height, 9 and 12 m, below the filling are shown in Figure 74. The results show a decrease in vibration velocity for the low frequencies, but an increase in the mid and high frequency range. This is as expected since arrays of LC-columns under the track reinforces the soil, making it stiffer and thereby alters the frequency spectra to be more high-frequent. Therefore, LC-columns under the track may need to be combined with other measures to avoid structure borne noise problems.
Figure 74 Model with clay in the filling and LC-columns below track. Ratio between evaluated model and the reference model (clay in the filling and without LC-columns).

In Figure 75 results for the model with a combination of Glasopor in the filling and LC-columns below the filling is shown. The figure shows that LC-columns combined with Glasopor result in lower vibration velocities for frequencies below 80 Hz.
Figure 75 Results for the model with Glasopor in the filling and LC-columns below. Ratio between evaluated model and the reference model.

Results from analyses of LC columns as a wall next to the railway tracks are shown in Figure 76 for clay in the filling and in Figure 77 for Glasopor in the filling. The model includes two walls with a height of 15 m and a spacing of 3,5 m. The closest wall is at a distance of 14 m from the track, see Figure 68.

The results show a reduction of vibration velocity for frequencies below 50 Hz for the model with clay in the filling. For the model with Glasopor in the filling the LC screen has good effect up to 80 Hz.
Figure 76 Model with clay in the filling and a LC-columns screen next to the track. Ratio between evaluated model and the reference model (clay in the filling and without LC-columns).

Figure 77 Model with Glasopor in the filling and a LC-columns screen next to the track. Ratio between evaluated model and the reference model (clay in the filling and without LC-columns).
4.2.4 Design and effect of absorbing boundary Layers

Since a numerical model needs to be truncated with particular conditions applied on the boundaries, a challenging aspect is the artificial reflections from these model boundaries. Since this is crucial for achieving correct results, efforts have been made to design the boundary layers and to verify that they work properly. To effectively absorb the elastic energy in the absorbing domains a combination of geometrical and complex scaling has been used. In Figure 78, results from an isotropic and homogeneous half-space model with a point source on the surface is shown. A sketch of the 2D model with absorbing domains is shown in Figure 78a. The geometric scaling means that the absorbing domains are numerically stretched such that the elastic waves travel 2-4 times the distance of a full wavelength, see e.g. Figure 78c.

![Figure 78a 2D model with absorbing domains (colored rectangles)](image1)

![Figure 78b Vertical displacement amplitude for three different thicknesses of absorbing domain.](image2)

![Figure 78c Absolute scaled x-coordinate. Black curve is the cartesian x-coordinate, the colored lines show the local x-coordinate in the absorbing domain.](image3)
The complex scaling means that as the wave travels in the absorbing domains, the elastic properties become increasingly complex, in effect dispersing the elastic wave also in the complex space resulting in an attenuation in the real space. As can be seen from Figure 78b, the applied transmitting boundary layer effectively absorbs the vibration energy and prevents reflections back to the model.

4.2.5 2D versus 3D model

For railway induced ground vibrations, the load is not a line source, but point sources where the train is in contact with the rails. To be precise, note that the line source is actually a boundary load, but the boundary is very narrow compared to the length and resembles in effect as a line. The same applies to the point source, which is actually a small surface. For a line source, a 2D and a 3D model will be equivalent and give the same solution, see Figure 79 top. However, for point source, a 3D model will attenuate the vibrations in all three dimensions and not only in two dimensions as in a 2D model. Therefore the 3D and 2D models give different responses for point sources, see Figure 79 (lower panels). Note that the displacement for the 3D model with point sources has been amplified 10 times to be comparable to the 2D solution.

![Figure 79](image-url) Comparison between 2D and 3D in a cross section (blue line in the left figures). Top left: Model with a line source. Bottom left: Model with point sources. Right: Difference between vertical displacement in 2D and 3D model.
However, looking at the results in a plan view the vertical displacement show similar trend or shape for a line source and for the point sources, see Figure 80. As can be seen from the figure the similarity will depend on the position of the comparison in the plan.

Figure 80 Surface plot, top view, of vertical displacement for a line source and point sources.

Another challenge is to approximate complex 3D features in 2D. Such features can be very thin plates, or very heterogeneous materials such as soil with LC-columns. LC-columns in a 2D model can be approximated as an effective media with volume average material properties. E.g. if 50% of the soil is replaced with LC-material, then the Young’s modulus, Poisson’s ratio and density of the resulting effective is the average of the respective properties of the LC-material and the soil. A comparison show that an effective media approximation is a good approximation, see Figure 81.

Figure 81 Comparing heterogeneous media in 3D with an equivalent effective media in 2D. Left: 3D-model with LC-columns below track. Right: Vertical displacement amplitude at 10 Hz for a line-load on the rails. Thick blue line is from the 2D model, thin line is from the 3D model.
There is also a concern that a 2D-model does not capture the effect of a finite structure as a screen correctly. This is because the angle of incidence of the propagating wave may affect the efficiency of the screen (in a 2D model the angle of incidence is 90 degree), and also because the screen has a finite length and hence propagating waves may bend around the corners of the screen. To investigate the effect of these phenomena 3D analyses of the LC columns screen solution has been performed. The model is without filling, and is shown in Figure 82. The results are compared with a 2D model, i.e. with a line source and infinite long screen, see Figure 83.

**Figure 82** 3D model with a LC screen. The model has a symmetry plane perpendicular to the tracks, thus only half the model needs to be solved (blue domains). Right: Top view of the surface (location of point sources marked with red).
The comparison shows similar results behind the screen for the two model but with clearly lower effect of the screen for the 3D model. The 3D model seems to capture the effect of the angle of incidence of the propagating wave towards the screen. The comparison indicates that 2D models may not be sufficient to evaluate structures like screens.
4.3 Case study 2: Transition zone

The main objective of this case study was to develop and calibrate a numerical 3D model of the transition zone between the open track on embankment and the bridge to help determine the best distribution of the soil layers and their mechanical characteristics within the transition zone area that enables the smoothest transition of the substructure and superstructure stiffness and deformation parameters during the train passage.

This research presents the extension of the research performed within the SMART RAIL project and the case study of “Buna” bridge in Croatia (Figure 84) where special construction technique for the transition zone have been used and analyzed (Pires et al, 2014 pp 55-77).

“Buna” bridge, situated at the railways track M104 Novska – Sisak – Zagreb at km 398+422, is selected for the case study because of the obvious problems in the transition zones, immediately before and after the bridge. During the SMART RAIL project the existing old steel bridge was replaced with the new prefabricated concrete bridge, and the new transition zones were constructed on both sides of the bridge in the length of 17 meters (Figure 85).

![Figure 84 Location of “Buna” bridge](image1)

![Figure 85 New “Buna” bridge - railways track M104 Novska – Sisak – Zagreb at km 398+422](image2)

On each side of the bridge different technical solution was implemented (Vajdić et al., 2014).
For the Zagreb direction, a solution was adopted based on the previous experience of different railway authorities as described in UIC CODE 719R: Earthworks and track bed on railway lines as can be seen on the Figure 86 (UIC, 2008b).

![Figure 86 Final design of the transition zone on Zagreb site](image)

For the Sisak direction, a more innovative approach was used since the stabilisation of the embankment is achieved by using prestressed geogrid reinforced soil (GRS) as can be seen on the Figure 87.

![Figure 87 Final design of the transition zone on Sisak site](image)

In terms of geotechnical parameters, each case is unique, and conclusions cannot be universally applied. Correctly defining model will allow us comparative analysis of different technical solutions, as well as a better understanding of the mechanisms that occur in the railway track structure (transition zone). This way we are able to confirm the methodology applied in solving specific problems in the transition zones increasing the likelihood of a positive outcome.

Calibration of the created new 3D numerical model will allow us further analysis of the transition zone open issues, as well as finding in the end optimal gradient of the linear change.
of stiffness values in longitudinal direction that causes minimal occurrence of dynamic excitation within the structure (Vajdić, 2016).

4.3.1 Methodology used in case study

Within the DESTination RAIL project, the focus is given to the creation and calibration of the numerical 3D model of the transition zones between the open track on embankment and the bridge in order to be able to analyze and in the end define the best practice to be used for achieving smoothest transition of the substructure and superstructure stiffness and deformation parameters during the train passage. The experimental results of the field tests performed during the SMART RAIL project on the “Buna” bridge together with the new inputs collected during the monitoring and measurements of deformations and stresses within DESTination RAIL project have been used to calibrate newly created 3D numerical model. The overall project performance methodology used can be seen in Figure 88.

Figure 88 Methodology used for calibration of the digital model – feedback analysis

In order to calibrate newly constructed 3D numerical model feedback analysis have been used. Inputs used from the previous SMART RAIL project are following:

1. Results obtained during construction by measurement of the characteristics of embedded materials (during construction phase):
   a. laboratory testing of samples of materials;
   b. testing the compactness with the static circular plate;
c. testing of compactness with a dynamic circular plate.
2. Results obtained during performed measurement of stresses and vertical displacements of the track structure using Benkelman beam (about 9 m long) and a wagon with a precisely defined axle load (approx. 20 t / axle) with locomotive (monitoring phase).
3. Track geometry measured with measuring trolley (monitoring phase).
4. Results obtained during performed measurement of settlement with inclinometer (monitoring phase).

Based on calibrated new 3D model other solutions have been numerically tested.

4.3.2 Numerical model

The efficiency of the transition zones of the Buna bridge were studied with a 3D model built in Abaqus software (ABAQUS, 2016). The model contained a railway track structure with a bridge and a terrain extending 20 meters on both sides of the bridge. The detailed model included bridge construction, rails, fastening system, sleepers, ballast, tampon layer and the lower layers of soil (embankment) to the depth of 3.3 m (measured from the lower surface of the ballast). Due to symmetry along the longitudinal axis, only one half of the railway track structure was considered (Figure 89) while symmetrical boundary conditions were applied at the border surface. At both ends of the rails axial springs were used as a replacement for the continuation of the rails. The stiffness of the springs was calibrated to fit the experimental results.

All parts (with the exception of bridge construction) were modelled with 3D solid elements (C3D8R and C3D6). The bridge construction was simplified and modelled with 2D shell elements (S4R). The soil below the depth of 3.3 m was modelled by 3D solid continuum infinite elements (CIN3D8) in order to provide “quiet” boundaries to the finite element model in
dynamic analyses. A special attention was paid to the modelling of the transition zones so different layers of soils were considered at different locations of the embankment (Figure 90). In order to capture the dynamic effects, the upper surface of the rail was considered uneven – i.e. the measured elevation of the rail was taken into account.

![Different soil layers in transition zones](image)

**Figure 90** Different soil layers in transition zones

All parts of the model were modelled with elastic materials. While standard properties were considered for steel (rails), concrete (sleepers) and fastening system (dampers), the properties of the ballast, tampon layers and soil were calibrated to fit the measured response of the static tests.

Two types of analyses were performed to calibrate the numerical model: static and dynamic. The analyses are described in the following chapters.

### 4.3.3 Static analysis

The static analysis was first used to calibrate the model and elastic properties of the soil components. The characteristics of the material used for modelling were the one obtained by measuring on site:

a) Transition zone on Zagreb side of the bridge (Figure 91, Table 7)

![Indexing of the areas of the railway track with the same homogeneous material characteristics on Zagreb side](image)

**Figure 91** Indexing of the areas of the railway track with the same homogeneous material characteristics on Zagreb side
### Table 7 Characteristics of the material for each index area of the railway track on Zagreb side

<table>
<thead>
<tr>
<th>No.</th>
<th>Type of the installed material</th>
<th>Density</th>
<th>Poisson's ratio</th>
<th>Young's Modulus of elasticity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Existing subgrade material</td>
<td>2,00</td>
<td>0,30</td>
<td>50.000,00</td>
</tr>
<tr>
<td>2a</td>
<td>Subgrade material of controlled composition and granulation</td>
<td>2,00</td>
<td>0,30</td>
<td>280.000,00</td>
</tr>
<tr>
<td>2b</td>
<td>Subgrade material of controlled composition and granulation</td>
<td>2,00</td>
<td>0,30</td>
<td>40.000,00</td>
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<tr>
<td>2c</td>
<td>Subgrade material of controlled composition and granulation</td>
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<td>0,30</td>
<td>25.000,00</td>
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<tr>
<td>2d</td>
<td>Subgrade material of controlled composition and granulation</td>
<td>2,00</td>
<td>0,30</td>
<td>50.000,00</td>
</tr>
<tr>
<td>3a</td>
<td>Cement stabilized base layer</td>
<td>2,70</td>
<td>0,15</td>
<td>8.000.000,00</td>
</tr>
<tr>
<td>3b</td>
<td>Embankment stabilized by hydraulic binder</td>
<td>2,10</td>
<td>0,30</td>
<td>5.000.000,00</td>
</tr>
<tr>
<td>4</td>
<td>Sub-ballast bed</td>
<td>1,90</td>
<td>0,20</td>
<td>340.000,00</td>
</tr>
<tr>
<td>5</td>
<td>Ballast bed</td>
<td>1,90</td>
<td>0,20</td>
<td>340.000,00</td>
</tr>
</tbody>
</table>

b) Transition zone on Sisak side of the bridge (Figure 92, Table 8)

![Indexing of the areas of the railway track with the same homogeneous material characteristics on Sisak side](image_url)
Table 8 Characteristics of the material for each index area of the railway track on Sisak side

<table>
<thead>
<tr>
<th>No.</th>
<th>Type of the installed material</th>
<th>Density</th>
<th>Poisson's ratio</th>
<th>Young's Modulus of elasticity</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Ballastbed</td>
<td>1,90</td>
<td>0,20</td>
<td>340.000,00</td>
</tr>
<tr>
<td>6a</td>
<td>Prestressed geogrid reinforced soil</td>
<td>2,00</td>
<td>0,30</td>
<td>150.000,00</td>
</tr>
<tr>
<td>6b</td>
<td>Prestressed geogrid reinforced soil</td>
<td>2,00</td>
<td>0,30</td>
<td>10.000,00</td>
</tr>
<tr>
<td>6c</td>
<td>Prestressed geogrid reinforced soil</td>
<td>2,00</td>
<td>0,30</td>
<td>270.000,00</td>
</tr>
<tr>
<td>6d</td>
<td>Prestressed geogrid reinforced soil</td>
<td>2,00</td>
<td>0,30</td>
<td>220.000,00</td>
</tr>
<tr>
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<td>0,30</td>
<td>200.000,00</td>
</tr>
<tr>
<td>6f</td>
<td>Prestressed geogrid reinforced soil</td>
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<td>0,30</td>
<td>180.000,00</td>
</tr>
<tr>
<td>6g</td>
<td>Prestressed geogrid reinforced soil</td>
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<td>0,30</td>
<td>160.000,00</td>
</tr>
<tr>
<td>6h</td>
<td>Prestressed geogrid reinforced soil</td>
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<td>0,30</td>
<td>140.000,00</td>
</tr>
<tr>
<td>6i</td>
<td>Prestressed geogrid reinforced soil</td>
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<td>0,30</td>
<td>120.000,00</td>
</tr>
<tr>
<td>7a</td>
<td>Subgrade material of controlled composition and granulation</td>
<td>2,00</td>
<td>0,20</td>
<td>60.000,00</td>
</tr>
<tr>
<td>7b</td>
<td>Subgrade material of controlled composition and granulation</td>
<td>2,00</td>
<td>0,20</td>
<td>55.000,00</td>
</tr>
<tr>
<td>7c</td>
<td>Existing subgrade material</td>
<td>2,00</td>
<td>0,20</td>
<td>45.000,00</td>
</tr>
</tbody>
</table>

In the static analysis, the rail was loaded with a force equal to one half of the weight of the test wagon (F = 94.4 kN), as shown in Figure 93. The force was introduced uniformly across the entire width of the rail, i.e. as a line load. The obtained results are compared to the measured data. After several iterations, relatively good matching can be observed, Figure 94. Figure 95 shows the vertical displacements analyzed by depth (rail level, sleeper level, tampon level, ballast level and the lower layers of soil) and different locations of the forces.

wagon type Kgs -z
The deviation from linear change of the measured results of the vertical displacement in some locations, as shown as location A, B and C in Figure 95, was caused mainly due to the fact that applied construction methodology was not considered within the design phase (construction in three phases by using temporary bridges) and lack of adequate time required for proper construction (compaction of the embankment layers) due to the short closer of the railway line, Figure 96. This caused that main elements of the embankment structure within the transition zones did not possess unique and homogeneous characteristics as predicted by the design.
Figure 94 Graphical representation of measured and calculated results of the vertical displacements

Figure 95 Vertical displacements at different height levels (static analysis)
Figure 96 The main reasons why there has been a point deviation from liner changes in the locations A, B and C
If we neglect the above mentioned defects which were caused due to wrong construction technology, and by using the characteristics of the material as estimated by the design (Figure 97, Table 9, Figure 98, Table 10) we got the following values of vertical displacements as shown in Figure 99.

**Table 9** Characteristics of the material for each indexed area of the railway track on Zagreb side used for modelling

<table>
<thead>
<tr>
<th>No.</th>
<th>Type of the installed material</th>
<th>Density</th>
<th>Poisson’s ratio</th>
<th>Young’s Modulus of elasticity</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Existing subgrade material</td>
<td>2,00</td>
<td>0,30</td>
<td>50.000,00</td>
</tr>
<tr>
<td>2a</td>
<td>Subgrade material of controlled composition and granulation</td>
<td>2,00</td>
<td>0,30</td>
<td>280.000,00</td>
</tr>
<tr>
<td>3a</td>
<td>Cement stabilized base layer</td>
<td>2,70</td>
<td>0,15</td>
<td>8.000.000,00</td>
</tr>
<tr>
<td>3b</td>
<td>Embankment stabilized by hydraulic binder</td>
<td>2,10</td>
<td>0,30</td>
<td>5.000.000,00</td>
</tr>
<tr>
<td>4</td>
<td>Sub-ballastbed</td>
<td>1,90</td>
<td>0,20</td>
<td>340.000,00</td>
</tr>
<tr>
<td>5</td>
<td>Ballast bed</td>
<td>1,90</td>
<td>0,20</td>
<td>340.000,00</td>
</tr>
</tbody>
</table>

**Figure 97** Indexing of the areas of the railway track with the same homogeneous material characteristics on Zagreb side used for modelling

**Figure 98** Indexing of the areas of the railway track with the same homogeneous material characteristics on Sisak side used for modelling
Table 10 Characteristics of the material for each indexed area of the railway track on Sisak side used for modelling

<table>
<thead>
<tr>
<th>No.</th>
<th>Type of the installed material</th>
<th>Density</th>
<th>Poisson’s ratio</th>
<th>Young’s Modulus of elasticity</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Ballast bed</td>
<td>1,90</td>
<td>0,20</td>
<td>340.000,00</td>
</tr>
<tr>
<td>6a</td>
<td>Prestressed geogrid reinforced soil</td>
<td>2,00</td>
<td>0,30</td>
<td>270.000,00</td>
</tr>
<tr>
<td>6d</td>
<td>Prestressed geogrid reinforced soil</td>
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<td>0,30</td>
<td>220.000,00</td>
</tr>
<tr>
<td>6e</td>
<td>Prestressed geogrid reinforced soil</td>
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<td>0,30</td>
<td>200.000,00</td>
</tr>
<tr>
<td>6f</td>
<td>Prestressed geogrid reinforced soil</td>
<td>2,00</td>
<td>0,30</td>
<td>180.000,00</td>
</tr>
<tr>
<td>6g</td>
<td>Prestressed geogrid reinforced soil</td>
<td>2,00</td>
<td>0,30</td>
<td>160.000,00</td>
</tr>
<tr>
<td>6h</td>
<td>Prestressed geogrid reinforced soil</td>
<td>2,00</td>
<td>0,30</td>
<td>140.000,00</td>
</tr>
<tr>
<td>6i</td>
<td>Prestressed geogrid reinforced soil</td>
<td>2,00</td>
<td>0,30</td>
<td>120.000,00</td>
</tr>
<tr>
<td>7a</td>
<td>Subgrade material of controlled composition and granulation</td>
<td>2,00</td>
<td>0,20</td>
<td>60.000,00</td>
</tr>
<tr>
<td>7b</td>
<td>Subgrade material of controlled composition and granulation</td>
<td>2,00</td>
<td>0,20</td>
<td>55.000,00</td>
</tr>
<tr>
<td>7c</td>
<td>Existing subgrade material</td>
<td>2,00</td>
<td>0,20</td>
<td>45.000,00</td>
</tr>
</tbody>
</table>

Figure 99 Results of the vertical displacement analysis by depth

After reviewing of the results calculated by using static analysis it can be concluded that certain linear change of the vertical displacement variable in the longitudinal direction within the transition zones has been achieved.
4.3.4 Dynamic analysis

Based on the calibrated model, a dynamic analysis was performed to verify the accuracy of the model in dynamic loading. For this purpose, implicit dynamic procedure was applied assuming different speeds of the train (v = 40 – 160 km/h) (Vajdić, 2017). During the monitoring phase, when the track was in operation, the following inputs were collected (Figure 100):

- stresses on the rail foot (where the Strain Gauges were installed);
- vertical displacement of the rail foot and the sleeper (with Linear Variable Differential Transformer).

![Figure 100 Measured strains and vertical displacements with Linear Variable Differential Transformer (LVDT) and Strain Gauges](image)

The measurements were taken each time the electrical train series 6111 passed by (Figure 101).
Strain on the rail foot was measured at 6 locations, three (3) locations on each side of the bridge, Figure 102. Vertical displacement of the rail foot and the sleeper was measured on two (2) locations, one on each side of the bridge, Figure 103.

**Figure 101** Electric train series 6111

-15.3m  -5.7m  -0.3m  12.3m  17.1m  27.3m
1  2  3  4  5  6

**Figure 102** Locations where strain was measured

-2.9m  18.3m
1  2

**Figure 103** Locations where vertical displacements were measured
The technical characteristics of the train can be found in the Figure 104:

![Figure 104 Technical characteristics of the EMU 6111 (Švaljek et al., 2003)](image)

Electric three-unit railcar series 6111 consist of two driver’s cars and the middle motor car. The train was in this case represented by a number of rigid wheels as shown in Figure 105. For the dynamic analysis two bogies were used as a critical load, one on the driver car and one on the motor car. Each bogie has two (2) axels. The weight of the carriage was equal to 83.38 kN per wheel. Frictionless, hard contact was assumed between the wheels and the rail.

Prior to the dynamic analysis the wheels were pressed against the rails outside of the “soil” model (to ensure the movement of wheels across the entire transit zone, the rails were extended as shown in Figure 105. The linear increase of the movement versus time was then prescribed to ensure the constant velocity of the carriage. The speed used was the one measured on the track, $v=111.6$ km/h (or 31 m/s).

In order to achieve better overlapping of the in situ measured results and the one calculated, additional modification of the characteristics of the material for each indexed area of the railway track were performed.
In the following figures from Figure 106 to Figure 109, the comparison between measured and calculated vertical displacement of the rail foot and sleeper can be seen:
**Figure 106** Vertical displacement of the rail foot on Zagreb side

**Figure 107** Vertical displacement of the sleeper on Zagreb side
Figure 108 Vertical displacement of the rail foot on Sisak side

Figure 109 Vertical displacement of the sleeper on Sisak side

In the following figures Figure 110 and Figure 111, the comparison between measured and calculated stresses in rail foot can be seen:
Figure 110  Measured and calculated stresses in rail foot on 3 locations on Zagreb site
Material hysteretic damping was considered in all tampon, ballast and soil layers. The value of Material hysteretic damping for each material used for dynamic analysis can be seen in the Table 11 and Table 12.
**Table 11** Characteristics of the material for each indexed area of the railway track on Zagreb side used for dynamic analysis

<table>
<thead>
<tr>
<th>No.</th>
<th>Type of the installed material</th>
<th>Layer thickness</th>
<th>Density</th>
<th>Poisson’s ratio</th>
<th>Young’s Modulus of elasticity</th>
<th>T</th>
<th>f</th>
<th>ksi</th>
<th>alpha</th>
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</thead>
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<td>0</td>
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<td>0,30</td>
<td>30.000.000,00</td>
<td>0,00</td>
<td>3.789,40</td>
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<td>2.380,95</td>
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<td>112,44</td>
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<tr>
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<td>Subgrade material of controlled composition and granulation</td>
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<td>100.000,00</td>
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<td>50,33</td>
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<td>Ballast bed</td>
<td>0,30</td>
<td>1,90</td>
<td>0,20</td>
<td>340.000,00</td>
<td>0,00</td>
<td>317,38</td>
<td>0,40</td>
<td>1.595,31</td>
</tr>
</tbody>
</table>

**Table 12** Characteristics of the material for each indexed area of the railway track on Sisak side used for dynamic analysis

<table>
<thead>
<tr>
<th>No.</th>
<th>Type of the installed material</th>
<th>Layer thickness</th>
<th>Density</th>
<th>Poisson’s ratio</th>
<th>Young’s Modulus of elasticity</th>
<th>T</th>
<th>f</th>
<th>ksi</th>
<th>alpha</th>
</tr>
</thead>
<tbody>
<tr>
<td>5</td>
<td>Ballasted</td>
<td>0,30</td>
<td>1,90</td>
<td>0,20</td>
<td>340.000,00</td>
<td>0,00</td>
<td>317,38</td>
<td>0,40</td>
<td>1.595,31</td>
</tr>
<tr>
<td>6c</td>
<td>Prestressed geogrid reinforced soil</td>
<td>1,00</td>
<td>2,00</td>
<td>0,30</td>
<td>270.000,00</td>
<td>0,01</td>
<td>82,78</td>
<td>0,30</td>
<td>311,77</td>
</tr>
<tr>
<td>6d</td>
<td>Prestressed geogrid reinforced soil</td>
<td>1,00</td>
<td>2,00</td>
<td>0,30</td>
<td>220.000,00</td>
<td>0,01</td>
<td>74,65</td>
<td>0,30</td>
<td>281,42</td>
</tr>
</tbody>
</table>
**D4.1 Guidelines on the Use of Novel Construction and Maintenance Techniques within the Operational Railway Environment**

**DESTination RAIL – Decision Support Tool for Rail Infrastructure Managers**

<table>
<thead>
<tr>
<th>Table</th>
<th>Description</th>
<th>Density</th>
<th>Width</th>
<th>Height</th>
<th>Stiffness</th>
<th>Young's Modulus</th>
<th>Price</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>6e</td>
<td>Prestressed geogrid reinforced soil</td>
<td>1.00</td>
<td>2.00</td>
<td>0.30</td>
<td>200,000.00</td>
<td>71.18</td>
<td>0.3</td>
<td>268.33</td>
</tr>
<tr>
<td>6f</td>
<td>Prestressed geogrid reinforced soil</td>
<td>0.88</td>
<td>2.00</td>
<td>0.30</td>
<td>180.000.00</td>
<td>76.73</td>
<td>0.3</td>
<td>289.27</td>
</tr>
<tr>
<td>6g</td>
<td>Prestressed geogrid reinforced soil</td>
<td>0.81</td>
<td>2.00</td>
<td>0.30</td>
<td>160.000.00</td>
<td>78.60</td>
<td>0.3</td>
<td>296.30</td>
</tr>
<tr>
<td>6h</td>
<td>Prestressed geogrid reinforced soil</td>
<td>0.70</td>
<td>2.00</td>
<td>0.30</td>
<td>140.000.00</td>
<td>85.07</td>
<td>0.3</td>
<td>320.71</td>
</tr>
<tr>
<td>6i</td>
<td>Prestressed geogrid reinforced soil</td>
<td>0.46</td>
<td>2.00</td>
<td>0.30</td>
<td>120.000.00</td>
<td>119.85</td>
<td>0.4</td>
<td>602.45</td>
</tr>
<tr>
<td>7a</td>
<td>Subgrade material of controlled composition and granulation</td>
<td>2.31</td>
<td>2.00</td>
<td>0.20</td>
<td>60,000.00</td>
<td>16.88</td>
<td>0.4</td>
<td>84.83</td>
</tr>
<tr>
<td>7b</td>
<td>Subgrade material of controlled composition and granulation</td>
<td>2.31</td>
<td>2.00</td>
<td>0.20</td>
<td>55,000.00</td>
<td>16.16</td>
<td>0.4</td>
<td>81.22</td>
</tr>
<tr>
<td>7c</td>
<td>Existing subgrade material</td>
<td>2.31</td>
<td>2.00</td>
<td>0.20</td>
<td>45,000.00</td>
<td>14.62</td>
<td>0.4</td>
<td>73.47</td>
</tr>
</tbody>
</table>

### 4.3.5 Conclusion

An extensive research on the Buna bridge reconstruction started already within the EU project SMART RAIL (Pires et al., 2014). Data received from that work were used in Destination Rail project case study for further FE numerical analysis. Several problems observed during the construction process give case study realistic framework within real environment.

Transition zones of Buna bridge were modelled initially in 2D with static loading conditions and extended further to 3D in an attempt to capture the dynamic responses as well. From the results obtained one can observe satisfactory matching of the numerical model with the measured values from analysed location. It proves capability of FE modelling to simulate real field conditions. Thus parametric analysis of various characteristics of the transition zone is enabled on such a calibrated model. Results of parametric analysis will enable optimization of transition zone areas, allowing the design of substructure and superstructure without interruptions. Thus, smooth deformability with no sudden soft bumps on the railway track within the transition on the bridge is expected.
5 Conclusions

This deliverable aims to support railway asset managers by outlining novel construction and maintenance techniques for railway infrastructure. Existing techniques for rail remediation works have been outlined and a review of their main characteristics has been presented. The use of expanding polyurethane resin is presented in detail since this technique minimises rail traffic disruption. This technique has been demonstrated for two operational rail locations in Slovenia. The two sections comprise a bridge transition zone and a railway embankment. The injection has been performed in the ballast layer and the subgrade in the case of transition zone. For the embankment, the injection was performed in the subgrade layer. This technique provides a convenient solution for the case of remediation of differential settlements and bearing capacity improvements for railway tracks that does not disrupt rail traffic.

Measurements were obtained for both case study locations to compare the track quality before and after the injection. The results demonstrated that the dynamic track performance along the bridge transition zone did not alter during the rehabilitation works. However, the technique resulted in an insignificant change in terms of the track geometry. For the rail embankment, the injection of high pressure expansion polyurethane resin decreased rail deflections and increased the track stiffness.

Novel construction techniques for railways have also been described in this report. Specifically, the use of secondary and locally sourced materials in railway construction works has been described. Additionally, a review of construction techniques for tracks, earthworks and transition zones has been conducted. The study specifically analysed the use of several new materials for earthworks and for the construction of tracks. Particular attention is paid to light weight materials, including light weight expanded clay aggregate and cellular glass aggregate, which have been characterised according to laboratory tests.

This report has also presented an analysis of the first GRS integrated bridge with FHR facings in Europe, which has been constructed across the Pavlovski potok stream in Slovenia at the end of 2014. Several significant advantages of the GRS bridge abutments compared to conventional steel-reinforced concrete cantilevered abutments have been demonstrated based on the analysis, including minimal environmental impacts and cost savings.

Finally, advanced FE modelling was conducted for the two case studies: 1) the transition zone for a railway bridge in Croatia, where pre-stressed reinforced soil was implemented, and 2) the use of light weight aggregates for the construction of railway embankments on poor ground in Norway. Overall, this report provides guidance for rail infrastructure designers and managers in relation to novel rail maintenance and construction techniques.
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