CHARACTERIZATION OF THE FOULED BALLAST LAYER IN THE SUBSTRUCTURE OF A 19TH CENTURY RAILWAY TRACK UNDER RENEWAL

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ABSTRACT

The purpose of these field and lab studies undertaken during rehabilitation work being done on an ancient railway line was to characterize a layer of ballast fouled with soil found in the track substructure. The field studies included the characterization of the thickness, grain size distribution and void ratio of the fouled ballast layer, as well as a large number of plate load tests, both on the fouled ballast layer and on the subgrade. The resilient behaviour of the fouled ballast was evaluated in the lab by cyclic triaxial tests on large size reconstituted specimens with distinct fouling indexes (different grain size distribution) and distinct humidity states (dry or wet). The results obtained were used as support for the decision to maintain the fouled ballast layer under the new sub-ballast in a number of stretches of the renewed line.

Key words: cyclic triaxial tests, fouled ballast, plate load tests, resilient modulus, track rehabilitation (IGC: C8/D6/H6)

INTRODUCTION

Many railway lines in different countries have been in operation for more than a hundred years. Their adaptation to modern trains requires a detailed characterization of the existing platform, by identifying the nature of the materials and their state and by evaluating their strength and stiffness.

A layer of ballast fouled with soil frequently occurs in the substructure of these lines, underlying the ballast (see Fig. 1(a)). Various mechanisms lead to the formation of this layer: grain size modification in ballast particles due to cracking, weathering and crushing; the infiltration of materials from the surface; the infiltration of materials from underlying layers; and, the weathering of the sleepers.

The interest in studying fouled ballast is related with the advantage of maintaining the layer under the new sub-ballast layer (see Fig. 1(b)), during the renewal process, thus reducing the cost, the construction period and the disturbance to line operation.

Nevertheless, the behaviour of fouled ballast is difficult to characterize, since it consists of a more or less dense mixture of materials—soil and ballast—in highly variable proportions and thickness. In addition, collecting undisturbed samples is not possible.

This paper presents field and lab studies for the renewal of an ancient Portuguese railway line which links the two main cities in the country—Lisbon (South) and Porto

(North) with a double track 336 km in length. The North-South track (N-S track) was built between 1856 and 1864, and the South-North track (S-N track) was settled between 1861 and 1930.

Fig. 1. Schematic representation of the rail track: (a) old railway track and (b) renewed railway track

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The field studies of the substructure included: i) the characterization of the thickness, grain size distribution and void ratio of the fouled ballast layer; ii) plate load tests, both on the fouled ballast layer and on the subgrade. In addition, a lab characterisation was developed, with cyclic triaxial equipment appropriate for testing large specimens (Fortunato, 2005). The density, grain size distribution and rock type of the fouled ballast specimens were established on the basis of the information from the field survey. Tests were performed on dry specimens and on wet specimens.

Those studies are part of a more comprehensive work the purpose of which was to establish the conditions that existing materials must fulfil in order to be maintained as part of the track substructure at the time of renewal.

IN SITU CHARACTERISATION OF THE RAILWAY SUBSTRUCTURE

Thickness, Fouling Index and Void Ratio of Fouled Ballast

The field survey consisted of 144 boreholes and 93 trenches carried out over a 125 km long stretch on both tracks.

At the surface two ballast materials were identified: granite and limestone. However, the rock material of the fouled ballast found under the ballast was limestone since this was the only type of rock used as ballast for many years. In some sites, significant differences in the quantity of fine particles in the fouled ballast were observed, depending on the location of the sampling points as to rail and to sleeper positions.

Figure 2 shows, as an example, graphs indicating the

depth (from the base of sleepers), corresponding to the bottom limit of both ballast and fouled ballast layers along the S-N track and the N-S track. Generally, the average thickness of the fouled ballast along this track is about 3 cm to 10 cm less than the one calculated for the S-N track. Since no significant differences were found between the type and condition of the foundation soils of both tracks, the observed difference is justified by the fact that the S-N track was typically used by trains transporting heavier products, such as cement, because of the concentration of plants in the south of the country.

The analysis of the 114 grain size curves of fouled ballast revealed the following: i) the maximum grain size usually ranges from 50 to 60 mm; ii) the percentage of grain size below 4.75 mm ranges from 3% to 30%; iii) the percentage of grain size below 0.075 mm ranges from 1% to 18% (see Fig. 3). The degree of the ballast fouling was evaluated by the fouling index, F_I , defined as (Selig and Waters, 1994):

$$F_I = P_4 + P_{200} \quad (1)$$

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

The void ratio of the fouled ballast (determined *in situ* by the membrane method developed by Yoo et al., 1978) ranged from 0.10 to 0.45 (see Fig. 4). In general, the lowest values occurred beneath the rails, where the influence of train loads is more significant. The results led us to conclude that, generally, the void ratio of fouled ballast decreases with the increase of the F_I .

Subgrade Modulus from Plate Load Tests

In accordance with the UIC CODE 719R (UIC, 1994),

Fig. 2. Depth of the bottom limit of both ballast and contaminated ballast layers: (a) along the S-N Track and (b) along N-S track

Fig. 3. Grain size curves of fouled ballast

Fig. 4. Void ratio of fouled ballast versus fouling index (F_I)

the renewed substructure must fulfil a number of requirements based on the deformation modulus in the second loading cycle of a plate load test, EV_2 (AFNOR, 2000). The sub-ballast layer is placed on a capping layer ($EV_2 \geq 80$ MPa) and at the top of the sub-ballast layer, EV_2 is required to be equal to or higher than 120 MPa.

After removing the ballast, 138 plate load tests (600 mm diameter plate) on the ancient substructure were carried out, 70 of which were performed on the fouled ballast layer, with a thickness ranging from 5 to 70 cm and an average value of 24 cm. The minimum value of EV_2 was higher than 40 MPa and, in 60% of the cases, values higher than 80 MPa were reached. When the tests were performed on the subgrade soil (where the fouled ballast layer was no longer present or had already been removed), the 80 MPa value was achieved only in 45% of the cases.

In about 75% of the cases, a rather homogeneous subgrade was found below the fouled ballast layer, i.e., only granular soils (less than 35% passing 0.075 mm sieve) or fine soils (35% or more passes a 0.075 mm sieve) were observed and the water content remained fairly constant in depth. An attempt was made to relate subgrade soil parameters with EV_2 . As was expected, it was only possible to detect fairly weak relations. In fact, bearing in mind that the plate was 600 mm in diameter and the thickness of the fouled ballast layer ranged very often from 5 to 70 cm, the effect on EV_2 is different in each case, depending on the fouled ballast and the subgrade soil.

Figure 5 illustrates the two relationships described above: one between the percentage of fines, and the other between the water content of the subgrade and EV_2 . The thin lines in the figures indicate the analogous relations

Fig. 5. Deformation modulus determined on the fouled ballast layer versus (a) P_{200} and (b) the water content in the subgrade soil

found for the substructure without fouled ballast between the ballast and the subgrade (the markers are not shown). The comparison of the curves reveals a favourable influence of the fouled ballast layer on EV_2 , with an increase of about 20 to 30 MPa observed, particularly when the subgrade consists of fine soils, or when the water content is higher than 10%. This behaviour is due to the modulus of fouled ballast being greater than that of the subgrade soils, as will be explained below.

RESILIENT BEHAVIOUR FROM CYCLIC LOAD TRIAXIAL TESTS

Equipment, Specimens and Tests

The proper evaluation of the stiffness of materials like the fouled ballast in the lab must respect their actual grain size distribution, since the way a load is propagated throughout a particulate medium depends not only on the structural arrangement of particles but also on their size (Kolisoja, 1997). As shown by Fig. 6, the specimens built in the lab were 0.30 m in diameter and 0.60 m in height, including particles with diameters of up to 60 mm. They were subject to triaxial cyclic loads on both principal directions through distinct stress paths. The system used allows the axial and radial strains to be recorded with a resolution of at least 10^{-5} on the central part of the specimen, ensuring negligible interference with the behaviour of the material to be tested.

The field characterisation showed that in many samples collected in the zones where the railway track revealed good performance, the fouled ballast consisted of limestone, the finest fraction of which mainly resulted from the weathering of the coarser particles and not from other sources, such as the subgrade. It was then considered that these would be the more relevant cases to be studied in order to provide the basis for support of the option of keeping the fouled ballast layer in place under the new sub-ballast layer in the course of renewal works.

Fouled ballast specimens with different grain size distributions were built, with the purpose of representing the

Fig. 6. A specimen prepared for testing

coarsest (C), the medium (M) and the finest (F) grain size distributions found in the fouled ballast layer. Figure 7 shows the corresponding grain size distribution curves, the fouling index and the value of the void ratio; the pictures included in the figure show the aspect of the specimens. The density obtained for the specimens C, M and F are 2.02, 2.16, and 2.33, respectively. The results obtained for the field density, for similar values of the fouling index (samples represented by the trendline presented in Fig. 4), were 2.11, 2.30 and 2.40, respectively.

The specimens were built with 0.06 m thick layers, which were compacted by vibration for nine minutes inside a cylindrical half split steel mould lined by a 1.5 mm thick rubber membrane and by two porous discs and geotextile blankets at the top and bottom. After completion of the specimens, these were kept in an oven at a 40°C temperature for 72 hours, so they could be subsequently tested with similar water content values, at approximately 1%.

The study of the fouled ballast under repeated loading was performed by taking into account the EN13286-7 (CEN, 2000) regarding the method of variable confining pressure, as follows:

- the 1st phase) cyclic triaxial loading (20000 cycles at a frequency of 0.25 Hz), with a high stress level (confining pressure ranging from 0 to 100 kPa and deviator stress ranging from 0 to 600 kPa) in order to stabilize the permanent strains and to attain a resilient behaviour (the so-called conditioning phase);
- the 2nd phase) study of the resilient behaviour by applying 100 load cycles, and then recording the stress and strain values from cycle number 90 to cycle number 100; loading was done through distinct stress paths, with the ratio between the cyclic deviator stress ($q = \sigma_1 - \sigma_3$) and the mean stress ($p = (\sigma_1 + 2\sigma_3)/3$) ranging from 0 to 2.5 (see Table 1).

The stress paths obtained by simulating the loading types usually observed in the platform of railway tracks

are compatible with the resistance of the materials typically employed for that purpose and, therefore, there is no apparent damage to the specimens.

Permanent Deformation Behaviour during the Conditioning Stage

The variation of the permanent strain during the 20000 cycles performed during the conditioning phase is characterised by a rapid increase during the first cycles, followed by progressive stabilisation. The maximum strain values obtained for the specimens C, M and F were 13.4×10^{-4} , 19.3×10^{-4} and 23.5×10^{-4} , respectively.

The first well known relationship describing the variation of permanent axial deformations, ε_1^p , with the number of load cycles, N , is that proposed by Barksdale (1972), who found a linear increase of ε_1^p with the logarithm of the number of load cycles:

$$\varepsilon_1^p = a + b \cdot \log(N) \quad (2)$$

Table 2 presents the values of the parameters of this model calculated from the triaxial test results. The high value of the determination coefficient reveals the appropriateness of the model for the studied material. As regards parameter b , which can be considered as an indicator of the behaviour of the material when subject to repeated loading, the conclusion is that it is low for the fouled ballast with grain size distribution M and C and reaches a fairly high value for the fouled ballast of grain size distribution F. These values point out the least resistance to permanent deformation of the F specimen.

In the first load cycles, the permanent deformation can be particularly conditioned both by the phenomena of the local arrangement of particles, particularly relevant when materials with sizes similar to those under study are tested, and by the initial state of compaction of the specimens. Therefore, a few models provide the reference for the permanent deformation associated with the number of cycles from which the long-term behaviour of the material is assessed, making the latter independent from

Fig. 7. Grain size curves, fouling index, void ratio and an aspect of tested fouled ballast

Table 2. Parameters of the Barksdale permanent deformation model

Specimen	$\varepsilon_1^p (10^{-4}) = a + b \cdot \log(N)$			
	a	b	R ²	$\Sigma (\varepsilon_m - \varepsilon_c)^2$
F	3.01	4.690	0.991	7.45
M	15.13	0.942	0.992	0.26
C	7.61	1.471	0.953	3.79

ε_m —measured permanent axial strain

ε_c —calculated permanent axial strain

Table 1. Stress paths of cyclic triaxial loading (σ_3 minimum = 0 and q minimum = 0)

	$q/p = 0.0$				$q/p = 0.5$				$q/p = 1.5$				$q/p = 2.0$			$q/p = 2.5$		
σ_3 (kPa)	50	100	175	250	50	100	175	250	50	100	150	200	30	60	100	10	15	20
q (kPa)	0	0	0	0	30	60	105	150	150	300	450	600	180	360	600	150	225	300

Table 3. Parameters of the Paute et al. permanent deformation model

Specimen	$\varepsilon_1^p(100)$ (10^{-4})	$\varepsilon_1^p(20000)$ (10^{-4})	A_1 (10^{-4})	B	R^2	$\varepsilon_1^{p*}(20000)$ (10^{-4})
F	11.9	23.5	37.3	0.069	0.994	11.6
M	17.1	19.3	17.3	0.024	0.987	2.0
C	10.6	13.4	2.9	0.489	0.998	2.7

Fig. 8. Rate of change of the permanent axial strain

the permanent deformation which occurs in the first cycles. In those circumstances, another way to model the permanent axial strain accumulated after N load cycles, higher than 100 ($\varepsilon_1^p(N)$) is the one proposed by Paute et al. (1994), which is represented as follows:

$$\varepsilon_1^p(N) = \varepsilon_1^p(100) + \varepsilon_1^{p*}(N) \quad (3)$$

with

$$\varepsilon_1^{p*}(N) = A_1 \cdot \left[1 - \left(\frac{N}{100} \right)^{-B} \right] \quad (4)$$

A_1 and B being parameters obtained from tests.

Table 3 includes the values of the permanent strain after the first 100 cycles and after 20000 cycles, the values obtained using a least squares method, for parameters A_1 and B of the model and the coefficient of determination associated, as well as the values of the strain estimated by the model after 20000 cycles.

It can be seen that the model makes it possible to obtain a good approximation of the behaviour of the materials. It is interesting to note that the value of the A_1 parameter is very small in specimen C, which implies good behaviour with regard to permanent deformation.

Figure 8 depicts the curves relating the evolution in the rate of change of the permanent axial strain (permanent axial strain/cycle) with the value of that strain after the first 100 load cycles. The rate of change of the strain decreases rapidly with the increase in the number of cycles in the C and M specimens. For specimen F, the rate of change of the strain gradually decreased. This behaviour is what was expected given the values of the b parameter of the Barksdale model.

Resilient Behaviour

The interpretation of the 2nd phase test results to estimate the resilient behaviour of the fouled ballast was

Fig. 9. Resilient modulus of the fouled ballast **Fig. 10. Variation of resilient modulus with fouling index ($q/p = 2.0$)**

done, firstly, on the basis of the theory of elasticity in order to obtain the resilient modulus (E) through the equation:

$$E = \frac{\sigma_{1r}^2 + \sigma_{1r}\sigma_{3r} - 2\sigma_{3r}^2}{\sigma_{1r}\varepsilon_{1r} + \sigma_{3r}\varepsilon_{1r} - 2\sigma_{3r}\varepsilon_{3r}} \quad (5)$$

where:

- σ_{1r} —resilient axial stress ($\sigma_{1\max} - \sigma_{1\min}$);
- σ_{3r} —resilient radial stress ($\sigma_{3\max} - \sigma_{3\min}$);
- $\sigma_{1\max}$, $\sigma_{1\min}$ —maximum and minimum total axial stresses in one load cycle;
- $\sigma_{3\max}$, $\sigma_{3\min}$ —maximum and minimum total radial stresses in one load cycle;
- ε_{1r} , ε_{3r} —resilient axial and radial strains.

Figure 9 presents all the E values together with the curves which best fit the experimental results, as a function of the mean stress. Those curves are represented by relations such as $E = a \cdot p^b$, in which a and b are constants.

The resilient modulus increases with the increase in the mean stress and with the decrease in F_1 , as shown in Fig. 10, established for stress ratio $q/p = 2.0$.

The results of the tests were further treated in accordance with theoretical models usually employed to represent the resilient behaviour of conventional aggregates (Lekarp et al., 2000). Among these models, the one that provided most satisfactory results was the Boyce model (Boyce, 1980) modified by Hornyh et al. (1998) in order to take into account anisotropy through the parameter γ , and represented by the following:

$$\varepsilon_v = p_a^{1-n} \cdot p^{*n} \left[\frac{\gamma+2}{3 \cdot K_a} + \frac{n-1}{18 \cdot G_a} (\gamma+2) \cdot \left(\frac{q^*}{p^*} \right)^2 + \frac{\gamma-1}{3 \cdot G_a} \left(\frac{q^*}{p^*} \right) \right] \quad (6)$$

$$\varepsilon_q = \frac{2}{3} p_a^{1-n} \cdot p^{*n} \left[\frac{\gamma-1}{3 \cdot K_a} + \frac{n-1}{18 \cdot G_a} (\gamma-1) \cdot \left(\frac{q^*}{p^*} \right)^2 + \frac{2\gamma+1}{6 \cdot G_a} \left(\frac{q^*}{p^*} \right) \right] \quad (7)$$

with

$$p^* = \frac{\gamma \cdot \sigma_1 + 2\sigma_3}{3} \quad (8)$$

and

$$q^* = \gamma \cdot \sigma_1 - \sigma_3 \quad (9)$$

where ε_v and ε_q are the resilient volumetric and shear strains, σ_1 and σ_3 are the principal stresses and K_a , G_a and n are model parameters.

Figure 11 compares the experimental results with the curves calculated by the Boyce model with anisotropy for different stress paths, q/p . In most cases, the curves obtained by the model closely fit the volumetric and shear strain results.

Fig. 11. Volumetric strain (ε_v) and shear strain (ε_q) observed and calculated by the anisotropic Boyce model: (a) grain size F, (b) grain size M and (c) grain size C

Behaviour of the Wet Material

After characterizing the resilient behaviour of the dry specimens, these were abundantly wetted by imposing a flow of water under a hydraulic head of 1 m and a confining pressure of 30 kPa. After stopping the flow, the specimens were left to freely drain for two hours and then the resilient modulus was determined in a way similar to the second phase, as explained in *Equipment, Specimens and Tests*. The values of the water content of wet specimens evaluated at the end of the tests were 2.1%, 5.5% and 4.9% for the specimens C, M and F, respectively.

Figure 12 summarizes the results obtained from wet specimens. By comparing this figure with Fig. 9, it can be

Fig. 12. Resilient modulus of the fouled ballast in wet condition

Fig. 13. Variation of resilient modulus with fouling index ($q/p=2.0$) – wet specimens

seen that the values of the resilient modulus of wet specimens are generally smaller than those for dry specimens. Moreover, it seems that the fouling index is not as influential on the resilient modulus as it was found to be in the case of the dry specimens (see Fig. 13).

RANKING OF MATERIALS BASED ON RESULTS OF THE TESTS

According to EN13286-7 (CEN, 2000), granular materials used for base layers of pavements are ranked on the basis of two parameters: i) a characteristic value of the resilient modulus, E_c , defined as the resilient modulus determined for the stress values $p = 250$ kPa and $q = 500$ kPa; ii) a characteristic permanent axial strain, ε_1^c , which defines the resistance of the material to permanent deformations, determined from the results of the conditioning. They were calculated as follows:

$$\varepsilon_1^c = \varepsilon_1^p(20000) - \varepsilon_1^p(100) \quad (10)$$

With regard to ε_1^c , the dry fouled ballast exhibited very low values: 2.8×10^{-4} , 2.2×10^{-4} and 11.6×10^{-4} for the specimens C, M and F, respectively. Taking into account the procedure followed in this study (the specimens were wetted after characterizing the resilient behaviour) it is not possible to calculate ε_1^c for wet fouled ballast, because there is no conditioning phase for the wet specimens.

The values of E_c estimated for dry and wet specimens using the Boyce model are presented in Table 4. The reduction in E_c induced by wetting is about 4%, 25% and 34% for F, M and C specimens, respectively. However, E_c values are quite considerable for all the specimens, even higher than typical values for conventional unbound granular materials. Paute et al. (1994) obtained E_c values ranging from 300 to 1000 MPa for well graded crushed aggregates, usually employed as supporting layers of transportation infrastructures.

Taking into account the classification of the unbound granular materials based on mechanical performance parameters E_c and ε_1^c (CEN, 2000), the fouled ballast is classified as a C1 material ($500 \text{ MPa} \leq E_c$ and $\varepsilon_1^c \leq 2.5 \times 10^{-3}$).

Table 4. Characteristic value of the resilient modulus

Specimen	E_c (MPa)		
	F	M	C
Dry material	941	959	1261
Wet material	902	722	838

CONCLUSIONS

Extensive field and lab studies have been carried out during the renewal of the main railway line in Portugal, with the purpose of characterizing the physical and mechanical properties of the existing substructure materials.

The field survey revealed a layer of fouled limestone ballast (ballast with soil), with a non-regular thickness and variable grain size distribution and void ratio.

Plate load tests performed on the substructure showed that the presence of the fouled ballast contributes to increase the deformation modulus, in comparison with the test results from the subgrade soils only.

The resilient behaviour of ballast with distinct fouling indexes (different grain size distribution) was evaluated by employing cyclic triaxial tests. During the conditioning phase of the samples, the permanent deformation obtained after 20000 load cycles was higher in samples with greater fouling index.

The results provide evidence that the resilient modulus of the fouled ballast depends not only on the applied stress but also on the fouling index and on the material state (dry or wet). When the specimens are dry, the resilient modulus obtained is higher in the coarser materials. When the specimens are wet, the resilient modulus decreases and this seems to be less dependent on the fouling index. This assumption needs to be confirmed by further research.

The resilient behaviour and the permanent deformation of the fouled ballast can be represented by well-known models commonly applicable to unbound granular materials. The results obtained, namely with regard to the characteristic resilient modulus, are similar to those for well graded crushed aggregates, usually employed as supporting layers of transportation infrastructures.

The overall evaluation of the results obtained in the field and in the lab tests provided the basis to support the decision of maintaining the fouled ballast layer under the new sub-ballast in a number of stretches of the renewed line.

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