

Case study on the rehabilitation of old railway lines: experimental field works

E. Fortunato; S. Fontul & A. Paixão

Laboratório Nacional de Engenharia Civil, Lisbon, Portugal

F. Asseiceiro

Mota-Engil, Lisbon, Portugal

ABSTRACT: In many countries, old railways have to be renewed in order to fulfil modern quality requirements, related either to higher travel speeds or to greater axle loads. When renewal works are required it is usual to replace the superstructure, laying new support layers and, in some cases, improving the subgrade soils. Preferably, such renewal works should be carried out in an economical and environmental sustainable process, taking into account the existing materials and the *in situ* conditions of the structure. It is necessary to carry out evaluation studies of the physical and mechanical properties of the existing elements as well as of the materials to be used in the renewal process. Different structural solutions can be established for rehabilitation purposes, regarding the existing track conditions. Among other aspects, the design of these structural solutions depends on the hydrogeological conditions that occur along the line, on the characteristics of the foundation soils and on the existing ballast layer. In old lines it is often observed a layer of fouled ballast on the top of the foundation soil. During the renewal there are some technical, economic and environmental advantages in maintaining the fouled ballast layer under the new reinforcement layers. These advantages are related to the high stiffness of that layer and to the reduction in supply, deposit and transportation of materials. This paper presents some results of experimental work carried out on a deactivated rail stretch, used as an experimental site. This study was performed in order to assess the feasibility of some structural solutions, using reinforcement layers built with unbound granular materials (UGM) and cement bound granular mixtures (CBGM).

KEY WORDS: railway track rehabilitation, *in situ* tests, fouled ballast.

1 INTRODUCTION

The modernization of under operation railway lines usually requires renewal of the track superstructure as well as improving its substructure. This process consists in modifying the technical characteristics of the components, so that a superior quality of the track can be provided, compared to the one existing initially. It may include the replacement of the superstructure, the construction of reinforcement layers or even replacement of foundation soils.

The main differences between a new construction project and a renewal project of a railway infrastructure are related to (Fortunato et al., 2001): i) the concern to maintain, as much as possible, the existing structure; ii) the need to reduce disturbances on the railway line

service, during the renewal works; iii) the physical constraints imposed by existing structures; iv) the failure to adopt some of the construction techniques commonly used when it comes to building a new track, for example improving the subgrade soils; v) longitudinal and transversal inhomogeneity of the materials present in the substructure, particularly when no appropriate maintenance and rehabilitation works were undertaken; vi) the maintenance of high safety standards during the renewal process.

The renewal process, besides considering the referred aspects, should be based on characterisation studies of the physical and mechanical properties of existing elements. In fact, this is absolutely necessary in order to provide efficient solutions both in structural terms (regarding the railway track layers and the superstructure) and, in terms of complementary works of different areas such as drainage structures.

The evaluation of physical and mechanical characteristics of the structure and its components can be made using different methods, depending particularly on the access conditions to the railroad. In what concerns the subgrade, it is common to characterize the geo-materials, determining intrinsic parameters and condition states (water content and degree of compaction).

In old railway tracks, particularly those in which there is no sub-ballast layer, it is often observed a layer of fouled ballast on the top of the subgrade. This contamination with fine particles is due to different causes, mainly resulting from “pumping” of fine material of the foundation due to cyclic loading resulting from passing trains and from the accumulation of water on the substructure. In addition, when the ballast has low resistance to abrasion and fragmentation, contamination may also result from the wear of the larger fraction.

There are some technical, economic and environmental advantages in maintaining the fouled ballast layer in the support track under the new reinforcement layers (Fortunato et al., 2010). These advantages are related to the high stiffness of that layer and with the reduction in supply, deposit and transportation of materials. Furthermore, maintaining that layer simplifies the construction process, with consequent reductions in costs, deadlines and interference in the operation of the line. In this perspective, it seems appropriate to assess the mechanical properties of materials through laboratory and *in situ* performance-based tests (Gomes Correia et al., 2012).

This paper presents some results of experimental works carried out on a deactivated rail stretch in Portugal, used as an experimental site. This study was performed in order to assess the feasibility of some structural solutions, using reinforcement layers built with unbound granular materials (UGM) and cement bound granular mixtures (CBGM).

2 STUDIES PERFORMED ON A OLD RAILWAY LINE

2.1 Introduction

The experimental studies presented in this work aimed to construct and evaluate various solutions for structural reinforcement of old railway tracks.

The selected ballasted track section to develop the studies is about 36 m long, with Iberian gauge (1.668 m) and with bi-block sleepers laid on a layer of limestone ballast. The soil of the foundation is sandy clay (Fig. 1) with plasticity index of 10. Despite being a granular soil, because of their intrinsic characteristics, namely a relatively high percentage of fine particles ($\%_{<#200}=23.2\%$), this soil may change substantially its consistency as a result of slight variations in water content, particularly when its water content is close to the optimum water content. The CBR value of the material is 9%.

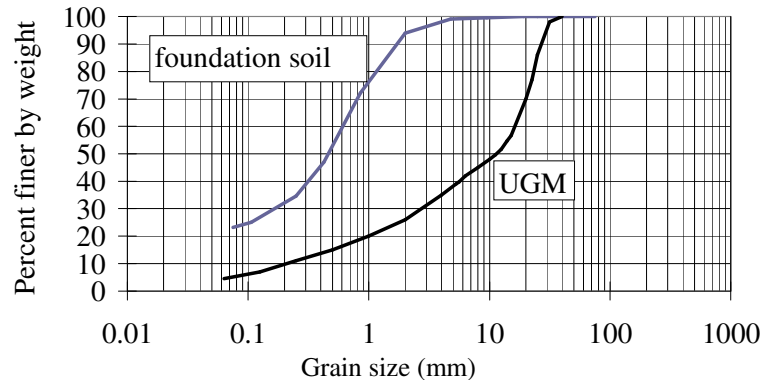


Figure 1: Grain size distribution of foundation soil and of UGM for reinforcement layer.

In terms of structural behaviour, it was established that the minimum deformation modulus to be obtained at the top of the renewed substructure was 120MPa (UIC, 2008).

Before any modification in the characteristics of the stretch under study, namely the removal of the existing track, Ground Penetrating Radar (GPR) tests were performed, with different setups. Plate Load Test (PLT) and Falling Weight Deflectometer (FWD) tests were also performed on the top of the ballast layer. These tests and the construction of the experimental section were made during dry season.

2.2 Ground Penetrating Radar tests before removing the old track

In this study the evaluation of thickness of ballast was made by the GPR method. The GPR transmits short duration electromagnetic pulses from the transmitter antenna into the materials being tested and picks up the reflected energy in the receiver antenna. The reflected wave gives information about the substructure. The wave amplitude is related to the difference in dielectric properties of the two adjacent layers, while the travel time gives the interface location beneath the surface. The GPR measures the travel time, which is post-processed and converted to layer thickness. The thickness can be estimated if the relative dielectric constant is known.

Two pairs of air-coupled (horn) antennas (1 GHz and 2 GHz) were used, with a maximum penetration depth of 0.80 m and 0.40 m, respectively (Fig. 2a). The calibration of the device was made in laboratory by determining the dielectric constants of the materials (Fontul et al., 2011) and by performing *in situ* test pits.

The results obtained with the GPR (Fig. 2b) allowed the identification of the interface between the ballast layer and the subgrade. The ballast layer ranged from 0.25 m to 0.30 m thick. The test pits showed a fouled ballast bottom layer.

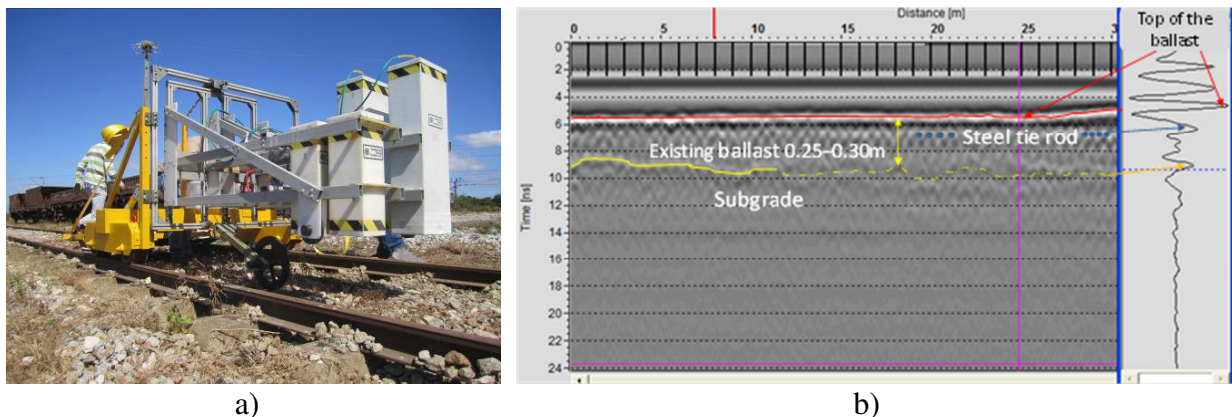


Figure 2: Tests with GPR before removing the old track: a) test equipment; b) results.

2.3 Load tests before removing the old track

Before removing the superstructure of the track, PLT tests were performed on the ballast layer between sleepers, according to the procedure recommended by the standard NF P 94-117-1, yet using a 0.45m diameter plate (Fig. 3a). FWD tests (Fig. 3b) were also performed in order to try to relate the values obtained by this method with those obtained with PLT.

To perform FWD tests on the ballast layer it was necessary to adapt the equipment to circulate on rails. Thus, the axle and the wheels of the equipment were changed. Moreover, a system to apply the load and measure the deflection was designed and constructed so that the geophone located at the centre of the load plate had an adequate support.

In the analysis of the FWD and PLT tests the deformation modulus was calculated using the following expression:

$$E = 0.75 d \frac{p}{\delta} \quad (1)$$

where E is the deformation modulus, d is the load plate diameter, p is the applied vertical pressure and δ is the deflection under the plate.

Two different load tests were performed: the PLT, applying a quasi-static pressure of 200 kPa on a 0.45 m load plate, and the FWD, applying a dynamic pressure of 500 kPa on a 0.40 m plate. Despite these differences, the deformation modulus values on the ballast layer at each test site were similar using both methods (Fig. 3c). In general, the modulus values ranged along the stretch between about 60 MPa and 80 MPa.

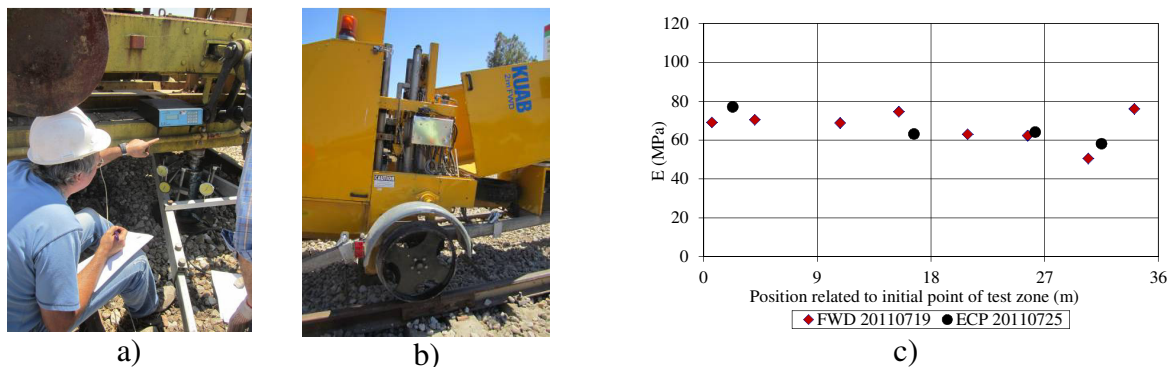


Figure 3: Load tests before removing the track: a) PLT; b) FWD; c) deformation modulus (E).

3 CONSTRUCTION OF EXPERIMENTAL SECTIONS

3.1 General aspects

After the previously described tests, rails and sleepers were removed and the existing ballast was levelled and compacted. The test zone was divided in four sections with 9 m long each, in order to build different structural solutions on top of old ballast, creating a new platform, as presented in Fig. 4. In Section 4 only the ballast was levelled and no other layer was built on.

In Section 1, a 0.10 m well graded crushed unbound granular material (UGM) layer was spread over the existing old ballast; this UGM was mixed with cement (5%) and then the layer was compacted (CBGM); in Section 2 a 0.10 m UGM layer was spread on the existing ballast and mixed with it at the surface (UGM+B); after the compaction of this layer, a 0.20 m thick UGM layer was spread and compacted; in Section 3 a 0.10 m UGM layer was spread on the existing ballast and mixed with it at the surface (UGM+B); after the mixing process the layer was compacted; in Section 4 only the compaction of the existing ballast was done.

Section 1 (9 m long)

- 10 cm of CBGM (5% of cement)
- 25 cm of Fouled Ballast (FB)
- Subgrade (SG)



d) Section 1

Section 2 (9 m long)

- 20 cm of UGM
- 10 cm of UGM mixed with ballast (UGM+B)
- 25 cm of fouled ballast (FB)
- Subgrade (SG)



b) Section 2

Section 3 (9 m long)

- 10 cm of UGM mixed with ballast (UGM+B)
- 25 cm of fouled ballast (FB)
- Subgrade (SG)



c) Section 3

Section 4 (9 m long)

- 25 cm of fouled ballast (FB)
- Subgrade (SG)



d) Section 4

Figure 4: Aspects of the experimental sections and pits after construction.

The granite UGM of the reinforcement layer, which grain size distribution is shown in Fig. 1, has high resistance to fragmentation (Los Angeles value <26%) and high resistance to abrasion (Micro-Deval value <10%). This material was characterized through laboratorial cyclic load triaxial tests, applying several stress paths on large samples with different state conditions (Fortunato et al., 2012). The resilient modulus (E) of this material depends on the state conditions, particularly the water content (w -wopm) and the degree of compaction (D_c). Moreover, the modulus increases with the increase of mean stress. This behaviour was more evident in specimens with higher E , considering the same stress level and different state conditions (Fig. 5a). During construction of the experimental sections, the material was moistened to increase its water content to the optimum water content value (about 5.0%).

On this track, the old ballast was essentially a limestone material. At the bottom of the layer, the ballast was fouled with fine particles from degradation of the ballast. This type of materials has been studied in recent years during railway renewal projects (Fortunato et al., 2010; Trinh et al., 2012). In particular, it was characterized by cyclic load triaxial tests performed on large specimens, using the full grain-size distribution. The resilient modulus depends on several parameters (Fig. 5b), including the fouling index, the degree of compaction and the stress level. The fouling index can be calculated by:

$$F_I = P_4 + P_{200} \quad (2)$$

in which P_4 and P_{200} are the percentage of material passing sieves 4.75 mm and 0.075 mm, respectively (Selig and Waters 1994).

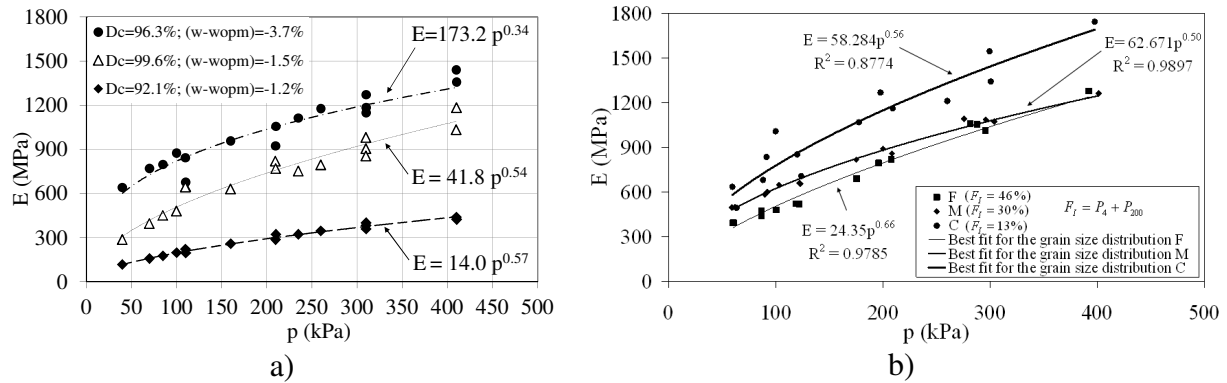


Figure 5: Resilient modulus obtained in cyclic triaxial load tests: a) UGM (Fortunato et al., 2012); b) Fouled Ballast (Fortunato et al., 2010).

3.2 GPR tests performed on new experimental sections

After the construction of reinforcement layers, GPR tests were performed on the top of the experimental sections, for layer thickness measurement purpose. Figure 6 presents some of the results obtained in each section after GPR interpretation. It can be observed that the interfaces between the reinforcement layers and the existing ballast are generally visible. Also, it is possible to detect in Section 2, interfaces between the UGM layers given the fact they were placed and compacted in three different layers of 0.10m thick each. The results obtained with GPR confirmed the design values for thicknesses. These results were used during the back-calculation of FWD load tests, as presented in part 4 of this paper.

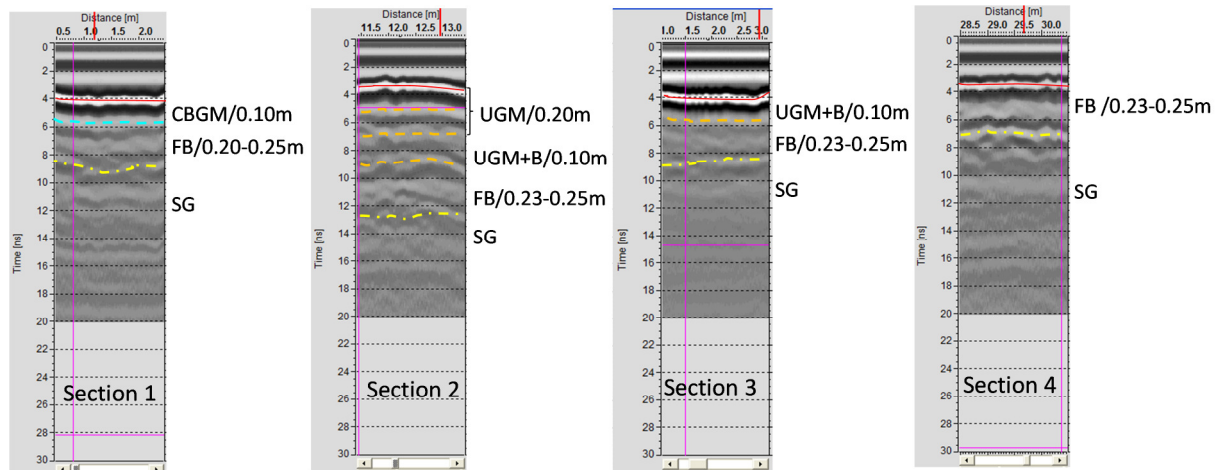


Figure 6: Tests with GPR on experimental sections.

3.3 Load tests after construction of the experimental sections

To measure the deformation modulus on top of the foundation, after laying and compaction of reinforcement layers, test pits were done, and loading tests were performed within, using a Light Falling Weight Deflectometer (LFD), as shown in Fig. 7a. This equipment was used as it was not technically possible to use the FWD, due the depth of the surface to be tested. The deformation modulus values obtained on the top of the foundation using a 0.30 m diameter plate, applying a vertical pressure of 200 kPa, were about 70-80 MPa. It should be noted that these results correspond to tests performed during dry season. Given the geotechnical characteristics of the foundation soils, deformation values may decrease with an increase in water content.

After construction, FWD tests were performed on the reinforcement layers (Fig. 7b), applying a vertical pressure of 400 kPa on a 0.30 m diameter plate. Deflections were measured using a set of seven geophones (the first was located at the centre of the load plate and the others at 0.30 m, 0.45 m, 0.60 m, 0.90 m, 1.20 m and 1.80 m). In Section 4, another test was performed on the top of the ballast using only the geophone located at the centre of the load plate. On the top of these layers more FWD tests were carried out two months after construction (September 2011), aiming at assessing whether the deformation modulus values changed over time, particularly due to the variation of the materials water content and change in stiffness of the cement bound aggregate mixture (in Section 1).



Figure 7: Load test equipment: a) LFD on the subgrade; b) FWD on the top of the reinforcement layer.

The measured deflections during the two series of tests are presented in Fig. 8a. It is possible to conclude that there are some differences, being higher the deflections measured in September, particularly under the load plate and at some geophones in Section 2.

The deformation modulus was calculated according to equation (1) considering only the deflection measured at the centre of the load plate. The results obtained (Fig. 8b) lead to the conclusion that: a) in Section 1 and Section 2 deformation modulus values were about 200 MPa. The values of the load tests obtained in September 2011 (20110928) were 15% to 20% lower compared to those obtained in August (20110804); b) in Section 3, the deformation modulus values measured in the August were about 130 MPa, and the measured values in September were about 15% lower; c) in Section 4, where no new layers were placed, the deformation modulus values measured in August on the top of the old ballast layer were about 80 MPa, and decreased about 30% in September measurements; it should be noted that, as presented earlier, before removing the track and before compacting the old ballast layer at Section 4, the values of deformation modulus ranged from 50 MPa to 75 MPa.

The lower values of deformation modulus obtained in September may be due to higher water content of materials (which was not possible to measure), either in the aggregates or in the foundation soils, since rainfall occurred between the two series of tests. Taking into account the results, it is possible to conclude that the structural solutions built in Sections 1 and 2 led to the deformation modulus values higher than 120 MPa, as was established. However, if the tests had been performed after the rainy season, with higher water content values of materials, the deformation modulus values could have been even lower. Although the deformation moduli values of Sections 1 and 2 are similar, Section 1 has the advantage of having a thickness of 0.10 m, comparing with the 0.30 m of Section 2 solution; this can be significant in the situations where there are geometrical constraints. The solution of Section 3 hardly achieved the minimum value established for the deformation modulus (particularly the values obtained in September tests), having however the advantage of being cheaper, easier and quicker to build.

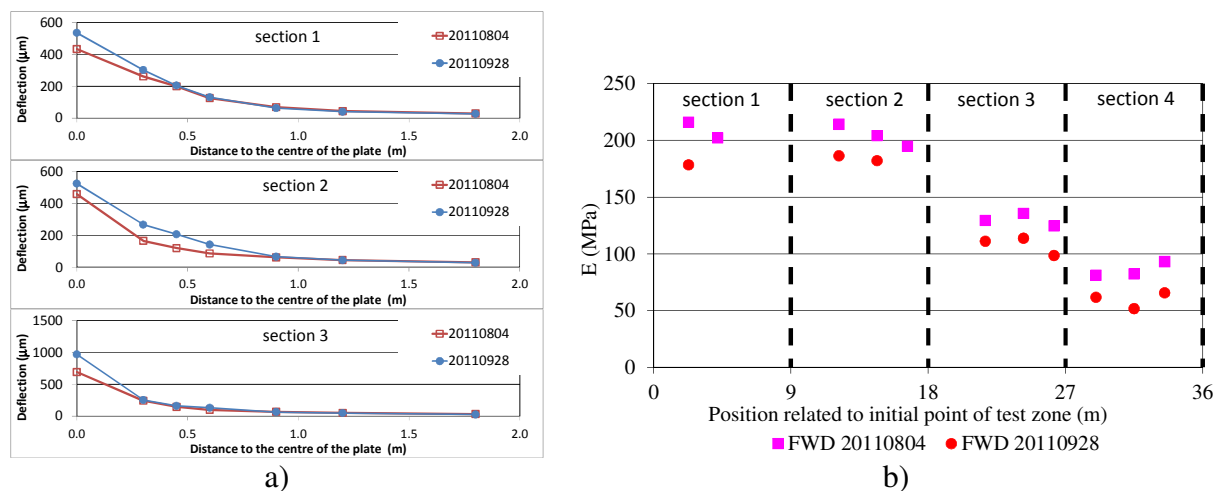


Figure 8: Results of the tests on top of the structures: a) deflections; b) resilient modulus.

4 BACK-CALCULATION OF ELASTIC MODULUS OF LAYERS

The deflections obtained from all geophones during FWD tests were used to estimate the elastic moduli of the layers at each section, through back-analysis (Fontul et al., 2011). In this procedure, the measured deflections are compared with the calculated deflections, obtained

through structural numerical modelling using multi-layer linear elastic theory. Therefore, neither the nonlinear behaviour nor the dynamic response of the materials was considered.

The back-calculation study was developed for the representative point of each section (near the middle point of the stretch), using the BISAR software. In general, it was possible to achieve a good fit to the measured values (Fig. 9).

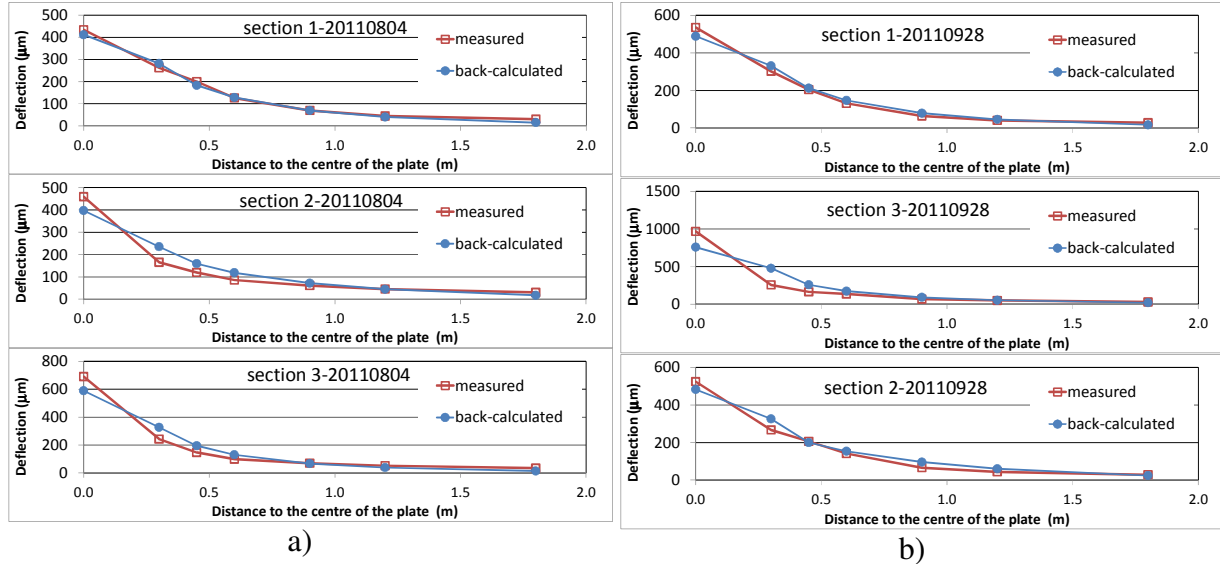


Figure 9: Measured and back-calculated deflections: a) first tests, August; b) second tests, September.

The calculated values for the elastic modulus, considering the deflections measured in the tests performed in the two series, in Sections 1, 2 and 3, are shown in Table 1. In Section 4, as already mentioned, only the deflection under the load plate was measured, due to the characteristics of the ballast layer and the device used.

Table 1: Deformation modulus obtained by back-analysis.

	FWD-20110804 E (MPa)	FWD-20110928 E (MPa)
Cement bound granular material (CBGM)	600	500
Unbound granular material (UGM)	150	120
UGM mixed with ballast (UGM+B)	180	160
Fouled Ballast (FB)	180	160
Subgrade (SG)	80	60

The values obtained by back-analysis seem to be adequate, namely the ones regarding UGM and fouled ballast, taking into account the expected stress distribution in depth in each of these layers (Fortunato and Resende, 2006) and the resilient modulus values measured in laboratory for UGM and fouled ballast. Regarding the subgrade soil, the calculated modulus was similar to the one obtained with LFWD load tests. The back-calculated modulus of the cement bound granular material seems low, taking into account values reported in the literature (Lim and Zollinger, 2003) for CBGM with similar binder content. It is likely that this result is due to the difficulty of adequately mix the materials on site.

When comparing the two series of tests, it should be noted the decrease in the modulus values after rainfall. The higher decrease (in terms of percentage) occurs in the subgrade soils, as it is expected. However, there is also an important decrease of the UGM resilient modulus. This behaviour was also observed in laboratory tests (Fortunato et al., 2012).

5 CONCLUSIONS

On the scope of track substructure renewal, the fouled ballast layer can play an important structural role, if it remains integrated in the new substructure, under the reinforcement layers. In this way, different solutions can be adopted along the track, depending on the design requirements, track condition and layout restrictions, in order to provide economic solution. To that end, it is necessary to perform a proper characterization of the structure. The non-destructive *in situ* tests represent useful tools for the evaluation of the existing railway track and allow for the characterisation of the reinforcement condition and its evolution in time.

The characteristics of the materials obtained with *in situ* testing and with structural modelling indicate that laboratory cyclic load triaxial tests can provide suitable resilient modulus values. To that end the parameters that influence the behaviour of materials must be taken into account, namely the water content, the degree of compaction and the stress levels.

From the results of this study it can be concluded that it is possible to reinforce the rail track substructure with a relatively thin layer of aggregate mixed with cement. However, it should be stressed that only the assessment of resilient behaviour of the materials was done with these studies. It is necessary to estimate the long term performance of each of the solutions tested. Performance evaluation should consider the permanent deformation. In the case of CBGM it is necessary to evaluate the possibility of cracking and degradation of the layer and the increase of the permeability, which is inadequate to the foundation behaviour.

REFERENCES

- Fontul, S.; Fortunato, E. and De Chiara, F., 2011. *Non-destructive tests for railway infrastructure stiffness evaluation*. The Thirteenth International Conference on Civil, Structural and Environmental Engineering Computing, Crete, Greece.
- Fortunato, E.; Pinelo, A. and Mateus da Silva, J., 2001. *The strengthening and upgrade of railway infrastructures: Geotechnical aspects*. Proc. of the XVth International Conference on Soil Mechanics and Geotechnical Engineering, Istanbul, Turkey.
- Fortunato, E. and Resende, R., 2006. *Mechanical behaviour of railway track structure and foundation – three dimensional numerical modelling*". Proc. of RailFound 06 - International Conference on Railway Track Foundations, Birmingham, UK.
- Fortunato, E.; Pinelo, A. and Matos Fernandes, M., 2010. *Characterization of the fouled ballast layer in the substructure of a 19th century railway track under renewal*. Soils and Foundations, Vol. 50(1).
- Fortunato E., Paixão, A. and Fontul, S., 2012. *Improving the use of unbound granular materials in railway sub-ballast layer*. Proc. of the Second International Conference on Transportation Geotechnics, Sapporo, Japan.
- Gomes Correia, A.; Roque, A. J.; Reis Ferreira, S. M. and Fortunato, E., 2012. *Case Study to Promote the Use of Industrial Byproducts: The Relevance of Performance Tests*. Journal of American Society for Testing and Materials (ASTM). Vol. 9(2).
- Lim, S. and Zollinger, D., 2003. *Estimation of the Compressive Strength and Modulus of Elasticity of Cement-Treated Aggregate Base Materials*. TRR: Journal of the Transportation Research Board; Vol. 1837/2003.
- Selig, E. T. and Waters, J. M., 1994. *Track Geotechnology and Substructure Management*, Thomas Telford Services Ltd., London.
- Trinh, V.N.; Tanga, A.M.; Cui, Y-J.; Dupla, J-C.; Canou, J.; Calon, N.; Lambertb, L., Robinet, A. and Schoen, O., 2012. *Mechanical characterisation of the fouled ballast in ancient railway track substructure by large-scale triaxial tests*. Soils and Foundations, Vol. 52(3).
- UIC, 2008. CODE 719R, Union Internationale des Chemins de Fer, Paris.